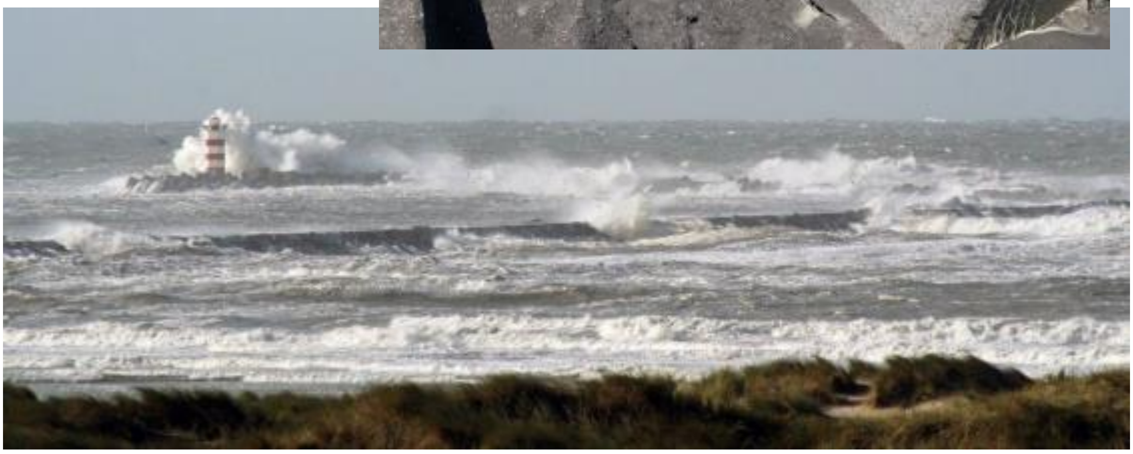
IJmuiden Breakwater

Upper image: Large blocks cover the breakwater slope, visible during calm weather conditions

Lower image [Ref. 11]: The breakwater has to endure extreme weather conditions and wave attack



EVALUATION OF NEW EUROCODE ‘PREN1991-1-8 ON WAVE AND CURRENT ACTIONS

CASE STUDY IJMUIDEN – DOES THE NEW EUROCODE IMPROVE THE CURRENT DESIGN PRACTICE OF BREAKWATERS AND OTHER COASTAL STRUCTURES?

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Evaluation of new Eurocode 'prEN1991-1-8' on wave and current actions:

Case study IJmuiden - Does the new Eurocode improve the current design practice of breakwaters and other coastal structures?

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In partial fulfilment of the demands for the

Master of Science of **Civil Engineering**

at Delft University of Technology,

Published on Thursday the 10th of March 2022

Publicly defended on Wednesday the 16th of March 2022

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PREFACE

This is the final progress report of the thesis that evaluates the draft version 'Eurocode 1: Actions on structures — Part 1-8: General actions — Actions from waves and currents on coastal structures'. The report has been written in partial fulfilment of the demands for the Master of Science of Civil Engineering at Delft University of Technology. In addition, Rijkswaterstaat is acknowledged as a third party involved in the realisation of this report

The report is mainly intended for reading by people concerned with the development of the draft Eurocode towards a definitive document, and by coastal engineers that want to know how this part of the Eurocode will work out for the future design practice. Knowledge regarding hydrodynamics and probabilistics, especially those aspects related to the design process of breakwaters, is considered to be known.

Readers interested in the differences and similarities of the draft Eurocode prEN1991-1-8 with respect to the general Eurocode EN1990 are referred to chapter 2. Readers interested in the case study, which is the re-design of the breakwaters of IJmuiden following the draft Eurocode, are referred to chapters 3, 4 and 5. Readers interested in the comparison between a deterministic design approach, semi-probabilistic design approach and full probabilistic design approach, treated in the case study, are referred to chapter 6.

A lot of gratitude is owed to the commission members dr. ir. B. Hofland, ir. W.F. Molenaar, ing. C. Kuiper and ir. J.P. van den Bos for their support during the trajectory of graduation, and for their valuable feedback.

Dordrecht, March 2022
Mats van Gemert

SUMMARY

CONTEXT

The Eurocodes are design codes, widely accepted throughout Europe, that provide guidelines on how to deal with loads, resistance, and uncertainty. For most type of structures, loads and materials, the Eurocode has been well developed. However, for the loads on coastal structures, caused by wave, water level and current actions, no such Eurocode exists. Therefore, a new Eurocode is in progress. The preliminary version of this Eurocode is titled prEN1991-1-8, and will describe how to deal with hydraulic loads on coastal structures.

The Eurocode has a safety philosophy based upon partial factors and consequence classes, but the extension of this method to hydraulic engineering is not so straightforward. A premature introduction of the new Eurocode could mean that important design aspects related to sea condition parameters, such as extreme value analysis, dependence and wave transformation, are underexposed. In turn, this might give way to unsafe structures. Because of this, it is important to investigate how the semi-probabilistic approach (proposed by prEN1991-1-8) compares to existing design methods, both in terms of safety and in terms of ease of application. The goal of this thesis is to acquire knowledge on how the introduction of the new Eurocode influences the design of coastal structures, and address any inconsistencies and issues in prEN1991-1-8 before the preliminary version is adopted as the definitive version.

METHOD

The main focus of the research is the re-design of a coastal structure following the instructions in prEN1991-1-8. The breakwaters of IJmuiden are used as a case study for this. A case study has been opted for so as to not only discover fundamental flaws that are present in the draft Eurocode, but to discover practical issues in working with the code as well.

Three different approaches are adopted for the design: a deterministic approach, a semi-probabilistic approach and a full probabilistic approach. The breakwater elements that will be extensively examined in this thesis are the armour layer, consisting of either rock or artificial units, the crest height and the crown wall.

Conclusions have been drawn based on any unclarities that are encountered in the design process, and based on comparisons between the semi-probabilistic approach and the two existing design methods, respectively.

RESULTS

The breakwater design that has been arrived at by following the semi-probabilistic approach described in prEN1991-1-8, did not resemble the design outcome obtained with the deterministic and full probabilistic approach, as can be seen from the design results in the table that is presented on the next page.

For the case study of IJmuiden, Design approach DA-1 structurally underestimated the required size or height of breakwater elements compared to the DA-2 approach for the Serviceability Limit State-(LD). For the Ultimate Limit State, there is no such similarity for the various breakwater elements when comparing DA-1 and DA-2. The rock armour layer provided an overestimation, the artificial unit size was spot on, and the base thickness of the crown wall gave an underestimation.

In addition, it can be noticed that DA-0 structurally overestimates the required size or height of breakwater elements compared to the DA-2 approach, for SLS-(LD) as well as ULS. Using this approach is thus perhaps conservative, but the structures that are designed with this approach will at least measure up to the target reliability.

Breakwater element	Limit state	Deterministic Result (DA-0)	Semi-probabilistic Result (DA-1)	Full probabilistic result (DA-2)
Armour layer – rock size (D_{n50})	SLS-(LD)	2.21 [m]	1.74 [m]	2.03 [m]
Armour layer – rock size (D_{n50})	ULS	1.63 [m]	1.85 [m]	1.52 [m]
Armour layer – artificial units (D_n)	SLS-(LD)	2.10 [m]	1.25 [m]	1.59 [m]
Armour layer – artificial units (D_n)	ULS	2.07 [m]	1.73 [m]	1.72 [m]
Crest height (A)	SLS-(LD)	13.27 [m+NAP]	10.35 [m+NAP]	12.42 [m+NAP]
Crown wall – base thickness (t_1)	ULS	2.25 [m]	1.39 [m]	1.89 [m]

The differences in these results do not necessarily mean that the entire DA-1 framework is wrong, as these differences may very well originate from simplifications that were made in the case study, or misinterpretations of what is in the draft Eurocode, which can still be fixed before the definitive version is introduced.

CONCLUSIONS

The introduction of the new document prEN1991-1-8 should result in more conformity regarding the treatment of sea condition parameters and their accompanying uncertainties in design calculations. However, the new (draft) Eurocode in its current form does not seem to achieve this objective.

In addition, the semi-probabilistic approach is still a time-consuming method when applied to breakwater elements, while it is relatively easy to set up a full probabilistic calculation. Hence, it is questionable whether the new (draft) Eurocode on wave and current actions has a positive impact on the design of coastal structures compared to commonly used existing design methods (see e.g. the Rock Manual [Ref. 6]).

Nevertheless, it can be viewed as a positive development that there will be one general document to consult for the design of coastal structures. Most importantly, standardised levels of safety and return periods are now available with the introduction of prEN1991-1-8, even though their interpretations are not always straightforward. With a couple of simple adjustments, many of the flaws of the draft Eurocode can already be taken away.

Three conclusive statements can be made with respect to the draft Eurocode and its shortcomings:

1) A consistent use of the semi-probabilistic approach in the draft Eurocode cannot be guaranteed, as one user might interpret its content differently than another user.
   - For the armour layer consisting of rock, these misinterpretations piled up to a factor of approximately 2.5 (D_{n50} of 1.20 m vs. D_{n50} of 2.90 m) in the case study of IJmuiden.

2) The descriptions in prEN1991-1-8 on how to determine wave properties and water levels are not specific enough.
   - This problem originates from the fact that it is not straightforward to translate the concepts of EN1990, which describes the basis of structural design, to hydraulic engineering (and thus to Eurocode prEN1991-1-8). Part of this problem is related to the necessity of performing an Extreme Value Analysis, the remaining part of this problem lies predominantly in the dependency between several sea condition parameters. There should be a clear instruction of how to deal with this, but instead concepts like correlation or the relationship between wave height and wave period are insufficiently explained.

3) While the partial safety factor method should deal with the uncertainties related to the estimation of sea condition parameters and design of coastal structures, the application of it raises some questions that are left unanswered by the draft Eurocode. Examples of issues that were encountered in the case study include:
    - The absence of an explicit partial resistance factor in the DA-1 framework.
    - The underexposure of the water level as a hydraulic action throughout the entire draft Eurocode, i.e. no clarity is given on how to properly treat the different components of the water level such as surge, tide and sea-level rise. Moreover, it is remarkable that, as opposed to the wave height, water levels are not factored.
    - The omission of specifications on how the partial load factor should properly be applied, i.e. whether it should be applied to the force or to the wave height in situations where both are possible, how to adequately take into account the phenomena that arise due to depth-limitations, etc.
    - The doubts that surround the newly introduced Serviceability Limit State-(LD) for coastal structures, i.e. its position relative to the other limit states and its concerns related to the achieval of the target reliabiltity that arise because of the (lack of) treatment of response model uncertainties and sea climate uncertainties.

RECOMMENDATIONS

Following the conclusions, it is advised to re-evaluate the relevance of a semi-probabilistic approach in the light of coastal structures.

One of the opportunities that is presented by prEN1991-1-8, is the inclusion of the full probabilistic approach into the design framework, which is given as an option in the draft Eurocode. It is recommended to shift (at least part of) the attention towards the full probabilistic approach, and explore the possibilities of making it the default approach for coastal structures, as it raises fewer questions than the semi-probabilistic approach and deals with uncertainties more extensively. For this to work, the description of distributions related to hydrodynamic loads and other model/resistance parameters needs to be more elaborate, and it should be described more clearly how to interpret the safety levels.

Instead of focusing on the full probabilistic approach, it is of course also possible to improve the semi-probabilistic approach as it is currently proposed by prEN1991-1-8. When this is preferred, it is recommended to include a more systematic explanation of the DA-1 format in EN1991-1-8, describing the characteristic values, partial factors and safety margins to be adopted in design without any ambiguity.

Besides that, it is recommended to work out more test cases with prEN1991-1-8, as it is difficult to assess the quality of the draft Eurocode based on just a single case study.

CONTENT

LIST OF DEFINITIONS AND SYMBOLS

DEFINITIONS

The list of definitions contains important terminology that you will come across when using either prEN1991-1-8 [Ref. 2] or EN1990 [Ref. 1]. These sources have been used in the creation of the list.

Accidental action	Action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life
Accidental design situation	Design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure
Basic variables	Part of a specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities, and material properties including soil properties
Characteristic value	Principal representative value of an action or of the strength/resistance
Characteristic load combination	The load combination that is used to determine the combined design load for persistent and transient design situations in SLS and/or SLS-(LD)
Coastal structure	Coastal structures are defined as those in the coastal zone and exposed to actions arising from environmental sea conditions, specifically waves, water-levels and currents and where those actions are likely to be the dominant action(s) affecting the load case of the structure. Examples include floating structures, breakwaters, coastal embankments, etc.
Combination of actions	Set of design values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions
Combination value	Value chosen so that the probability that the effects caused by the combination will be exceeded is approximately the same as by the characteristic value of an individual action
Consequence class	Specification of the potential consequences of damage and/or failure
Design Approach	Risk related to structure/wave interaction; collective description of the four methods presented in the draft Eurocode which can be applied in the design for assessing actions on coastal structures to achieve a predefined safety level against sea-borne hydrodynamic actions
Design service lifetime	Assumed period for which a structure or part of it is to be used for its intended purpose
Design situations	This refers to circumstances under which the structure is required to fulfil its function. Sets of physical conditions representing the real conditions occurring during a certain time interval are used to demonstrate that relevant limit states are not exceeded for the design

List of definitions and symbols

Design value	Value that is obtained after the partial factor is applied to the representative value
Design wave height	Wave height selected for functional design, structural design and stability analysis of coastal structures
Design water level	Water level selected for functional design, structural design and stability analysis of coastal structures
Extreme value	Representative value of a parameter used in limit state checks
Fatigue design situation	Design situation that is relevant in case of cyclic loading
Frequent load combination	See characteristic load combination, but with combination values replaced by frequent values
Frequent value	Value determined so that either the total time, within the reference period, during which it is exceeded is only a small given part of the reference period, or the frequency of it being exceeded is limited to a given value
Fundamental load combination	The load combination that is used to determine the combined design load for persistent and transient design situations in ULS
Hydraulic Action	Hydraulic actions refer to hydraulic sea conditions such as wave height, water level and current velocity
Hydraulic Action Effect	Hydraulic action effects refer to the pressure distributions or total forces moments as a result of hydraulic actions
Hydrodynamic Estimate Approach	Relates to uncertainty of structure exposure and environmental sea conditions and consequence class
Hydrodynamic uncertainty	Hydrodynamic uncertainty ranges from low to high, and is governed by the quality/quantity of the data and the level of understanding of the sea conditions
Limit states	States beyond which the structure no longer fulfils the relevant design criteria
Permanent action	Action for which the variation in magnitude with time is negligible
Persistent design situation	Design situation that is relevant during a period of the same order as the design working life of the structure
Quasi-permanent value of a variable action	Value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period
Quasi-permanent load combination	See characteristic load combination, but with combination values replaced by quasi-permanent values
Reference period	Chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions
Reliability	Reliability is the ability of a structure or a structural member to fulfil the specified requirements for which it has been designed.

List of definitions and symbols

Return period	The period that (on average) separates two occurrences of equal or greater magnitude
Representative value	A single value that is adopted to represent the entire load or resistance distribution; either the characteristic value, the combination value, the quasi-permanent value or the frequent value
Seismic design situation	Design situation involving exceptional conditions of the structure when subjected to a seismic event
Serviceability Limit State	State that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met
Serviceability Limit State-(Limited Damage)	Limit state where the structure is only slightly damaged and it is deemed appropriate to undertake repairs; used mainly for the design of engineered mound protection
Structure response uncertainty	Uncertainty that depends on the acceptance of the design formulae and methods used and the knowledge of the response properties of the materials to be used
Transient design situation	Design situation that is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence
Ultimate Limit State	State associated with collapse or with other similar forms of structural failure
Variable action	Action for which the variation in magnitude with time is neither negligible nor monotonic

SYMBOLS AND ABBREVIATIONS

The symbols and abbreviations used in this document are presented below, though this list might not be exhaustive. For any missing symbols, the reader is redirected to the documents in the list of references.

A	Accidental action [terminology in EN1990]
A	Crest height [in design of breakwaters]
A_c	Armour crest freeboard; distance between water level and level of the armour crest
α	Armour slope angle [in design of breakwaters]
α	Influence parameter [in probabilistic calculation]
β	Reliability index
C_b	Width of the crown wall
C_h	Height of the crown wall
C_{pl}	Empirical coefficient in the Van der Meer equations
C_1	Empirical coefficient in the overtopping formula
C_2	Empirical coefficient in the overtopping formula
CC	Consequence class
CMU	Model uncertainty in empirical design model of crown wall
CSE	Central statistical estimate
CV	Coefficient of variation
γ_β	Factor taking into account oblique wave attack
γ_f	Factor taking into account the roughness of artificial units
γ_E/γ_S	Partial action factor
γ_G	Partial factor to be applied to a permanent action
γ_Q	Partial factor to be applied to a variable action
γ_{Qz}	Partial factor to be applied to the wave height
γ_R/γ_M	Partial resistance factor
d	Used as a subscript to indicate that the design value is referred to
D	Storm duration
D_n	Nominal diameter of artificial units
D_{n50}	Nominal rock diameter

List of definitions and symbols

$DA-0$	Design approach 0; deterministic method
$DA-1$	Design approach 1; semi-probabilistic method
$DA-2$	Design approach 2; full probabilistic method
$DA-3$	Design approach 3; economic optimisation method
$DA-4$	Design approach 4; method assisted by testing
Δ	Relative buoyant density
EVA	Extreme Value Analysis
f	Friction factor
F_c	Distance between the base level of the crown wall and the water level
F_G	Downward acting force on the crown wall as a result of its self-weight
$F_{h0.1\%}$	Overturning moment acting on the crown wall caused by the wave height that is exceeded by only 0.1% of the waves
F_u	Up-lift force
$FORM$	First-order reliability method
g	Gravitational acceleration
G	Permanent action
h	Water depth
h_t	Water depth on top of the toe
H_{m0}	Significant wave height determined with spectral analysis
H_s	Significant wave height
$H_{s,max}$	Maximum significant wave height in depth-limited conditions
$H_{1/3}$	Significant wave height determined with time-series analysis
HEA	Hydrodynamic Estimate Approach
η	Water level
k	Used as a subscript to indicate that the characteristic value is referred to
K_F	Consequence factor
L_m	Wavelength based on mean wave period
L_{0m}	Deep-water wavelength based on mean wave period
L_{0p}	Deep-water wavelength based on peak wave period

List of definitions and symbols

M_{cw}	Mass of the crown wall
M_G	Resisting moment of the crown wall as a result of its self-weight
$M_{h0.1\%}$	Overturning moment acting on the crown wall caused by the wave height that is exceeded by only 0.1% of the waves
M_U	Overturning moment acting on the crown wall caused by the up-lift force
M_{50}	Median rock mass
MCM	Monte Carlo Method
μ	Mean value of a distribution
μ_1	Statistical uncertainty related to the uncertainty in the fit of the extreme wave distribution
μ_2	Statistical uncertainty related to the uncertainty in the fit of the extreme water level distribution
N	Number of waves
N_{od}	Damage number expressed by the relative number of displaced units out of armour layer
N_s	Number of storms per year [in Peak-over-Threshold method]
N_s	Hydraulic stability number [in design of breakwaters]
NAP	Normaal Amsterdams Peil [datum level]
ξ	Surf similarity parameter
P	Notional permeability [in design of breakwaters]
P	Exceedance probability [in Peak-over-Threshold method]
p_f	Probability of failure
$P_{bFh0.1\%}$	Up-lift pressure at the seaward side of the crown wall, generated by the same wave that caused $F_{h0.1\%}$
ρ_c	Density of concrete
ρ_s	Density of rock
ρ_w	Density of water
q	Mean overtopping discharge
q_{tol}	Tolerable overtopping discharge
Q	Variable action [Terminology in EN1990]
Q	Dimensionless overtopping discharge [in design of breakwaters]
Q	Non-exceedance probability [in Peak-over-Threshold method]
R_c	Freeboard; distance between water level and level of top of the crest
R_d	Design value of resistance

List of definitions and symbols

R_k	Characteristic value of resistance
$R_{u0.1\%}$	Run-up level exceeded by only 0.1% of the waves at the toe of the structure
$RMSE$	Root-mean-square error
RP	Return period
s_{0m}	Fictitious wave steepness corresponding to mean wave period
s_{0p}	Fictitious wave steepness corresponding to peak wave period
$S_{(d)}$	Damage parameter expressed by the non-dimensional eroded area
S_d	Design value of action
S_k	Characteristic value of action
SLS	Serviceability Limit State
$SLS-(LD)$	Serviceability Limit State-(Limited Damage)
σ	Standard deviation of a distribution
T_{life}	Design service lifetime of the structure
T_m	Mean wave period determined with time-series analysis
T_{m02}	Estimation of mean wave period with spectral analysis
T_p	Peak wave period
t_1	Thickness of the bottom slab of the crown wall
t_2	Thickness of the vertical face of the crown wall
ULS	Ultimate Limit State
V	Unit volume of artificial units
V_{cw}	Volume of the crown wall
X	Design point in full probabilistic method
ψ	Reduction factor applied to the characteristic value to obtain either the combination value (with subscript 0), the frequent value (with subscript 1) or the quasi-permanent value (with subscript 2) of a variable action
Z	Limit state function

List of definitions and symbols

1. INTRODUCTION

1.1 PROBLEM DESCRIPTION

The Eurocodes are design codes, widely accepted throughout Europe, that provide guidelines on how to deal with loads, resistance, and uncertainty. Currently, there are a total of 10 Eurocodes in use. For many types of structures, loads and materials, these Eurocode have been well developed. However, until now for the loads on coastal structures, caused by wave, water level and current actions, no such Eurocode exists. Therefore, a new Eurocode is in progress and a draft version has come available in June 2020. This document is titled prEN1991-1-8 [Ref. 2], and will describe how to deal with these types of loads on coastal structures. In the absence of a Eurocode that describes wave and current actions, many countries currently have their own regulations for the design of coastal structures. Hence, the current design practice in hydraulic engineering is far from standardised. The main goal of the code is to uniform design methods and safety standards.

The new draft Eurocode[1] on wave and current action is intended to fit into the design framework of the already existing Eurocodes, which has a safety philosophy based upon partial factors and consequence classes. After years of development the first draft has been issued to designers. Some of the methods in the code are still under discussion and it is not clear how the new Eurocode on wave and current actions will work out in actual design practice. The extension of the partial safety factor method as applied in other Eurocodes is not yet implemented in the current design practice in hydraulic engineering. How the introduction of the partial safety factor method influences the design of coastal structures is not known, as nobody has a working experience with the code yet.

In addition to the TU Delft, the institution that has commissioned the thesis, Rijkswaterstaat is also keen on investigating the effect of the new code on the existing design manuals for coastal and inland waterways as water safety is of key priority for the Netherlands. Rijkswaterstaat is a governmental organisation, and is a major client of Dutch (water-related) projects. If the new Eurocode, which will be introduced in the near future, greatly deviates from the present Dutch standards or turns out to be impractical to work with, this will become a source of potential conflict between Rijkswaterstaat and its contractors.

Therefore, the main goal of this thesis is to have a critical look at the new standardised approach (proposed by prEN1991-1-8) for designing coastal structures and bring any uncertainties and inconsistencies to light, so that they can still be addressed before the draft version of the new Eurocode is adopted as the definitive version. It should be noted that the focus of the new Eurocode is on creating a standard framework that establishes how to determine the hydraulic boundary conditions near the site of interest, and not so much on the design rules and equations itself.

[1] The terms new Eurocode, draft Eurocode and new draft Eurocode have been used interchangeably throughout this document, and all refer to prEN1991-1-8

1.2 RESEARCH QUESTION AND SUB-QUESTIONS

The main research question of this thesis is:

How does the new Eurocode on wave and current action change the design of coastal structures, such as breakwaters and embankments, compared to commonly used existing design methods?

In order to answer this question, some sub-questions are formulated:

(1) How much room for interpretation exists in the new Eurocode, that could possibly lead to varying designs, and how does this compare to conventional existing design methods?

(2) How are the uncertainties related to the estimation of sea condition parameters and design of coastal structures dealt with in the new Eurocode?

(3) How does a design following the partial safety factor method compare to a design following a conventional deterministic design method?

(4) How does a design following the partial safety factor method compare to a full probabilistic design?

(5) *Does the new Eurocode improve the current design practice of breakwaters and other coastal structures?*

1.3 RESEARCH METHOD AND OUTLINE

The effect of the new draft Eurocode on wave and current actions will be evaluated using a case study: The design of the breakwater of IJmuiden, which protects the major lock entrance to the port of Amsterdam. The historical background of this case study will be elaborated upon in Section 1.4.

Figure 1.1: The IJmuiden breakwaters as seen from above [Ref. 4]

A case study has been opted for so as to not only discover fundamental flaws that are present in the draft Eurocode, but to discover practical issues in working with the code as well.

The approach for the evaluation follows the scheme presented in Figure 1.2:

Figure 1.2: Research method steps

The outline of this thesis follows the research method steps in Figure 1.2, which is why an elaboration of these steps is described here in combination with the outline.

The literature study is the starting point of the research and is required to gain knowledge on the topic. A separate chapter that serves as a summary of the literature study has been included in the thesis. Chapter 2 describes, in more detail, the framework of the Eurocode and explains how the new draft Eurocode fits into this. This will be done by finding the most important concepts in the basis of the structural design of the Eurocode, described in EN1990 [Ref. 1], and examine how this has been translated to prEN1991-1-8, in order to find any knowledge gaps. Moreover, it explains additions to this framework by the (new) draft Eurocode, that are specifically relevant for designing coastal structures. Finally, the formulae that will be used for the design of the breakwater elements are presented. Chapter 2 is the primary chapter that tries to answer sub-question 1, but in fact this sub-question is treated throughout the entire document.

The next steps focus on the acquisition and analysis of data, and the derivation of the boundary conditions near the design site with the aid of the (new) draft Eurocode. Chapter 3 first establishes the structure and design specification and defines the basic variables. With the help of data and the descriptions in the draft Eurocode, the hydraulic boundary conditions that will be used for design purposes are determined. Furthermore, it describes the bathymetry near the breakwaters and deals with wave height-water level correlation and offshore-nearshore wave transformation. By systematically following the draft Eurocode, any unclarities in the design process are discovered.

The design will then be executed threefold. Chapter 4 sets out the breakwater design following the partial safety factor method. Any uncertainties that are not resolved by the new draft Eurocode are addressed, resulting in a range of possible design outcomes presented in a sensitivity analysis. Sub-question 2 is primarily treated in chapters 2, 3 and 4.

Chapter 5 uses alternative design approaches to come up with a design for the various breakwater elements. A deterministic approach will firstly be adopted. Secondly, a full probabilistic calculation is executed, which demonstrates the parameters that exert most influence on the design outcome.

The final steps consist of the comparative analysis and the conclusions. Chapter 6 compares the results found in chapter 4 to those found in chapter 5. It reports the similarities and differences, and the findings that can be derived from this. Finding an answer to sub-questions 3 and 4 is a combined effort of chapter 4 up to and including chapter 6.

Finally, the main question of the thesis is answered in the conclusions, and recommendations are given for further research. The answer to sub-question 5 can be found in the conclusions and recommendations, as is the case for the main research question.

1.4 TEST CASE IJMUIDEN

The new Eurocode is evaluated by applying its content to a specific design case. To this end, the breakwaters at IJmuiden will be used as a test case. The site was chosen as a test case because several years of data are available at a point nearby, and its site conditions fall into the range of application of prEN1991-1-8. Figure 1.3 indicates the design site of the breakwaters, together with the cross-section of interest:

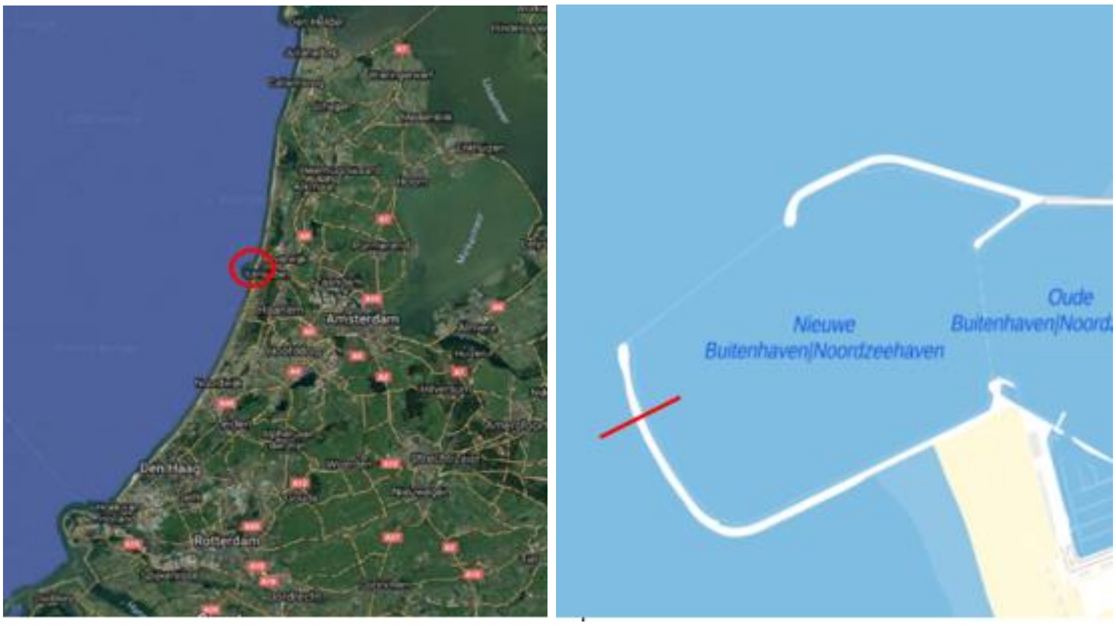


Figure 1.3: Location (left; taken from Google Maps) and orientation (right) [Ref. 15] of the IJmuiden breakwaters. The red line indicates the cross-section that will be designed

The IJmuiden breakwaters have been there since the late 19th century, and were extended into the sea in the 1960's. The extension was necessary to provide shelter for the vessels that were becoming increasingly larger in size. The function of the breakwaters is therefore to create a safe harbour basin for ships seeking entrance to the port of Amsterdam [Ref. 3].

The cross-section of the existing breakwater consists, on the seaward slope, of single layer cubes on an impermeable asphalt layer. There have been many struggles since the breakwaters were extended seawards. Figure 1.4 shows how the design of the breakwaters came about:

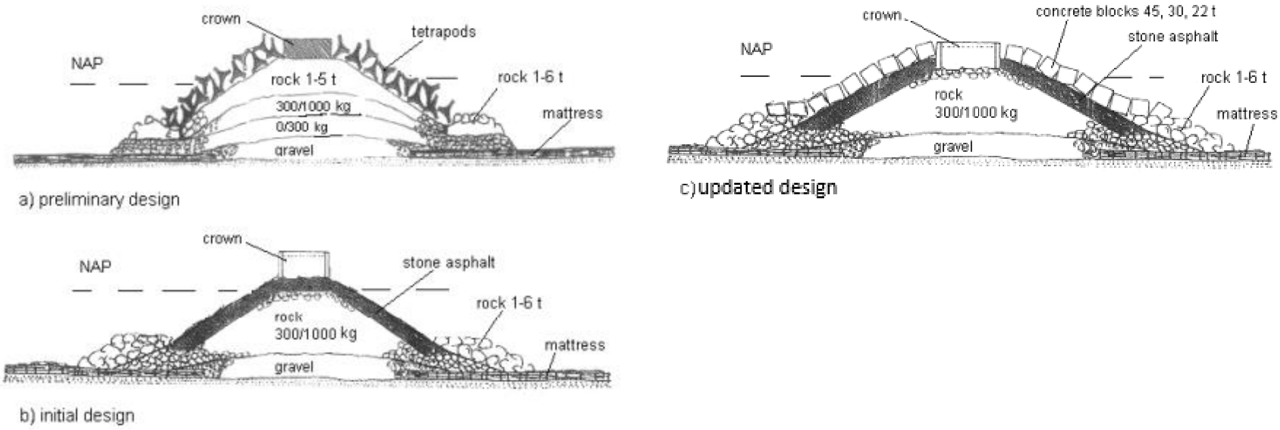


Figure 1.4: IJmuiden breakwaters design process in the 1960's [Ref. 15]

The initial design was a closed structure in which an asphalt layer had to ensure stability of the breakwater. However, the slope of the breakwater was not altered compared to the preliminary design. This led to major stability problems for the asphalt layer during construction because it was too steep. Large concrete cubes were placed on top of the asphalt layer, but this did not improve the stability during design conditions. In addition, the blocks were subject to deterioration [Ref. 3].

Differentiation occurs along the length of the breakwater. There are lighter concrete blocks closer to the shore, since the wave attack is smaller there. The slope of the breakwater had to be decreased at for example the heads to ensure stability. In Table 1.1, values for several hydrodynamic conditions that were used for the original design are listed, together with some of the dimensions of the breakwater. The intention of these values is to give an idea of typical orders of magnitude.

Parameter	**Value**
Design wave height H_s	7 m
Design (peak) wave period T_p	9 s
Water depth	Order of magnitude of 15 m near breakwaters
Water level	NAP +2.5 m
Frequency design conditions	1/50 year
Length of extension into sea	2 km
Thickness asphalt layer	2.25 m
Slope	1:1.5
Width crown element	7 m
Height crown element	2.5 m (NAP +2.2 m till NAP +4.7 m)

Table 1.1: Important design parameters and dimensions for the original breakwaters [Ref. 15]

In the past, the breakwaters have been damaged to quite a large extent. Discussions are still ongoing on whether a reconstruction is necessary or whether just continuing with doing maintenance works suffices. An important lesson that can be learnt from history is that the breakwaters should just be an open structure. A closed structure, an asphalt layer with concrete blocks on top of them, is not an ideal solution. Furthermore, it is evident that the armour layer will consist of rock or concrete units that are quite large, in order to be able to withstand the design waves.

1.5 SCOPE

Below, a list of statements is presented that delimit the scope of the thesis:

- The version of the new (draft) Eurocode prEN1991-1-8 that is referenced to throughout the thesis is the one issued in June 2020. Any relevant tables from this document have been added to Appendix A.
- Apart from this document, the new Annex A.6 that will be added as a supplement to EN1990, issued in March 2021, is also taken into account. This Annex contains important information (provided in Table A.6.8 in said Annex) on the application of partial factors. This table can be found in Appendix B.
- Any other later issues of or adjustments to prEN1991-1-8 are left outside the scope of the thesis.
- The semi-probabilistic partial factor approach (DA-1) will be taken as the design approach that forms the baseline, to which the other design approaches are compared.
- The deterministic approach (DA-0) as described by the draft Eurocode, has been adopted to represent the conventional deterministic method that is applied in current design practice.
- The full probabilistic approach (DA-2) as described by the draft Eurocode, has been adopted to represent the probabilistic method that is applied in current design practice.
- For the purpose of this thesis, it is not necessary to design every element of the breakwater in great detail. Figure 1.5 presents the main elements and failure modes of a breakwater:

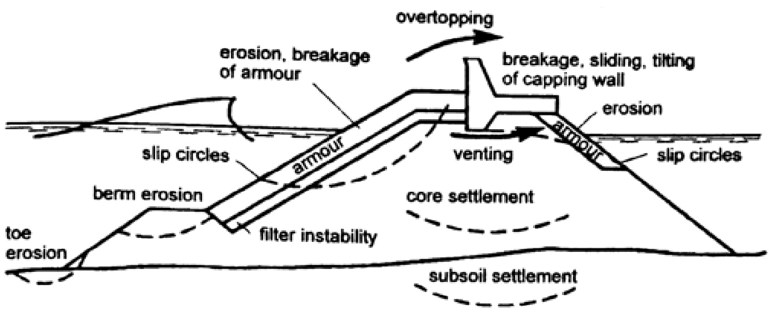


Figure 1.5: Schematisation of breakwater elements and its main failure mechanisms [Ref. 21]

The elements which are part of the scope of the thesis are:
- the seaward armour layer, consisting of either rock or artificial units (failure mechanism erosion)
- the crest height (failure mechanisms related to overtopping)
- stability of the crown wall (failure mechanisms sliding and tilting).

Considerations for any other elements that are required to complete the cross-sectional design will not be as extensive.

- Only the preliminary design stage is considered, which means physical modelling will not be carried out, even though this might be prescribed by the new Eurocode.
- The location and orientation of the breakwaters will be the same as that of the original. Apart from this aspect, the re-design will not take into account the existing situation/breakwater.
- One cross-section of the Southern breakwater will be designed. The cross-section will be located at a point furthest away from the shore on the Southern breakwater, such that the largest possible wave attack is considered. This particular cross-section has been highlighted in Figure 1.3.
- The rock-armoured slope is a standard two-layer armour layer.
- The slope with artificial units is a modern randomly placed single-layer armour layer (e.g. Accropode, Cubipod or Xbloc).
- It will be assumed that waves are not limited by depth at the toe of the breakwaters. A simplified offshore-nearshore wave transformation will be performed to confirm this assumption. The intention

of this offshore-nearshore transformation is not to transfer all design variables to the site of interest. The analysed data at the location of measurements will simply be adopted to represent the conditions at the design site, as the hydraulic data was retrieved close to the site of interest.

- The influence of currents on the wave height is neglected.
- A simplified Extreme Value Analysis will be performed on the wave heights (and water levels). No segmentation of directions, and the wave attack is assumed to be perpendicular to the slope of the breakwater.
- It is assumed that wave height and water level can be treated independently of one another for the purpose of this thesis. This means no joint probability analysis will be executed. Even though a joint probability analysis could provide valuable insights, there is no accepted way to do this, and working with more advanced tools such as copulas would take too much time.
- The accidental design situation is left outside the scope of the thesis, as accidental actions are already covered by EN1991-1-7.

1. Introduction

2. LITERATURE

The purpose of this chapter is to elaborate on the present state of knowledge of the fields that are important for this study. The literature can be divided into roughly three categories: codes and design guidelines, documents containing information on probabilistics, and theory on hydrodynamics. As the research is design-oriented, the majority of the chapter consists of the explanation of codes and design guidelines. The most relevant code in this respective is of course the Eurocode. The first section will describe the framework of the Eurocode and the introduction of the new draft Eurocode. The second section dives into uncertainties and the way they are treated in various existing design methods. The final section presents the design formulae that will be used in the succeeding chapters.

2.1 THE EUROCODE

The Eurocode consists of a total of ten main documents, listed below [Ref. 2]:

- EN 1990 Eurocode 0: Basis of structural and geotechnical design
- EN 1991 Eurocode 1: Actions on structures
- EN 1992 Eurocode 2: Design of concrete structures
- EN 1993 Eurocode 3: Design of steel structures
- EN 1994 Eurocode 4: Design of composite steel and concrete structure
- EN 1995 Eurocode 5: Design of timber structures
- EN 1996 Eurocode 6: Design of masonry structures
- EN 1997 Eurocode 7: Geotechnical design
- EN 1998 Eurocode 8: Design of structures for earthquake resistance
- EN 1999 Eurocode 9: Design of aluminium structures

Most of these Eurocodes are in turn subdivided into more specific parts. The 'EN1991 Eurocode 1: Actions on structures' is split up into the following parts:

- EN1991-1-1: Densities, self-weight, imposed loads for buildings
- EN1991-1-2: Actions on structures exposed to fire
- EN1991-1-3: General actions - Snow loads
- EN1991-1-4: General actions - Wind actions
- EN1991-1-5: General actions - Thermal actions
- EN1991-1-6: General actions - Actions during execution
- EN1991-1-7: General actions - Accidental Actions
- **EN1991-1-8: General actions – Actions from waves and currents on coastal structures**
- EN1991-2: Traffic loads on bridges
- EN1991-3: Actions induced by cranes and machinery
- EN1991-4: Silos and tanks

The new Eurocode, *'prEN1991-1-8: General actions — Actions from waves and currents on coastal structures'* [Ref. 2] has been added as a first draft to Eurocode 1. The EN1990 [Ref. 1] is the governing code and is therefore also relevant to this new code, as it provides the basis of design. In addition to these documents, countries may also specify National Annexes as an extension of the general documents, or to specify specific values for parameters which are valid for only a certain country.

The remaining Eurocodes all deal with specific types of structures, and are less relevant for the purpose of this thesis.

2.1.1 EN1990 EUROCODE 0: BASIS OF STRUCTURAL AND GEOTECHNICAL DESIGN

The design method described in EN1990 is a semi-probabilistic approach. The main principle behind such an approach is that a single design value is adopted that represents the variable distribution of a parameter. The calculations will then be performed with these design values, which should guarantee the required level of safety. This as opposed to simply specifying a somewhat randomly chosen global safety factor, as one would do in a deterministic approach, or working with distributions of the relevant variables, as one would do in a full probabilistic approach.

A structure is safe when the resistance exceeds the load. When performing design calculations according to EN1990, this means that the design value of the resistance should exceed the design value of the load. This statement can be translated into a limit state function:

$$Z = R_d - S_d \geq 0$$

In this limit state function Z, R represents the resistance or strength whereas S represents the load. The subscript d indicates that the design value referred to, whereas a subscript k refers to the characteristic value. The design value can be split up into two parts: a characteristic value of the load and resistance, multiplied or divided by a partial factor, respectively. The equation then reads:

$$\frac{R_k}{\gamma_R} \geq \gamma_S * S_k$$

The above is illustrated in Figure 2.1:

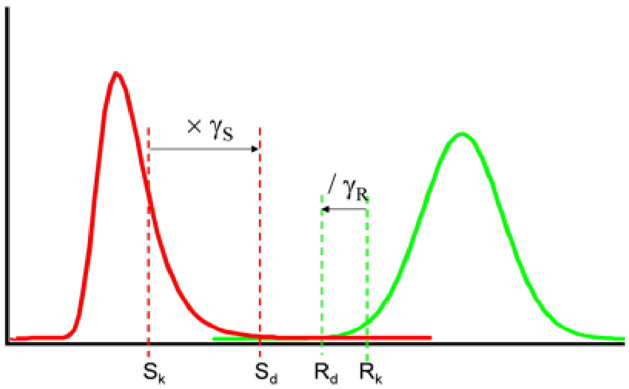


Figure 2.1: Safety philosophy of EN1990

The most important concepts of EN1990 will be elaborated upon in the next paragraphs.

2.1.1.1 LIMIT STATES DESIGN

An element of the safety philosophy in EN1990 is the use of limit states in design practice [Ref. 1]. The limit state of a structure is the point at which the structure can just fulfil its functions, but with an increase of the load (or decrease of the resistance for that matter) failure occurs. A distinction has been made between two limit states:

- the ultimate limit state ULS
- the serviceability limit state SLS

The ULS is the design situation for the safety of (one of the elements of) the structure and its users at the maximum load just before failure.

The SLS refers to a design state in which the structure loses its operational function, for example because of deformations or vibrations.

Within the ultimate limit state, different failure mechanisms are discerned. These failure mechanisms include:

- EQU: failure relating to loss of equilibrium of the structure as a whole
- STR: failure relating to insufficient strength or exorbitant deformations of internal elements
- GEO: failure relating to insufficient strength or exorbitant deformations of soil / rock
- FAT: failure relating to fatigue
- UPL: failure relating to uplift because of for example water pressure
- HYD: failure relating to hydraulic gradients

It should be verified that each element of a structure meets the required safety levels for every relevant limit state. When this is the case, it is assumed that the entire structure is safe.

2.1.1.2 CHARACTERISTIC VALUES AND DESIGN VALUES

In order to comprehend the concepts of characteristic values and design values, it is firstly necessary to explore the classification of actions (i.e. loads). Actions are sorted according to their fluctuation in time, and can be classified as either permanent, variable or accidental. Table 2.1 provides some examples:

Type of action	**Symbol**	**Example**
Permanent	G	Self-weight of a structure
Variable	Q	Wind action
Accidental	A	Explosion

Table 2.1: Classification of actions according to EN1990 [Ref. 1]

Accidental actions are left outside the scope of this thesis.

The characteristic action value is the value that is adopted to represent the assumed load distribution. As can be seen in Figure 2.1, this value does not necessarily equal the mean or most likely value of the distribution. To understand why a certain value is chosen, the uncertainty that comes with the action needs to be considered.

A distinction should be made between permanent and variable actions:

- Characteristic value of permanent actions (G_k):
  The uncertainty accompanied by a permanent action is often low. For example, the self-weight of a structure can be determined with a fair amount of accuracy. Because of this, the mean is usually taken as the characteristic value. For those particular cases in which the variations are larger, an upper and a lower value for G_k may be used.

- Characteristic value of variable actions (Q_k):
  If the mean of the distribution were to be used in design calculations with variable loads, this would almost definitely lead to failure, as the occurring loads can be considerably higher than the mean. That's why, according to Clause 4.2.1 in EN1990, *'the characteristic value for variable actions shall correspond to either an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period'*.

Characteristic action values are thus intended to cover (part of) the physical uncertainty related to the loads. For the characteristic value of resistance models, such as material and product properties, the same principle applies. When a low (respectively high) resistance value is unfavourable, a characteristic value with a probability of non-exceedance of 5% (respectively 95%) is adopted.

The characteristic value is one of the four representative values that can be selected to represent a variable action. Other options are the combination value, the frequent value and the quasi-permanent value. See the list of symbols and definitions for their definitions.

A design value is then arrived at by combining the representative values with partial factors. These partial factors are introduced to cover the remainder of the uncertainty and achieve the required level of safety. The design values shall be determined by multiplication with a partial load factor for actions, or division by a partial resistance factor for materials. The partial factors consist of two separate shares, one to cover the physical and statistical uncertainty, and the other share to cover the model uncertainty. Together they make up the total partial load or resistance factor. This is illustrated in Figure 2.2:

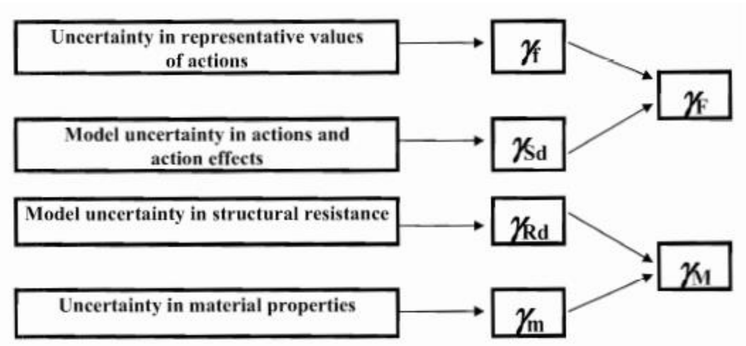


Figure 2.2: Decomposition of partial factors [Ref. 1][2]

[2] As you may notice, the subscripts in Figure 2.2 differ from those in Figure 2.1. The subscripts F and S for actions, and M and R for resistance are interchangeable.

The value of the partial factors may vary depending on a number of aspects. Firstly, EN1990 differentiates between permanent and variable actions. Since variable actions come with more uncertainty, the magnitude of this partial factor is also larger. Typical values for the partial load factors are **γ_G = 1.35** and **γ_Q = 1.5** for permanent actions and variable actions, respectively. Partial resistance factors vary depending on the type of material being used in the construction.

Secondly, EN1990 differentiates between the ultimate limit state and serviceability limit state. The consequences of failure of the ultimate limit state-type are more severe, and therefore ULS requires a larger margin of safety. This is reflected in the magnitude of the partial factors, as the value of partial factors in SLS is set to 1.

Thirdly, the value of a partial factor may differ depending on the required target reliability. This will be explained in Paragraph 2.1.1.3.

2.1.1.3 TARGET RELIABILITY

Before start of design, it needs to be established what reliability is designed for. The Eurocode prescribes the required safety by means of β-values. This β-value is equal to the mean of the limit state function divided by its standard deviation, if the limit state function is schematised by a Gaussian distribution. The larger this value, the larger the margin of safety becomes.

The reliability depends on the consequence class. The purpose of defining a consequence class is to ensure that structures are constructed with the appropriate level of quality control. Three main consequence classes have been defined in the Eurocode:

- CC3: High consequences in case of failure (in terms of loss of human life and social, environmental and economic damage)
- CC2: Medium consequences in case of failure
- CC1: Low consequences in case of failure

CC0 and CC4 are left outside the scope of this thesis. Table 2.2 shows the magnitude of β-values per consequence class and limit state prescribed by EN1990 (in table C-3 of said document):

Consequence class	ULS	SLS
CC3	4.3	1.5
CC2	3.8	
CC1	3.3	

Table 2.2: Target reliability for β for a 50 year reference period

It is visible that the target reliability, and thus the β-values for a 50 year reference period, is higher in ULS compared to SLS. The required level of safety also increases as the consequences of failure do.

These β-values have been translated by EN1990 into a standardised set of partial factors in order to, together with the characteristic value, guarantee the same minimum level of safety. As β-values are higher in ULS than in SLS, the partial factors are so as well. Moreover, β-values also increase as the consequence class increases. That’s why a consequence factor has been introduced that should be applied to the partial load factor. This multiplication factor K_F equals 0.9, 1.0 and 1.1, for CC1, CC2 and CC3, respectively.

2.1.1.4 COMBINATION OF ACTIONS

Another important concept in the Eurocode, is the combination of actions. When more than one action governs a certain failure mechanism / limit state, they should be combined according to the prescribed load combinations. Combination of actions by using the characteristic value of each load, would lead to highly conservative designs, as the maxima of the loads do not (always) act on the structure simultaneously. Which load combination should be used depends on the limit state and the design situation under consideration.

Five different design situations exist:

- Persistent design situation, for conditions under normal use and exposure (including extreme weather conditions)
- Transient design situation, for temporary conditions such as maintenance works
- Accidental design situation, for extraordinary conditions such as a collision
- Seismic design situation, for earthquake events
- Fatigue design situation, for repeated loading

For each design situation, there are one or more load combinations that correspond to it. In this thesis, only the persistent design situation will be considered. The **fundamental load combination** applies to the ultimate limit state of a persistent (or transient, as a matter of fact) design situation, and is used to compute the combined effect of the actions (E_d):

$$E_d = E\left\{\sum_{j\geq 1}^{n} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1}^{n} \gamma_{Q,i} \psi_{0,i} Q_{k,i}\right\}$$

Four different components can be recognised:

- The sum of the characteristic values of the permanent loads multiplied by their corresponding partial factors.
- The pre-stressing load.
- The characteristic value of the dominant variable load multiplied by its corresponding partial factor.
- The sum of the remaining variable loads.

The fourth term requires some additional explanation. It is too conservative to use the characteristic value of all variable loads when combined, which is why a reduction factor is applied to it. The magnitude of the reduction factors is prescribed in EN1990 as well. Multiplication of the reduction factor and characteristic value yields the combination value of a load: $\psi_0 Q_k$. When designing with the combination of actions, each variable load should alternately be assumed to be the dominant variable load, and the most conservative load combination is then adopted for design purposes.

The **characteristic load combination** applies to the serviceability limit state of a persistent design situation:

$$E_d = E\left\{\sum_{j\geq 1}^{n} G_{k,j} + P + Q_{k,1} + \sum_{i>1}^{n} \psi_{0,i} Q_{k,i}\right\}$$

As one may notice, it resembles the fundamental load combination, only lacking partial factors.

2.1.2 PREN1991-1-8 DRAFT EUROCODE ON WAVE AND CURRENT ACTIONS

The new Eurocode has been introduced so as to include coastal structures into the Eurocode framework, as up until now the Eurocodes predominantly dealt with ‘dry’ structures like bridges and buildings. The difference between them is the presence or absence of hydrodynamic loads on design, respectively.

The translation of the concepts described in Subsection 2.1.1 towards coastal structures is not straightforward. In this subsection, any new concepts that have been introduced by the new Eurocode will firstly be explained. Next, the same concepts as in Subsection 2.1.1 will be treated, but now to see how prEN1991-1-8 has adapted them.

2.1.2.1 ADDITIONAL DESIGN CONCEPTS

There are two new design concepts in the new Eurocode on wave and current actions that deserve specific attention:

- **Hydrodynamic Estimate Approach:**

The Hydrodynamic Estimate Approach (HEA) has been introduced because of the variety of wave/current climates and water level conditions across the globe. The HEA-level ranges from 1 to 3, and depends on the selected consequence class and the hydrodynamic uncertainty, see Table 2.3:

Consequence Class	**Hydrodynamic uncertainty**		
	LOW	**MEDIUM**	**HIGH**
CC1	1	1	2
CC2	1	2	2
CC3	2	2	3

Table 2.3: Selection of the HEA-level, taken from Table 4.6 in prEN1991-1-8

The hydrodynamic uncertainty depends on the availability of data and the complexity of hydraulic processes at the project location. In essence, a HEA-level of 1 means that it is less important to have a very accurate estimate of the sea condition parameters at the project site, as the consequences of failure are not too great and the level of understanding of the relevant hydrodynamic processes is high.

- **Design Approach:**

The new Eurocode mentions a total of five design approaches that can be adopted. These are listed below:

- DA-0: The deterministic approach in which the Return Periods together with a global safety factor should provide the required level of safety
- DA-1: The semi-probabilistic approach that uses characteristic values and partial factors
- DA-2: The full probabilistic approach for which a limit state function and complete distributions are required
- DA-3: The approach in which the design is optimised based on risk, social and economic considerations
-DA-4: The approach that uses physical modelling to assist the design; should be used in combination with one of the other design approach

The design approach that should at least be used depends on the HEA-level and the structure response uncertainty, see Table 2.4:

HEA Level	Structure response uncertainty	
	Low-to-Medium	**High**
HEA-1	DA-0	DA-1 or DA-2
HEA-2	DA-1 or DA-0	DA-2 or DA-4
HEA-3	DA-2 or DA-4	DA-2 and DA-4

Table 2.4: Selection of the Design Approach, taken from Table 4.8 in prEN1991-1-8

The structure response uncertainty is governed by the acceptance and understanding of the response formulae. Although it is pointed out that DA-1 is the default approach for coastal structures, it may be the case that DA-2 is required when the HEA-level and/or the structure uncertainty are high, or that DA-0 is sufficient when they are low. This differs from EN1990, which predominantly considers a semi-probabilistic approach.

2.1.2.2 LIMIT STATES DESIGN

EN1990 distinguishes two different limit states, being the ultimate limit state and the serviceability limit state. PrEn1991-1-8 introduces a new limit state, the Limited Damage-serviceability limit state (SLS-(LD)). This new limit state is mainly used for mound-like structures, such as breakwaters and coastal embankments. The reason this new limit state has been introduced is because of the specific damage behaviour of these engineered mound protections. In most Building Codes and the new Flood Risk standards of The Netherlands, ULS refers to start of damage to indicate that the structure has failed. For example, it is undesirable for any column or beam in a building to give way. This is not in line with the design of mound-like structures, as start of damage is not that bad when it comes to for example breakwaters, since it is fairly easy to execute the necessary repairs.

2.1.2.3 CHARACTERISTIC VALUES AND DESIGN VALUES

In the different parts of EN1991, the magnitude of the characteristic value that should be used in design calculations, is described for the different types of actions. Hence, the new Eurocode on wave and current actions should do the same. The difficulty in defining a general characteristic value for water actions arises from two different aspects. First of all, there is the issue that many analytical models work directly with hydraulic conditions instead of with forces. The characteristic value thus does not necessarily consist of a force or pressure, as is mentioned in Clause 4.3.1:

'The characteristic value of actions from waves, water-levels or currents on coastal structures can be the sea condition parameter in some cases, the action effect in others, and in some the action induced by the sea condition or action effect.'

Secondly, there is the issue that sea conditions show enormous variability, both spatial and temporal. What is then the upper value that should be taken to ensure that the probability of failure is sufficiently low?

The new draft Eurocode tries to tackle these problems by providing return periods for the characteristic value, depending on the design service life and the consequence class. The default DA-1 approach (with partial factors) specifies the return periods as listed in Tables 2.5 and 2.6:

Consequence Class	**T_{life}**			
	≤10 years	**25 years**	**50 years**	**100 years**
CC3	40 y	100 y	200 y	400 y
CC2	20 y	50 y	100 y	200 y
CC1	10 y	25 y	50 y	100 y

Table 2.5: Return Period of dominant component to be used in fundamental load combination, taken from Table A.4 in prEN1991-1-8

Consequence Class	**T_{life}**			
	≤10 years	**25 years**	**50 years**	**100 years**
CC3	4 y	10 y	20 y	40 y
CC2	2 y	5 y	10 y	20 y
CC1	1 y	2.5 y	5 y	10 y

Table 2.6 Return Period of dominant component to be used in characteristic load combination, taken from Table A.5 in prEN1991-1-8

The dominant component in the fundamental load combination is equal to the characteristic value in ULS, whereas the dominant component in the characteristic load combination is equal to the characteristic value in SLS-(LD), as can be derived from the equations in Paragraph 2.1.1.4.

In addition, new partial factors need to be defined that belong to water actions. In Clause 4.5.1 of [Ref. 2], the following is stated regarding partial factor values: *'Applicable values of those factors can be found in Annex B (and Appendix A6 of EN1990).'*

The table with the partial factors presented in Annex A6[3] of EN1990 is shown in Appendix B of this document. From this table, it is visible that waves are classified as variable actions, to which a value of $\boldsymbol{\gamma_{Qz} = 1.35}$ belongs. The wave height should be multiplied with this partial factor.

[3] NOTE: this is the updated version of the Annex, as was explained in Section 1.5.

2.1.2.4 TARGET RELIABILITY

The new draft Eurocode describes safety levels as well. These safety levels are expressed as either return periods (to be used in DA-0), probabilities of failure (to be used in DA-1) or β-values (to be used in DA-2). These values can be found in Annex A of the new Eurocode and are shown in Table 2.7:

Consequence Class	Return Period	P_f	β
	Limit state: ULS		
CC3	1000	0.05	4.1
CC2	400	0.12	3.8
CC1	150	0.28	3.5
	Limit state: SLS-(LD)		
CC3	500	0.10	3.8
CC2	100	0.39	3.5
CC1	50	0.64	3.2
	Limit state: SLS		
CC-1/2/3	-	-	1.6

Table 2.7: Safety levels defined in prEN1991-1-8 for a reference period of 50 years, taken from Table 13.1

In order to ensure that the new Eurocode fits into the framework of EN1990, it has been described how to classify coastal structures into the different consequence classes. Examples are given of coastal structures that belong to each consequence class. Moreover, it has been specified how to select a design service life for coastal structures. Appendix A shows the tables that are used for this in prEN1991-1-8.

2.1.2.5 COMBINATION OF ACTIONS

Apart from the characteristic value of sea condition parameters, the other representative values need to be defined for hydraulic actions as well. These other representative values comprise the frequent value, the quasi-permanent value and the combination value. The frequent value and quasi-permanent value are defined in Clauses A.6.3 and A.6.4, respectively, but will not be further treated here. Nevertheless, the combination value is an aspect of interest, namely because of the following two reasons:

- In EN1990, a combination value was arrived at by applying a reduction factor to the characteristic value. For hydraulic actions, this method does not suffice. This is caused by the fact that for certain models, a combination of a wave height with another sea condition parameter such as current velocity or water level, is required. Overtopping is for instance a failure mechanism where both wave height and water level are relevant. The combination value of the water level cannot be obtained by multiplying the wave height with a reduction factor. That's why, again, return periods have been specified in prEN1991-1-8 for the combination value, as listed in Tables 2.8 and 2.9 below.
- When determining the combination value for a hydraulic action, correlation becomes a relevant notion as well. Depending on the circumstances, it may be the case that one parameter is more likely to have a high value if the other parameter has a high value as well. It would then not be sufficient to work with the values in Tables 2.8 and 2.9. Turkstra's rule should be applied, according to Table A.6 in the new Eurocode (see Appendix A).

Consequence Class	T_{life}			
	≤10 years	**25 years**	**50 years**	**100 years**
CC3	4 y	10 y	20 y	40 y
CC2	2 y	5 y	10 y	20 y
CC1	1 y	2.5 y	5 y	10 y

Table 2.8: Return Period of accompanying component to be used in fundamental load combination, taken from Table A.4 in prEN1991-1-8

Consequence Class	T_{life}			
	≤10 years	**25 years**	**50 years**	**100 years**
CC3	2 y	5 y	10 y	20 y
CC2	1 y	2.5 y	5 y	10 y
CC1	1 y	1 y	2.5 y	5 y

Table 2.9: Return Period of accompanying component to be used in characteristic load combination, taken from Table A.5 in prEN1991-1-8

It should also be possible to combine water level, wave and current actions with actions outside the field of hydraulic engineering. Clause A.6.8 in prEN1991-1-8 prescribes the combination factors that should then be used for the hydraulic action effects. Combining them is of course only possible if the analytical model in play translates the hydraulic action to a force or pressure.

2.1.3 DISCUSSION OF PREN1991-1-8 AND COMPARISON TO EN1990

The knowledge gaps that are still present despite, or due to, the introduction of prEN1991-1-8 will be addressed. Anything that stands out when considering the transition of the concepts in EN1990 to the design of coastal structures, is discussed in this subsection.

2.1.3.1 HYDRODYNAMIC ESTIMATE APPROACH AND DESIGN APPROACH

The Hydrodynamic Estimate Approach should perhaps be expanded upon before it can really be a useful addition. Less ambiguity in the definition of hydrodynamic uncertainty would be helpful. A question that one may ask is for instance: if there is sufficient data available but the hydrodynamic processes are complex, is the hydrodynamic uncertainty then low or high?

It is also paradoxical that low quality/quantity of data leads to high hydrodynamic uncertainty and thus a higher HEA-level, for which more accurate estimates of sea condition parameters should be obtained, but in order to achieve this you would need high-quality long-term data.

Furthermore, it remains unclear what the exact implications are of the HEA-level. Is it only used for selecting the Design Approach, or does a lower HEA-level also mean that less extensive analyses may be performed, for instance by using marginal distributions instead of a joint distribution? And what other requirements are demanded on e.g. hydraulic models?

Finally, a point of discussion remains regarding the Design Approaches. On the one hand, differentiation of the minimum required design approach as a function of HEA-level and structure response uncertainty may save costs in case of simple design projects and increase reliability in case of complex ones. However, at the moment it seems that the user has large freedom to choose the exact HEA-level. In fact, the HEA-level can also be entirely neglected, as it is stated that the semi-probabilistic approach will be the default approach.

2.1.3.2 LIMIT STATE SLS-(LD)

PrEN1991-1-8 introduces a new limit state SLS-(LD). It remains unclear whether this new limit state should be placed in between the existing limit states ULS and SLS, or that it replaces the SLS (or even the ULS). Judging by the reliability index (see Paragraph 2.1.2.4), the SLS-(LD) is a completely new one, as different β-values are defined for SLS-(LD) and SLS. On the other hand, it is prescribed that the characteristic, quasi-permanent and frequent combinations are used in SLS-(LD), which were reserved for SLS in EN1990. This pushes it more into the direction of being a replacing limit state.

2.1.3.2 CHARACTERISTIC VALUES

The return periods that are specified in prEN1991-1-8 correspond to offshore conditions. It is stated that they **can** be based on the upper limit of a certain confidence interval, but it is not stated that this **shall** be done. This raises confusion when carrying out the design. Which confidence interval is also not specified.

Furthermore, two matters stand out when examining Tables 2.5 and 2.6:

- There is a difference in the magnitude of the return periods when comparing ULS to SLS-(LD). This difference is non-existent in EN1990, where the only difference between ULS and SLS is the presence of partial factors or not, not the magnitude of the characteristic value.
- The characteristic value differs depending on the consequence class. This as opposed to EN1990, in which the partial factors differ depending on the consequence class, not the characteristic value.

Another crucial point of discussion is the discrepancy between the return period values presented in Table 4.3 and Tables A.4&A.5 in prEN1991-1-8. Both tables indicate that the specified return periods may be used in DA-1, but the values are different. One interpretation is that Table 4.3 is only intended for single loads, whereas Tables A.4&A.5 are intended for the combination of actions. However, according to the fundamental load combination described in EN1990, this difference should be non-existent as the dominant variable in this load combination equals the characteristic value of a single load. Another interpretation is that the values presented in Table 4.3 are only intended for use in DA-0, whereas Tables A.4&A.5 describe DA-1. A third interpretation is that the discrepancy is simply a typo in the draft. Anyway, this point of discussion should be resolved in a definitive version.

2.1.3.3 PARTIAL FACTORS

There are some issues that arise because the partial factor is applied to a wave height instead of a load:

1) Most types of loads described in EN1990 have an unlimited range of possible values that the load can assume. Waves, however, are limited by physical processes.

Waves break when the steepness becomes too large or when the depth becomes too small. Especially the latter phenomenon, depth-limited breaking, gives rise to problems with respect to the application of partial factors. The following quote from Clause 5.1 from the new Eurocode shows that thought has been given to this particular process:

'Evaluation of hydrodynamic actions should be made with due consideration of dependence (joint probability) between design parameters, e.g. wave height and water level at a site where the water is relatively shallow and breaker heights are controlled by the depth of water.'

The draft Eurocode does not mention, however, how this process should be treated in combination with a partial factor. In the updated Annex A.6 (see Appendix B), it is specified that the partial factor is given for offshore waves. Nevertheless, if the offshore wave is translated nearshore to shallow water, all of the uncertainty built into the partial factor will have disappeared. Of course, the wave height uncertainty is also greatly reduced for shallower water, but the partial factor also covers other uncertainties. The alternative is to apply the partial factor on the nearshore wave, but then you obtain an unrealistic wave height, and the question remains whether that would be allowed.

2) Another issue that originates from the fact that a wave height is factored and not a load, is related to the relation with wave properties.

These properties, in particular the wave period and wavelength, increase or decrease as the wave height does. In prEN1991-1-8, it is not settled whether these wave properties should be adjusted to correspond to the factored wave height or not.

In addition, two remarks can be made about the magnitude of the partial factor:

3) The magnitude of the partial factor is smaller than the one used for variable loads in general.
4) Most distribution types for wave and current actions show asymptotic behaviour towards higher return periods, as they are to some extent limited in their magnitude because of physical limits. A partial load factor is prescribed in prEN1991-1-8 regardless of the magnitude of the load, but it is debatable whether it is fair to apply the same factor to a wave with a 10-year return period as to a wave with a 200-year return period.

Other matters that are noteworthy:

5) The new Eurocode only discusses a partial load factor to be applied on the wave height. There is no mention of a partial resistance factor[4]. This does make sense, as a partial resistance factor in the Eurocodes is related to the strength of a certain material, while failure in coastal structures is often not related to material strength but to hydraulic stability. Nevertheless, the lack of a partial resistance factor is a point of discussion, as it is an integral part of the semi-probabilistic approach. Without it, the partial load factor serves more as a global safety factor.
6) The new draft Eurocode does not elaborate on how the partial load factor can be combined with the water level. Table A.6.8 in the updated Annex A.6 even states that water levels are under no circumstances factored. This seems strange, as the water level is the underlying sea condition parameter for all hydraulic loads.

[4] NOTE: characteristic resistance values are also not described properly

2.1.3.4 REQUIRED LEVELS OF SAFETY

Several remarks can be made when looking at Table 2.7 and comparing it to Tables 2.2, 2.5 and 2.6:

- The return periods mentioned in Table 2.7 don't match up with the return periods specified in Table 2.5 and Table 2.6. This relates to the same issue as described in Paragraph 2.1.3.2, discussing that the values in Table 2.7 may be intended for use in DA-0 whereas the values in Tables 2.5 and 2.6 are reserved for DA-1.
- The specified β-values slightly deviate from what is prescribed in EN1990 (see Table 2.2). Although the new Eurocode claims they are compatible, which may be the case because of different statistical definitions, it is confusing for the user. A decent explanation of how various safety levels are equivalent, for example due to a probability being expressed per storm or per lifetime, lacks anyway in the new Eurocode.
- SLS-(LD) has been assigned higher β-values than SLS. This makes sense, as you do not want damage, though to a limited extent, to occur often. Each time it happens repairs must be performed, whereas in SLS there is only some discomfort which is not as detrimental. This seems to indicate that SLS-(LD) is indeed a new limit state of its own, but some questions remain unanswered. Why should one use the characteristic combination for SLS-(LD), while it was originally drawn up for SLS? Why are the prescribed return periods for this lower than one would expect given EN1990, while the reliability indices are stricter? And if one were to design something in DA-1 for SLS, which return periods would then have to be used?

Another issue that is left open to discussion is the description of consequence classes for coastal structures. Though this is a useful addition, it is not immediately evident from the descriptions how to treat breakwaters. They can be put into either CC3 or CC2, depending on whether they are protecting nationally significant ports or not. The use of this term, 'nationally significant', makes that there is still room for interpretation of the designer. The same can be said for the descriptions of the design service life of coastal structures.

2.1.3.5 COMBINATION ACTIONS

The designer should be careful in applying the partial factors when combining actions, for the following two reasons:

- PrEN1991-1-8 lacks to describe whether a partial factor should be applied to a force or to a wave height, in an analytical model where both are possible. The draft Eurocode does distinguish between actions and action effects, but the application of a partial factor to either of them is not described. The descriptions of actions action effects are not unambiguous anyway when placed into the framework of EN1990. For several coastal structure response models, there won't be a force to consider as an action, which means that the wave height must definitely be factored.
- Caution is recommended if a force/pressure is computed as a result of a hydraulic action, and a partial factor is then applied to this force. The characteristic value of the hydraulic action varies depending on the consequence class but the magnitude of a partial factor for the action effect does so as well. If you are designing a structure in CC3, both of these safety mechanisms come into play (without depth-limitations) which might result in an overestimation of the design parameters.

2.2 CURRENT DESIGN PRACTICE

This section starts by explaining the different uncertainties that a designer needs to deal with. Afterwards, the different design methods that are currently available for the design of breakwaters are elaborated upon. Moreover, equivalence between the existing methods and the new Eurocode is explained.

2.2.1 UNCERTAINTIES

It is useful to gain insight into the nature of uncertainties, as it helps understand how different design methods try to cope with them.

When speaking about uncertainties, they can roughly be categorised into three main types [Ref. 10]:

- Physical or intrinsic uncertainties
- Statistical uncertainties
- Model uncertainties[5]

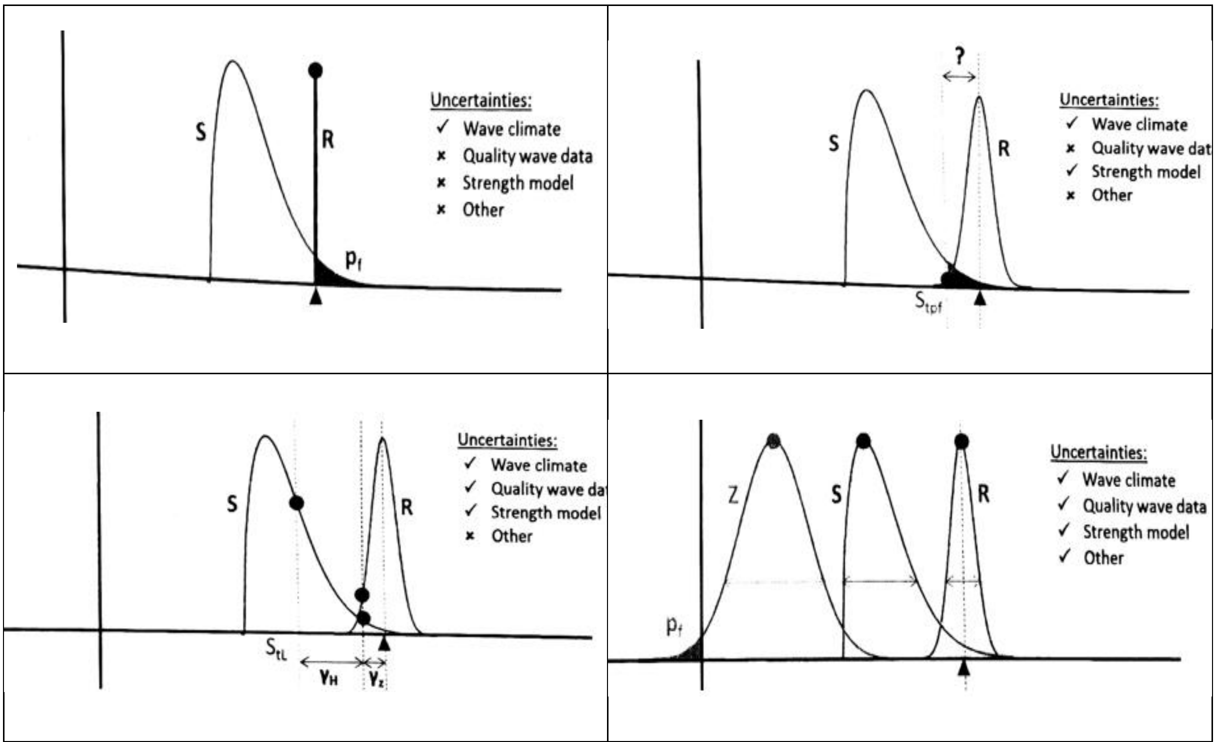
Physical uncertainties relate to the random character of load and resistance. For example, it is uncertain what the maximum load/wave height will be that the structure needs to endure during its lifetime, or what the exact properties are of the materials that need to provide the resistance. A certain distribution (e.g. Gaussian, Lognormal) can be assigned to a load or resistance parameter to describe this variability.

Statistical uncertainties arise due to the limited amount of data that is available to estimate the parameters of said distributions. For example, inaccuracy is involved when estimating the load or wave height that belongs to a given return period.

Model uncertainties come into play because analytical or numerical models are often used to simplify reality, but this can never be done without making some assumptions that cause the computed response to deviate from the actual response. For example, it is uncertain whether the chosen distribution is the correct one, and the results obtained from a design equation come with a certain degree of inaccuracy.

[5] The term ‘schematisation uncertainties’ probably fits the description that is given here better. Model uncertainties are better described by the scatter in the empirical response model. No further attention has been paid to this distinction in the remainder of this thesis.

The ways a designer can deal with these uncertainties are beautifully visualised by the Breakwater Design Lecture Notes [Ref. 7], see Table 2.10:



Top left: Level 0, deterministic or conventional approach; An upper value for the load, corresponding to a certain failure probability, is applied in design calculations. This should cover the physical uncertainty.

Top right: Conventional approach with additional safety; An upper value for the load is adopted, and in addition safe values are taken for empirical coefficients in the design formula. Besides the physical uncertainty, this should also cover the model uncertainty.

Bottom left: Level I approach; Characteristic values for the load and resistance are assumed, which are then multiplied with and divided by a partial factor. Besides the physical and model uncertainty, this should also cover statistical uncertainty.

Bottom right: Level II or III approach; The load distribution is subtracted from the resistance distribution to arrive at a distribution for the limit state function Z. Any other remaining uncertainties have been taken into account here as well.

Table 2.10: Different ways of dealing with uncertainty as described by Breakwater Design Lecture Notes [Ref. 7]

2.2.1 CONVENTIONAL DETERMINISTIC METHOD

The conventional method is a (quasi-)deterministic approach. The target probability of failure is converted into a return period, for instance the wave height that occurs once every 100 years. This means that all of the uncertainty should be covered by the magnitude of the return period, so only the uncertainty on the load side is considered. An addition to this method is the application of a global safety factor, to cover any remaining uncertainties. This global safety factor can either come in the form of an actual factor, or by using conservative values for coefficients of the formula in play.

The conventional method is a method that is frequently used in current breakwater design practice. Helpful documents in this respect are the Rock Manual (stability) and the EurOtop Manual (overtopping).

From this point on, the conventional deterministic method that is used in current breakwater design practice, will be referred to as DA-0. DA-0 is the deterministic approach of the new Eurocode and includes additional uncertainty by using conservative coefficient values, as is stated in Clause 4.5.1 of prEN1991-1-8:

'DA-0: The applicable margin is related to the response formula used. Semi-empirical formulae commonly incorporate a safety margin (but this is often not explicitly stated). If no safety margin is stated for the formula employed (or a deterministic version of the formula not available) then a safety margin of one Standard Deviation (SD) above the CSE should be applied to the sea condition parameter.'

The draft Eurocode prescribes that DA-0 can be used when the combination of HEA-level and structure uncertainty is low, and is very similar to the current breakwater design practice. Whereas the designer used to evaluate the desired probability of failure himself for the conventional method, the new Eurocode now aims to provide guidelines for this.

2.2.2 PIANC METHOD

A partial safety factor system specific to breakwaters, has already been developed by PIANC [Ref. 12]. This approach is semi-probabilistic, just as the basis of structural design as described in EN1990.

The design should then abide by the following function [Ref. 7]:

$$\frac{(\Delta D) * f_d}{\gamma_R} - \gamma_H * H_s \leq 0$$

The resistance is described as a critical wave height, so as to be able to include a partial resistance factor. The characteristic value for the load is the wave height which has a return period that equals the lifetime of the structure. The partial load factor that should be applied to the wave height is shown below:

$$\gamma_H = \frac{H_{ss}^{tpf}}{H_{ss}^{tL}} + \sigma'^{\left(1+\left(\frac{H_{ss}^{3tL}}{H_{ss}^{tL}}-1\right)k_\beta p_f\right)}_{F_{Hs}} + \frac{0.05}{\sqrt{p_f N}}$$

It is not so important to know the exact meaning of all parameters in the expression, reference is made to the PIANC report [Ref. 12] in case one is interested. It is more relevant to comprehend what each term in the expression deals with. The first part of this term ensures that the characteristic wave height is brought back to the wave height that has the proper target reliability, thus ensuring that the physical uncertainties have been taken into account. This is just a matter of definition. The second term focuses on the quality of the data (physical uncertainty), whereas the third term focuses on the quantity of the data (statistical uncertainty).

The characteristic value for the resistance is obtained by filling in mean values for coefficients in the relevant design formula. The partial resistance factor that should then be applied is as follows:

$$\gamma_z = 1 - (k_\alpha p_f)$$

In this expression, the coefficient k_α is related to the design formula being used (e.g. Hudson, Van der Meer) and therefore deals with model uncertainty.

The PIANC method can be compared to the newly proposed DA-1 in prEN1991-1-8, of which the description can be found in Clause 4.5.1:

'DA-1: This Design Approach refers to the principal design format in the Eurocodes associated to the use of partial factors that cover the required safety margin for a range of design cases.'

As was already established, DA-1 does not specify partial resistance factors for coastal structures, so the PIANC method is an improvement to this. Moreover, it does a better job at targeting the different uncertainties for specific wave climates. One might argue that it requires some effort to compute the partial load factor, whereas in DA-1 these values are standardised. Nevertheless, an extreme value analysis must be performed for DA-1 anyway, so if the wave climate has been analysed this far you could easily go all the way and compute the magnitude of the partial factors as well. Last but not least, the partial factors take into account the probability of failure as well. In Paragraph 2.1.3.5, it was discussed that partial factors should perhaps vary for different return periods, which is the case in the PIANC method.

Although the application of the method still comes with a lot of uncertainty, it can be concluded that this method definitely has its advantages when compared to DA-1 in the new Eurocode.

2.2.3 FULL PROBABILISTIC METHOD

In order to carry out a full probabilistic method, the limit state function must be considered, Z = R – S. In the full probabilistic method, all the distributions of the relevant parameters should be combined so as to arrive at a distribution for Z. The probability of failure then equals that part of the distribution for which Z < 0. As the number of relevant parameters increases, it becomes more and more complex to derive an analytical expression for the distribution of Z. Methods that circumvent the need for such an expression are the FORM analysis or the Monte Carlo analysis.

From this point on, the full probabilistic method that is used in current breakwater design practice, will be referred to as DA-2. DA-2 is the full probabilistic approach of the new draft Eurocode and incorporates uncertainty through the target reliability levels, as is stated in Clause 4.5.1 of prEN1991-1-8:

'DA-2 (and DA-3): This Design Approach does not require the use of extra safety margin since the latter has been incorporated in the target reliability levels given in Table 13.1.'

The draft Eurocode prescribes that DA-2 should be used when the combination of HEA-level and structure uncertainty is high. The new draft Eurocode aims to unify the target reliability levels.

Interestingly, the ROM [Ref. 5], the Spanish code for the design of maritime structures, already includes an elaborate description of the full probabilistic method, whereas the main focus of EN1990 is on the semi-probabilistic approach. In prEN1991-1-8, a full probabilistic approach has been given more attention, but the default approach is still the partial safety factor method. For breakwaters, it is often easy to set up a full probabilistic calculation, and because of this it might be superior to a method with partial factors.

2.3 BREAKWATER RESPONSE FORMULAE

This Section presents the formulae that will be used for designing the main breakwater elements. The main breakwater elements that will be designed in this thesis are a rock-armour layer, a concrete-armour layer, the crest height and the crown wall. Clause 7 and Annex E specifically deal with the design of mound breakwaters, and should therefore specify how to design for all the possible failure mechanisms. Average values and standard deviations of empirical model coefficients will be discussed in those parts where the design of the element itself is treated.

2.3.1 STABILITY OF ROCK-ARMOURED SLOPE

In Clause E.3.4, the new Eurocode mentions three formulae related to rock stability in the armour layer, being the Van Gent formula, the Van der Meer formula and the Hudson formula. The Van der Meer formula will be used. The Van der Meer formula (as described in the Rock Manual [Ref. 6]) consists of three parts:

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

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

For surging waves ($\xi_m \geq \xi_{cr}$)

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^P$$

Critical surf similarity parameter

$$\xi_{cr} = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

Parameters

$H_s = significant\ wave\ height\ at\ the\ toe\ of\ the\ structure\ [m]$

$\Delta = relative\ buoyant\ density\ [-];\ \frac{\rho_s}{\rho_w} - 1$

$D_{n50} = nominal\ rock\ diameter\ [m]$

$P = notional\ permeability\ of\ the\ breakwater\ [-]$

$N = number\ of\ waves\ [-]$

$S = damage\ level\ parameter\ [-]$

$c_{pl} = Van\ der\ Meer - coefficient\ for\ plunging\ waves\ [-]$

$c_s = Van\ der\ Meer - coefficient\ for\ surging\ waves\ [-]$

$\xi_m = surf - similarity\ parameter\ using\ mean\ wave\ period\ [-]$

$\alpha = slope\ angle\ [^o]$

2.3.2 STABILITY OF SLOPE OF ARTIFICIAL UNITS

Clause E.3.5 in the new Eurocode states the following with regards to the design of artificial units:

‘When seaward slopes of a mound breakwater are armoured by prefabricated concrete armour units, the required mass of the latter may be estimated for several unit shapes through the stability number Hs/ΔDn or the stability coefficient KD in the Hudson formula. Values of those numbers are suggested along with other design information for concrete armour layers in The Rock Manual, § 5.2.2.3.’

This suggests that you should either work with the Hudson formula or with a fixed value for the stability number. Since these fixed values are well established for single-layer concrete armour units, this is what will be worked with (as described in the Rock Manual):

$$\frac{H_s}{\Delta D_{n50}} = N_{s,d}$$

Parameters

$H_s = significant\ wave\ height\ at\ the\ toe\ of\ the\ structure\ [m]$
$\Delta = relative\ buoyant\ density\ [-];\ \frac{\rho_c}{\rho_w} - 1$
$D_{n50} = nominal\ diameter\ of\ concrete\ unit\ [m]$
$N_{s,d} = fixed\ stability\ number\ [-]; acceptable\ damage\ level$

2.3.3 CREST HEIGHT BASED ON WAVE OVERTOPPING

Clause E.3.7 states the following:

'Wave overtopping is nowadays the prime parameter to decide on the crest elevation of a conventional mound breakwater with or without a crest wall. Empirical formulae to calculate overtopping discharge based on physical model and field experiments may be used for conventional mound breakwaters, see EurOtop Manual §6.3.1.'

The crest height should be designed in such a way that a certain tolerable overtopping discharge is not exceeded. In EurOtop Manual §6.3.1 [Ref. 11] the following formula is presented:

$$\frac{q}{\sqrt{gH_{m0}^3}} = c_1\ exp\left[-\left(c_2 \frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)^{1.3}\right]$$

Parameters

$H_{m0} = significant\ wave\ height\ at\ the\ toe\ of\ the\ structure\ [m]$
$g\ = gravtational\ acceleration\ [m/s^2]$
$q = mean\ wave\ overtopping\ discharge\ [m^2/s]$
$R_c = freeboard\ [m]$
$c_1 = empirical\ overtopping\ coefficient\ [-]$
$c_2 = empirical\ overtopping\ coefficient\ [-]$
$\gamma_f = influence\ factor\ roughness\ [-]$
$\gamma_\beta = influence\ factor\ oblique\ wave\ attack\ [-]$

2.4.4 WAVE FORCING ON CROWN WALL

In order to determine the required strength of the crown wall, the forces acting on the crown wall first need to be computed. Various methods are available, in Clause E.3.10 the following can be found with respect to this:

'Wave action on crown walls, including uplift, may be calculated through Martin's method (1999) for waves that do not break directly on the crown wall. For other cases Pedersen's method (1996) may be employed. For further guidance on the structural response of crown walls refer to The Rock Manual §5.2.2.12.'

However, the situation at the IJmuiden breakwaters does not fall into the application range of either of these methods, and will also not yield stable solutions. The new Eurocode hence does not provide sufficient information on the design of crown walls, as it is unclear what should now be done. Nevertheless, other design methods exist for which the application range is adequate for the considered location. The crown wall design will be performed under the assumption that it is allowed to select an alternative design method for the calculation of wave actions on crown walls. The method estimates wave forces based on overtopping rates [Ref. 22]:

$$\frac{Fh_{0.1\%}}{(0.5\rho_w g C_h^2)} = \left((0.27 * \ln(\xi_{0p}) + 0.1)(\log Q + 6) + 0.23\right)\left(0.5 * \frac{R_c - A_c}{C_h} + 1\right) - 0.15$$

$$\frac{PbF_{h0.1\%}}{(0.5\rho_w g C_h)} = 0.02 * \left(\frac{F_c}{L_{0p}}\right)^{-1/2}$$

$$\frac{Mh_{(Fh0.1\%)}}{(\rho_w g C_h^3)} = 1.08 + 0.18 * \log Q$$

In these equations, $F_{h0.1\%}$refers to magnitude of the horizontal force, generated by waves, with an exceedance probability of 0.1%, $PbF_{h0.1\%}$ refers to the uplift pressure at the seaward side generated by the same wave that caused the horizontal force and $Mh_{(Fh0.1\%)}$ refers to the destabilising moment caused by this wave. The wave forces as calculated by the equations above can then be used to design the crown wall against sliding and overturning. Apart from the parameters presented below, the width of the crown wall and the friction factor are needed to do so.

Parameters

$\rho_w = density\ of\ sea\ water\ [kg/m^3]$
$g\ = gravtational\ acceleration\ [m/s^2]$
$C_h = height\ of\ the\ crown\ wall\ [m]$
$\xi_{0p} = surf - similarity\ parameter\ in\ deep\ water\ using\ peak\ period\ [-]$
$Q = dimensionless\ overtopping\ discharge\ [-];\ \frac{q}{\sqrt{gH_{m0}^3}}$
$R_c = freeboard\ [m]$
$A_c = distance\ between\ armour\ crest\ and\ water\ level\ [m]$
$F_c = distance\ between\ base\ level\ and\ water\ level\ [-]$
$L_{0p} = deep - water\ wavelength\ based\ on\ peak\ period\ [-]$

A more elaborate discussion on the choice of these breakwater response formulae, based on the application ranges of the formulae under consideration, can be found Appendix C.

2.4 MAIN FINDINGS FROM CHAPTER 2

The main findings from studying the Eurocodes and other literature are listed below:

- Physical, statistical and model uncertainties need to be tackled during design. The main way to do this in EN1990 is a semi-probabilistic approach. Characteristic values should be determined for all relevant load and resistance variables, and multiplied with or divided by a partial factor, respectively. PrEn1991-1-8 tries to extend this method towards hydraulic engineering.
- The introduction of the partial factors for wave and current actions on coastal structures comes with some flaws:
    - It is not clear how to take the physical limitations of the actions into account.
    - It is not clear whether other wave properties should change along with the wave height, so as to maintain a physically realistic wave.
    - It is not clear whether a partial factor should be applied to the wave height or to the force, in case of an analytical model that computes the force as a result of the wave height.
- A poor translation of several concepts from EN1990 to prEN1991-1-8 has been noticed, which requires the user to proceed the design with great care. Several examples:
    - A new limit state has been introduced (SLS-(LD)), but it is confusing whether it serves as a replacement of SLS/ULS or as an additional limit state.
    - The characteristic return periods of the variable actions to be used in the fundamental load combination and the characteristic load combination do not match up in the draft Eurocode, whereas this should be the case according to the basis of structural design described in EN1990.
    - The difference in target reliability level for different consequence classes is all of a sudden reflected in the characteristic value instead of in the magnitude of the partial factor.
- It remains to be seen whether designing coastal structures according to DA-1 provides the required level of safety, because of the following reasons:
    - For hydraulic failure mechanisms, no partial resistance factor is available to cover for the physical and model uncertainties related to the resistance.
    - The new draft Eurocode does not elaborate on how the partial load factor can be combined with the water level. Table A.6.8 in the updated Annex A.6 even states that water levels should not be factored, which is doubtful as water levels are also surrounded by physical uncertainties, and it may have a great influence on other hydraulic actions.
    - Especially for SLS-(LD), there does not seem to be a safety mechanism that will provide for the desired target reliability.
- Although the addition of the concept of Hydrodynamic Estimate Approach seems valuable, the ambiguous description of it raises more questions than it answers. More clarity for the user is desired.
- Additional uncertainties arise because of the fact that the hydraulic conditions will differ for each project location. An advantage of a semi-probabilistic approach should be that it is faster than a full probabilistic calculation. However, if a comprehensive study of the environmental sea parameters is required for each specific case in DA-1, you could just as well go all the way and set up a design calculation according to DA-2. The new document then seems to miss its target by stating that DA-1 is still the default approach.
- The new Eurocode refers a lot to formulae in other documents without stating these formulae themselves. Although it would perhaps not make sense to repeat all these formulae, it does leave room for human error when picking the proper method and finding the correct formulae. This implies that designs should still be made by experienced engineers.

3. BOUNDARY CONDITIONS OF BREAKWATER DESIGN

In this chapter, the Basis of Design for the IJmuiden breakwaters will be established, based on the fourth chapter of prEN1991-1-8. Figure 3.1 below shows the steps that need to be undertaken:

Structure specification	•Determine consequence class (Table 4.1 in prEN1991-1-8) •Determine design service life (Table 4.2 in prEN1991-1-8)
Design specification	•Select the relevant limit states (Table 4.4 in prEN1991-1-8) •Select the relevant design situations (Table 4.5 in prEN1991-1-8) •Specify Hydrodynamic Estimate Approach (Table 4.6 in prEN1991-1-8) •Specify Design Approach (Table 4.8 in prEN1991-1-8)
Definition of Basic Variables	•Define actions (See Clause 4.3.1 in prEN1991-1-8) •Define material and product properties (See Clause 4.3.2 in prEN1991-1-8) •Define geometrical parameters (See Clause 4.3.3 in prEN1991-1-8)
Collection of data	•Collect water level data •Collect wave action data (See Clause 5 and Clause C in prEN1991-1-8) •Collect bathymetric data
Correlation analysis	•Determine the level of correlation between water level and wave height (See Clause 5 and Clause 13 in prEN1991-1-8)
Hydraulic Boundary Conditions	•Select the required Return Periods (See Table 4.3 or Tables A.4 & A.5 in prEN1991-1-8) •Perform Extreme Value Analysis (See Clause 5 and Clause C in prEN1991-1-8) •Compute the design variables (design water level, design wave height and design wave period) (See Clause 5 and Clause C in prEN1991-1-8)
Offshore-nearshore transformation	•Transform the design variables towards the site of interest (See Clause 4 and Clause 5 in prEN1991-1-8)

Figure 3.1: Steps for Basis of Design

The order of the required steps, presented in the flow-chart in Figure 3.1, needs to be decided by the user of prEN1991-1-8 him- or herself. This becomes evident when looking at the numbers of the Tables and Clauses, which are not ascending along with the design steps. The fact that the necessary information is scattered throughout the document, makes it more difficult for the designer to properly understand and apply the guidelines provided by the draft Eurocode.

The prEN1991-1-8 tables that are mentioned in Figure 3.1, can be consulted in Appendix A.

The structure of this chapter will follow the design steps in Figure 3.1. This means that Section 3.1 treats the structure specification, Section 3.2 deals with the design specification, etcetera. Moreover, any uncertainties that are not clarified by the introduction of the new Eurocode are elaborated upon.

3.1 STRUCTURE SPECIFICATION

The structure specification comprises the determination of the consequence class and the design service life.

3.1.1 CONSEQUENCE CLASS

Coastal structures shall be classified into CC3, CC2 or CC1 according to Table 4.1 in prEN1991-1-8. This table, however, is sensitive to the interpretation of the user. For the breakwaters of IJmuiden, the consequences in case of failure with respect to human life can be classified as ‘medium’, because of the public accessibility (though access can be prohibited in case of storm events). Failure could lead to considerable economic damage, if due to failure the port of Amsterdam cannot be operational temporarily. From this perspective, the selected consequence class is CC2.

The description in the new Eurocode also mentions breakwaters protecting a nationally significant port as an example of a structure belonging to CC3. As the IJmuiden breakwaters are protecting the entrance towards the port, and not the port itself, it has been deemed appropriate to classify the structure as **CC2** and not CC3.

The fact that the breakwaters fulfil a function with respect to the IJmuiden sea sluices, lowering the wave conditions at this location, has not been taken into account. If you would do this, the breakwaters might even be regarded as belonging to CC4 (which is outside the scope of this thesis), since the sluices are a primary sea defence according to the Water Act [Ref. 28].

3.1.2 DESIGN SERVICE LIFE

The design service life shall be determined based on Table 4.2 in prEN1998-1-8. T_{life} ranges from ≤10 to 100 years. Breakwaters are mentioned both in the category of structures with a T_{life} of 50 years as well as in the category of structures with a T_{life} of 100 years, with the difference being the strategic or economic value of the port they are protecting. The question thus comes down to the following: *‘When does a port classify as a port of nationally significant strategic or economic value?’.* The port of Amsterdam is the second largest port of the Netherlands, but relative to the port of Rotterdam it is fairly small. Taking this into consideration, a design lifetime of **50 years** has been selected for the IJmuiden breakwaters.

3.2 DESIGN SPECIFICATION

The design specification comprises the selection of limit states, design situations and the specification of the Hydrodynamic Estimate Approach and Design Approach.

3.2.1 LIMIT STATES

The limit states that need to be considered can be selected from Table 4.4 in prEN1991-1-8. Here it can be found that the two-layer rock armour should be checked at SLS-(LD), the single-layer concrete armour should be checked at ULS and SLS-(LD)[6], and the crown wall should primarily be checked at ULS. Moreover, in Clause 4.5.1 the following is stated:

'Structures designed primarily at SLS-(LD), e.g. rubble mound breakwaters, should be checked also at ULS when they belong to the CC2 class (or higher) and/ or where they involve a single armour layer only (or in some other way have limited resilience following a design event to a subsequent event without repairs having been carried out).'

'Structures designed primarily at ULS, e.g. vertical face breakwaters, should be checked also at the SLS-(LD) when they belong to the CC3 class.'

The first statement holds, which means that the rock armour will also be checked at ULS. The second statement does not hold, which means that the crown wall does not have to be checked at SLS-(LD).

For which limit states the crest height should be designed is not specified in Table 4.4 in the draft Eurocode. However, overtopping is described in Clause 7.4.3, in which it is stated that:

'The effect of overtopping water and spray should be considered in relation to the function of the breakwater and the activities on and behind the breakwater.'

Considering this, it can be linked to SLS. In addition, Clause 7.4.4 in prEN1991-1-8 states:

'The stability of the rear armour layer shall be considered. Where significant overtopping occurs, the rear slope should be designed by taking into account the loading due to the overtopping water.'

If this is considered, it can also be linked to damage, and then the SLS-(LD) seems most appropriate.

Table 3.1 provides an overview:

Breakwater element	Limit states
Armour layer – rock	SLS-(LD), ULS
Armour layer – concrete units	SLS-(LD), ULS
Crest height	SLS, SLS-(LD)
Crown wall	ULS

Table 3.1: Limit states to be checked in the design of the various breakwater elements

[6] It is not clear what the physical meaning is of limited damage to a single-layer armour of concrete units. It is a possibility that Table 4.4 in the draft Eurocode has been interpreted incorrectly, for which the conclusion could then be that the table is not straightforward enough.

3.2.2 DESIGN SITUATION

The relevant design situations can be selected from Table 4.5 in prEN1991-1-8. In this case, only the **persistent design situation** will be considered. The transient, accidental and fatigue design situations are left outside the scope of this research (though their descriptions can be found in the list of definitions), whereas the seismic design situation is irrelevant for the location. In Table A.3 in the new Eurocode, it can be identified that in the persistent design situation for ULS, the **fundamental load combination** must be used. For SLS-(LD) either the characteristic, the frequent or the quasi-permanent combination must be used. It is strange that this choice is left up to the designer. Anyway, it has been chosen to continue working with the **characteristic load combination**.

3.2.3 HYDRODYNAMIC ESTIMATE APPROACH AND DESIGN APPROACH

Next, the Hydrodynamic Estimate Approach and Design Approach should be specified. However, it has been chosen to work with DA-1 regardless of what the new Eurocode prescribes to be minimally required, as DA-1 is the default approach which becomes obvious from the following quote in Clause 4.51 of the draft Eurocode:

'The semi-probabilistic design approach associated with the partial factors format as proposed in EN1990 shall be the default approach for coastal structures (DA-1)'

This is also in line with the basis of structural design as described in EN1990. If you do want to work out the HEA-level and Design Approach for the case study of IJmuiden, you must consult Tables 4.6 and 4.8 in prEN1991-1-8.

In Table 4.6 in prEN1991-1-8, examples are given of sea conditions belonging to either low or high hydrodynamic uncertainty. For this particular test case, one might consider the hydrodynamic uncertainty to be low, as high-quality time series data of waves, water levels and currents are available. On the other hand, there are surges of over 1 m and currents of over 1 m/s at the project site, which are mentioned in Table 4.6 as examples of high hydrodynamic uncertainty. It would probably be best to go with medium hydrodynamic uncertainty, which together with CC2 leads to HEA-2.

Table 4.8 in prEN1991-1-8 then displays how the HEA-level should be combined with the structure response uncertainty, to arrive at a Design Approach. Although a lot can still be learnt about the behaviour of breakwaters, quite some research has already been done looking into this, which is why a 'Low-to-medium structure response uncertainty' is selected. Together with HEA-2, this leads to either DA-1 or DA-0.

It is peculiar that the Hydrodynamic Estimate Approach is introduced as a new concept in prEN1991-1-8, but that the designer can still take the semi-probabilistic design approach by default, regardless of the HEA-level.

3.3 DEFINITION OF BASIC VARIABLES

The basic variables can be categorised into three different parameter types:

- The actions (See Subsection 3.3.1)
- The material and product properties (See Subsection 3.3.2)
- The geometrical parameters (See Subsection 3.3.3)

A good starting point for identifying all basic variables is to look at the parameters that are required for the relevant design formulae[7].

3.3.1 ACTIONS

For the stability of a rock-armoured slope, information is required about the wave height and the wave period (which occurs in the surf similarity parameter). The deep-water wavelength is also needed, but this can directly be determined from the wave period. For the stability of a slope of artificial units, information is required about the wave height. For wave overtopping, information is required about the wave height, the wave direction and the water level, as it governs the magnitude of the freeboard. For wave forcing on a crown wall, information is required about the wave height, the wave period and the water level. In general, it can be said that information is required about:

- Water levels: tide and surge
- Wave characteristics: wave height and wave period
- Bathymetry

Although bathymetry is not strictly speaking an action, it does determine the magnitude of the action since the wave height near the breakwater may be limited by depth. Moreover, self-weight of the crown wall could be considered an action as it provides resistance, but it is not included here as it depends on the material properties and geometry.

3.3.2 MATERIAL AND PRODUCT PROPERTIES

The relative buoyant density Δ is relevant for the stability of the armour slope. In order to compute this variable, information on sea water density, rock density (in case of rock-armoured slope) and concrete density (in case of concrete-armoured slope) is required. The concrete density is also required to calculate the self-weight of the crown wall. Reasonable values have been assumed for these variables based on expertise.

The breakwater will have a permeable core with a filter layer between the armour and the core, so a notional permeability of 0.4 should be adopted in the Van der Meer-formula [Ref. 7]. The type of single-layer concrete armour units that have been selected are Accropodes. The roughness factor for Accropodes equals 0.46 [Ref. 11]. For the friction between the crown wall and the breakwater core, a logical value is approximately 0.6, according to The Rock Manual.

[7] In fact, the selection of the design formulae is also an important step in the design process, but this has been treated separately in Section 2.3, as this was also not explicitly stated as design step in the draft Eurocode.

The material and product properties have been summarised in Table 3.2:

Description	Symbol	Value	Unit
Density of sea water	ρ_w	1025	kg/m^3
Density of rock	ρ_s	2650	kg/m^3
Density of concrete	ρ_c	2400	kg/m^3
Notional permeability	P	0.4	-
Roughness factor	γ_f	0.46	-
Friction factor	f	0.6	-

Table 3.2: Overview of relevant material and product properties

The nominal diameter $D_{n(50)}$ is one of the main material properties to design, both for rock and concrete units. It should be mentioned that, given the description of the test case in Section 1.4, it is expected that the required rock size is not available. It is not a problem if the design calculations yield unrealistic rock sizes, as it will still give insight into aspects in the new Eurocode that need more clarity.

3.3.3 GEOMETRICAL PARAMETERS

The geometrical parameters of importance are:

- The slope of the breakwater face
- The level of the crest, i.e. top of the crown wall
- The level of the armour crest
- The height of the crown wall
- The width of the crown wall
- The thickness of the crown wall

The slope of the breakwater in case of a rock armour layer has been chosen to equal 1:3. The slope of the breakwater in case of an armour layer of artificial units has been chosen to equal 1:1.5. The level of the crest will be designed based on overtopping requirements. The level of the armour crest, and the height, width and thickness of the crown wall will be defined in the design process of the crown wall.

3.4 COLLECTION OF DATA

From the basic variables, it has been concluded that information about waves, water levels and bathymetry is necessary for design calculations. Subsection 3.4.1 discusses the retrieval of the data, together with some questions that might arise when doing so. Subsection 3.4.2 provides an overview of the wave and water level conditions at the site of interest. Subsection 3.4.3 examines the bathymetry near the breakwaters.

3.4.1 DATA RETRIEVAL

3.4.1.1 SOURCE

For the collection of data, use has been made of the website Waterinfo [Ref. 15]. This is a platform run by the governmental institution Rijkswaterstaat, which provides measurements of several water-related topics such as waves, water levels, currents, discharges, etc.

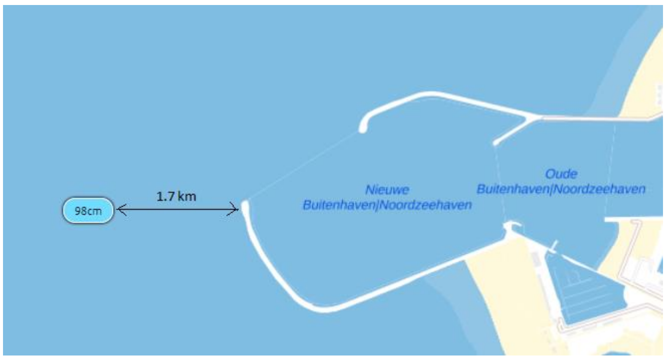
Data has been obtained on:

- Wave height
- Wave period
- Water level

No failure mechanisms involving current velocity will be investigated, hence there is no need for data on currents[8]. It is possible for currents to interact with waves, but it has been assumed that the effects of this on the waves are negligible. Apart from the data mentioned above, it is also useful to obtain data of the astronomical tide [Ref. 15] and the bathymetry. As the platform Waterinfo only provides data on water-related quantities, the bathymetric data has been gathered from Waterinfo Extra [Ref. 16], which contains data on morphology.

3.4.1.2 LOCATION

It is preferrable that the location of the measurements is as close as possible to the location of the breakwaters, since this would mean that the measurements reflect the actual conditions at the site of interest well. Figure 3.2 shows the point where the data has been retrieved with respect to the head of the southern breakwaters[9].



[8] It should in fact be investigated whether this is indeed justified by considering typical values of spring tide currents accompanied by an explanation of why they are neglected, but this has been left out of the scope of this thesis.
[9] The location of the cross-section that is considered in the design will be slightly further than the head, but the intention is to show the order of magnitude.

Figure 3.2: Location of retrieval of wave and water level data [Ref. 15]

It is not unusual for the location of sea condition measurements to lie several tens of kilometres away from the site of interest. However, for this case (as is visible in Figure 3.2) this point only lies approximately 1.7 km away from the breakwaters. Therefore, the measured wave and water level data will probably represent the actual situation at the toe of the structure quite well. It is thus assumed that it is not necessary to perform an extensive offshore-nearshore wave transformation study. In Section 3.7 it will briefly be explained that this decision is indeed justified. Another simplification that has been made is to disregard the influence of wave direction, since this information was not available for the location of measurements. When the wave direction is relevant, it will be assumed that the wave attack is perpendicular to the cross-section of interest.

3.4.1.3 LENGTH

With the help of the ServiceDesk Data of the platform Waterinfo, data were retrieved from November 2002 till March 2021, a period of approximately 18.4 years. The new Eurocode provides guidance on the required length of the data record, but this guidance is not unambiguous. Throughout the document, terms as 'sufficient length' and 'long-term time-series data' are used, without conclusively defining what this means. In Clause 5.3.1 in prEN1991-1-8 the following phrase can be found:

'Statistics of extreme sea conditions at a specific site should be established on the basis of instrumentally measured and/or hindcast data, coupled with necessary transformation analysis to represent key physical processes that may influence conditions between the measurement/ hindcast location and structure location, that cover the duration as long as possible and not less than 15 years.'

This seems to indicate that the length of the data record of 18.4 years is sufficient. Nevertheless, the following is stated in Clause C.2.4.1:

'The length of data record is preferably 30 years or longer. A long record is needed so as to reduce the effect of sample variability and to minimize the influence of wave climatic changes on the prediction of extreme wave heights for a long return period such as 100 years.'

This somewhat negates the previously made statement. The breakwater design will be continued on the premise that the length of the data record is sufficient, as the question what should be done when the length of the data record is insufficient remains unanswered.

3.4.1.4 SPECTRAL VS. TIME-DOMAIN ANALYSIS

The final aspect related to data collection that should be addressed is how to deal with short-term wave statistics. There are two ways in which the significant wave height H_s can be estimated from the recorded data. On the one hand you can use time-domain analysis, and on the other hand you can use spectral analysis. For the data point near IJmuiden, both $H_{1/3}$ (following from time-domain analysis) and H_{m0} (following from spectral analysis) measurements are available. If both estimations are available, which one should then be used for design purposes?

The Eurocode does not give much clarity on this topic. In clause 5.5.1.1 the following is mentioned:

'For an accurate analysis of wave conditions, a spectral approach should be used. If required, regular wave parameters for design may be derived from spectra.'

This seems to indicate that a spectral analysis is preferred over a time-domain analysis. However, in clause 5.5.1.5 this is contradicted by stating:

'The characteristic heights of wind waves and swell for evaluation of the actions from waves should be the significant wave height $H_{1/3}$ and the highest wave height Hmax, which are defined by the zero-crossing method in the time domain analysis. Other definitions of wave heights may be used as the characteristic wave heights when a method of evaluation requires the use of such wave heights. The significant wave height may be estimated from the zero-th moment of wave spectrum, m0, as being equal to 4.0 $m0^{1/2}$. When this estimation is employed, the symbol Hm0 should be used instead of $H_{1/3}$ so as to clarify the estimation method of the significant wave height, because they can differ by several percent or more.'

According to this passage, it is accepted to use either of the two estimation methods. This choice does, however, influence the magnitude of the wave height that will be used in the design formulae, even though the difference will be negligibly small in deep water.

A similar consideration applies to the wave period. The wave period occurs in the surf similarity parameter, and its index shows that the mean wave period has to be worked with. The mean wave period is generally a parameter belonging to time-domain analysis, but it too can be estimated by spectral analysis. This is explained in the new Eurocode in Clause 5.5.1.5:

'The mean period may be estimated for a narrow spectrum from the zero-th and second moments of wave spectrum as being equal to $2\pi\ (m0/m2)^{1/2}$. When this estimation is employed, the symbol Tm0,2 should be used so as to clarify the estimation method of the mean wave period, because the spectrally estimated mean period is generally smaller than the individually counted mean period.'

In this case, only the mean period estimated from a spectral analysis was available for the data point near IJmuiden, which will therefore be used for the design. In order to be consistent, the significant wave height will be derived from spectral analysis as well.

In reality, the appropriate estimation method depends on the formula that is used, which should be explicitly stated in prEN1991-1-8, but this is currently not evident.

3.4.2 OVERVIEW WAVE AND WATER LEVEL CONDITIONS

This Subsection visually presents the obtained wave and water level data, so as to give an idea of the order of magnitude of the sea condition parameters near the IJmuiden breakwaters. Any erroneous or unrealistic measurements have been filtered out of the data. Figure 3.3 shows the obtained wave height data for the time period of 2002-2021, Figure 3.4 shows the obtained water level data for the same time period, and Figure 3.5 plots wave height versus wave period:

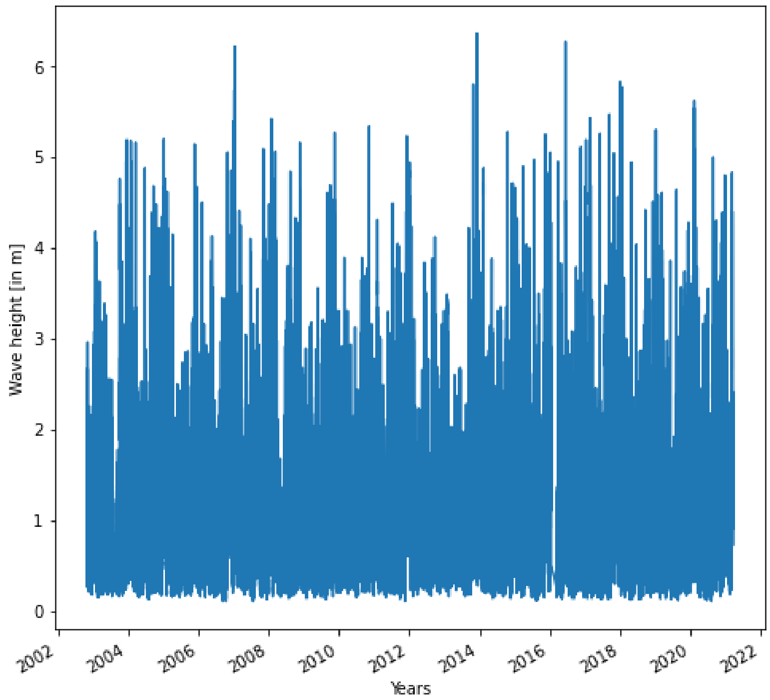


Figure 3.3: Time series of significant wave height

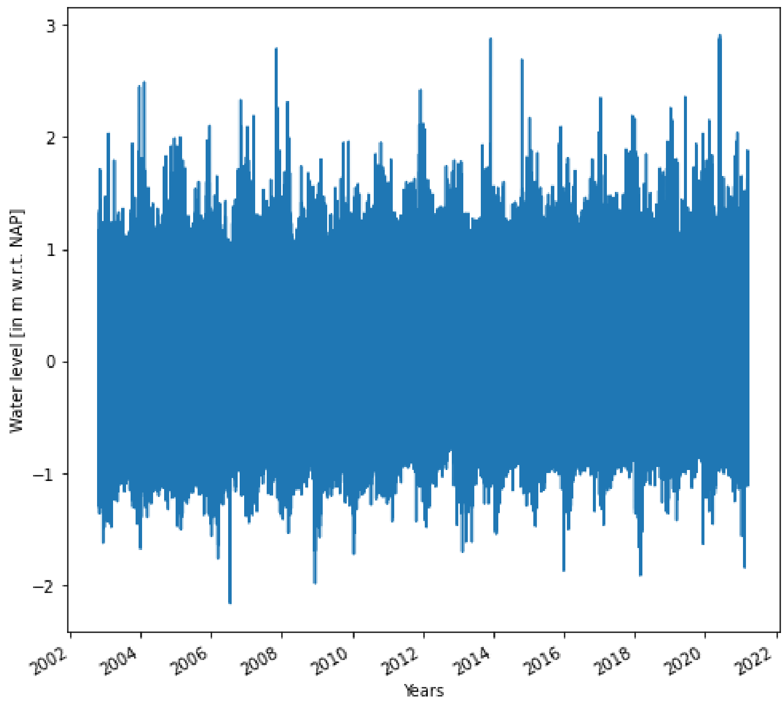


Figure 3.4: Time series of water level

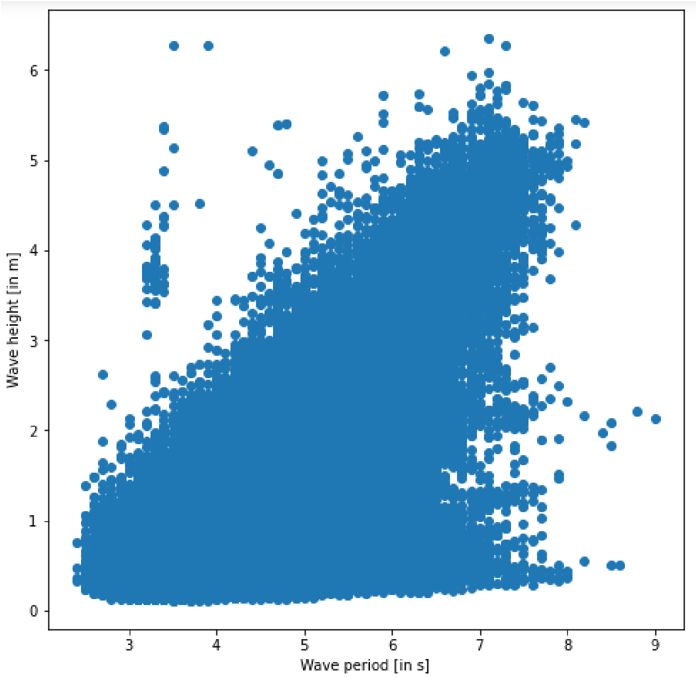


Figure 3.5: Scatter plot of significant wave height versus mean wave period

From these Figures, it can already be deduced that the breakwaters need to be able to withstand waves higher than 6 m. Moreover, it can be noticed that water levels of almost 3 m+NAP can be reached, which is relevant for the combined action of water level and waves. Finally, it can be concluded that the waves are locally generated wind sea waves, and not swell waves. The fictitious wave steepness namely has a magnitude of 0.05 or even higher.

3.4.3 BATHYMETRY

In this subsection, it will be examined what the bathymetry near the breakwaters at IJmuiden looks like. A fictitious line has been drawn between the two main locations of interest: the point of data retrieval and the toe of the structure, see Figure 3.6:

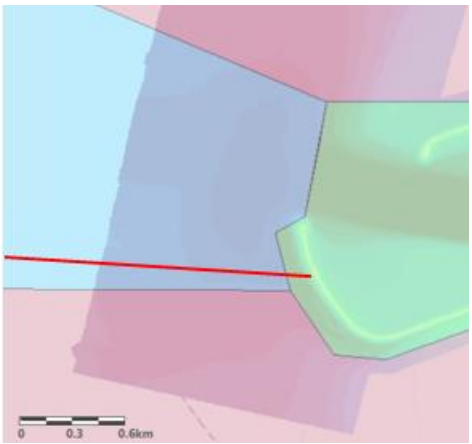


Figure 3.6: Section (red line) for which bathymetric profile is drawn up

For this fictitious line, the bottom levels have been schematised in Figure 3.7:

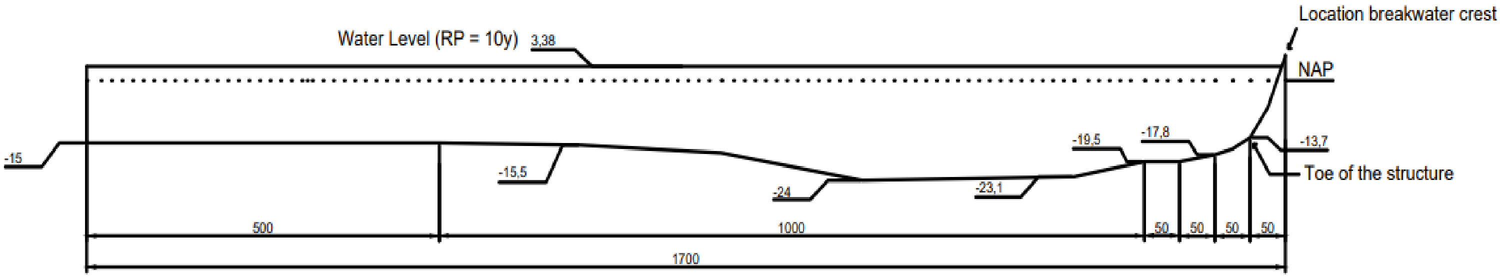


Figure 3.7: Bottom levels of fictitious line [Ref. 16]

The bathymetric profile can be used to see how the depth develops towards the breakwaters. This is necessary for an offshore-nearshore wave transformation. Moreover, the bottom level at the toe of the structure in combination with the water level can be used to say something about wave breaking. This will be discussed in Section 3.7.

The bottom level at the point of data retrieval is also interesting, to see whether the wave measurements are affected by depth-limitations or not. An expression that relates the significant wave height and the depth to each other, can be found in the Lecture Notes of Coastal Dynamics [Ref. 8]:

$$H_{s,max} \approx 0.45 * d$$

The bottom level of -15 m with respect to NAP, combined with a water level of 3.38 m+NAP, gives a depth of 18.38 m at the point of data retrieval. The maximum possible significant wave height for this depth is then computed to be 8.27 m. The highest wave that was measured in the 19-year long wave record had a height of 6.4 m, which is lower than the limit imposed on it by the depth. It is therefore safe to perform an Extreme Value Analysis on the wave data without getting biased results.

For more information about how the bathymetric profile has been drawn up, reference is made to Appendix D.

3.5 CORRELATION ANALYSIS

The combination of wave height and water level becomes relevant for the armour layer when the wave height is limited by depth. It has been assumed that this is not the case, since depth-limitation does not apply to the waves at the measurement location, and these waves will directly represent the wave conditions at the site of the breakwaters. Section 3.7 further explores this. Anyway, for the determination of the crest height and the crown wall design, the combination of the two parameters is most definitely relevant. The following quote with respect to the combination of sea state parameters can be found in Clause 5.3.3 in the new Eurocode:

'When the combination of certain parameters, particularly wave height and water level, (and also current for some applications), are important in the design of coastal structures, it should be established from the available data whether large wave heights and high water-levels (or other combinations of parameters, e.g. wave orbital velocities and currents or water-levels and currents) tend to be dependent or independent of one another.'

Hence, it needs to be investigated whether a certain degree of dependence exists between wave height and water level for the location of IJmuiden. In Clauses A.6.5, A.6.6. and A.6.7 in the new Eurocode, a distinction is made between 'less than moderately correlated environmental parameters' and 'moderately or strongly correlated environmental parameters'. It can be deduced that correlation is an adequate criterion for the dependence of two parameters. Unfortunately, this still does not mean that it can conclusively be decided which of the two categories should be worked with. Three aspects are left unanswered:

- How should the correlation be determined?
- What are the boundaries when speaking of 'less than moderately' and 'moderately or strongly' correlated?
- Should the wave height be correlated to the absolute water level? Or to the storm surge only?

Despite the lack of clear instructions by the draft Eurocode on these aspects, it will be attempted to quantify the correlation by making several assumptions.

Ad. 1 The Pearson product-moment correlation coefficient will be used to quantify the correlation. It shows how strong the linear relationship between two parameters is, and can take values between -1 and 1.

Ad. 2 The boundaries that are worked with are the following: a correlation coefficient lower than 0.40 will indicate weak correlation, a correlation coefficient higher than 0.40 will indicate moderate correlation, and a correlation coefficient higher than 0.60 will indicate strong correlation (see Appendix E).

Ad. 3 The storm surge levels are obtained by subtracting the tide from the water level data. Intuitively, it is better to correlate the wave height to the storm surge only, as waves and storm surges can have the same driving parameter, being the wind. The tide does not have such a relationship with the waves, and would therefore only act as a disturbance.

In Figure 3.8, the dependence between wave height and storm surge level is illustrated:

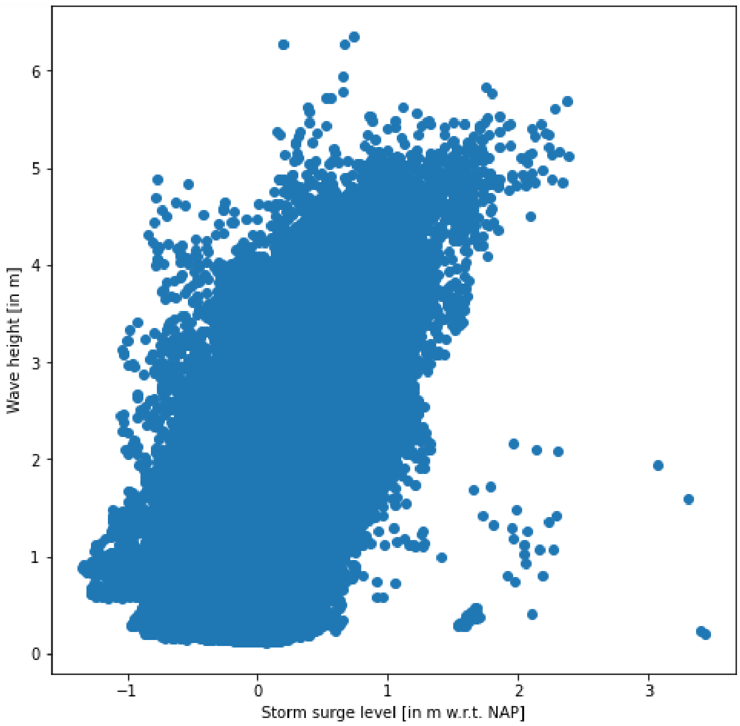


Figure 3.8: Scatter plot of wave height versus storm surge level for the location of IJmuiden

Figure 3.8 shows a positive correlation between the storm surge level on the X-axis and the wave height on the Y-axis. That is, a high wave height is more likely to be accompanied by a high water level. This were to be expected, as the North Sea has a wave climate dominated by wind [Ref. 8] that generates both high waves and water levels.

The data in Figure 3.8 has a Pearson correlation coefficient equal to 0.59, which confirms the positive correlation. The magnitude of this coefficient indicates that we are dealing with moderate (and almost strong) correlation. If the wave height were to be correlated with the absolute water level, a correlation coefficient of 0.30 is found, indicating a weak correlation.

It is assumed that the proper way to determine the dependence between the two parameters is correlating the wave height with the storm surge level instead of the water level. So, for the design of breakwater elements for which a combination of these two sea state parameters is required, the wave height and water level should be treated as being ‘moderately or strongly correlated environmental parameters’.

According to Clause A.6.7 in prEN1991-1-8, this would mean that a joint probability distribution for wave height and water level should be drawn up. However, the construction of a joint probability distribution has been left outside the scope of this thesis. It is questionable anyway if a joint probability analysis should be demanded in the DA-1 format. The semi-probabilistic approach should be a fast way to achieve the desired reliability, but if you have to set up an entire joint probability distribution, one could just as well turn to a full probabilistic approach.

Instead of the joint probability analysis, full correlation will be assumed for the relevant failure mechanisms in the remainder of this document, by using marginal distributions for both the wave height and water level at the same design Return Period. Clause 5.3.3 in the new Eurocode mentions this as an alternative but also warns that it is very conservative. Sensitivity of variations in the water level will therefore be investigated in Chapter 4 by considering the design outcome for weakly correlated variables.

Appendix E treats the analysis of correlation in more detail.

3.6 HYDRAULIC BOUNDARY CONDITIONS

In Subsection 3.4.2, some light was already shed on the order of magnitude of wave height, wave periods and water levels near IJmuiden. However, to perform design calculations we need more precise information on the wave and water level conditions, in particular the extreme conditions. Subsection 3.6.1 will discuss the wave height, Subsection 3.6.2 will establish the accompanying wave period, whereafter subsection 3.6.3 will treat the water level.

3.6.1 WAVE HEIGHT

We must select the return period that will be designed for, so as to be able to implement the semi-probabilistic design approach. Subsequently, the wave data needs to be analysed to compute which wave height belongs to the selected return period.

3.6.1.1 SELECTION OF RETURN PERIOD

In Clause 5.3.1 of the new draft Eurocode the following statement is made:

'The probability of an event should be characterized by its return period, T_R, a statistical definition, being the period that (on average) separates two occurrences of equal or greater magnitude.'

In the new Eurocode, the return periods are prescribed based on the consequence class, the design service life and the limit state. It has been established that the breakwaters belong to consequence class 2 and have a design service life of 50 years. However, question marks have been put to these specifications in Section 3.1, in which it was argued that CC3 and T_{life} = 100 years would also be valid choices.

For the failure mechanisms that will be discussed in the design, the wave height will be the dominant component. This is especially true, since the wave height will be factored whereas this does not hold for the water level. Tables 2.5 and 2.6 (originally Tables A.4 and A.5 in prEN1991-1-8) can then be consulted for the selection of the appropriate return period. Table 3.3 repeats the most relevant values from these tables:

Consequence Class	**T_{life}**	
	50 years	**100 years**
	Limit state: ULS	
CC3	RP = 200 [y]	RP = 400 [y]
CC2	RP = 100 [y]	RP = 200 [y]
	Limit state: SLS-(LD)	
CC3	RP = 20 [y]	RP = 40 [y]
CC2	RP = 10 [y]	RP = 20 [y]

Table 3.3: Characteristic return period as a function of consequence class, design service life and limit state from draft Eurocode [Ref. 2]

From Table 3.3 it becomes clear that the uncertainties concerning the consequence class and design service life make quite the difference, as the return period that is consequently selected may range from 100 to 400 years. Nevertheless, the classifications made in Section 3.1 will be stuck to. It can therefore be concluded that the return periods of interest are **RP=100 years** for ULS and **RP=10 years** for SLS-(LD). In the remainder of this subsection, when uncertainties need to be illustrated, a return period of 100 years will be used.

3.6.1.2 PEAK-OVER-THRESHOLD

As the record of available hydraulic data is shorter than the prescribed return period, extrapolation of this data through means of an extreme value analysis is required. For this, a dataset with extreme storm events should be generated from the gathered wave data. The new Eurocode mentions the following in Clause 5.3.1:

'The peaks-over-threshold (POT) method should be preferred to produce the data set of extreme values. To apply this method, suitable threshold and event separation values should be determined.'

The data set of extreme values depends on two variables: the threshold above which waves are counted as storm events, and the required time between waves in order for them to be counted as separate storm events. The new Eurocode, however, fails to discuss what is meant by 'suitable threshold and event separation values'.

From Figure 3.3, which shows the time series of significant wave height, it is visible that choosing a threshold of 6.0 m would give a data set with only very few extreme events. On the other hand, choosing a threshold of 2.0 m would result in a data set with so many events that you could barely refer to all these events as being extreme. Anything in between, however, could be chosen, as prEN1991-1-8 does not specify how to deal with these variables.

Below it is investigated how choices with regards to the threshold value and event separation time affect the Extreme Value Analysis. Since the type of distribution has not yet been discussed, a Weibull distribution has been assumed for illustration purposes:

Threshold value [m]	Event separation time [h]	Distribution	RP	Number of storms per year	H_s [m]	$\%\Delta H_s$
4.00	24	Weibull	100	8.4	7.01	-
4.00	36	Weibull	100	7.7	7.03	0.29
4.00	48	Weibull	100	7.3	7.02	0.14

Table 3.4: Impact of event separation time on significant wave height

From the results in Table 3.4 it can be deduced that the declustering time span has little influence on the significant wave height, it only has a small effect on the number of storms per year.

Threshold value [m]	Event separation time [h]	Distribution	RP	Number of storms per year	H_s [m]	$\%\Delta H_s$
4.50	36	Weibull	100	4.0	7.01	-
4.00	36	Weibull	100	7.7	7.03	0.29
3.50	36	Weibull	100	13.0	7.19	2.6
3.00	36	Weibull	100	21.6	7.44	6.1

Table 3.5: Impact of threshold value on significant wave height

From the results in Table 3.5 it can be found that for slightly lower threshold values the significant wave height barely changes, but that generally speaking the choice of the threshold value can lead to a difference in wave heights of up to several percent. Moreover, from Table 3.5 it is also evident that the threshold value has an influence on the number of storms per year.

In paragraph 4.3.2 of the Breakwater Design Lecture Notes [Ref. 7], the following is mentioned:

'A good rule of thumb for a first approach is to select the threshold level such that approximately N_s=10 storms per year of data remain.'

This is of course not a strict requirement, the number of storms per year may also be somewhat lower or higher, but at least it gives guidance on the selection of the threshold and event separation values. Currently, the new Eurocode does not provide sufficient information on this matter. Based on the rule of thumb described above, it has been chosen to adopt a **threshold value of 4 m** and a **declustering time span of 36 h**.

3.6.1.3 (INDEPENDENT) EXTREME VALUE ANALYSIS

For the performance of the Extreme Value Analysis, a distinction can be made between two steps that need to be undertaken: the distribution fit and the extrapolation.

DISTRIBUTION FIT

Once the data set of extreme events has been constructed, an extreme value distribution should be fitted to the data set. Multiple extreme value distributions exist that can represent a wave climate, and it is unclear which distribution is the 'true' distribution for a certain climate. The new Eurocode mentions the following in Clause C.2.4.2:

'Commonly employed distributions in extreme wave analysis are the Fisher-Tippett type I (double exponential or Gumbel) and the Fisher-Tippett type II (Frechét) and the Weibull distributions (see Clause 11 of Goda 2000 for their functional forms). However, other distributions such as the Generalized Extreme Value and the log-normal distributions can also be used.'

This suggests that there are various distributions that may be used for design purposes. In this case, four candidate distribution types have been used to demonstrate the uncertainty related to this aspect. These four distribution types are: the Exponential distribution, the Gumbel distribution, the Weibull distribution and the Generalised Pareto distribution.

In Clause C.2.4.3 of prEN1991-1-8 the following two statements are then made:

'The Least Squares Method (LSM), the Maximum Likelihood Method (MLM), and other valid methods may be employed for distribution fitting.'

'Appropriate criteria of best fitting and/or rejection should be chosen and applied for the data set, depending on the methodology of data fitting.'

The methodology of data fitting that has been used is the Least Squares Method, also known as linear regression. The results of this can be found in Figure 3.9:

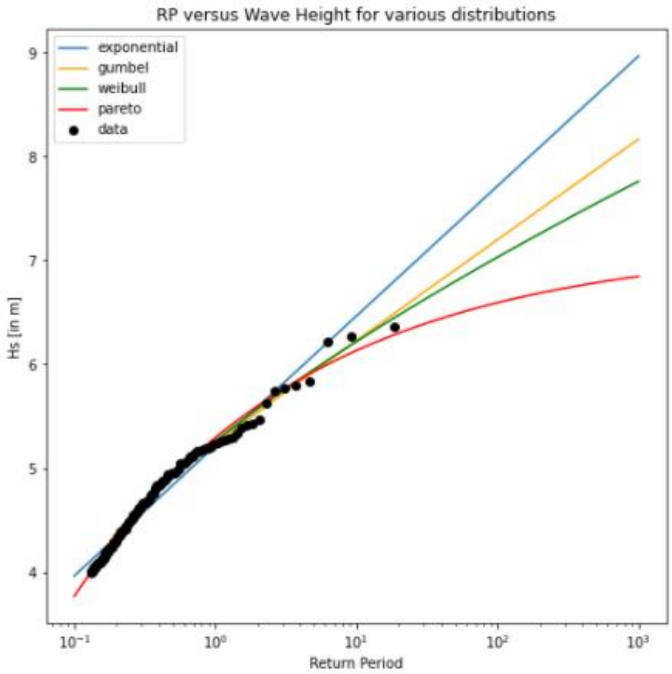


Figure 3.9: Fitting of distributions to data set of extreme events

The numerical values belonging to the graph in Figure 3.9 are presented in Appendix E, along with a description of the distributions.

The 'true' distribution should then be chosen according to appropriate criteria of best fitting and/or rejection. One tool to evaluate how good a certain fit is, is the root-mean square error. The value of the RMSE for each distribution is shown in Table 3.6:

Distribution	**RMSE [m]**
Exponential	0.0919
Gumbel	0.0630
Weibull	0.0429
Generalised Pareto	0.0451

Table 3.6: Comparison of root-mean square error for various distributions

From both Figure 3.9 and Table 3.6 you can conclude that the exponential distribution should not be used as it is too conservative. However, the other three distributions may all very well be the 'true' distribution. One could argue that Weibull is the best because it has the lowest RMSE, but it is questionable whether this can be claimed with certainty as the number of data points is low. Moreover, a different choice for the threshold level and/or event separation time could even lead to another distribution having the lowest RMSE. Below it is explored by how much the wave height differs if either of the three remaining distributions is employed for a return period of 100 years:

Distribution	**RP**	**H_s [m]**	**%ΔH_s**
Generalised Pareto	100	6.59	-
Weibull	100	7.03	6.7
Gumbel	100	7.20	9.3

Table 3.7: Influence of use of various distributions on design wave height

As Table 3.7 demonstrates, the choice for a certain extreme distribution may have a significant effect on the wave height that will be used in design. The new Eurocode should describe, in more detail, on the basis of what criteria a distribution should be selected. In this test case, the **Weibull distribution** has been adopted as the true distribution.

EXTRAPOLATION

The limited amount of data leads to uncertainties when extrapolating. PrEN1991-18 proposes to take this into account by working with confidence intervals. In Table A.4 in the new Eurocode the following is stated:

'The statistical estimation of the RP value can be based on the upper bound of a given confidence interval, considering the uncertainty due to the limited sampling size.'

Nevertheless, the information is incomplete as it is not specified which confidence interval should be used. Moreover, in Clause 4.5.1 the following three citations can be found:

'Values to be applied in design can be higher than their Central Statistical Estimate (CSE). The following safety format should be followed in this respect.'

'DA-0: If no safety margin is stated for the formula employed (or a deterministic version of the formula not available) then a safety margin of one Standard Deviation (SD) above the CSE should be applied to the sea condition parameter.'

'DA-1: This Design Approach refers to the principal design format in the Eurocodes associated to the use of partial factors that cover the required safety margin for a range of design cases.'

These statements seem to imply that only in the DA-0 format the designer is in particular circumstances allowed to use confidence intervals, and in the DA-1 format the central statistical estimate can be used as the required safety margin is covered by the use of partial factor. Figure 3.10 demonstrates the uncertainty related to the extrapolation of the wave height data, computed by means of bootstrapping[10]:

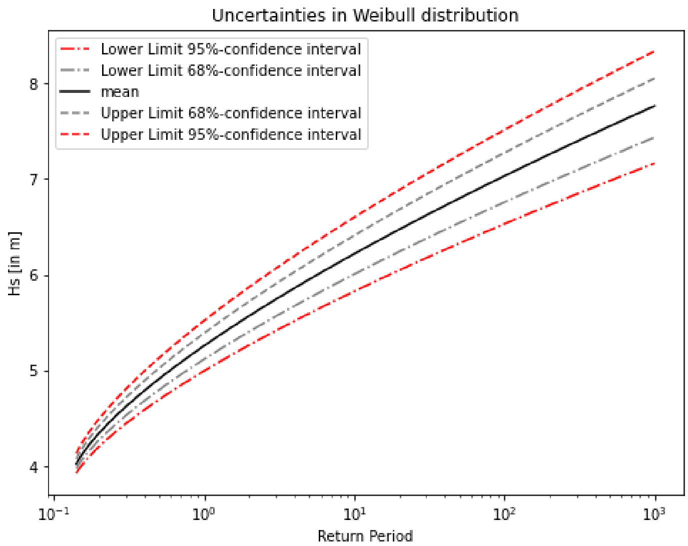


Figure 3.10: Graphical display of uncertainties in Weibull distribution

[10] This procedure usually results in larger relative uncertainties for larger return periods. It could be that the method has been incorrectly implemented. Nevertheless, the uncertainty predicted with the help of the Goda equations set out in [Ref. 7] was of the same order of magnitude.

The upper limit of the 68%-confidence interval is by definition 1 standard deviation above the mean and corresponds to an 84%-non-exceedance value, whereas the upper limit of the 95%-confidence interval is by definition 2 standard deviations above the mean and corresponds to a 97.5%-non-exceedance value. Other confidence intervals, such as the 90%-confidence interval, are common as well. In Table 3.8 the impact of this choice on the wave height is presented:

RP	**%-non-EV**	**H_s [m]**	**%ΔH_s**
100	50	7.03	-
100	84	7.26	3.3
100	97.5	7.50	6.7

Table 3.8: Impact of using certain exceedance values for the wave height

A consistent use of prEN1991-1-8 is only possible when it is evident whether the central statistical estimate or a certain non-exceedance value should be used when statistically extrapolating. In addition, it should also be specified which non-exceedance value or confidence interval should be applied. In this test case, the **CSE** of the wave height will be worked with.

The numerical values belonging to the graphs in Figure 3.9 and Figure 3.10 are presented in Appendix F, along with a description of the distributions and a more elaborate description of the methods in the Extreme Value Analysis.

3.6.2 WAVE PERIOD

Apart from the wave height, information on the wave period is also required for several design formulae. Only a few short passages about this are stated in the new Eurocode, one of which can be found in Clause C.2.4.6:

'The information of wave period associated with the R-year return wave height is often needed when evaluating actions from waves. However, no established method is currently available to estimate such the wave period. Often a joint distribution of storm wave heights and periods is prepared to find out a meaningful correlation between the height and period. For fully-grown wind waves in deep water, the following mean relationship can be quoted: $T_{1/3} \cong 3.3 * H_{1/3}{}^{0.63}$*.'*

The wave period should thus be determined as a function of the wave heights. First, it needs to be investigated whether the mean relationship presented in the citation can be used, or whether a joint distribution of storm wave heights and periods should be prepared. It turns out that the mean relationship is not valid for the location of IJmuiden, see Appendix F.

Hence, we need to find a meaningful correlation between the height and period. There is no description of a method on how to draw up a joint distribution of storm wave heights and period, so a certain relationship needs to be assumed. In the Lecture Notes of Breakwater Design [Ref. 7] it is proposed to fit the following relation to the data:

$$T_m = a * \sqrt{H_s}$$

In this expression, it is assumed that the wave period has a linear relationship with the square root of the wave height. An alternative to this expression is one in which the wave period has a linear relationship with the wave height to an arbitrary power, instead of a square root. This will introduce an extra fitting parameter into the distribution:

$$T_m = a * H_s^b$$

In Figure 3.11 you will find the result of fitting both of these relations to the wave data:

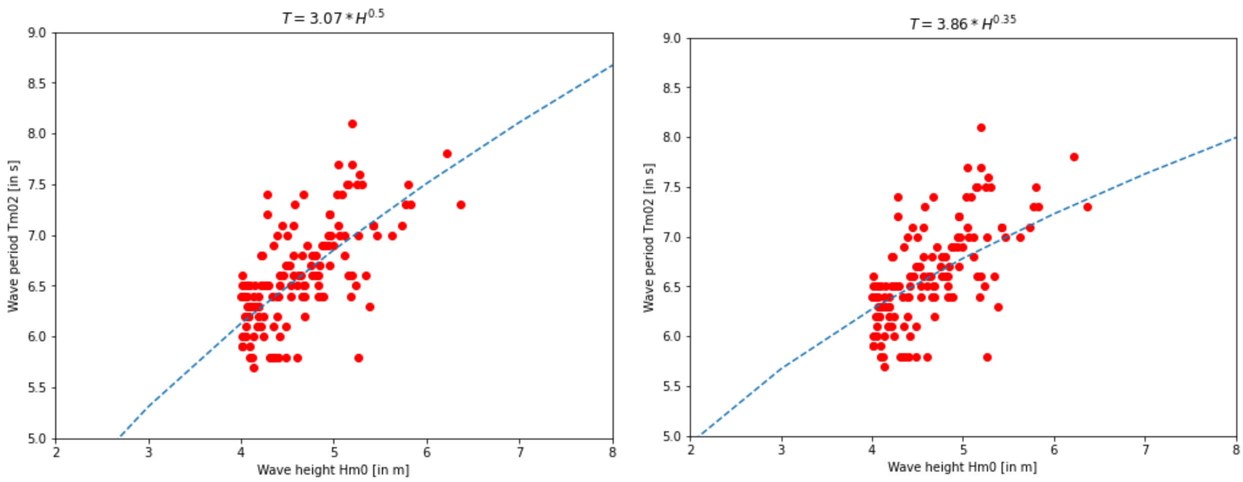


Figure 3.11: Fitting a square root-function (left) and an arbitrary power-function (right) to the wave data

At first glance, both alternatives seem to be reasonable. The fictitious wave steepness will be considered to determine which of the two options is physically most realistic. The expression for the fictitious wave steepness is as follows:

$$s_{0m} = \frac{2\pi H_s}{g T_m^2}$$

Table 3.9 explores the wave steepness for both joint distributions of storm wave heights and periods:

Wave height raised to the power of …	**Coefficient a**	**RP**	**H_s [m]**	**T_m [s]**	**S_{0m}**
0.5	3.07	100	7.03	8.13	0.068
0.35	3.86	100	7.03	7.64	0.077

Table 3.9: Consideration of the wave steepness for a square root-function and an arbitrary power-function

An expected value for the wave steepness of storm waves is approximately 0.05. It should be noted that the values in Table 3.9 represent the fictitious wave steepness and not the actual wave steepness. Nevertheless, as the wave measurements stem from intermediate to deep water, the actual wave steepness will not deviate much from these values. Both values are quite high, but the value of 0.077 is perhaps a bit too extreme. Therefore, the following expression that relates the wave period to the wave height is adopted:

$$T_m = 3.07 * \sqrt{H_s}$$

The statistical uncertainty related to the fit has also been investigated by means of bootstrapping and is shown in Figure 3.12. It is visible that the uncertainty is very low. This can be explained by the fact that the only coefficient that can vary is the multiplication factor, the square root itself is a given.

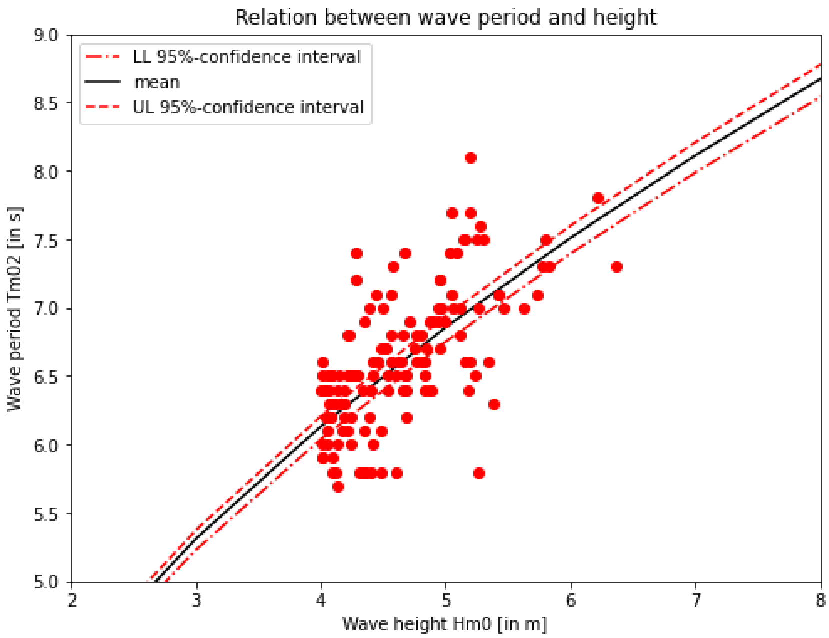


Figure 3.12: Square-root relation between wave period and height, together with lower and upper limit of 95%-confidence interval

3.6.3 WATER LEVEL

In order to determine the design water level, three components [11]will be considered. These three components are the tide, storm surges and sea level rise. The first paragraph deals with the combined effect of the tide and surges, after which the second paragraph dives deeper into sea level rise.

3.6.3.1 TIDE & SURGE

According to the new Eurocode, tides and surges can either be treated separately or in combination with each other depending on their relative magnitude. The following two statements can be found in Clause 5.4.4:

'In locations where the tidal range is small relative to the design event surge magnitude, a statistical Extreme Values Analysis (EVA) may be undertaken on the surge component only, i.e. where the astronomical tidal component is separated from a record of total water-levels and the surge records are treated independently. The extreme surge values may then be combined with a selected tide level (and other components of the design water level).'

'In locations where the tidal range is large relative to the design event surge magnitude, careful consideration, selection and justification of whether tide and surge are treated separately or in combination should be made, e.g. to avoid a potential over-estimate of a design water level by addition of an extreme surge value and a high astronomical tide condition.'

It happens to be the case that the maximum tidal range and storm surge are more or less of the same magnitude. As prEN1991-1-8 fails to be more specific regarding the desired method, it is difficult to conclusively state whether the tide and surge should be treated together or apart from each other.

Consequently, a choice needs to be made by the designer. An Extreme Value Analysis can be performed to compute the design water level. Although it is not strictly correct in a probabilistic sense to treat tide and storm surge simultaneously in an EVA, this has been opted for in this case. The new Eurocode allows it, as can be read in Clause C.1.2:

'Extreme statistical analysis of storm surge levels: This can be done for both the absolute level of the highest water above the datum level and the deviation of highest water level from the astronomical tide at the times of storm surges. The analysis can yield the return water level corresponding to a designated return period.'

If you were to treat the tidal component and the storm surge level separately, this would give rise to another challenge, namely which tidal level the storm surge event should be added to. Clause C.1.1 gives some guidance with respect to this, but the designer is still left with several options.

The Extreme Value Analysis was thus performed for the absolute level of the water above the datum. The analysis follows the same steps as was set out in Subsection 3.6.1. That is, the Peak-over-Threshold method has been applied to the water level data, after which several distributions were tried to find the best fit, whereafter the function is extrapolated to the return period of interest.

[11] The wave set-up has been neglected, as its effect is usually small, and prEN1991-1-8 provides few guidance on this nearshore process.

A summary of the design considerations that were made for the Extreme Value Analysis of absolute water levels are displayed in Table 3.10:

Parameter	**Value**	**Unit**
Threshold value	1.80	m+NAP
Event separation time	36 h	h
Number of storms (2002-2021)	86	-
Number of storms per year	4.7	-
Distribution type	Weibull	-
Root-mean-square error	0.033	m

Table 3.10: Summary of important input and output parameters in the EVA

The choice for the distribution type was quite arbitrary. It has been assumed that the extremal distribution functions that were considered for wave heights, may also be employed for the water levels. This is not self-evident, as Clause C.2.4.2 in prEN1991-1-8 only speaks of *'extremal distribution functions for storm wave heights'*. In addition, experience has taught us that water levels are usually Gumbel distributed[12]. For this particular case, the Gumbel distribution and Weibull distribution are quite different, so it is a shortcoming of the draft Eurocode that no guidance on this is given whatsoever.

Table 3.11 shows the eventual outcome of the Extreme Value Analysis:

Return Period [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Water level [m+NAP]	2.20	2.40	2.67	2.88	3.10	3.39	3.61	3.83

Table 3.11: Water level for various return periods, based on EVA of total water level signal (combined tide and surge)

It is interesting to see how this might change when treating the tide and storm surge separately. The values that you would then obtain are presented in Table 3.12:

Return Period [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Water level [m+NAP]	2.73	2.95	3.23	3.45	3.67	3.96	4.19	4.41

Table 3.12: Water level for various return periods, based on EVA of surge only and adding tidal level MHWS

The values in Tables 3.11 and 3.12 differ by a reasonably large amount, so it is a shortcoming that the draft Eurocode is not more specific when it comes to determining the design water level.

Appendix F explains in more detail the considerations that were made in the Extreme Value Analysis for water levels.

[12] Based on the expertise provided by ing. C. Kuiper and Ir. J.P. van den Bos

3.6.3.2 SEA-LEVEL RISE

The new Eurocode mentions the following about climate change in Clause 5.2.1:

'Design of coastal structures should take account of the fact that climate change can cause sea-level rise and changes in storm intensity and direction (potentially affecting coastal surge/wave and pluvial/fluvial events). The design should take into account region-specific guidance where available, as well as the probability level associated with published or predicted changes, e.g. due to inherent uncertainties in future emissions scenarios and climate modelling generally.'

Sea-level rise thus needs to be taken into account. This can be done by considering the various climate scenarios. In order to do this adequately, it should be described in prEN1991-1-8 how to deal with the future emissions scenarios and climate modelling, but this lacks. An average climate scenario has been assumed, for which it can be deduced that a sea-level rise of 50 cm is a reasonable value for the end of design lifetime, although this value is prone to a lot of uncertainty.

If the allowance of 50 cm sea-level rise is added to the already determined values in Table 3.11, it results in the values shown in Table 3.13:

Return Period [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Water level [m+NAP]	2.70	2.90	3.17	3.38	3.60	3.89	4.11	4.33

Table 3.13: Design water level for various return periods

In Appendix F, a more elaborate description of the treatment of sea-level rise is presented.

3.7 OFFSHORE-NEARSHORE TRANSFORMATION

In Paragraph 3.4.1.2, it was established that an extensive offshore-nearshore wave transformation study would not be necessary, as the measured wave and water level data will probably represent the actual situation at the toe of the structure quite well. This assumption was made based on the fact that the location of data retrieval is close to the breakwater site[13].

The assumption will be explored in this section. In Table 4.7 in the draft Eurocode, the following statement can be found describing the required pathway assessment for a structure designed at an HEA-2 level:

Numerical wave transformation model representing (with reasonable accuracy) all key physical processes expected. Adjustment of (statistically estimated) sea-level or current values to account for site-specific physical processes either by empirical or numerical model.

So prEN1991-1-8 actually prescribes to transform the waves nearshore with the help of a numerical model, but not much is said about this afterwards. A good starting point would be to provide the user with examples of a program that can be used for this, and describe the key physical processes that should be included, but this is only briefly touched upon in the draft Eurocode without giving specific recommendations.

Nevertheless, an attempt has been made to set up an appropriate numerical wave transformation model. A simple offshore-nearshore wave transformation has been executed. Simple in this case means a single run, for a one-dimensional instead of a two-dimensional set-up. The program SwanOne has been used to perform the calculation. The graphical output is visible in Figure 3.13[14]:

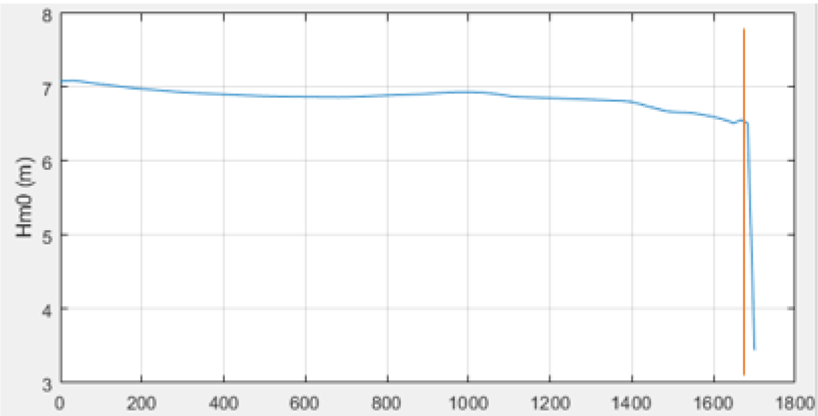


Figure 3.13: Output offshore-nearshore transformation using SwanOne

It is visible that the wave height has decreased by ca. 5 percent at the toe of the structure (indicated with the vertical line in Figure 3.13), which was estimated to be located at approximately 50 m from the breakwater crest. This is not strange, as some extent of breaking, either because of wave steepness or shoaling, is to be expected. What is strange, however, is that the wave height immediately starts to decrease whereas the bottom levels remain constant for the first stretch, and even increase after a while. With this atypical depth development towards the shore, an accurate input is required of e.g. how much wind is put on the simulation to obtain a realistic nearshore wave transformation, but no information is provided on this topic in prEN1991-1-8. Had this been done properly, it would probably have resulted in a slightly higher wave height at the toe.

[13] The distance of 1.7 km is in fact still quite far away, but this assumption has been made to simplify the case study

[14] An alternative would be to transfer the factored wave height nearshore, but in this thesis the considerations on depth limitation have been detached from the application of the partial factor.

Because of the uncertainty in the accuracy of the numerical wave transformation model, a consideration has also been made based on empirical wave breaking limits.

It has already been determined that the wave measurements at the point of data retrieval were not limited by depth. The expression used for this was the following:

$$H_{s,max} \approx 0.45 * d$$

If another look is now taken at Figure 3.7, the same consideration can be repeated for the toe of the structure. The bottom level of -13.7 m with respect to NAP, combined with a water level of 3.38 m+NAP, gives a depth of 17.08 m at the toe of the structure. The maximum significant wave height that is possible for this depth equals $H_{s,max}$ = 7.69 m. This is an important indicator to predict whether waves have started breaking yet at the toe of the structure. In Subsection 3.6.1, the characteristic wave height for a return period of 100 years was computed to be 7.03 m. This wave height is smaller than the limit of 7.69 m. It can therefore be concluded that, although being close to the limit, the waves are not limited by depth near the breakwater for the cross-section defined in Figure 1.3.

This strengthens the statement that the wave properties have not undergone large adjustments compared to the wave measurements when arriving at the toe of the structure. All in all, it is confirmed that the decrease of wave height towards the shore is insignificant for the purpose of this thesis.

The input that was given to the program SwanOne is laid out in Appendix G.

3.8 MAIN FINDINGS FROM CHAPTER 3

The most important characteristics of the design that was made in the chapter are summarised in Table 3.14:

Description	**Outcome**
Consequence class	CC2
Design service life	50 years
Primary limit states	SLS-(LD) and ULS
Design situation	Persistent design situation
Design approach	DA-1
Actions	Wave actions, water level
Material and product properties	Density of sea water, density of rock, density of concrete, notional permeability, roughness factor, friction factor
Geometrical parameters	Slope of 1:3 for rock-armour layer
	Slope of 1:1.5 for armour layer of artificial units
Data collection	Wave height, wave period, water level, tidal component, bathymetry
Degree of correlation	Moderate to strong
	NOTE: no joint probability analysis
Return period ULS	100 years
Return period SLS-(LD)	10 years
H_s (RP=100)	7.03 m
H_s (RP=10)	6.22 m
Relation H_s – Wave period	$T_m = 3.07 * \sqrt{H_s}$
Water level (RP=100)	4.11 m
Water level (RP=10)	3.38
Information on EVA of H_s and η	Weibull distributions, threshold values and event separation times assumed based on number of storms per year, CSE

Table 3.14: Summary of relevant aspects for continuation of the design

The shortcomings regarding the introduction of prEN1991-1-8 for the design conditions that were analysed for this location are listed below:

- A better structured method of describing the required design steps for determining the site specific boundary conditions would benefit the user of the draft Eurocode.
- The selection of the consequence class and design service lifetime was sensitive to the interpretation of the user, which should be resolved, for instance by incorporating more specific examples of coastal structures.
- The limit states related to overtopping are not clearly defined.
- It is strange that a HEA-level is introduced, but that you can still just go with DA-1 as it is the default approach. The relevance of the HEA-level was difficult to comprehend, which should be elaborated upon for it to be of added value.
- It is not clearly specified how to analyse the available data, both in terms of length as well as type of analysis (spectral vs. time-domain). It seems that the draft Eurocode demands the use of long data

records, but the uncertainty related to this is not covered in prEN1991-1-8. It is recommended to include more specific suggestions on this, e.g. a minimum length of 10 years. If this would be done, it should also be more elaborately described how to proceed when the required length of the data record is not available.

- Almost no information provided on how to deal with correlation. Several questions that are left unanswered (or open to the interpretation of the user):
  - How should the correlation be determined?
  - What are the boundaries when speaking of 'less than moderately' and 'moderately or strongly' correlated?
  - Should the wave height be correlated to the absolute water level? Or to the storm surge only?

For the case study of IJmuiden, the room for interpretation resulted in two possible ways of treating the dependence between the wave height and the water level.

A correlation coefficient (e.g. Pearson product-moment correlation coefficient) should be specified, specific boundaries should be given (e.g. a value higher than 0.4 indicates 'moderate to strong correlation') and the considered components need to be described (e.g. correlate wave height to storm surge only).

- The semi-probabilistic approach should be a fast way to achieve the desired reliability, but if you have to set up an entire joint probability distribution, one could just as well turn to a full probabilistic approach. Alternative options would be to also define a set of return periods for 'moderately to strongly' correlated variables, or to draw up a relation between the two correlated variables by applying linear regression to the data, and selecting the value of the dependent parameter based on the value of the dominant parameter.

- A certain degree of human interpretation was allowed for when selecting the return period as a designer, varying between 100 and 400 years, as a result of the poorly defined structure specifications.
- Poor description in the new Eurocode of how an Extreme Value Analysis on the wave height should be performed. Several questions that are left unanswered (or open to the interpretation of the user):
  - What do suitable threshold and event separation values mean?
  - On the basis of what criteria should an extreme distribution be selected?
  - Should the central statistical estimate be used, or should an upper limit of a certain confidence interval be used when extrapolating? And if so, shouldn't it be specified which confidence interval to use?

For the case study of IJmuiden, the room for interpretation resulted in variations of the design wave height of up to 10%, as will be demonstrated in Subsection 6.4.1.

The method of determining the threshold value should be specified (e.g. aim for 10 storms per year, or iterate to find the smallest confidence bounds for the distribution), adequate criteria of selection of the extreme distribution should be provided (e.g. base your choice on RMSE, on the method with which the data set of extreme values has been produced, or on typical distributions for the sea condition parameter of interest) and a specific confidence interval to be used needs to be defined.

- The same issues arise when performing an EVA on the water level. In addition, for the case study of IJmuiden, significantly varying water levels were found when treating the tide & surge in combination or separately. An actual value should be given that suggests the appropriate method (e.g. it is allowed to treat the two components separately when the magnitude of the tidal range divided by the magnitude of extreme surges is larger than 2).

- Other water level components have also been given too little attention in the draft Eurocode. The wave set-up is poorly described, and the instructions related to sea-level rise are vague. such as the wave set-up whi
- Very little information is provided on the relationship between wave height and wave period in prEN1991-1-8. It should either be recommended to compute the wave period by means of a consideration on the wave steepness, or by providing an expression that can be used to fit the combined data of wave period and wave height to.

4. DA-1 SEMI-PROBABILISTIC BREAKWATER DESIGN

This chapter focuses on the design of the breakwater following the DA-1 method described in the draft Eurocode. As it is a draft code with minor design experience, this chapter focuses on unclarities with regards to design choices that need to be made. The sections describe an armour layer with rock, concrete artificial units, the crest height, the cross-sectional design and the crown wall. Firstly, each section presents the design result that you would arrive at by following the draft Eurocode. Whenever a design choice needs to be made because prEN1991-1-8 does not give clarity, it will be explicitly stated what the choice is that will be proceeded with. Secondly, each section investigates other possible design outcomes if several aspects are interpreted differently, by means of a sensitivity analysis.

Before the design of the various breakwater elements is demonstrated, three major unclarities are firstly addressed that hold in general.

AMBIGUITY IN PR-EN1991-1-8

1) **Selection of return periods:**

In prEN1991-1-8, Table 4.3 and Tables A.4 & A.5 show a discrepancy regarding the return periods to be used. It can be interpreted as if Table 4.3 should be used in case of using only one variable, and Tables A.4 & A.5 when more than one variable is required. If the framework of EN1990 is followed, this discrepancy should not exist as the characteristic value (as described in Table 4.3) should have the same magnitude as the dominant variable in the combination of actions (as described in Tables A.4 & A.5).

It has been chosen to follow Tables A.4 & A.5, even when the response of the structure depends on one variable only, based on the following citation in Clause A.5.4: *'Structures designed using DA-1 shall follow this Annex, in particular Table A.1 and Clause A.6.'*

2) **The use of average or safe values:**

There are several design formulae, which already include a safety margin for the strength. It is also not unusual to base the design variable on an upper limit of a certain confidence interval, as the data set of load measurements is often limited. The latter is even stated in Tables A.4 & A.5 as a possibility. In the draft Eurocode it is not conclusively mentioned how to deal with this in DA-1. To assist in this choice, it is useful to compare three different design approaches, as described in Clause 4.5.1:

'DA-0: The applicable margin is related to the response formula used. Semi-empirical formulae commonly incorporate a safety margin (but this is often not explicitly stated). If no safety margin is stated for the formula employed (or a deterministic version of the formula not available) then a safety margin of one Standard Deviation (SD) above the CSE should be applied to the sea condition parameter. When the latter is not available, as typically in HEA-1, an estimate of SD can be assumed based on experience with the formula used. Further guidance for particular structures is given in Clauses 6 to 11.'

'DA-1: This Design Approach refers to the principal design format in the Eurocodes associated to the use of partial factors that cover the required safety margin for a range of design cases. Applicable values of those factors can be found in Annex B (and Appendix A6 of EN1990).'

'DA-2 (and DA-3): This Design Approach does not require the use of extra safety margin since the latter has been incorporated in the target reliability levels given in Table 13.1 (NDP).'

The most logical interpretation is that only DA-0 allows the user to add safety either via the response formula or the design variable. DA-1 should cover any uncertainties by the use of the partial factors, and DA-2 incorporates this in the target β-values.

3) Application of partial factors:

Table A.7 in prEN1991-1-8 and Table A.6.8 in the updated Annex A6, issued March 2021, of EN1990 show a discrepancy regarding the partial factors to be used in ULS. This value could either be 1.00 (according to Table A.8 in prEN1991-1-8, for hydraulic limit states) or 1.35 (according to Table A.6.8, see Appendix B). The latter has been chosen, as the guidance on the application of a partial factor on the wave height in the draft Eurocode is deemed insufficient.

A partial factor needs to be applied on the wave height. This may result in a design wave that can physically not exist because of e.g. depth-limitations. This thesis applies the partial factor on wave heights also if this may result in unrealistic wave heights[15].

Since prEN1991-1-8 focuses on actions and not on resistance, it is not clear what the partial resistance factor in the hydraulic limit states should be. A partial resistance factor is therefore not considered, and the lack of a description of how to deal with is a severe shortcoming of the draft Eurocode.

Clause A.7 mentions that *'values of partial factors for limit state functions of breakwater types can be found in PIANC Report No.* 196'. These partial factors will not be considered in this chapter, as it is not evident that these factors should indeed be used.

[15] The result of what would happen if this assumption would not be made is investigated for a rock armour layer in Subsection 6.4.3

4.1 ARMOUR LAYER – ROCK

This section deals with the design of the armour layer of the breakwater, assuming a rock-armoured slope. Both the Serviceability Limit State-(Limited Damage) (SLS-(LD)) as well as the Ultimate Limit State (ULS) will be checked.

In Subsection 2.3.1 it was explained that the Van der Meer-formula will be used for the design of the rock armour layer.

Box 5.13 in The Rock Manual [Ref. 6] sets out how the Van der Meer-formula should be applied. The steps that need to be undertaken to determine the required rock diameter of the armour layer are enlisted in Figure 4.1. Step 7 is a new step and is necessary because of the introduction of partial safety factors in the draft Eurocode.

Step 1	• Define wave conditions H_s and T_m at the toe of the structure
Step 2	• Define acceptable values of damage level parameter S_d
Step 3	• Determine number of waves N
Step 4	• Determine surf similarity parameter
Step 5	• Determine whether waves are plunging or surging
Step 6	• Compute stability number N_s
Step 7	• Compute the design wave height by multiplication of characteristic wave height with partial factor
Step 8	• Compute required rock size D_{n50}

Figure 4.1: Design steps for application of Van der Meer-formula

4.1.1 DESIGN STEPS

In this subsection, the elaboration of the steps in Figure 4.1 is presented.

STEP 1: DEFINE WAVE CONDITIONS H_S AND T_M AT THE TOE OF THE STRUCTURE

The hydraulic boundary conditions have been assessed in Chapter 3, and the outcome is repeated below in Table 4.1:

Limit state	Return Period [y]	H_s [m]	T_m [s]
SLS-(LD)	10	6.22	7.64
ULS	100	7.03	8.13

Table 4.1: Significant wave height and mean wave period at the breakwater toe

STEP 2: DEFINE ACCEPTABLE VALUES OF DAMAGE LEVEL PARAMETER S_D

The following values can be found in Table 7.1 (see Appendix A) of the draft Eurocode:

Sub-system	Damage parameter	Slope	SLS-(LD)	ULS
Two-layer armour	S_d	1:1.5-1:2	2	8
	S_d	1:3	2	12
	S_d	1:4-1:6	3	17
Single-layer armour	S_d	1:1.3-1:1.4	2	8

Table 4.2: Values of damage parameters for rock as specified in Table 7.1 in prEN1991-1-8

This means that for a two-layer armour with a slope of 1:3 **values of S_d of 2 and 12** should be used for SLS-(LD) and ULS, respectively.

When a look is taken at the Rock Manual [Ref. 6], one may notice that the same values can be found, with S_d = 2 belonging to 'start of damage', and S_d = 12 belonging to 'failure'. The Rock Manual, however, also specifies damage values for 'intermediate damage'. This again raises the question of the exact meaning of the limit state SLS-(LD), when compared to the other limit states.

STEP 3: DETERMINE NUMBER OF WAVES N

The number of waves attacking the breakwater slope during a storm needs to be determined. This number can be determined with the following expression:

$$N = \frac{storm\ duration\ D * 3600}{T_m}$$

It is not straightforward which value should be taken for the storm duration D. The draft Eurocode mentions a storm duration of 12 hours in the legend of Table A.2, though this table is not specifically intended for design purposes but for illustrating the equivalence of failure probabilities. Nevertheless, it has been decided to continue with a **storm duration of 12 hours**, as this is the only value mentioned in the draft Eurocode itself.

STEP 4: DETERMINE SURF SIMILARITY PARAMETER

The surf similarity parameter can be determined with the following expression:

$$\xi_m = \frac{\tan\alpha}{\sqrt{s_{0m}}}$$

In this expression, the subscript '0' indicates that the deep water wavelength should be used to calculate the wave steepness, and the subscript 'm' indicates that the mean wave period should be used for determining this deep water wavelength.

This leads to the following values for the surf similarity parameter:

Return Period [y]	H_s [m]	T_m [s]	s_{0m}	tan(α)	ξ_m
10	6.22	7.64	0.068	1/3	1.28
100	7.03	8.13	0.068	1/3	1.28

Table 4.3: Value of surf similarity parameter for two different return periods

We do not yet include the partial factor on H_s in this calculation. The draft Eurocode does not describe how to deal with this, but doing so would result in an unrealistic wave steepness.

STEP 5: DETERMINE WHETHER WAVES ARE PLUNGING OR SURGING

The average values for the empirical coefficients are c_{pl} = 6.2 and c_s = 1. Since prEN1991-1-8 does not provide clarity on the use of average or safe values, it has been assumed to work with the average values, as was explained in the beginning of the chapter. The other variables in the Van der Meer-equations, as given in Subsection 2.3.1, have already been defined.

This results in a value for the critical surf similarity parameter of ξ_{cr} = 3.01. From this value it can be concluded that the waves on the slope will be **plunging waves**, as the surf similarity parameter is smaller than this critical value.

STEP 6: COMPUTE STABILITY NUMBER N_S

Next, the magnitude of the stability number can be calculated, as all parameter values in the right-hand side of the Van der Meer equation for plunging waves are known. Average values will be used to compute the magnitude of the stability number. That is, the average values of the coefficient c_{pl} and the slope. This results in a **stability number of 2.25 for SLS-(LD) and 3.24 for ULS**.

STEP 7: COMPUTE THE DESIGN WAVE HEIGHT BY MULTIPLICATION OF CHARACTERISTIC WAVE HEIGHT WITH PARTIAL FACTOR

The second-to-last step is to compute the design wave height. It depends on the characteristic wave height as defined in step 1, and the magnitude of the partial factor. In SLS-(LD), the magnitude of the partial factors is set to 1. For ULS, partial factor values can be found in Table A.6.8 in EN1990 (see Appendix B). The partial load factor depends on the type of action and the design case. The type of action that is considered is waves. Design Case 1 applies to this problem, which refers to *'ultimate limit states that involve the hydraulic resistance of coastal structures loaded by waves and currents'*.

The partial load factor that should then be used equals 1.35. A partial resistance factor is not specified, or at least it is not self-evident that such a factor should indeed be used. The design wave height is presented in Table 4.4:

Limit state	Return Period [y]	H_s [m]	γ_{Qz}	$H_{s,d}$ [m]
SLS-(LD)	10	6.22	1.00	6.22
ULS	100	7.03	1.35	9.49

Table 4.4: Design wave height that should be applied for SLS-(LD) and ULS

It has been chosen[16] to only factor the wave height in the stability number, thus not in other expressions in the Van der Meer-formula where the wave height emerges, because this avoids the consideration of whether or not the wave period should be adjusted alongside the wave height.

[16] This is not described in the draft Eurocode; this is an assumption that seems logical.

STEP 8: COMPUTE REQUIRED ROCK SIZE D_{N50}

After the right-hand side has been calculated and the design wave height determined, one can rewrite the Van der Meer-equation and obtain the required rock size:

$$D_{n50} = \frac{H_{s,d}}{\Delta * N_s}$$

In this equation, N_s equals the stability number computed in step 6. The final results are shown in Table 4.5:

Limit state	SLS-(LD)	ULS
Return Period	10 [years]	100 [years]
Wave Height (H_s)	6.22 [m]	7.03 [m]
Wave Period (T_m)	7.64 [s]	8.13 [s]
Slope (tan α)	1:3	
Damage parameter (S_d)	2	12
Storm duration (D)	12 [h]	
Number of waves (N)	5654	5314
Notional Permeability (P)	0.4 [-]	
Density of Water (ρ_w)	1025 [kg/m^3]	
Density of Rock (ρ_s)	2650 [kg/m^3]	
Van der Meer-coefficient (c_{pl})	6.2 [-]	
Partial factor	1.00	1.35
Rock size (D_{n50})	1.74 [m]	1.85 [m]
Median rock mass (M_{50})	14.0 [t]	16.7 [t]

Table 4.5: Design results following DA-1 for rock-armoured slope in SLS-(LD) and ULS

4.1.2 SENSITIVITY ANALYSIS

Several design choices have been made in the previous subsection to arrive at the design outcome as presented in Table 4.5. Nevertheless, there are certain aspects that could have been interpreted differently, which would have resulted in alternative design outcomes. The most important of these aspects are listed below:

- The selection of return periods as a function of consequence class and T_{life}
- The choice of the extreme wave distribution function
- The use of a certain non-exceedance value for the extrapolation of wave heights
- The storm duration that has been opted to calculate with
- The wave period that has been reasoned to accompany the wave height
- The coefficient in the response formula that can be altered as a way of incorporating safety on the resistance side, as there is no partial resistance factor
- The application of a partial factor to the wave height in other expressions apart from the stability number
- The way the discrepancies in Tables 4.3/A.4&A.5 and Tables A.6.8/A.7 are interpreted

In the tables that follow, it is shown how these aspects influence the design. Here, only the deviating parameter is shown together with its influence on the outcome of the design for ULS, whilst keeping the other parameters equal to their values as determined in Subsection 4.1.1.

Return Period [y]	H_s [m]	D_{n50} [m]	%ΔD_{n50}
100	7.03	1.85	-
200	7.26	1.90	2.7
400	7.48	1.96	5.9

Table 4.6: Influence of selection of return period on design

Distribution	RP [y]	H_s [m]	D_{n50} [m]	%ΔD_{n50}
Generalised Pareto	100	6.59	1.74	-
Weibull	100	7.03	1.85	6.3
Gumbel	100	7.20	1.89	8.6

Table 4.7: Influence of use of various distributions on design

Non-EV [%]	RP [y]	H_s [m]	D_{n50} [m]	%ΔD_{n50}
50	100	7.03	1.85	-
84	100	7.26	1.90	2.7
97.5	100	7.50	1.96	5.9

Table 4.8: Impact of using certain non-exceedance values for the wave height, linked to confidence intervals of the wave height exceedance curves

D	RP	H_s	N	D_{n50}	%ΔD
6 h	100	7.03	2657	1.72	-
12 h	100	7.03	5314	1.85	7.6

Table 4.9: Effect of storm duration choice

T_{m02} [s]	RP [y]	H_s [m]	s_{0m}	ξ_m	D_{n50} [m]	$\%\Delta D_{n50}$
8.13	100	7.03	0.068	1.28	1.85	-
8.54	100	7.03	0.062	1.34	1.88	1.6
8.94	100	7.03	0.056	1.40	1.92	3.8

Table 4.10: Influence on design when using larger wave period

Coefficient C_{pl}	RP [y]	H_s [m]	Stability number	D_{n50} [m]	$\%\Delta D_{n50}$
6.2	100	7.03	3.24	1.85	-
5.8	100	7.03	3.03	1.97	6.5
5.5	100	7.03	2.88	2.08	12.4

Table 4.11: Influence of using different values for Van der Meer-coefficient

Situation	RP [y]	H_s [m]	T_m [s]	s_{0m}	D_{n50} [m]	$\%\Delta D_{n50}$
H_s factored in all expressions; wave period not adjusted	100	7.03	8.13	0.092	1.71	-
H_s factored in all expressions; wave period adjusted to keep constant wave steepness	100	7.03	9.44	0.068	1.82	6.4
H_s factored in stability number only	100	7.03	8.13	0.068	1.85	8.2

Table 4.12: Influence on design of factoring the wave height in different ways

Return Period [y]	H_s [m]	T_m [s]	Partial Factor	D_{n50} [m]	$\%\Delta D_{n50}$
100	7.03	8.13	1.0	1.37	-
400	7.48	8.38	1.0	1.45	5.8
100	7.03	8.13	1.35	1.85	35.0
400	7.48	8.38	1.35	1.96	43.1

Table 4.13: Influence on design as a result of inconsistencies in various draft Eurocode Tables

The full set of parameters used for these calculations, along with the SLS-(LD) calculations and further explanatory tests, can be found in Appendix H.

The Table below shows the possible variation in the design outcome for a rock armour layer, as a result of different interpretations:

	Alternative 1	**Alternative 2**
Limit state	ULS	ULS
Return Period	100 [y]	400 [y]
Distribution	Generalised Pareto	Gumbel
Non-exceedance value	50 [%]	97.5 [%]
Wave height H_s	6.59 [m]	8.27 [m]
Wave period T_m	7.87 [s]	8.81 [s]
Empirical coefficient c_{pl}	6.2	5.5
Storm duration D	6 [h]	12 [h]
Partial factor	1.00	1.35
Nominal rock diameter D_{n50}	1.20 [m]	2.90 [m]
Median rock mass M_{50}	4.6 [t]	37.9 [t]

Table 4.14: Extremely optimistic design interpretations (left) vs. Extremely conservative design interpretations (right), following from the sensitivity analysis

This is of course an extreme example, but it goes to show that there is still a large degree of ambiguity in the draft Eurocode that needs to be addressed.

4.2 ARMOUR LAYER – ARTIFICIAL UNITS

This section deals with the design of the armour layer of the breakwater, assuming a slope armoured with artificial units. The stability formula for the design of artificial units has been explained in Subsection 2.3.2, and contains much fewer parameters than the ones for rock design. This also means that fewer steps need to be undertaken in the design. Figure 4.2 displays the necessary design steps:

Step 1	•Define wave condition H_s at the toe of the structure
Step 2	•Define acceptable values of fixed damage level parameter $N_{s,d}$
Step 3	•Compute the design wave height by multiplication of characteristic wave height with partial factor
Step 4	•Compute required size of artificial units D_n

Figure 4.2: Design steps for application of stability formula for artificial units

4.2.1 DESIGN STEPS

In this subsection, the elaboration of the steps in Figure 4.2 is presented.

STEP 1: DEFINE WAVE CONDITIONS H_S AT THE TOE OF THE STRUCTURE

The same values as in Table 4.1 apply (see step 1 in Subsection 4.1.1). For the design of artificial units, the wave period is not of interest (for most current design formulas for artificial single layer units).

STEP 2: DEFINE ACCEPTABLE VALUES OF FIXED DAMAGE LEVEL PARAMETER $N_{S,D}$

The draft Eurocode mentions the following damage parameter values for Accropodes:

Sub-system	**Damage parameter**	**Slope**	**SLS-(LD)**	**ULS**
Single-layer armour	N_{od}	1:1.33	0	0.5

Table 4.15: Value of damage parameter for Accropodes according to Table 7.1 in draft Eurocode

The damage parameter N_{od} is defined, but it is the damage parameter $N_{s,d}$ that needs to be inserted into the formula. According to the Rock Manual [Ref. 6], the value of N_{od} = 0 corresponds to the start of damage and is equivalent to an acceptable **stability number of 3.7**. The value of N_{od} > 0.5 corresponds to failure and is equivalent to an acceptable **stability number of 4.1**. The draft Eurocode specifies that artificial units should also be checked at SLS-(LD), but what this means (i.e. how this can be observed) remains unclear.

STEP 3: COMPUTE THE DESIGN WAVE HEIGHT BY MULTIPLICATION OF CHARACTERISTIC WAVE HEIGHT WITH PARTIAL FACTOR

The same values as in Table 4.4 apply (see step 7 in Subsection 4.1.1).

STEP 4: COMPUTE REQUIRED SIZE OF ARTIFICIAL UNITS D_N

One can rewrite the equation in Subsection 2.3.2 and obtain the required artificial unit size:

$$D_n = \frac{H_{s,d}}{\Delta N_{s,d}}$$

The final results are shown in Table 4.16:

Limit state	**SLS-(LD)**	**ULS**
Return Period	10 [years]	100 [years]
Wave Height (H_s)	6.22 [m]	7.03 [m]
Slope (tan α)	1:1.5	
Unit type	Accropode	
Stability number ($H_s/\Delta D_n$)	3.7	4.1
Density of Water (ρ_w)	1025 [kg/m^3]	
Density of Concrete (ρ_c)	2400 [kg/m^3]	
Partial factor	1.00	1.35
Unit size (D_n)	1.25 [m]	1.73 [m]
Unit volume (V)	2.0 [m^3]	5.2 [m^3]

Table 4.16: Design results following DA-1 for concrete unit armour layer in SLS-(LD) and ULS

4.2.2 SENSITIVITY ANALYSIS

The uncertainties related to the wave height discussed in Subsection 4.1.2 also hold for the design of artificial units, but they will not be treated again. The uncertainty related to the wave period is irrelevant for the conceptual design of artificial units, as this parameter does not occur in the formula. The most important aspect that is unclear and may lead to an alternative design outcome, is the treatment of the fixed damage level parameter $N_{s,d}$. Even though it was agreed upon to not use a safety margin in the response formula in the DA-1 format, for this particular formula this choice seems questionable, given the statement in §5.2.2.3 of the Rock Manual [Ref. 6]:

'Note that these are empirical data based on model tests – thus not meant for design without first applying a safety factor.'

Below it is investigated how this alternative, i.e. incorporating a safety factor into the formula, leads to a different design. The design outcome is shown for ULS only.

Stability number N_s	**RP [y]**	**H_s [m]**	**Partial Factor**	**D_{n50} [m]**	**%ΔD_{n50}**
4.1	100	7.03	1.35	1.73	-
2.7	100	7.03	1.35	2.62	51.4

Table 4.17: Influence of safety margin in the response formula for concrete armour design

For the same consideration in SLS-(LD) and the full set of parameters used in the calculations, reference is made to Appendix I.

4.3 CREST HEIGHT

This section deals with the design of the crest height of the breakwater. The crest height is determined for the armour layer of artificial units. The overtopping formula for the calculation of the minimum required crest height is presented in Subsection 2.3.3. In Figure 4.3, the design steps that should be undertaken are set out:

Step 1	• Define wave condition H_{m0} and water level η at the toe of the structure
Step 2	• Determine the magnitude of the coefficients in the overtopping formula
Step 3	• Define tolerable overtopping discharge q_{tol}
Step 4	• Compute the design wave height and water level by multiplication of characteristic values with partial factor
Step 5	• Compute the minimum required crest height

Figure 4.3: Design steps for application of overtopping formula

4.3.1 DESIGN STEPS

In this subsection, the elaboration of the steps in Figure 4.3 is presented.

STEP 1: DEFINE WAVE CONDITION H_{M0} AND WATER LEVEL H AT THE TOE OF THE STRUCTURE

We established in Subsection 3.2.1 that the relevant limit states for the crest height are not clearly defined in prEN1991-1-8. Both SLS and SLS-(LD) will be considered. A return period of 10 years is used for SLS-(LD), but a return period is not specified for SLS. Based on common sense, a return period of 1 year will be assumed. Under the assumption of full correlation (conservative) between wave height and water level, this yields the values in Table 4.18:

Limit state	Return Period [y]	H_{m0} [m]	η [m+NAP]
SLS	1	5.26	2.70
SLS-(LD)	10	6.22	3.38

Table 4.18: Significant wave height and absolute water level at the breakwater toe

STEP 2: DETERMINE THE MAGNITUDE OF THE COEFFICIENTS IN THE OVERTOPPING FORMULA

In the overtopping formula, there are two empirical coefficients with average values c_1 = 0.09 and c_2 = 1. In the DA-1 format, the average values of these coefficients will be used. Other coefficients are the influence factor on obliqueness γ_β, which is set to 1 because of the assumption of perpendicular wave attack, and the roughness factor γ_f, which has a value of 0.46 for concrete elements as already presented in Table 3.2.

STEP 3: DEFINE TOLERABLE OVERTOPPING DISCHARGE Q_{TOL}

There are two limit states to be considered, and therefore also two different tolerable overtopping discharges that might apply. The draft Eurocode, in Clause 7.4.3, refers to the wave overtopping manual [Ref. 11] for the appropriate values:

'Threshold overtopping values are given in the EurOtop.'

In this manual, three tables are presented that discuss tolerable discharges with respect to structural design, property and people, respectively. These tables are shown in Appendix J.

Operational functioning of the breakwater coincides with the serviceability limit state. There are no ships moored behind the breakwater, so the overtopping limits with respect to property are irrelevant. The breakwater is accessible to the public, but this is not the main function so entrance can simply be prohibited in case of storm conditions. This is actually the case for the current breakwaters. Overtopping limits with respect to people are therefore not considered to be relevant either. The main function of the breakwater relates to limiting the wave height behind the breakwater to guarantee safe passage of vessels. No such limits are described in the overtopping manual, but an estimate can be made based on the current situation. At present, severe overtopping is allowed for, which makes that a high tolerable overtopping discharge of **q_{tol} = 100 l/s/m is assumed for SLS**.

For the serviceability limit state with limited damage, we have to consider the structural resistance of the breakwaters. The overtopping volumes may cause damage to the rear side. Considering this aspect, the discharge q may approximately be 5 to 10 l/s/m, given that the rear side has been designed for wave overtopping. It is not clear whether these values relate to no damage at all, or whether some damage already occurs. An even less strict tolerable overtopping discharge is therefore adopted of **q_{tol} = 20 l/s/m for SLS-(LD).**

STEP 4: COMPUTE THE DESIGN WAVE HEIGHT AND WATER LEVEL BY MULTIPLICATION OF CHARACTERISTIC VALUES WITH PARTIAL FACTOR

This step has been included in the design process as it is an essential concept in the Eurocode. However, partial factors are only specified for ultimate limit states, for serviceability limit states these factors are equal to 1. Since overtopping is not checked for at ULS, the design wave height and water level will simply be the same as their characteristic values.

STEP 5: COMPUTE THE MINIMUM REQUIRED CREST HEIGHT

The minimum crest height that is required can now be calculated by rewriting and filling in the overtopping formula as given in Subsection 2.3.3, using the tolerable overtopping discharges as specified in step 3. The crest height is implicitly part of the freeboard R_c, which is equal to the crest height minus the water level.

The result is displayed in Table 4.19:

Limit state	**SLS**	**SLS-(LD)**
Return Period H_s	1 [year]	10 [years]
Wave Height (H_s)	5.26 [m]	6.22 [m]
Return Period η	1 [year]	10 [years]
Water Level (η)	2.70 [m+NAP]	3.38 [m+NAP]
Slope (tan α)	1:1.5	

Unit type	Accropode	
Empirical coefficients (c_1 and c_2)	0.09 and 1.5	
Roughness factor (γ_f)	0.46	
Obliqueness factor (γ_β)	1.0	
Tolerable overtopping discharge (q_{tol})	0.100 [m^3/s per m]	0.020 [m^3/s per m]
Partial factor	1.00	1.00
Freeboard (R_c)	4.26 [m]	6.97 [m]
Calculated overtopping discharge (q)	0.992 [m^3/s per m]	0.0199 [m^3/s per m]
Crest height (A)	6.96 [m+NAP]	10.35 [m+NAP]

Table 4.19: Design results following DA-1 for crest height in SLS and SLS-(LD)

4.3.1 SENSITIVITY ANALYSIS

Alternative design outcomes will be shown for SLS-(LD) only, as it is by far the normative limit state. The most important aspect to consider is, just as was the case for artificial units, the use of the response formula. In EurOtop Clause 6.3.1 [Ref. 11], the following is stated: *'For a design and assessment approach it is strongly recommended to increase the average discharge by about one standard deviation.'*

Another reason why it would make sense to incorporate this safety margin in this case, is that the crest height is only designed for at serviceability limit states. Partial factors should make up for the absence of a safety margin in the response formula, but these are only used in ultimate limit states. Table 4.20 shows how the crest height would change if this alternative design choice were to be made:

Parameter	**Symbol**	**Value**	**Value**
Tolerable overtopping discharge	q_{tol}	0.020 [m^3/s per m]	0.020 [m^3/s per m]
Overtopping coefficients	c_1 and c_2	0.09 and 1.5	0.1035 and 1.35
Return period wave height and water level	RP	10 [y]	10 [y]
Wave height	H_s	6.22 [m]	6.22 [m]
Water level	η	3.38 [m+NAP]	3.38 [m+NAP]
Influence factors roughness and obliqueness	γ_f and γ_β	0.46 and 1.0	0.46 and 1.0
Freeboard	R_c	6.97 m	7.90 m
Crest height	A	10.35 m+NAP	11.28 [m+NAP]

Table 4.20: Alternative crest height design when adding safety to the response formula

Other alternative design outcomes, which deal with the choice of the tolerable overtopping discharge and the degree of correlation, are elaborated upon in Appendix J.

4.4 CROSS-SECTIONAL DESIGN

In order to proceed with the design of the crown wall, it is necessary to make a cross-sectional design. The cross-sectional design mainly depends on geometrical considerations and not only on hydraulic forcing. As the draft Eurocode is primarily intended to deal with the latter, it does not specify design recommendations for many of the elements that follow in this section. Therefore, it was frequently necessary to consult other documents. These documents will be referenced in the corresponding subsections. The eventual cross-sectional design will give insight into geometrical properties of the breakwater.

4.4.1 ARMOUR LAYER

In Section 4.1 and 4.2 the required diameter of armour rock and concrete armour units have been determined, respectively. For the rock armour layer, a value of d_{n50} = 1.85 m (i.e. M_{50} = 16.7t) was computed. If one consults Table A-2 in Bed, Bank & Shore Protection [Ref. 9], you can find a maximum rock size of d_{n50} = 1.44 m. Although it was very useful to investigate how different choices led to various diameters for the armour rock, the eventual result is apparently a d_{n50} that is too large to belong to a standard grading.

Hence, the design will be continued with concrete armour units. For Accropodes, standard unit sizes are collected from its Design Guide Table [Ref. 32]. The first page of this design guide has been added to Appendix K. An armour unit size of 1.73 m was computed. If you round this up to the nearest unit standard size you will find **Accropodes with D_n = 1.82 m (i.e. V = 6 m^3)**

4.4.2 ARMOUR LAYER THICKNESS

For the armour layer thickness, use can be made of the following expression, found in Clause 10.2.2 of the Breakwater Design Lecture Notes:

$$t = n * k_t * d_{n50}$$

In this expression, n is the number of stones in the layer, which equals 1 for single-layer elements like Accropodes. The layer coefficient k_t for Accropodes is equal to 1.29. Together with the nominal diameter of 1.82 m, this gives a **layer thickness of 2.35 m**.

4.4.3 UNDER-LAYER

From Clause 7.4.8 in the draft Eurocode:

'Empirical formulae for the gradation of filters and the relative stone sizes between adjacent layers including the foundation soil, based on physical tests and prototype observations, should be used.'

'Further guidance on filter layers can be found in the Coastal Engineering Manual VI.5-3 and the Rock Manual § 5.2.2.10.'

In the specified paragraph in the Rock Manual [Ref. 6], you will find the following relationship:

$$\frac{M_{50u}}{M_{50a}} = \frac{1}{10} to \frac{1}{15}$$

In Table 5.36 in the Rock Manual you can also find the following relation:

$$M_{50u} = 0.1 * M_a$$

The latter expression translates to a d_{n50} ratio of approximately 2, meaning that for the first under-layer the following calculation can be made: d_{n50} = 1.82 m / 2 = 0.91 m. In the design, it is then opted to use the standard stone grading **HM_A 1000-3000, which has a d_{n50} of 0.90 m**.

4.4.4 UNDER-LAYER THICKNESS

In order to compute the thickness of the under-layer, Bed, Bank & Shore Protection [Ref. 9] has been consulted. In this book, Table A-2 deals with standard rock gradings and their properties. The table mentions a typical layer thickness of 1.5 * d_{n50}[17]. For the stone class HM_A 1000-3000, the **under-layer thickness will then equal 1.5 * 0.9 m = 1.35 m**.

4.4.5 CORE

It is assumed that the weight ratio of under-layer and core will approximately be 1/15. This results (using ρ_s = 2650 kg/m^3) in a d_{n50} for the core of 0.365 m. The material that is used for the core will then be LM_A 40-200[18], which has **a d_{n50} of 0.34 m**.

4.4.6 CREST HEIGHT

From Section 4.3 a crest height of 10.35 m+NAP was determined. The crest height here means the top of the crown wall. However, this crest height was a minimum crest height based on the overtopping criterium. Another consideration has to be made as well, which arises because of the desire to include a crown wall in the design. As the wave height is fairly large, it may very well be that the base level of the crown wall (and thus the entire height of the breakwater) needs to be raised in order to obtain a stable solution. This is investigated in Appendix L. The **crest has eventually been set at a height of 10.5 m+NAP**.

4.4.7 CREST WIDTH

From the Breakwater Design Lecture Notes, it is found that the width of the crest must be at least 3 armour units:

$$B = n * k_t * d_{n50} = 3 * 1.29 * 1.82 = 7.04\ m$$

This is a minimum, other considerations come into play as well. Firstly, it is necessary to keep some space in front of the crown wall. In the Accropode Design Guide Table one can find that this should preferably be at least 2.8 * D_n = 5.1 m. Therefore, a space of 5 m will be reserved in front of the crown wall, which translates to approximately 4 m at the base level of the crown wall.

Furthermore, space is required for the bottom slab of the crown wall. In Appendix L it is explained that this width will take on a value of 4.5 m. Finally, some space behind the crown wall should be provided in order to prevent geotechnical failure, which is estimated to be 1.5 m. The **total width of the crest will then equal 10 m**.

[17] This multiplication is a minimum, to achieve this minimum under layer-thickness, a layer thickness of 2d_{n50} should actually be applied. This was only discovered after the crown wall design had been finished, and it would have been too time-consuming to adjust this. That's why this value has been stuck to, the principles of the crown wall design do not change because of this different value.
[18] Such a large gradation would in reality probably not be implemented for the core, because of the high costs. Nevertheless, for the purpose of this thesis it is assumed to be okay.

4.4.8 TOE DIMENSIONS

Guidance is given on actions on the seaward toe in prEN1991-1-8 in Clause E.3.6:

'The stability of the seaward toe under wave action may be estimated through the Van der Meer et al. formula 1995. Design issues including toe berms in shallow water and gently sloping foreshores are dealt with in The Rock Manual, §5.2.2.9.'

The equation that has been selected to do the calculations with is the following:

$$\frac{H_s}{\Delta d_{n50}} = \left(6.2\frac{h_t}{h} + 2\right) N_{od}^{0.15} \quad if \quad 0.4 < h_t/h < 0.9$$

The damage level parameter N_{od} equals 0.5 for SLS-(LD) and 4.0 for ULS, according to Table 7.1 in prEN1991-1-8.

The draft Eurocode mentions in Clause 7.4.2 that *'in case of non-depth-limited waves the most critical situation will generally be associated with low water levels'*.

However, no guidance is given in the draft Eurocode on how to determine these low water levels. Clause C.1.2 in prEN1991-1-8 states the following:

'For a certain type of structures such as retaining walls, exceptionally low water at the ebb of tsunami can cause its seaward collapse owing to the earth and residual water pressures behind it. When scour of the seabed in front of a structure is apprehended, a low water level can become a critical condition. Impulsive breaking wave pressures can be exerted on a vertical or composite breakwater when the water level is intermediate or low, depending on the geometry of the breakwater. However, most cases of structural designs set the design water level at a rather high elevation.'

Hence, the draft Eurocode acknowledges the importance of low water levels, but it does not specify how to determine them. An assumption has therefore been made, combining the 10-year wave height for SLS-(LD) or the 100-year wave height (multiplied with the partial factor) for ULS with the Mean Low Water, which has a value of -0.69 m+NAP (see Figure F.3). The sea-level rise has not been included in the water level values, as the toe should also be stable at the start of its lifetime.

These simplified calculations pointed out that ULS is the leading limit state for the toe design. When a toe height of 2.5 m is assumed, this yields a d_{n50} of 0.69 m. **The standard rock grading HM_A 1000-3000 has been selected for the toe, which has a d_{n50} of 0.90 m.** This nominal rock diameter is then also in line with the assumed toe height of 2.5 m, since a toe typically has a thickness of 2-3 d_{n50}. The width of the toe will equal 3.5 m, such that it abides by a typical width of 3-5 d_{n50}.

4.4.9 DRAWING CROSS-SECTIONAL DESIGN

The cross-sectional design is sketched in Figure 4.4:

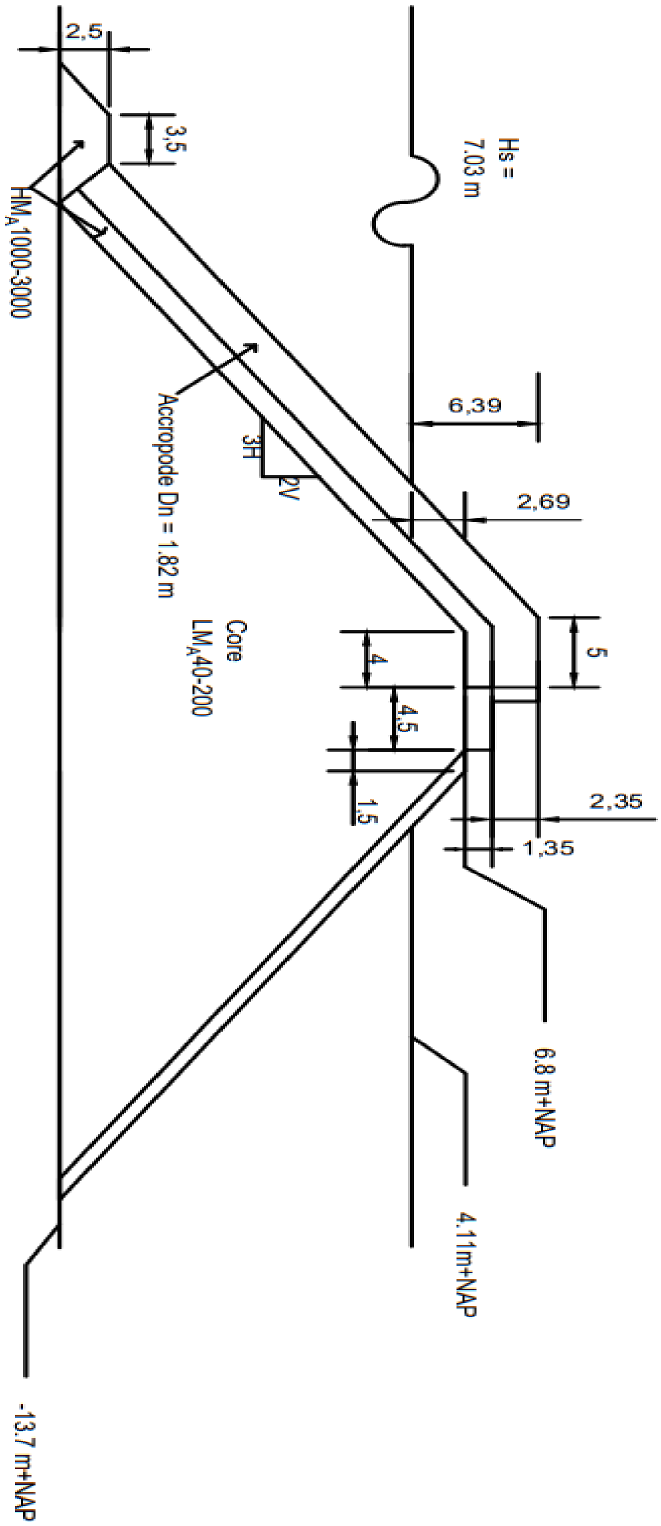


Figure 4.4: Drawing of cross-sectional design resulting from the considerations in Chapter 4 (breakwater dimensions in [m], RP of water level and wave height of 100 [y])

4.5 CROWN WALL

This section deals with the design of the crown wall of the breakwater. The wave forces acting on the crown wall are based on overtopping rates. The corresponding formulae were stated in Subsection 2.3.4. The parameter that will be designed for is the thickness of the crown wall bottom slab. Given the forcing on the crown wall, it should be able to resist against sliding and overturning. Figure 4.5 lists the necessary design steps:

Step 1	• Define wave conditions H_{m0} and T_p and water level η at the toe of the structure
Step 2	• Define the geometrical parameters in the formulae
Step 3	• Compute the design wave height and water level by multiplication of characteristic values with partial factor
Step 4	• Calculate the hydraulic parameters in the formulae
Step 5	• Translate the hydraulic parameters to forces and moments on the crown wall
Step 6	• Determine the forces and moments as a result of the self-weight of the crown wall
Step 7	• Compute the design values of these forces and moments by multiplication with partial factor
Step 8	• Test the crown wall against sliding and overturning

Figure 4.5: Design steps for determining the crown wall dimensions

4.5.1 DESIGN STEPS

In this subsection, the elaboration of the steps in Figure 4.5 is presented.

STEP 1: DEFINE WAVE CONDITIONS H_{M0} AND T_P AND WATER LEVEL H AT THE TOE OF THE STRUCTURE

The governing limit state for the design of the crown wall is the ultimate limit state. A return period of 100 years is used for ULS. Under the assumption of full correlation (conservative) between wave height and water level, this yields the values in Table 4.21. The wave period to be used in the formulae is the peak period, but no data is available on this. However, in Clause C.2.1.4 in the draft Eurocode, the following two relations are mentioned: $T_{1/3} \cong 1.2T_m$ and $T_{1/3} \cong 0.9T_p$. Combining these expressions gives an expression with which the peak wave period can be estimated given the mean wave period: $T_p \cong \frac{1.2}{0.9}T_m$. These estimates have been included in Table 4.21 as well.

Limit state	Return Period [y]	H_{m0} [m]	η [m+NAP]	T_p [s]
ULS	100	7.03	4.11	10.84

Table 4.21: Significant wave height, absolute water level and peak wave period at the breakwater toe

STEP 2: DEFINE THE GEOMETRICAL PARAMETERS IN THE FORMULAE

From Figure 4.4, the following geometrical parameters can be defined:

Symbol	Description	Value	Unit
A	Crest height	10.5	m+NAP
A_{base}	Level on which the crown wall base rests	6.8	m+NAP
C_h	Crown wall height	3.7	m
C_b	Crown wall width	4.5	m

Table 4.22: Geometrical parameters to be used in crown wall force equations

STEP 3: COMPUTE THE DESIGN WAVE HEIGHT AND WATER LEVEL BY MULTIPLICATION OF CHARACTERISTIC VALUES WITH PARTIAL FACTOR

This is the step where it gets interesting. Clause 7.4.7 in prEN1991-1-8 describes the design of crown walls, and frequently refers to loads and pressures. Hence, it is plausible that the action in this case should not be the wave height, but the force. This is compatible with the standard approach in EN1990. However, it is not explicitly stated in the draft Eurocode that this should indeed be done. When seeking for guidance on the application of partial factors, Table A.6.8 of the updated Annex A.6 (which will be added as a supplement to EN1990) has been consulted.

Another look is taken at this Table A.6.8 (see Appendix B) to determine the magnitude of the partial factors. The crown wall is designed to resist against sliding and overturning. Instead of Design Case 1, it might very well be that Design Case 2 then holds, as it *'is typically used for the combined verification of strength and static equilibrium'*. On the other hand, DC2 is applied *'when the structure is sensitive to variations in permanent action arising from a single-source',* and no further explanations are given of when this situation applies to a structure. For the continuation of the calculation it is irrelevant whether DC1 or DC2 holds, as in both cases the wave height should be factored with 1.35 and the water level is not factored. This results in **a design wave height $H_{s,d}$ of 9.49 m and a design water level η_d of 4.11 m+NAP**.

STEP 4: CALCULATE THE HYDRAULIC PARAMETERS IN THE FORMULAE

The hydraulic parameters [Ref. 22] are the following:

- Breaker parameter based on peak period: $\xi_{0p} = \frac{\tan\alpha}{\sqrt{s_{0p}}}$ $with\ s_{0p} = \frac{H_s}{L_{0p}}$ $and\ L_{0p} = \frac{9{,}81 * T_p^2}{2\pi}$
- Dimensionless wave overtopping discharge: $Q = 0.09\ exp\left[-\left(1.5\frac{R_c}{H_{s,d}\gamma_f\gamma_\beta}\right)^{1.3}\right]$
- Wall protection ratio: $\frac{R_c - A_c}{C_h}$
- Relative foundation elevation: $\frac{F_c}{L_{0p}}$

It has been chosen to only use the factored wave height in the expression for the dimensionless wave overtopping discharge. The other expression in which the wave height occurs is the fictitious wave steepness, but factoring this wave height would lead to a distorted ratio with the wavelength. Table 4.23 displays their values:

Variable	Value
L_{0p}	183.5 [m]
s_{0p}	0.038
ξ_{0p}	3.4
Q	0.00558
$(R_c - A_c) / C_h$	0
F_c / L_{0p}	0.015

Table 4.23: Values of hydraulic parameters to be used in crown wall force equations

STEP 5: TRANSLATE THE HYDRAULIC PARAMETERS TO FORCES AND MOMENTS ON THE CROWN WALL

Reference is made to the equations presented in Subsection 2.3.4 for the formulae that can be used to translate the hydraulic parameters to forces and moments on the crown wall.

Obtaining the horizontal force, the up-lift pressure at the outer base corner and the horizontal moment around the inner base corner is just a matter of filling in the formulae with the values presented in Tables 4.22 and 4.23.

The up-lift pressure is translated to an uplift force through the expression $F_u = 0.5 * PbF_{h0.1\%} * C_b$, assuming a triangular pressure distribution. The moment as a result of this is determined by multiplying F_u with 2/3 of the crown wall width. The results are displayed below in Table 4.24:

Description	Notation	Value	Unit
Dimensionless horizontal force	$\frac{Fh_{0.1\%}}{(0.5\rho_w g C_h^2)}$	1.69	-
Horizontal force exceeded by 0.1% of the waves	$Fh_{0.1\%}$	116.6	kN
Dimensionless up-lift pressure	$\frac{PbF_{h0.1\%}}{(0.5\rho_w g C_h)}$	0.165	-
Up-lift pressure simultaneous with $F_{h0.1\%}$	$PbF_{h0.1\%}$	3.07	kN/m
Up-lift force	F_u	6.91	kN
Dimensionless overturning moment	$\frac{Mh_{(Fh0.1\%)}}{(\rho_w g C_h^3)}$	0.674	-
Overturning moment simultaneous with $F_{h0.1\%}$	$Mh_{(Fh0.1\%)}$	343.5	kNm
Overturning moment due to up-lift force	M_u	20.7	kNm

Table 4.24: Forces and moments acting on the crown wall

A point of discussion is that the analytical model that we are working with in this section for the design of the crown wall (based on overtopping rates) specifies to work with a value of $F_{h0.1\%}$. In Section 4.1, a storm duration of 6 to 12 hours was assumed. With a wave period of approximately 10 seconds, this would result in 2000 to 4000 waves, and hence there are 2 to 4 waves per storm higher than this value. It is questionable whether $F_{h0.1\%}$ is the appropriate characteristic value for the force. The draft Eurocode should mention this issue and specify how to deal with this, for example by stating that the forces are calculated with your maximum (depth-limited) wave height.

Anyway, the calculations will be proceeded with the characteristic force $F_{h0.1\%}$ as proposed by the empirical model.

STEP 6: DETERMINE THE FORCES AND MOMENTS AS A RESULT OF THE SELF-WEIGHT OF THE CROWN WALL

The self-weight of the crown wall is the stabilising force. This downward acting force can be found by multiplying the mass of the crown wall with the gravitational acceleration:

$$F_G = M_{cw} * g \quad with \quad M_{cw} = V_{cw} * \rho_c \quad and \quad V_{cw} = t_1 * C_b + (C_h - t_1) * t_2$$

The dimensioning of the crown wall is sketched in Figure 4.6:

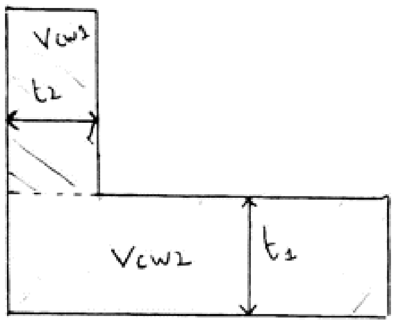


Figure 4.6: Symbols to be used in crown wall dimensioning

The thickness of the vertical wall face (t_2) will equal 1 m, whereas the thickness of the bottom slab (t_1) may vary in order to reach a stable solution. The thickness t_2 will equal 1.39 m. **The vertical force as a result of the self-weight then equals F_G = 201.7 kN.**

The stabilising moment due to the self-weight of the crown wall has been computed by splitting up the crown wall in two separate volumes and multiplying the resulting forces by the distance between their respective centres of gravity and the outer corner base. The expression to do so is shown here:

$$M_G = \rho_c * g * \left(V_{cw1} * (C_b - t_2 + 0.5 * t_2) + V_{cw2} * \left(\frac{C_b}{2}\right)\right)$$

The stabilising moment as a result of the self-weight then equals M_G = 548.9 kNm.

STEP 7: COMPUTE THE DESIGN VALUES OF THESE FORCES AND MOMENTS BY MULTIPLICATION WITH PARTIAL FACTOR

Another look is taken at Table A.6.8 (see Appendix B) to determine the magnitude of the partial factors. When either Design Case 1 or 2 holds, it is stated that the effects of the actions, i.e. the forces and moments, should not be factored, since we have already factored H_S. To the self-weight, which is a favourable force, a partial factor of 1 should be applied. This results in the following design values:

Force/Moment	Design value	Unit
$F_{H,d}$	116.6	kN
$F_{U,d}$	6.91	kN
$F_{G,d}$	201.7	kN
$M_{H,d}$	343.5	kNm
$M_{U,d}$	20.7	kNm
$M_{G,d}$	548.9	kNm

Table 4.25: Design values for forces and moments acting on the crown wall

STEP 8: TEST THE CROWN WALL AGAINST SLIDING AND OVERTURNING

The dimensions of the crown wall should be such that it is able to resist the failure mechanisms sliding and overturning. The following should hold against sliding:

$$f(F_G - F_U) \geq F_H$$

With a friction factor of 0.6 (see Table 3.2) the left-hand side of the equation is equal to 116.8 kN, and the right-hand side is equal to 116.6 kN.

Against overturning, the following should hold:

$$M_G - M_U \geq M_H$$

The left-hand side of the equation is equal to 548.9 kNm, and the right-hand side is equal to 343.5.

The thickness of the bottom slab of 1.39 m that was chosen in Step 6 is thus sufficient to resist both sliding and overturning. It is of course an iterative process to select the proper thickness.

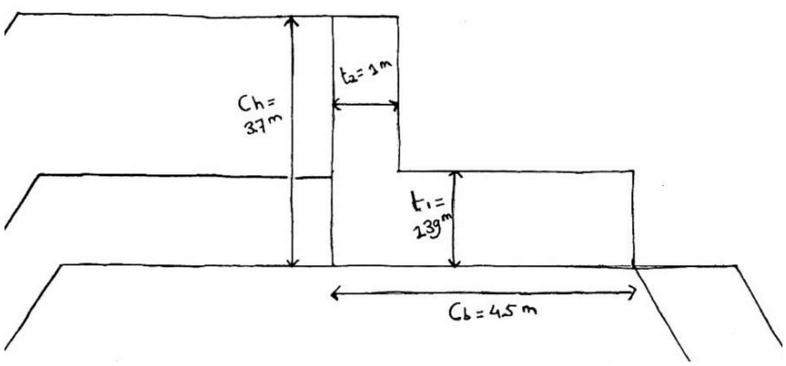


Figure 4.7: Final dimensions of crown wall following DA-1

4.5.2 SENSITIVITY ANALYSIS

The main uncertainty in the design of the crown wall lies in the application of the partial factors. It may seem logical to apply the partial factor on the wave height, as the wave height occurs earlier in the design process, and you want to tackle uncertainties as soon as you encounter them. Moreover, the other breakwater elements have been designed with a factored wave height as well. On the other hand, the Eurocode (EN1990) has been based on working with forces, so it would make sense to apply the partial factors on the forces when the analytical model allows it.

Two issues arise when working with partial factors on the wave height:

- Validity ranges:

When the wave height is factored, this may lead to parameters that are suddenly outside the ranges of validity that were specified for a particular formula. This will not be further investigated, but it is worth mentioning.

- Adjustment of wave parameters:

This issue is the same as the one described in Subsection 4.1.2. For the crown wall, it is best explained by considering the up-lift pressure. The up-lift pressure depends on the parameter $\frac{F_c}{L_{op}}$. This means that if solely the wave height is factored, the up-lift pressure remains the same since the deep-water wavelength does not directly depend on the wave height, but only indirectly through the wave period. However, it is not specified how to deal with this in the draft Eurocode. If you would work with wave parameters (wave period and wavelength) that are adjusted along with the design wave height, the resulting thickness would barely change in this particular case as the influence of the up-lift force on the design outcome is small, but this confusion should not exist at all in the draft Eurocode.

Most interesting is to see how the design outcome changes when not the wave height is factored, but the forces are. To the forces, a partial factor of 1.5 (see Table A.6.8) should be applied, as they are classified as variable actions. This is referred to as option 2 in Table 4.26.

Another alternative is to select Design Case 4, for which Table A.6.8 in the updated Annex A.6 states:

'Design Case 4 (DC4) is typically used when it is relevant to apply partial factors on actions together with a partial factor on effects of actions (see EN 1997 for details). It is used for the structural and geotechnical design of coastal structures loaded by waves and currents.'

The user of the draft Eurocode is left in the dark for which particular cases it is indeed relevant to apply partial factors on actions as well as action effects[19]. As a result of the poor description of the Design Cases, an alternative choice is to factor both the wave height and the force. The force should then be multiplied with a value of 1.5/1.35 (see Table A.6.8 in Appendix B). This is referred to as option 3 in Table 4.26.

[19] The forces and moments are in fact also referred to as actions in EN1990 [Ref. 1], but in prEN1991-1-8 they are classified under hydraulic action effects. Anyway, the same alternative of factoring both the wave height and the forces is still valid, as can be seen under Design Case 4 in Table A.6.8.

Table 4.26 shows the alternative design outcomes next to option 1, which follows from the reasoning in Subsection 4.5.1. Only the most relevant parameters are presented, reference is made to Appendix L for the full calculation.

Description	Notation	Option 1	Option 2	Option 3	Unit
Characteristic value wave height	H_{m0}	7.03	7.03	7.03	m
Partial factor on wave height	γ_{Qz}	1.35	1.00	1.35	-
Design value wave height	$H_{s,d}$	9.49	7.03	9.49	m
Horizontal force	$F_{h,0.1\%}$	116.6	99.5	116.6	kN
Up-lift force	F_u	6.91	6.91	6.91	kN
Overturning moment	$M_{h(Fh0.1\%)}$	343.5	290.7	343.5	kNm
Partial factors on forces and moments	γ_E	1.0	1.5	1.11	-
Factored horizontal force	$F_{h,0.1\%,d}$	116.6	149.3	129.6	kN
Factored up-lift force	$F_{u,d}$	6.91	10.4	7.68	kN
Factored overturning moment	$M_{h(Fh0.1\%),d}$	343.5	436.1	381.7	kNm
Required bottom slab thickness to achieve stability	t_1	1.39	2.09	1.66	m

Table 4.26: Alternative design outcome when factoring the forces instead of the wave height, or factoring both

Finally, different crown wall design formulae could be implemented. The draft Eurocode itself mentions Pedersen and Martin, but the site conditions lie outside the specified ranges of validity (see Appendix C). Still, it could be informative to consider the design result with Pedersen [Ref. 24]. The second design formula that is considered is Molines [Ref. 23].

Designing with Pedersen does not yield a stable design solution, which can be explained by the fact that the maximum up-lift force is considered, regardless of whether this force will act on the structure simultaneously with the maximum horizontal force, as opposed to the other 2 crown wall design methods considered in this thesis.

Molines [Ref. 23] provides a design solution with a thickness of the crown wall slab of 3.20 m, significantly larger than the design solution obtained with [Ref. 22]. Noteworthy is that a stable design solution could only be reached when applying the partial factor to the force instead of the wave height. Apparently, the wave height had much more influence on the outcome than was the case for the crown wall design based on overtopping rates [Ref. 22].

The relevant variables used in these calculations are set out in Appendix L. In addition, this Appendix shows a consideration of the degree of correlation.

It is stressed that the design of the crown wall is prone to a lot of uncertainty, that is not sufficiently dealt with in prEN1991-1-8.

4.6 MAIN FINDINGS FROM CHAPTER 4

The following observations can be made when scrutinising the DA-1 design calculations:

- The required steps (Figures 4.1, 4.2, 4.3 and 4.4) that should be undertaken to design the various breakwater elements according to the response formulae are not set out in prEN1991-1-8. Figure 4.1 was found in Chapter 5 of the Rock Manual, whereas the other Figures were self-invented. It is recommended that a more systematic description of the design formula steps is included in prEN1991-1-8.
- The use of a safety margin in the response formula, which seems to be undesired in DA-1, had significant consequences on the design outcome of the various breakwater elements in the case study, as was demonstrated by the sensitivity analyses for this aspect. If in fact it is the intention of prEN1991-1-8 that the safety margin in the response formula is also included in DA-1, then this should be written down without any room for speculation. Otherwise, it is highly questionable whether the partial factor of 1.35 will suffice, as the wave height is also surrounded by many uncertainties. The full probabilistic calculations in Chapter 5 will hopefully provide insight in this.
- A better description of how to apply the partial factor on the wave height is desired. Primarily the issues whether other wave parameters should be adjusted to the factored wave height accordingly, and whether it is allowed to compute with unrealistic wave heights. When the latter is not allowed, there are barely any safety mechanisms left in the DA-1 format in depth-limited conditions. This could be realistic, since the uncertainty in the wave height is also gone then and is replaced by an uncertainty in the water level. Nevertheless, when the water level is subsequently not factored (as is not allowed according to Table A.6.8 of the updated Annex A.6) either it seems as if there will be a certain degree of uncertainty left that is unaccounted for.
- There is no partial resistance factor specified for the mound structures. This does not seem to fit in the framework of the existing Eurocodes, which has both partial load and partial resistance factors. It is not clear if the partial factors specified by PIANC should instead be used. Explicit statements on the characteristic resistance value and magnitude of a partial resistance factor, for response mechanisms relevant to coastal structures, should be included in prEN1991-1-8.
- The sensitivity analysis for the rock armour layer demonstrates that too much room for interpretation in prEN1991-1-8 is still present. For the case study, the ratio between the nominal rock diameters for the most conservative design choices and the most optimistic design choices was well over 2.
- The sensitivity with respect to the storm duration was considerable in the case study. A quite extreme design storm duration of 12 hours was adopted, as this value is mentioned in Table A.2 of the draft Eurocode. Nevertheless, it should be made obvious whether a standardised value needs to be applied, or whether (and how) this parameter should be extracted from the available data.
- The introduction of the partial factor on the load side led to ULS becoming the leading limit state for the required nominal rock diameter, whereas one would expect SLS-(LD) to be leading for a breakwater armour layer, due to the low level of damage that is accepted.
- The exact meaning of SLS-(LD) is still vague. Its definition should be explicitly included in the draft Eurocode, in particular its relation to the other limit states and how it can be observed in physical model tests.
- It is unclear how to deal with overtopping for the design of the crest height (and the rear-armour stability). Its limit states are poorly defined, especially in combination with the tolerable overtopping discharge that should be used in the response formula. Moreover, no return periods are specified for SLS in an overtopping context, which means that the designer has to assume those values him- or herself.
- It is poorly described how to deal with low water levels, for example in the design of the toe. The draft Eurocode does acknowledge the relevance of low water levels for certain failure mechanisms, but it only describes how to compute the extreme high values for a certain return period, not how to do this

for low values. This should be included in prEN1991-1-8, or otherwise guidance should be provided on where to find this information in other documents.

- Insufficient information is provided on the choice of the equations for the design of the crown wall. Since there is not really ‘a single approved method' (as opposed to for example the rock armour layer for which the Van der Meer equations are widely accepted), this is a serious shortcoming. The case study showed that the variations in the design outcome due to this aspect are significant, even leading to a situation for which a stable solution could not be achieved.
- Insufficient information is provided, with respect to crown wall design, on the application of partial factors on either the wave height, the forces, or both. The descriptions of Design Cases in Table A.6.8 of the updated Annex A.6 are not easy to interpret, and the distinction between actions and action effects is ambiguous. For the case study of IJmuiden, the different interpretations led to different required dimensions of the crown wall, indicating the importance of explicitly treating this matter in prEN1991-1-8. Of course it may be the case that this difference in design results as a consequence of factoring the action (effects) in different ways is less obvious for different design equations or different site conditions.
- The characteristic value that should be used for forces, that are generated by waves, needs to be defined, as this currently lacks in the draft Eurocode.

5. ALTERNATIVE DESIGN APPROACHES

This chapter describes alternative approaches for the design of the main breakwater elements. These will be the deterministic approach, as described by DA-0 in prEN1991-1-8, and the full probabilistic approach, as described by DA-2 in prEN1991-1-8. The same breakwater elements as in Chapter 4 will be designed, with the exception of the ones only treated in the cross-sectional design. The results from these design calculations will be used in Chapter 6, with the goal of making statements about the quality of the draft Eurocode by comparing the results of Chapter 4 to the results of Subsections 5.1 and 5.2. The same breakwater elements as in Chapter 4 will be designed, with the exception of the ones only treated in the cross-sectional design.

5.1 DA-0 DETERMINISTIC BREAKWATER DESIGN

The DA-0 format uses a return period directly linked to a certain failure probability, and incorporates a safety margin in the response formula. This approach is similar to the current design practice of breakwaters. The draft Eurocode does not specify the magnitude of the safety margin. For each response formula, a separate consideration will therefore be made based on literature in which the formula is described.
Chapter 4 discussed the issue of contradictory return period values in Table 4.3 compared to Tables A.4 & A.5. In DA-0, this is not so much of an issue, as Clause A.6 is clearly reserved for use in DA-1. The proper return periods should thus be selected from Table 4.3, which gives a **return period of 100 years for SLS-(LD) and 400 years for ULS**.

5.1.1 ARMOUR LAYER – ROCK

The response formula used is the Van der Meer formula, and a safety margin can be applied by taking a safe value for the model coefficient c_{pl}. According to the Rock Manual, it is common to adopt a 5 percent limit value for this coefficient, resulting in c_{pl} = 5.5 (lower means safer in this case). The results are displayed in Table 5.1:

Limit state	**SLS-(LD)**	**ULS**
Return Period	100 [years]	400 [years]
Wave Height (H_s)	7.03 [m]	7.48 [m]
Wave Period (T_m)	8.13 [s]	8.38 [s]
Slope (tan α)	1:3	
Damage parameter (S_d)	2	12
Storm duration (D)	12 [h]	
Number of waves (N)	5314	5155
Notional Permeability (P)	0.4 [-]	
Density of Water (ρ_w)	1025 [kg/m^3]	
Density of Rock (ρ_s)	2650 [kg/m^3]	
Resistance coefficient (c_{pl})	5.5 [-]	
Rock size (D_{n50})	2.21 [m]	1.63 [m]
Rock mass (M_{50})	28.4 [t]	11.6 [t]

Table 5.1: Design results following DA-0 for rock-armoured slope in SLS-(LD) and ULS

For the Van der Meer formula, an alternative is to adopt a value for c_{pl} of 5.8, which lies one standard deviation below the average [Ref. 7]. This would yield rock sizes of D_{n50} = 2.09 m and 1.55 m for SLS-(LD) and ULS, respectively.

5.1.2 ARMOUR LAYER – ARTIFICIAL UNITS

The acceptable stability number values can adopt a safe value in the response formula for the design of artificial units. A safety factor of 1.5 is usually applied to the values that were used in DA-1. The results are displayed in Table 5.2:

Limit state	**SLS-(LD)**	**ULS**
Return Period	100 [years]	400 [years]
Wave Height (H_s)	7.03 [m]	7.48 [m]
Slope (tan α)	1:1.5	
Unit type	Accropode	
Acceptable stability number ($N_{s,d}$)	2.5	2.7
Density of Water (ρ_w)	1025 [kg/m^3]	
Density of Concrete (ρ_c)	2400 [kg/m^3]	
Unit size (D_n)	2.10 [m]	2.07 [m]
Unit volume (V)	9.3 [m^3]	8.9 [m^3]

Table 5.2: Design results following DA-0 for concrete unit armour layer in SLS-(LD) and ULS

5.1.3 CREST HEIGHT

The response formula used is the overtopping formula, in which the empirical coefficients c_1 and c_2 can adopt a safe value. According to the EurOtop Manual, these values should lie one standard deviation away from their mean for design purposes. Only SLS-(LD) is considered, as this limit state was by far the leading limit state considered in Section 4.3. Full correlation has been assumed, in line with what has been done in DA-1.

The results are displayed in Table 5.3:

Limit state	SLS-(LD)
Return Period H_s	100 [years]
Wave Height (H_s)	7.03 [m]
Return Period η	100 [years]
Water Level (η)	4.11 [m+NAP]
Slope (tan α)	1:1.5
Unit type	Accropode
Empirical coefficients (c_1 and c_2)	0.1035 and 1.35
Roughness factor (γ_f)	0.46
Obliqueness factor (γ_β)	1.0
Tolerable overtopping discharge (q_{tol})	0.020 [m^3/s per m]
Freeboard (R_c)	9.16 [m]
Calculated overtopping discharge (q)	0.0198 [m^3/s per m]
Crest height (A)	13.27 [m+NAP]

Table 5.3: Design results following DA-0 for crest height in SLS-(LD)

5.1.4 CROWN WALL

The wave forces acting on the crown wall are based on overtopping rates, as described in [Ref. 22]. The study that is mentioned in this document also provided confidence bands for the dimensionless forces. In this deterministic calculation the upper limit of the 90%-confidence interval will be used to describe the forces and moments. To arrive at these upper limits, a margin of 0.57 should be added to the mean dimensionless horizontal force, a margin of 0.45 to the mean dimensionless up-lift pressure and a margin of 0.19 to the mean dimensionless overturning moment [Ref. 22]. In symbols:

$$\sigma_{Fh} = 0.57/1.64$$

$$\sigma_{PbF} = 0.45/1.64$$

$$\sigma_{MhF} = 0.19/1.64$$

The configuration of the crown wall has been left intact, so the same values for crest height, base level, crown wall height and width. The only parameter that will be re-designed for is the thickness of the crown wall bottom slab. The results are displayed in Table 5.4:

Limit state	**ULS**
Return Period H_s	400 [years]
Wave Height (H_s)	7.48 [m]
Return Period η	400 [years]
Water Level (η)	4.56 [m+NAP]
Breaker parameter peak period (ξ_p)	3.40
Dimensionless wave overtopping discharge (Q)	0.00287
Wall protection ratio ((R_c-A_c)/C_h)	0
Relative foundation level (F_c/L_{0p})	0.011
Dimensionless horizontal force (Fh)	1.57
Upper limit 90%-CI Fh	2.14
Horizontal force ($F_{h,0.1\%}$)	147.3 [kN]
Dimensionless up-lift pressure (PbF)	0.19
Upper limit 90%-CI PbF	0.64
Up-lift pressure ($PbF_{h,0.1\%}$)	11.8 [kN/m]
Up-lift force (F_u)	26.6 [kN]
Dimensionless overturning moment (MhF)	0.62
Upper limit 90%-CI MhF	0.81
Overturning moment ($Mh_{(Fh0.1\%)}$)	413.8 [kNm]
Thickness crown wall base slab (t_1)	2.25 [m]
Sliding resistance (f*(F_G-F_U))	150.0 [kN]
Overturning resistance (M_G-M_U)	600.2 [kNm]

Table 5.4: Design results following DA-0 for thickness crown wall base slab in ULS

Instead of using the confidence bands presented in [Ref. 22], it would also be possible to use safe values for the empirical overtopping coefficients c_1 and c_2, since these coefficients indirectly have an influence on the forces through the wave overtopping formula. This would yield a thickness of the crown wall base slab of 1.38 m.

5.2 DA-2 FULL PROBABILISTIC BREAKWATER DESIGN

In Clause J.3.1 in prEN1991-1-8, the following statement can be found about DA-2:

‘Examples of probabilistic methods appropriate for the full application of DA-2 are the Direct Integration Method (DIM) and the Monte Carlo Method (MCM). It is noted that DIM requires more resources than MCM for its application.’

It has been chosen to work with the Monte Carlo method, as it is easier to implement. For the application of a Monte Carlo analysis, a limit state function needs to be defined first. Moreover, the *‘probabilistic distributions of all resistance and load variables should be known, as well as the correlation level between them, if any’*, according to Clause J.3.1. The Monte Carlo method then works as follows:

- For each variable, a value is randomly selected.
- This value is then entered into the limit state function.
- When the outcome is negative (i.e. R < S) the result is counted as ‘failure’.
- The probability of failure can be calculated by repeating this process, counting the number of failures and dividing this by the total number of simulations. Over the entirety of simulations, the randomly selected variable values should conform to their probabilistic distribution.
- When the total amount of simulations is large enough, the probability of failure converges to a constant value.

Before setting up the calculations, the ROM [Ref. 5] has been consulted, which has already embraced the full probabilistic calculation into its design framework as one of the most important approaches. Compared to prEN1991-1-8, the ROM contains much more information with respect to this approach. For instance, it systematically specifies the design steps to be undertaken, it gives recommendations on numerical codes that can be used for the computation and it provides an expression with which you can calculate the required number of samples to reach convergence. This all lacks in the draft Eurocode. For example, a statement that can be found in Clause J.3.3 of the draft Eurocode is:

‘N should be sufficiently large for P_f to attain acceptable convergence.’

However, this statement still gives no guidance whatsoever. Another interesting aspect is that the ROM also mentions level II methods (such as FORM), which can assist in the full probabilistic calculation by investigating which parameters exert little influence on the design, so that they can be assumed as deterministic and reduce computation time. PrEN1991-1-8 does not mention FORM at all.

Moreover, a description of how to acquire the probabilistic distributions of the resistance and load variables is lacking in prEN1991-1-8, but the ROM does not treat this extensively either. It is thus necessary to make some assumptions regarding these distributions.

In this chapter, the program Prob2B [Ref. 19] has been used to aid with the Monte Carlo calculations.

5.2.1 PROB2B INPUT

Prob2B takes as input the limit state function, complementary expressions and the distributions of all individual parameters. If relevant, the degree of correlation between parameters can also be specified. Furthermore, the method of calculation and the number of calculations is required as input.

5.2.1.1 LIMIT STATE FUNCTIONS AND COMPLEMENTARY EXPRESSIONS

In order to perform a full probabilistic calculation for a rock armour layer, the Van der Meer formula presented in Subsection 2.4.1 needs to be rewritten to a limit state function. It has been assumed that only the formula for plunging waves is relevant, as this was used in DA-1 and the surf similarity parameter was well below the critical value. The limit state function then becomes:

$$Z = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \left(\frac{\tan \alpha}{\sqrt{s_{0m}}}\right)^{-0.5} - \frac{\mu_1 H_s}{\Delta D_{n50}}$$

It is noticed that there is a variable in the limit state function that was not present in the original Van der Meer formula, namely μ_1. This variable will be explained in the next paragraph that elaborates on the distributions.

Complementary expressions are required for the number of waves, the fictitious wave steepness and the relative buoyant density, as they are all dependent on other parameters. Furthermore, the relationship between the wave height and the mean wave period (as determined in Subsection 3.6.2) needs to be specified. These expressions are listed in Appendix M.

In order to perform a full probabilistic calculation for an armour layer with artificial units, the stability number formula presented in Subsection 2.4.2 has to be rewritten to a limit state function. The concrete armour layer fails when the stability number is exceeded, which means the limit state function becomes:

$$Z = N_{s,d} - \frac{\mu_1 H_s}{\Delta D_n}$$

A complementary expression is required for the relative buoyant density, see Appendix M.

In order to perform a full probabilistic calculation for the crest height, the overtopping formula presented in Subsection 2.4.3 must be rewritten to a limit state function. The crest height is insufficient when the overtopping discharge is greater than the tolerable overtopping discharge, which means the limit state function becomes:

$$Z = q_{tol} - q$$

In the limit state function, the mean overtopping discharge is computed with the following equation:

$$q = c_1\, exp\left[-\left(c_2 \frac{R_c}{(\mu_1 H_{m0})\gamma_f \gamma_\beta}\right)^{1.3}\right] * \sqrt{g(\mu_1 H_{m0})^3}$$

A complementary expression is required for the freeboard, see Appendix M. In the expression for the freeboard, a variable is incorporated that was not present in the original overtopping formula, namely μ_2. This variable will be explained in the next paragraph that elaborates on the distributions.

In order to perform a probabilistic calculation for the crown wall, the method with wave forces based on overtopping rates, presented in Subsection 2.4.4, needs to be rewritten to a limit state function. For the DA-2 calculation, only the failure mechanism sliding will be considered, as this mechanism has the strictest requirement. The limit state function against sliding looks as follows:

$$Z = f(F_G - F_U) - CMU * F_H$$

It is noticed that there is a variable in the limit state function that was not present in the original formula, namely CMU. This variable will be explained in the next paragraph that elaborates on the distributions.

In the limit state function, the following forces are active:

$$F_H = \left(\left(\left(0.27 * \ln(\xi_{0p}) + 0.1\right)(\log Q + 6) + 0.23\right)\left(0.5 * \frac{R_c - A_c}{C_h} + 1\right) - 0.15\right) * (0.5\rho_w g C_h^2)$$

$$F_U = \frac{1}{2} * P_b * C_b$$

$$F_G = M_{cw} * g$$

Complementary expressions are required for the surf similarity parameter based on the peak period, the fictitious wave steepness based on the peak period, the peak period, the dimensionless overtopping discharge, the freeboard, the up-lift pressure and the mass of the crown wall. Moreover, the relationship between the wave height and the peak wave period needs to be specified. The expressions are listed in Appendix M.

5.2.1.2 DISTRIBUTIONS

The distributions that will be assumed for the parameters in the various limit state functions are listed in Table 5.5, together with the type of uncertainty that is addressed by the parameter.There are several parameters for which the distribution needs to be altered when doing the calculation for the Ultimate Limit State instead of the Serviceability Limit State-(Limited Damage). A distinction has therefore been made between general parameters (used in both SLS-(LD) and ULS), and parameters that differ for the different limit states.

The mean values for the nominal rock diameter, the nominal diameter of the concrete armour, the crest height and the thickness of the bottom slab are those calculated with DA-1 in Chapter 4. These parameters will be adjusted according to DA-2 later on in this chapter.

Parameter	Symbol	Unit	Distribution	Mean (μ)	Standard Deviation (σ)	Type of uncertainty
			General			
Wave height	H_s / H_{m0}	m	Weibull	u=4.73; k=1.42; ε=3.9		Physical
Water level	η	m+NAP	Weibull	u=2.54; k=0.92; ε=2.29		Physical
VdM-coefficient	C_{pl}	-	Normal	6.2	0.4	Model
Overtopping coefficient 1	c_1	-	Normal	0.09	0.0135	Model
Overtopping coefficient 2	c_2	-	Normal	1.5	0.15	Model
Crown wall model uncertainty	CMU	-	Normal	1	0.2	Model

Notional permeability	P	-	Deterministic	0.4	-	-
Storm duration	D	h	Normal	9	2	Physical
Slope (rock)	tan α	-	Normal	1:3	0.013	Physical
Slope	tan α	-	Normal	1:1.5	0.0267	Physical
Density of water	ρ_w	kg/m^3	Normal	1025	2	Physical
Density of rock	ρ_s	kg/m^3	Uniform	a=2637; b=2677		Physical
Density of concrete	ρ_c	kg/m^3	Normal	2400	10	Physical
Roughness factor	γ_f	-	Uniform	a=0.43; b=0.49		Physical
Obliqueness factor	γ_β	-	Deterministic	1.0	-	-
Gravitational acceleration	g	m/s^2	Deterministic	9.81	-	-
Tolerable overtopping discharge	q_{tol}	m^3/s/m	Deterministic	0.020	-	-
Friction factor	f	-	Normal	0.6	0.05	Physical
Crown wall height	C_h	m	Deterministic	3.7	-	-
Crown wall width	C_b	m	Deterministic	4.5	-	-
Thickness vertical face	t_2	m	Deterministic	1.0	-	-
			SLS-(LD)			
Uncertainty wave climate	μ_1	-	Normal	1	0.04	Statistical
Uncertainty sea climate	μ_2	-	Normal	1	0.06	Statistical
Damage parameter	S	-	Deterministic	2	-	-
Acceptable stability number	$N_{s,d}$	-	Normal	3.7	0.35	Model
Nominal rock diameter	D_{n50}	m	Triangle	a=1.71; b=1.74; c=1.77		Physical
Nominal diameter concrete armour	D_n	m	Deterministic	1.25	-	-
Crest height	A	m+NAP	Deterministic	10.35	-	-
			ULS			
Uncertainty wave climate	μ_1	-	Normal	1	0.05	Statistical
Uncertainty sea climate	μ_2	-	Normal	1	0.07	Statistical
Damage parameter	S	-	Deterministic	12	-	-
Acceptable stability number	$N_{s,d}$	-	Normal	4.1	0.4	Model
Crest height	A	m+NAP	Deterministic	10.5	-	-
Nominal rock diameter	D_{n50}	m	Triangle	a=1.81; b=1.85; c=1.89		Physical
Nominal diameter concrete armour	D_n	m	Deterministic	1.73	-	-
Thickness bottom slab	t_1	m	Deterministic	1.39	-	-

Table 5.5: Distributions of parameters used in probabilistic calculations

NOTE 1: Two slopes have been specified, because the slope used in the limit state function for the rock armour layer has a different value than the one used for the concrete armour layer, the crest height and the crown wall.

NOTE 2: The magnitude of the crest height has a different value under SLS-(LD) and ULS. This is because the crest height that followed from the DA-1 calculations for overtopping (presented under SLS-(LD)), was altered for the design of the crown wall (presented under ULS).

NOTE 3: The wave height is Weibull distributed. This means that a shape and scale parameter has been added here in Table 5.5. A uniform distribution has been assumed for the density of rock and roughness factor, and a triangular distribution for the nominal rock diameter.

Below, a list is given of all the stochastic variables in Table 5.5, along with a brief explanation of what the parameter comprises and how its distribution has been determined.

- **Wave height H_s / H_{m0}:** This parameter deals with the uncertainty in the weather conditions, related to the wave height, during the lifetime of the breakwater. Its distribution has been determined based on the wave data.
- **Water level η:** This parameter deals with the uncertainty in the weather conditions, related to the water level, during the lifetime of the breakwater. Its distribution has been determined based on the water level data.
- **Uncertainty wave climate μ_1:** This parameter deals with the uncertainty in the extreme value distribution of the wave height, that arises due to the limited number of data samples on which the distribution is fitted, and because various distribution functions can be selected. Its distribution is an estimate, substantiated with the available data.
- **Uncertainty sea climate μ_2:** This parameter deals with the uncertainty in the extreme value distribution of the water level, that arises due to the limited number of data samples on which the distribution is fitted, and because various distribution functions can be selected. Its distribution is an estimate, substantiated with the available data.
- **Van der Meer-coefficient c_{pl}:** This parameter accounts for the deviations between the measurements and the fit of the formula. Its distribution originates from literature.
- **Overtopping coefficient c_1:** This parameter accounts for the deviations between the measurements and the fit of the formula. Its distribution originates from literature.
- **Overtopping coefficient c_2:** Same as for overtopping coefficient c_1.
- **Crown wall model uncertainty CMU:** This parameter accounts for the deviations between the measurements and the fit of the formula, as no such model coefficient is included in the formula itself. Its distribution is an estimate, substantiated with literature.
- **Storm duration D:** This parameter deals with the uncertainty in the weather conditions, related to the duration of storms, during the lifetime of the breakwater. Its distribution is an estimate.
- **Slope tan α:** This parameter accounts for variations in the slope due to construction inaccuracies. Its distribution is an estimate.
- **Density of water ρ_w:** This parameter accounts for the randomness of substance properties. Its distribution is an estimate, substantiated with literature.
- **Density of rock ρ_s:** This parameter accounts for the randomness of material properties. Its distribution is an estimate, substantiated with literature.
- **Density of concrete ρ_c:** Same as for density of rock ρ_s.
- **Roughness factor γ_f:** Same as for density of rock ρ_s.
- **Friction factor f:** This parameter deals with the uncertainty related to friction, due to lack of knowledge on its exact value. Its distribution is an estimate, substantiated with literature.

- **Acceptable stability number $N_{s,d}$:** This parameter deals with the uncertainty related to failure of Accropodes, due to lack of knowledge on its exact value. Its distribution is an estimate, substantiated with literature.
- **Nominal rock diameter D_{n50}:** This parameter accounts for the randomness of material dimensions. Its distribution is an estimate, substantiated with literature.

The other parameters necessary for the probabilistic calculations have been assumed to be deterministic. That's because these parameters are either a dimension, a physical constant or a matter of definition.

Reference is made to Appendix M for more elaborate derivations of these distributions.

5.2.1.3 ADDITIONAL SETTINGS

Apart from the limit state functions, expressions and distributions, there are some other Prob2B input settings that require attention, in order for the probabilistic calculations to be done properly. These are presented in Table 5.6:

Method of calculation	Crude Monte Carlo
Number of calculations	$1*10^5$
Target reliability SLS-(LD)	$\beta = 3.02$
Target reliability ULS	$\beta = 3.41$
Correlation	Yes
Wave breaking	Irrelevant

Table 5.6: Prob2B input settings

The method of calculation that has been opted to work with is Crude Monte Carlo, as was explained in the introduction of Section 5.2. A high, though quite arbitrary, number of calculations[20] has been opted for. This requires a lot of computation time, but also results in a more accurate estimate of the probability of failure.

TARGET RELIABILITY

The target reliability is not actually an input variable, but it does provide information on the required magnitude of the parameters that we want design for (nominal rock/concrete diameter, crest height, thickness bottom slab) in Subsection 5.2.2.

Table 13.1 in prEN1991-1-8 displays the target safety levels expressed in β-values, as a function of the limit state and the consequence class. Nevertheless, these values should not blindly be adopted in the probabilistic calculations. Caution is needed, as the β-values in this table are for a 50-year reference period, *'with the exceedance probability based on a sample representative of all values of the relevant parameter over a suitable record length'*.

However, when using the extreme wave and water level distributions as presented in Table 5.5, the probability of failure per storm event will be calculated, which is not in accordance with the description above. Table A.2 in

[20] To verify that convergence was reached for the various calculations, the simulations have been done multiple times so as to evaluate whether the same order of magnitude was found for the reliability index each time.

prEN1991-1-8 shows equivalence between safety levels, from which the proper target reliability can be selected. The β-value that corresponds to a reference period of 1 year when extreme events are considered, for CC2 and SLS-(LD), equals 2.33. This annual failure probability can then be converted to a failure probability per storm event, using the Poisson distribution and the fact that there are 7.7 storms per year according to our Extreme Value Analysis. The eventual target β-value then becomes 3.02. The same procedure can be followed for ULS, resulting in a target β-value of 3.41.

DEGREE OF CORRELATION

For the design of the crest height and the crown wall, the correlation between wave height and water level should also be taken into account. This can be done by setting the dependence between the two parameters to a value of 0.59 in Prob2B. This correlation coefficient actually belongs to the dependence between wave height and surge, whereas the input in Prob2B is the absolute water level. However, as the input does focus on extreme events, this degree of correlation has been assumed.

INCLUSION OF WAVE BREAKING

In Prob2B, the phenomenon of wave breaking can be taken into account by adding two expressions to the model, which together impose an upper limit on the significant wave height:

$$h = \eta + 13.7$$

$$maximum\ H_s = \min\,(0.45H_s; h)$$

It was discovered that adding this expression to the Prob2B did not alter the outcome of the calculations. Apparently, it is rare that the randomly drawn wave height exceeds the upper limit because of depth considerations. Hence, this phenomenon will further not be discussed for the probabilistic calculation.

5.2.2 PROB2B OUTPUT

The output of Prob2B consists of three main elements: the computed reliability expressed in a β-value, the influence that each parameter has on the design outcome expressed in an α-value, and the design point in which the structure is most likely to fail.

For the various failure mechanisms, the reliability that follows from the DA-1 results is firstly calculated. Thereafter, a probabilistic calculation is made in which the magnitude of the parameter that follows from DA-1 has been adjusted in order to meet the target reliability.

5.2.2.1 ARMOUR LAYER – ROCK

For the rock armour layer, calculations have been performed for both SLS-(LD) as well as ULS.

SLS-(LD)

With the limit state function presented in Paragraph 5.2.1.1 and the parameters presented in Table 5.5, the reliability can be calculated that follows from the nominal rock diameter calculated in DA-1. The computed reliability equals β[21] = 2.13.

It is evident that the D_{n50} determined with DA-1 does not provide for the required reliability in SLS-(LD) (or the target reliability is too large).

By means of trial and error, it has been determined that for a **D_{n50} with a=1.99, b= 2.03 and c= 2.07** the required safety level is met, as the computed reliability equals **β = 3.02.**

The output, in terms of α-values and design points, of the Prob2B computation with the adjusted rock diameter is displayed in Table 5.7:

Parameter	Symbol	α	Design point X
Wave height	H_s	-0.80	6.43 [m]
Uncertainty wave climate	μ_1	-0.17	1.021
VdM-coefficient	C_{pl}	0.40	5.72
Notional permeability	P	-	0.4
Storm duration	D	-0.09	9.53 [h]
Slope (rock)	tan α	-0.04	0.33
Density of water	ρ_w	0.05	1025 [kg/m^3]
Density of rock	ρ_s	0.35	2643 [kg/m^3]
Damage parameter	S	-	2
Nominal rock diameter	D_{n50}	0.13	2.02 [m]

Table 5.7: Result probabilistic calculation of rock armour layer in SLS-(LD)

[21] This value has been calculated using FORM, in order to save time. For this breakwater element/limit state, the MCM has been performed as well to see whether their magnitudes are approximately equal, which was the case. For the other breakwater elements, the reliability following from DA-1 has been computed with FORM only. The adjusted breakwater dimensions have all been determined with MCM.

ULS

The same calculation can be made, but now with the parameters belonging to ULS. The computed reliability equals $\beta = 4.52$.

It is noticed that the D_{n50} determined with DA-1 does provide for the required reliability. Since the computed reliability is larger than the target reliability, the required stone size was overestimated. It is therefore possible to select a smaller nominal rock diameter. It has been determined that for a **D_{n50} with a=1.49, b=1.52 and c=1.55** the required safety level is still met. For this distribution, the computed reliability equals **$\beta = 3.41$.**

The output of the Prob2B computation with the adjusted rock diameter is displayed in Table 5.8:

Parameter	Symbol	α	Design point X
Wave height	H_s	-0.79	6.67 [m]
Uncertainty wave climate	μ_1	-0.26	1.045
VdM-coefficient	C_{pl}	0.30	5.79
Notional permeability	P	-	-
Storm duration	D	-0.23	10.6 [h]
Slope (rock)	tan α	-0.25	0.34
Density of water	ρ_w	-0.06	1025 [kg/m^3]
Density of rock	ρ_s	-0.06	2660 [kg/m^3]
Damage parameter	S	-	12
Nominal rock diameter	D_{n50}	-0.21	1.53 [m]

Table 5.8: Result probabilistic calculation of rock armour layer in ULS

5.2.2.2 ARMOUR LAYER – ARTIFICIAL UNITS

Just as in Paragraph 5.2.2.1, a distinction is made between SLS-(LD) and ULS.

SLS-(LD)

With the limit state function presented in Paragraph 5.2.1.1 and the parameters presented in Table 5.5, the reliability can be calculated that follows from the nominal diameter of the concrete armour calculated in DA-1. The computed reliability equals $\beta = 1.80$

It is evident that the D_n determined with DA-1 does not provide for the required reliability in SLS-(LD).

By means of trial and error, it has been determined that for a **D_n of 1.59** the required safety level is met, as the computed reliability equals **$\beta = 3.02$.**

The output, in terms of α-values and design points, of the Prob2B computation with the adjusted rock diameter is displayed in Table 5.9:

Parameter	**Symbol**	**α**	**Design point X**
Wave height	H_s	-0.70	6.14 [m]
Uncertainty wave climate	μ_1	-0.32	1.039
Density of water	ρ_w	-0.06	1025 [kg/m^3]
Density of concrete	ρ_c	-0.01	2400 [kg/m^3]
Acceptable stability number	$N_{s,d}$	0.63	3.04
Nominal diameter concrete armour	D_n	-	1.59 [m]

Table 5.9: Result probabilistic calculation of concrete armour units in SLS-(LD)

ULS

The same calculation can be made, but now with the parameters belonging to ULS. The computed reliability equals $\beta = 3.44$. It is noticed that the D_n determined with DA-1 provides a reliability that lies very close to the target reliability.

The value for D_n, selected after iteration, needs to be **$D_n = 1.72$** to exactly match the target reliability. The computed reliability then equals **$\beta = 3.41$** and the corresponding output is displayed in Table 5.10:

Parameter	**Symbol**	**α**	**Design point X**
Wave height	H_s	-0.74	6.53 [m]
Uncertainty wave climate	μ_1	-0.23	1.038
Density of water	ρ_w	0.17	1024 [kg/m^3]
Density of concrete	ρ_c	0.05	2398 [kg/m^3]
Acceptable stability number	$N_{s,d}$	0.61	2.98
Nominal diameter concrete armour	D_n	-	1.72 [m]

Table 5.10: Result probabilistic calculation of concrete armour units in ULS

5.2.2.3 CREST HEIGHT

It is not necessary to make a distinction between various limit states, as only SLS-(LD) is considered for the design of the crest height in DA-2.

With the limit state function presented in Paragraph 5.2.1.1 and the parameters presented in Table 5.5, the reliability can be calculated that follows from the crest height calculated in DA-1. The computed reliability equals $\beta = 2.13$.

The crest height, determined with DA-1, is thus not sufficient. The value of **A should be 12.42 m+NAP** to meet the target reliability. The computed reliability then equals **β = 3.03** and the corresponding output is displayed in Table 5.11:

Parameter	**Symbol**	**α**	**Design point X**
Wave height	H_{m0}	-0.06	6.19
Uncertainty wave climate	μ_1	-0.15	1.019
Water level	η	-0.78	3.37
Uncertainty sea climate	μ_2	0.02	0.997
Overtopping coefficient 1	c_1	-0.14	0.096
Overtopping coefficient 2	c_2	0.44	1.30
Roughness factor	γ_f	-0.24	0.44
Obliqueness factor	γ_β	-	1.0
Gravitational acceleration	g	-	9.81
Tolerable overtopping discharge	q_{tol}	-	0.020
Crest height	A	-	12.42

Table 5.11: Result probabilistic calculation of crest height

5.2.2.4 CROWN WALL

It is not necessary to make a distinction between various limit states, as only ULS is considered for the design of the crown wall.

With the limit state function presented in Paragraph 5.2.1.1 and the parameters presented in Table 5.5, the reliability can be calculated that follows from the thickness of the crown wall base slab calculated in DA-1. The computed reliability equals $\beta = 2.77$.

The thickness of the crown wall base slab, determined with DA-1, is thus not sufficient. The value of **t_1 should be 1.89 m** to meet the target reliability. The computed reliability then equals **$\beta = 3.44$** and the corresponding output is displayed in Table 5.12:

Parameter	Symbol	α	Design point X
Wave height	H_{m0}	0.12	5.70
Uncertainty wave climate	μ_1	-0.22	1.037
Water level	η	-0.60	3.37
Uncertainty sea climate	μ_2	0.18	0.957
Crest height	A	-	10.5
Overtopping coefficient 1	c_1	0.12	0.084
Overtopping coefficient 2	c_2	0.31	1.34
Roughness factor	γ_f	-0.21	0.48
Obliqueness factor	γ_β	-	1.0
Gravitational acceleration	g	-	9.81
Crown wall height	C_h	-	3.7
Crown wall width	C_b	-	4.5
Slope	tan α	-0.09	0.675
Density of water	ρ_w	-0.19	1026
Density of concrete	ρ_c	-0.05	2402
Crown wall model uncertainty	CMU	-0.43	1.30
Friction factor	f	0.35	0.54
Thickness vertical face	t_2	-	1.0
Thickness bottom slab	t_1	-	1.89

Table 5.12: Result probabilistic calculation of crown wall

5.3 MAIN FINDINGS FROM CHAPTER 5

The following remarks can be made about the design calculations following DA-0:

- There are few unclarities and/or inconsistencies in the new draft Eurocode when designing in DA-0.
- The most relevant issue is the magnitude of the safety margin that should be applied in the response formula, as each design equation has its own customs. PrEN1991-1-8 should give suggestions for this, for example by giving the statistic to use, and not only referring to the response formulae.
- Another unclarity surrounding DA-0, is whether its intention is to also add a standard deviation to the sea condition parameters used in design, or only treat a safety margin in the response formula.
- A non-standard source was used in combination with the draft Eurocode for the crown wall. This resulted in some additional effort in DA-0, because of the non-standard[22] safety margins.

The following remarks can be made about the design calculations following DA-2:

- For DA-2, some questions arose because of the non-standard safety margins, as it was not straightforward how to include the model uncertainty into the probabilistic calculation for the crown wall design.
- The explanation of the design steps to be undertaken in DA-2 in prEN1991-1-8 could be more extensive. No recommendations are given on numerical codes that can be used for the computation, and the required number of samples to reach convergence is also not specified.
- Some guidance on how to acquire the probabilistic distributions of the resistance and load variables is lacking in prEN1991-1-8. For some parameters the parameterisation of the distribution is straightforward, but for others not so much. For instance, it has been struggled with how to treat the notional permeability parameter. Eventually it has been taken as deterministic, but a distribution could have also been assigned to it, for example to deal with schematisation uncertainties. Clear instructions on how different uncertainties should be incorporated into the full probabilistic calculation should be added to the draft Eurocode.
- The equivalence of the safety levels to be used as target reliabilities, presented in Table A.2 in prEN1991-1-8, is difficult to comprehend. It could be useful if the draft Eurocode specifies in more detail what target reliability should be aimed for in combination with how the loads (i.e. wave height, water levels) have been entered into the probabilistic calculation.
- For the rock-armour layer, the wave height (related to uncertainty in the weather conditions) and the c_{pl}-coefficient stood out as the parameters with the most influence on the reliability.
- For the armour layer of artificial units, the wave height (related to uncertainty in the weather conditions) and the acceptable stability number $N_{s,d}$ stood out as the parameters with the most influence on the reliability.
- For the crest height, the water level (related to uncertainty in the weather conditions) and one of the empirical overtopping coefficients (c_1) stood out as the parameters with the most influence on the reliability.
- For the crown wall, the water level (related to uncertainty in the weather conditions), one of the empirical overtopping coefficients (c_1) and the crown wall model uncertainty (CMU) stood out as the parameters with the most influence on the reliability.

[22] Non-standard in this case means that it was not as easy as just taking one (or more than one) standard deviation away from the mean for a particular parameter, as can for example be done in the Van der Meer formula or overtopping formula.

- A common theme is thus that a sea condition parameter (either wave height or water level) affects the design outcome to a high degree on the one hand, and a coefficient determined from model tests (either Van der Meer-coefficient, stability number, overtopping coefficients or model uncertainty related to crown wall design) on the other hand.
- The calculations demonstrated that the uncertainty related to the wave climate has a certain degree of influence on various elements/limit states as well.
- Other parameters with a certain degree of influence in the case study calculations included the roughness factor, the storm duration and the density of rock. This could perhaps be resolved by adding
- Performance of the DA-1 results concerning ULS was quite okay. Contrary to this, DA-1 results concerning SLS-(LD) did not result in the required target reliability.

6. COMPARATIVE ANALYSIS

The goal of this chapter is to compare the design solutions of chapters 4 and 5 and to provide qualitative statements about the draft Eurocode based on the results.

Firstly, three design approaches have been treated, i.e. DA-0 (deterministic), DA-1 (semi-probabilistic) and DA-2 (fully probabilistic). The DA-2 calculations can be taken as the 'best' design outcome, as this method deals with uncertainties most extensively. It should be noted that some assumptions were necessary, i.e. distribution type and reliability of the parameters, in this calculation. Section 6.1 discusses the results from chapters 4 and 5, and points out the most striking differences.

Secondly, the three design approaches have been compared in more detail. The main focus of the comparison is on DA-1, as this is the method that has been most thoroughly investigated in this thesis. Section 6.2 compares the results from design approach DA-1 and DA-0, while Section 6.3 examines the outcome of DA-1 with respect to DA-2. This chapter concludes with considerations of the magnitude of the partial factor and a scenario study, in which different interpretations of the semi-probabilistic approach are compared to the full probabilistic design outcome.

6.1 SUMMARY OF RESULTS

Breakwater element	Limit state	DA-0 Deterministic Result	DA-1 Semi-probabilistic Result	DA-2 Full probabilistic Result
Armour layer – rock size	SLS-(LD)	2.21 [m]	1.74 [m]	2.03 [m]
Armour layer – rock size	ULS	1.63 [m]	1.85 [m]	1.52 [m]
Armour layer – artificial units	SLS-(LD)	2.10 [m]	1.25 [m]	1.59 [m]
Armour layer – artificial units	ULS	2.07 [m]	1.73 [m]	1.72 [m]
Crest height	SLS-(LD)	13.27 [m+NAP]	10.35 [m+NAP]	12.42 [m+NAP]
Crown wall – base thickness	ULS	2.25 [m]	1.39 [m]	1.89 [m]

Table 6.1: Design outcomes in DA-0, DA-1 and DA-2 for the various breakwater elements and limit states

If we make a first comparison, it can be concluded that design approach DA-1 structurally underestimates the required size or height of breakwater elements compared to the DA-2 approach for the Serviceability Limit State-(LD). This notification has been mentioned in Chapter 4, in which it was concluded that few safety mechanisms are in play for SLS-(LD) because of the lack of partial factors. This resulted in a rock size requirement that is stricter for ULS than for SLS-(LD) in DA-1. This is counterintuitive, as the serviceability limit state is usually governing for the design of a rock armour layer, as can be seen in Table 6.1 for both DA-0 and DA-2.

For the Ultimate Limit State, there is no such similarity for the various breakwater elements when comparing DA-1 and DA-2. The rock armour layer provides an overestimation, the artificial unit size is spot on, and the base thickness of the crown wall gives an underestimation.

Finally, it can be noticed that DA-0 structurally overestimates the required size or height of breakwater elements compared to the DA-2 approach, for SLS-(LD) as well as ULS. Using this approach is thus perhaps conservative, but the structures that are designed with this approach will at least measure up to the target reliability.

It should be noted that full correlation has been assumed in DA-0 and DA-1 (which is a conservative assumption), the actually intended outcome of these design approaches will be somewhat lower. Taking this into account, you might be able to conclude that the deterministic (DA-0) approach gives results closer to the results of the full probabilistic (DA-2) approach than the semi-probabilistic (DA-1) approach does. The next sections will have to reveal why this is the case.

6.2 COMPARISON OF DA-1 AND DA-0

It is interesting to investigate how DA-0 and DA-1 both deal with uncertainties in their own way, and how this difference in approach can explain the difference in the design solutions described in Section 6.1. It is even more interesting as DA-0 greatly resembles the conventional existing design methods. The introduction of DA-1 should result in estimates that resemble, or provide at least very similar estimates, of the DA-2 design outcomes, which are considered to be the most correct values. Nevertheless, from the values in Table 6.1 it was concluded that this does not appear to be the case for DA-1 in the way that it is currently described.

6.2.1 PARAMETER COMPARISON

The two methods will be further analysed by not only looking at the design results, but also at the way these were reached. The two methods are relatively easy to compare, as both methods work with single values that need to be inserted into the design formula, and not with distributions. For the different breakwater elements, Table 6.2 up to and including Table 6.5 show the parameters that have a deviating value in DA-1 compared to DA-0, for all relevant limit states. The tables are then followed by a discussion on their differences for that particular breakwater element.

ARMOUR LAYER-ROCK

Limit state	**SLS-(LD)**		**ULS**	
Design approach	DA-0	DA-1	DA-0	DA-1
Return period	100 [y]	10 [y]	400 [y]	100 [y]
Wave height (H_s)	7.03 [m]	6.22 [m]	7.48 [m]	7.03 [m]
Wave period (T_m)	8.13 [s]	7.64 [s]	8.38 [s]	8.13 [s]
Partial factor	-	1.00	-	1.35
Design wave height ($H_{s,d}$)	7.03 [m]	6.22 [m]	7.48 [m]	9.49 [m]
Number of waves (N)	5314	5654	5155	5314
Resistance coefficient (c_{pl})	5.5 [-]	6.2 [-]	5.5 [-]	6.2 [-]
Rock size (D_{n50})	2.21 [m]	1.74 [m]	1.63 [m]	1.85 [m]

Table 6.2: Parameter comparison for the rock armour layer

From Table 6.2, the most important difference between DA-0 and DA-1 can immediately be observed. DA-0 tackles uncertainties by choosing a safe value for the coefficient c_{pl}, which more or less acts as a safety factor on the strength side, and by selecting a high return period, which is a safety mechanism on the load side. Contrary to this, DA-1 tackles uncertainties by factoring the wave height, which acts on the load side only. This difference on itself is not a problem, as for example in ULS the higher design wave height compensates for the lack of a safety mechanism on the strength side, resulting in a very reasonable design outcome in DA-1. Nevertheless, it does become a problem for SLS-(LD). Table 6.2 shows that the design wave height in SLS-(LD) is higher in DA-0 than in DA-1, and the lower resistance coefficient even provides for additional safety in DA-0. The eventual computed rock size in DA-1 is therefore also far too low (compared to DA-2).

ARMOUR LAYER – ARTIFICIAL UNITS

Limit state	**SLS-(LD)**		**ULS**	
Design approach	DA-0	DA-1	DA-0	DA-1
Return period	100 [y]	10 [y]	400 [y]	100 [y]
Wave height (H_s)	7.03 [m]	6.22 [m]	7.48 [m]	7.03 [m]
Partial factor	-	1.00	-	1.35
Design wave height ($H_{s,d}$)	7.03	6.22 [m]	7.48	9.49 [m]
Acceptable stability number ($N_{s,d}$)	2.5	3.7	2.7	4.1
Unit size (D_{n50})	2.10 [m]	1.25 [m]	2.07 [m]	1.73 [m]

Table 6.3: Parameter comparison for the armour layer with artificial units

Table 6.3 shows the same pattern for the armour layer of artificial units. In SLS-(LD), the built-in safety in DA-1 is lower for the resistance (higher stability number) and for the action (lower design wave height). In the ULS, the design wave height is higher for DA-1, but apparently the uncertainty on the strength side is such that the magnitude of the partial factor cannot compensate this.

CREST HEIGHT

Limit state	**SLS-(LD)**	
Design approach	DA-0	DA-1
Return period H_s	100 [y]	10 [y]
Wave height (H_s)	7.03 [m]	6.22 [m]
Partial factor on H_s	-	1.00
Design wave height ($H_{s,d}$)	7.03	6.22 [m]
Return Period η	100 [years]	10 [years]
Water Level (η)	4.11 [m+NAP]	3.38 [m+NAP]
Overtopping coefficient c_1	0.1035	0.09
Overtopping coefficient c_2	1.5	1.5
Freeboard (R_c)	9.16 [m]	6.97 [m]
Crest height (A)	13.27 [m+NAP]	10.35 [m+NAP]

Table 6.4: Parameter comparison for the crest height

The crest level calculation shows similar behaviour. Uncertainties in DA-1 are included by the partial load factor, but as this factor is set to 1 in SLS-(LD), the crest height that is determined in DA-1 is significantly smaller than in DA-0.

CROWN WALL

Limit state	**ULS**	
Design approach	DA-0	DA-1
Return period H_s	400 [y]	100 [y]
Wave height (H_s)	7.48 [m]	7.03 [m]
Partial factor on H_s	-	1.35
Design wave height ($H_{s,d}$)	7.48 [m]	9.49 [m]
Return Period η	400 [years]	100 [years]
Water Level (η)	4.56 [m+NAP]	4.11 [m+NAP]
Dimensionless overtopping discharge	0.00287	0.00558
Relative foundation level (F_c/L_{0p})	0.011	0.015
Horizontal force ($F_{h,0.1\%}$)	147.3 [kN]	116.6 [kN]
Up-lift force (F_u)	26.6 [kN]	9.1 [kN]
Overturning moment ($Mh_{(Fh0.1\%)}$)	413.8 [kNm]	343.5 [kN]
Thickness crown wall base slab (t_1)	2.25 [m]	1.39 [m]

Table 6.5: Parameter comparison for the base thickness of the crown wall

For the base thickness of the crown wall, the uncertainty is dealt with differently in DA-0 than was the case for the previous breakwater elements. For the armour layer and the crest height, uncertainties were dealt with by applying safe values for the model resistance uncertainties, but in this case the uncertainty is tackled by increasing the forces. Still, this differs from DA-1 in which the uncertainty is attached to the sea conditions. The DA-0 result is an overestimation, whereas the DA-1 result an underestimation. The magnitude of the mechanisms coping with the uncertainty is thus very delicate.

Another aspect that is noteworthy is the fact that the water level is not factored in DA-1. This leads to the design water level being higher in DA-0, even though DA-1 copes with uncertainties through the sea condition parameters.

6.2.2 CONCLUSIONS

Below, some general remarks derived from the comparison between DA-1 and DA-0 are listed:

- DA-0 incorporates safety through the coefficients in the response formulae and thus indirectly through the strength side of the limit state. The approach also incorporates safety by selecting higher return periods and therefore through the load side of the limit state as well.
- DA-1 incorporates safety through the sea conditions and thus through the load side of the limit state. Differences in approach seems fine, however, the partial factor that is applied to the wave height then not only has to deal with uncertainties belonging to the wave height, but actually to all the uncertainties that are in play in the particular failure mechanism. Another reason why it is not logical is that the uncertainty in strength and load might differ between different cases.
- DA-0: Since the coefficients in the response formulae change regardless of the limit state, this safety mechanism works for both SLS-(LD) and ULS.
- DA-1: The safety because of partial factors only holds for ULS and disappears in SLS-(LD) where these factors are set to 1.

6.3 COMPARISON DA-1 AND DA-2

In Section 6.1, it was established that the general Eurocode approach by using DA-1, as adopted in the draft Eurocode for waves and current actions, seems to provide unsafe results compared to DA-0/DA-2. In this section, it is investigated where these dissimilarities originate from. Firstly, the sources of uncertainty are scrutinised, after which some general findings are presented.

6.3.1 DEALING WITH UNCERTAINTIES

It is more difficult to compare the individual parameters of DA-1 and DA-2, as the former works with single design values while the latter uses distributions. However, the statistical analysis provides information about the variables that exert most influence on the design outcome in DA-2, something can be said about the variables that exert most influence on the design outcome in DA-2, by looking at the absolute value of the parameter α. Moreover, the design point of these variables might contain some valuable information.

Table 6.6 up to and including Table 6.9 rank the parameters that have the highest absolute α-value, with a minimum of 0.20, accompanied by their design point. These values were produced with the Prob2B calculations performed in Section 5.2. The design solution is also presented in the tables. The content of the tables is examined and placed in perspective relative to DA-1.

A distinction is made between the various breakwater elements and relevant limit states.

ARMOUR LAYER - ROCK

Limit state: SLS-(LD)					
Order	**Variable**	**Symbol**	**|α|**	**Design point X**	**DA-1 value**
1	Wave height	H_s	0.80	6.43 [m]	6.22 [m]
2	VdM-coefficient	C_{pl}	0.40	5.72	6.2
3	Density of rock	ρ_s	0.35	2643 [kg/m^3]	2650 [kg/m^3]
-	Nominal rock diameter	D_{n50}	0.13	2.02 [m]	1.74 [m]
Limit state: ULS					
Order	**Variable**	**Symbol**	**|α|**	**Design point X**	**DA-1 value**
1	Wave height	H_s	0.79	6.67 [m]	9.49 [m]
2	VdM-coefficient	C_{pl}	0.30	5.79	6.2
3	Uncertainty wave climate	μ_1	0.26	1.045	-
4	Slope (rock)	tan α	0.25	0.34	0.33
5	Storm duration	D	0.23	10.6 [h]	12 [h]
-	Nominal rock diameter	D_{n50}	0.21	1.53 [m]	1.85 [m]

Table 6.6: Parameters with most influence on design outcome rock armour layer

The wave height is the parameter with the most influence on the design solution for both SLS-(LD) and ULS. Given this fact, it makes sense to factor the wave height in DA-1. On the other hand, the empirical coefficient c_{pl} also exerts a lot of influence, which indicates that the presence of a partial resistance factor would also be plausible.

In SLS-(LD), the design point of the wave height is slightly larger than the design value in DA-1. Hence, the wave height in DA-1 cannot cover for any other uncertainties in the Van der Meer-equation.

In ULS, the design point of the wave height is a substantial amount smaller than the design wave height in DA-1 (including the partial factor). Hence, the wave height in DA-1 is able to cover for any other uncertainties as well, resulting in a reliable estimate.

The wave climate uncertainty will not be treated as it does not have an equivalent in DA-1.

ARMOUR LAYER – ARTIFICIAL UNITS

Limit state: SLS-(LD)					
Order	**Variable**	**Symbol**	**|α|**	**Design point X**	**DA-1 value**
1	Wave height	H_s	0.70	6.14 [m]	6.22 [m]
2	Acceptable stability number	$N_{s,d}$	0.60	2.85	3.7
3	Uncertainty wave climate	μ_1	0.32	1.039	-
-	Nominal diameter concrete armour	D_n	-	1.59 [m]	1.25 [m]
Limit state: ULS					
Order	**Variable**	**Symbol**	**|α|**	**Design point X**	**DA-1 value**
1	Wave height	H_s	0.74	6.53 [m]	9.49 [m]
2	Acceptable stability number	$N_{s,d}$	0.61	2.98	4.1
3	Uncertainty wave climate	μ_1	0.23	1.038	-
4	Density of concrete	ρ_c	0.20	2392 [kg/m^3]	2400
-	Nominal diameter concrete armour	D_n	-	1.72 [m]	1.73 [m]

Table 6.7: Parameters with most influence on design outcome concrete armour layer

Tables 6.7 shows that both the wave height and the stability number greatly affect the eventual design solution in DA-2. In SLS-(LD), the influence of the acceptable stability number ensures an underestimation of the DA-1 result. In ULS, the design point of the wave height lies below the design wave height used in DA-1, which compensates for the influence of $N_{s,d}$. The design outcomes in ULS match, but for the 'wrong' reason, as the uncertainty on the resistance side (in DA-2) is accounted for by adding safety on the load side (in DA-1).

CREST HEIGHT

Order	Variable	Symbol	|α|	Design point X	DA-1 value
1	Water level	η	0.78	3.37 [m+NAP]	3.38 [m+NAP]
2	Overtopping coefficient 2	c_2	0.44	1.30	1.50
3	Roughness factor	γ_f	0.24	0.44	0.46
-	Crest height	A	-	12.42 [m+NAP]	10.35 [m+NAP]

Table 6.8: Parameters with most influence on design outcome crest height for SLS-(LD)

The wave height becomes less important, and the water level takes over as the most dominant variable. Since the design point of the water level is almost equal to the design value of the water level in DA-1, this variable is not able to compensate for other uncertainties in the overtopping formula, reflected by the overtopping coefficients.

CROWN WALL

Order	Parameter	Symbol	$\lvert\alpha\rvert$	Design point X	DA-1 value
1	Water level	η	0.60	3.37 [m+NAP]	3.38 [m+NAP]
2	Crown wall model uncertainty	CMU	0.43	1.30	-
3	Friction factor	f	0.35	0.54	0.6
4	Overtopping coefficient 2	c_2	0.31	1.34	1.50
5	Uncertainty wave climate	μ_1	0.22	1.037	-
6	Roughness factor	γ_f	0.21	0.48	0.46
-	Thickness bottom slab	t_1	-	1.89 [m]	1.39 [m]

Table 6.9: Parameters with most influence on design outcome thickness of the crown wall bottom slab for ULS

The water level is the most important parameter. Therefore, it is doubtful that the water level is not factored[23] in DA-1. Although the design point of the water level is similar to the design value in DA-1, there are so many other uncertainties not covered for (such as the overtopping coefficient, the crown wall model uncertainty and the friction factor), that the ultimate answer in DA-1 is not sufficient.

6.3.2 CONCLUSIONS

Below, some general remarks derived from the comparison between DA-1 and DA-2 are listed:

- For the armour layer, the wave height is one of the most important parameters, so it makes sense to apply a partial factor to it and in this way deal with a large part of the uncertainty.
- For the crest height and the crown wall, the water level becomes highly important, so it would perhaps be wise to include some additional safety for this sea condition parameter in DA-1, either by allowing it to be factored or by adding a standard deviation to the design value found with the EVA.
- For all failure mechanisms, the resistance model uncertainties, reflected by the empirical coefficients (c_{pl}, $N_{s,d}$, c_2, and CMU) exert great influence on the design results. This shows once again that it would be logical to (in some way) include a partial resistance factor in the safety framework.
- Model uncertainties and uncertainties in the wave climate are largely ignored in SLS-(LD), and the magnitude of the return period is not sufficient to compensate for this. In ULS this is not so much of an issue, as partial factors are present in this limit state to provide additional safety.

[23] REMINDER: As is specified by Table A.6.8 in the updated Annex A.6, shown in Appendix B of this document.

6.4 ANALYSIS OF DA-1

A closer look is taken at the partial factor that is applied to the wave height in DA-1, by consecutively examining the uncertainty in the wave height and by making a comparison to PIANC [Ref. 12]. Next, a scenario study will be executed, after which the section finishes off with suggestions for improvement of the draft Eurocode.

6.4.1 MAGNITUDE OF PARTIAL LOAD FACTOR

There is no partial resistance factor in DA-1. We interpreted this such that this implies that the partial load factor of 1.35 should cover for all the uncertainties in the calculation. In Subsection 3.6.1, we discussed the most important uncertainties surrounding the wave height. These comprise: the selection of return periods, the choice of threshold value and event separation time, the fitting of a distribution and the extrapolation towards higher return periods. The first two items will not be considered, since a better description in the draft Eurocode would mean that there is not a lot of uncertainty related to these aspects. However, there will always be a certain degree of model uncertainty because the true distribution is unknown, and statistical uncertainty because of the limited amount of data. An example is illustrated in Table 6.10, based on the extreme distributions and non-exceedance values that were already treated in Subsection 3.6.1:

Return period	**Distribution**	**Non-EV [%]**	**H_s [m]**	**Ratio**
100 [y]	Generalised Pareto	50	6.59	-
100 [y]	Gumbel	97.5	7.62	1.16

Table 6.1: Example of uncertainty in the wave climate due to optimistic versus conservative design choices

Of course this is an extreme comparison, but it shows that is possible that almost half of the partial factor is required to handle the uncertainty in the wave height. It is imaginable that the remaining portion of the partial factor is not sufficient to cover for the rest of the uncertainties.

6.4.2 COMPARISON TO PIANC

Another comparison to make with respect to DA-1, is comparing with an existing method developed within the safety framework drawn up by PIANC [Ref. 12]. PIANC developed a semi-probabilistic approach, as was described in Subsection 2.2.2. This may be an alternative approach to determine the magnitude of partial factors and return periods in this safety format. For the PIANC-method, a different definition for the characteristic wave height was proposed which has a return period equal to the design lifetime, so comparing partial factors alone is not of much use. It is better to make a comparison between the design values and the design results instead of solely considering the magnitude of the partial safety factors. A comparison is made in Table 6.11, for the full elaboration of the PIANC-calculation see Appendix N. In this table, the characteristic resistance value is found by simply filling in the right-hand side of the Van der Meer-equation using average values. It should be noted that this comparison is only possible for the rock-armour layer, as the PIANC method has been developed for a few specific cases only.

Limit state	SLS-(LD)		ULS	
Method	DA-1	PIANC	DA-1	PIANC
Characteristic wave height ($H_{s,k}$)	6.22 [m]	6.80 [m]	7.03 [m]	6.80 [m]
Partial load factor (γ_S)	1.00	1.057	1.35	1.170
Design wave height ($H_{s,d} = H_{s,k} * \gamma_S$)	6.22 [m]	7.19 [m]	9.49 [m]	7.95 [m]
Characteristic resistance ($R_k = N_s$)	2.25	2.25	3.24	3.24
Partial resistance factor (γ_R)	-	1.025	-	1.057
Design resistance ($R_d = N_s / \gamma_R$)	2.25	2.20	3.24	3.07
Nominal rock diameter (D_{n50})	1.74 [m]	2.07 [m]	1.85 [m]	1.64 [m]

Table 6.11: Comparison of the design values and required rock diameter for DA-1 and PIANC

When comparing both calculation methods, three matters stand out. Firstly, one can notice that the rock diameters computed with the PIANC-method are very similar to the ones that follow from DA-2, which were 2.03 m and 1.52 m for SLS-(LD) and ULS, respectively.

Secondly, one can notice that a partial resistance factor is included in the safety framework of PIANC, while it is missing in DA-1. It has already been concluded by looking at the DA-2 computations that it would make sense to incorporate such a factor in DA-1 as well, and PIANC shows that this is a possibility.

Thirdly, the PIANC method prescribes the use of partial safety factors regardless of the limit state under consideration. This is in conflict with the way that the Eurocodes deal with partial factors, but it does result in a more reliable outcome for the SLS-(LD) limit state, though this is also partly caused by the use of higher return periods in this limit state.

6.4.3 SCENARIO STUDY

The scenario study focuses on the design assumptions that have been made in Chapter 4, because of the lack of clear instructions in prEN1991-1-8. Just as in Subsection 6.4.2, only the rock armour layer has been considered[24]. The most important design aspects that could have been interpreted differently are listed below:

- Using a return period of 400 years (ULS) and 100 years (SLS-(LD)), so following Table 4.3 in prEN1991-1-8, instead of return periods of 100 years (ULS) and 10 years (SLS-(LD)), taken from Table A.4 & Table A.5 in prEN1991-1-8.
- The use of a safe value for the empirical coefficient (c_{pl} = 5.8) instead of an average value (c_{pl} = 5.8).
- Taking depth-limited wave breaking into account instead of assuming that unrealistic wave heights may be calculated with. The partial factor is applied on the offshore wave height, after which an upper limit of 0.45h is imposed on the nearshore wave height.
- Treating the variables as being ‘less than moderately correlated’ instead of using the assumption of full correlation.

Based on the different design assumptions sketched above, various scenarios have been drawn up in which the various unclarities, that still exist in the draft Eurocode, have been combined.

[24] In lack of time only the rock armour layer has been considered. For the other breakwater elements a similar scenario study would also provide valuable information, so it is recommended to follow the same procedure in future studies for these elements and investigate how the different design assumptions affect the eventual design outcome.

ARMOUR LAYER – ROCK – SLS-(LD)

For the Serviceability Limit State-(LD), only 4 scenarios will be considered. The offshore-nearshore transformation is not relevant, as the partial factor has a magnitude of 1.00 and the 10y wave and 100y wave can then still physically exist at the toe of the breakwater. In case of SLS-(LD), the level of correlation is not relevant either, at least for this failure mechanism. The scenarios are briefly explained below:

1) Return period of 10y for offshore wave height, and average value for empirical coefficient adopted in response formula.
2) Return period of 100y for offshore wave height, and average value for empirical coefficient adopted in response formula.

Scenarios 3 and 4 equal to scenarios 1 and 2 respectively, with the only difference being the use of safe value for the empirical coefficient adopted in the response formula.

In all scenarios, the following design variables are equal: CSE of wave height, Weibull distribution, slope of 1:3, Sd = 2, D = 12, partial factor magnitude of

Table 6.12 illustrates how the various scenarios affect the design wave height or empirical model resistance, and how large the calculated rock diameter then becomes:

Scen-ario	Return Period [y]	Nearshore (equal to offshore values)		T_m [s]	C_{pl}	D_{n50} [m]
		$H_{s,k}$ [m]	$H_{s,d}$ [m]			
1	10	6.22	6.22	7.64	6.2	1.74
2	100	7.03	7.03	8.13	6.2	1.96
3	10	6.22	6.22	7.64	5.8	1.86
4	100	7.03	7.03	8.13	5.8	2.09

Table 6.12: Required rock diameter for the various scenarios for SLS-(LD)

For ULS, the computed nominal rock diameter in DA-2 was equal to 2.03 m. That means that scenarios 2 and 4, which work with a higher return period compared to the design calculations made in Section 4.1, show the best performance.

ARMOUR LAYER – ROCK - ULS

For the Ultimate Limit State, 8 scenarios will be considered. The scenarios are briefly explained below:

1) Return period of 100y for offshore wave height, transformed nearshore, after which partial factor of 1.35 is applied to the wave height which ignores depth-limitations. Full correlation assumed[25], and average value for empirical coefficient adopted in response formula.
2) Return period of 400y for offshore wave height, transformed nearshore, after which partial factor of 1.35 is applied to the wave height which ignores depth-limitations. Full correlation assumed (irrelevant for this scenario), and average value for empirical coefficient adopted in response formula.
3) Return period of 100y[26] for offshore wave height, partial factor is applied to offshore wave height and transformed nearshore, taking into account depth-limitations. Full correlation assumed (water level of 4.11m+NAP determines maximum wave height), and average value for empirical coefficient adopted in response formula.
4) Return period of 100y for offshore wave height, partial factor is applied to offshore wave height and transformed nearshore, taking into account depth-limitations. Weak correlation assumed (water level of 3.38m+NAP determines maximum wave height), and average value for empirical coefficient adopted in response formula.

Scenarios 5, 6, 7 and 8 are equal to scenarios 1, 2, 3, and 4 respectively, with the only difference being the use of safe value for the empirical coefficient adopted in the response formula.

In all scenarios, the following design variables are equal: CSE of wave height, Weibull distribution, slope of 1:3, Sd = 12, D = 12, partial factor magnitude of 1.35.

Table 6.13 illustrates how the various scenarios affect the design wave height or empirical model resistance, and how large the calculated rock diameter then becomes:

Scen-ario	**Return Period [y]**	**Offshore**		**Nearshore**		**T_{m02} [s]**	**C_{pl}**	**D_{n50} [m]**
		$H_{s,k}$ [m]	**$H_{s,d}$ [m]**	**$H_{s,k}$ [m]**	**$H_{s,d}$ [m]**			
1	100	7.03	7.03	7.03	9.49	8.13	6.2	1.85
2	400	7.48	7.48	7.48	10.10	8.38	6.2	1.96
3	100	7.03	9.49	8.01	8.01	8.13	6.2	1.51
4	100/10	7.03	9.49	7.69	7.69	8.13	6.2	1.46
5	100	7.03	7.03	7.03	9.49	8.13	5.8	1.97
6	400	7.48	7.48	7.48	10.10	8.38	5.8	2.09
7	100	7.03	9.49	8.01	8.01	8.13	5.8	1.61
8	100/10	7.03	9.49	7.69	7.69	8.13	5.8	1.56

Table 6.13: Required rock diameter for the various scenarios in ULS

[25] The assumption of full or weak correlation is mentioned in each scenario, though it is irrelevant for the rock armour layer in the scenarios for which depth-limitations are ignored.
[26] In the scenarios for which depth-limitations have been taken into account, it does not matter whether you select the 100y or 400y RP for the offshore wave, because these waves will break towards the same physical limit which is determined by the water level (not taking into account the extreme scenario in which you combine the 400y wave with the 400y water level).

For ULS, the computed nominal rock diameter in DA-2 was equal to 1.52 m. That means that scenarios 3, 4, 7 and 8, which take depth-limitations into account, show the best performance.

Based on the considerations in the previous subsections, the following remarks can be made:

- PIANC shows that the inclusion of a partial resistance factor into the semi-probabilistic approach can, if applied properly, assist in creating an accurate design framework.
- For SLS-(LD), additional safety should be introduced so as to obtain more reliable estimates, i.e. closer to the 'most correct' DA-2 results. This can either be achieved by allowing the partial factors to be larger than 1 in SLS-(LD), by selecting higher return periods for the wave height, or by a combination of the two.
- For ULS, the draft Eurocode probably intended to take depth-limitations into account, though this was not immediately evident from its explanations.

It should be noted that these suggestions have been made based on a single case study and only for one breakwater element, so their implications should be carefully evaluated. The statements can be supported (or refuted) by looking at other case studies. Moreover, a scenario study for other breakwater elements, in which the combination of waves and water levels become more relevant, can perhaps teach us a bit more about the degree of correlation. It can then also be investigated whether applying a partial factor on the water levels should be added to the draft Eurocode. This seems to be a promising addition to the design framework, given the large influence that the water level has on the design outcome as was discovered in Section 6.3, but it still needs to be substantiated with data.

7. DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

This final chapter contains the discussion, the conclusions and the recommendations. Before jumping to any conclusions, the findings should be put into an appropriate context, which is done in the discussion. The answer to the main research question is presented in the conclusions, after which recommendations are given on how to improve the draft Eurocode and what cases should be investigated hereafter.

7.1 DISCUSSION

The content of this discussion will be twofold: on the one hand, it is valuable to comment on the validity of the obtained results in the case study; on the other hand, it is valuable to comment on the usefulness of the draft Eurocode in general.

In Chapter 6 it was found that the results obtained with the semi-probabilistic approach DA-1 do not resemble the results obtained with the full probabilistic approach DA-2 for the case study of IJmuiden. However, based on a single case study it would be premature to conclude that this means that the semi-probabilistic approach does not yield reliable results in general.

Firstly, it should be noted that prEN1991-1-8 is a draft, and it should be expected that following drafts will show improvements, which may be based on this report. Some of these flaws related to DA-1 (semi-probabilistic design) can very easily be fixed. For example, the discrepancy between the return periods presented in the draft Eurocode in Table 4.3 compared to those in Tables A.4&A.5, is an issue that can be resolved by adjusting one set of return periods to match the other. The lack of clarity whether to use safe or average empirical coefficient values in the response formulae, is an issue that can simply be resolved by adding a few lines of explanation. Had these design aspects been chosen differently, it could very well be that the DA-1 results are closer to the DA-2 (full probabilistic design) results.

A similar remark can be made for the DA-2 calculations. These are also surrounded by uncertainties, as the draft Eurocode does not provide clarity on how to properly perform such a calculation.

This shows that the design framework in prEN1991-1-8 is not per se fundamentally wrong. However, the possibilities for multiple and wrong interpretations currently present in the draft Eurocode should be tackled to create more consensus for the user.

Secondly, some design assumptions were made in the DA-1 design process, such as ignoring depth-limitations for the (factored) wave height, and treating wave height independently of the water level. These assumptions were made in order to simplify the case study. Some assumptions were made to reduce the time required for the calculations, but others because of the lack of clear instructions in the draft Eurocode on certain essential concepts (e.g. correlation between variables, selection of appropriate return periods). In the DA-2 design process, it was not necessary to make these assumptions. For example, it was easy to take correlation into account, depth-limitations became irrelevant without the application of a partial factor, and it was not necessary to deal with all the issues surrounding the selection of return periods. This indicates that, although it is meaningful to compare the DA-1 and DA-2 results, it is not entirely a fair comparison.

The validity considerations of the design outcome, discussed up until this point, demonstrate that both the DA-1 and DA-2 results are quite uncertain. This suggests that the conclusions that are drawn based on these results should be presented with some caution, especially when trying to interpret what the case study results mean for prEN991-1-8 in general.

The final question that is still left unanswered, is whether the proposed semi-probabilistic approach is a useful design method for coastal structures at all. One major advantage of the Eurocode framework for structures like buildings and bridges, is that it is widely accepted and hence leads to conformity in the design of these structures. Currently, there is no code that is widely accepted for coastal structures, and the draft Eurocode could fill in this void. This will result in more conformity if the room for interpretation in the final version of the new Eurocode is reduced to a minimum, which can quite easily be achieved with some small adjustments. From this perspective, you can say that prEN1991-1-8 looks promising.

Another major advantage of the Eurocode framework for structures like buildings and bridges, is that it is very quick to work with because the loads and combination factors are all standardised. This is one of the reasons why the semi-probabilistic approach is preferred over a full probabilistic calculation for these structures. When translating this to coastal structures, this is where prEN1991-1-8 falls short. It is difficult to standardise the loads and especially the dependence between the loads, due to the varying sea climate for different locations. From the experience gained by working on the case study, it has been discovered that the effort that it takes to analyse the data and compute the design variables to be used in the semi-probabilistic calculation, is almost equal to the effort that it takes to set up a full probabilistic calculation, if not greater. I would say that DA-2 should be preferred over DA-1 for coastal structures, even though this opinion is based on a single case study only. It is worthwhile to mention that the semi-probabilistic approach which has been developed by PIANC several years ago has also not been widely accepted yet, probably for the same reason.

The case study in this report and best practice in the field both indicate a preference of DA-2 over DA-1 for the design of coastal structures.

7.2 CONCLUSIONS

This research set out to answer the following question:

How does the draft Eurocode on wave and current action change the design of coastal structures compared to commonly used existing design methods?

The introduction of the new document prEN1991-1-8 should result in more conformity regarding the treatment of sea condition parameters and their accompanying uncertainties in design calculations. However, the new (draft) Eurocode in its current form does not seem to achieve this objective.

In addition, the semi-probabilistic approach is still a time-consuming method when applied to breakwater elements, while it is relatively easy to set up a full probabilistic calculation. Hence, it is questionable whether it has a positive impact on the design of coastal structures compared to commonly used existing design methods (see e.g. the Rock Manual [Ref. 6]).

Nevertheless, it can be viewed as a positive development that there will be one general document to consult for the design of coastal structures. Most importantly, standardised levels of safety and return periods are now available with the introduction of prEN1991-1-8, even though their interpretations are not always straightforward. With a couple of simple adjustments, many of the flaws of the draft Eurocode can already be taken away.

Four conclusive statements can be made with regards to the draft Eurocode and its shortcomings.

CONSISTENCY IN USE OF PREN1991-1-8

A consistent use of the semi-probabilistic approach in the draft Eurocode cannot be guaranteed, as one user might interpret its content differently than another user.

For the armour layer consisting of rock, these misinterpretations might pile up to a factor of approximately 2.5 (D_{n50} of 1.20 m vs. D_{n50} of 2.90 m). For the case study of IJmuiden, the interpretations that seem most logical would lead to an unsafe design outcome.

Some of the room for interpretation arises from inconsistencies within the document itself. Examples include:

- There is a difference in the return period values presented in Table 4.3 compared to Tables A.4&A.5
- The magnitude of the partial factor that should be used is poorly explained
- There is ambiguity in the description of the consequence class that needs to be worked with
- It is unclear whether the wave height or force should be implemented as the action variable (and should thus be factored) when both are possible
- It not explained what the characteristic value should be in case of a force being the action variable

Although most of these items can quite easily be repaired, they should definitely be addressed before prEN1991-1-8 can be adopted as a definitive version.

Moreover, inconsistencies between the draft Eurocode and other documents are not resolved. For example, from the prescription in the draft Eurocode it is deduced that no safety margins should be incorporated in response formulae for the design format DA-1, as uncertainties are covered for by partial safety factors. Nevertheless, for some design equations it is highly recommended to do so in the manuals in which these equations are described, and it is unclear which of the two should then be followed.

DESCRIPTION OF HYDRAULIC ACTIONS

The descriptions in prEN1991-1-8 on how to determine wave properties and water levels are not specific enough.

The translation of the concepts in Eurocode EN1990, which describes the basis of structural design, to hydraulic engineering (and thus to Eurocode prEN1991-1-8) comes with unclarities. The combination of actions is not so straightforward, as there is often a dependency between several sea condition parameters. There should be a clear instruction of how to deal with this, but instead concepts like correlation or the relationship between wave height and wave period are poorly described.

In addition, sea conditions will vary across the globe, which makes it impossible to work with standardised loads, as can be done for most types of loads in EN1991. This requires analysis of the available data, but a solid description of how this must be performed lacks. Based on the test case it appeared that the following aspects that are relevant to the final design are important to describe more clearly:

- The extreme distributions that may be applied, especially for water levels
- On the basis of which criteria an extreme distribution should be selected
- The threshold level and event separation time to be applied in the extreme value analysis
- Whether the storm duration is a given, or whether this should be extracted from the data (and how this should then be done)
- In what cases storm surge and tide should be treated independently and in what cases they may be treated in combination
- The climate scenario that should be considered when determining the sea-level rise
- Whether to use the central statistical estimate or a certain non-exceedance value when extrapolating
- Etcetera

It is true that these matters are also play a role in existing design methods, but when the purpose of prEN1991-1-8 is to increase uniformity regarding the determination of hydraulic actions, it can not be the case that there is still so much unclarity.

APPLICATION OF PARTIAL SAFETY FACTOR METHOD

While the partial safety factor method should deal with the uncertainties related to the estimation of sea condition parameters and design of coastal structures, the application of it raises some questions that are left unanswered by the draft Eurocode.

This conclusion can be supported from multiple perspectives. Firstly, the basis of structural design described in EN1990 uses both a partial load factor as well as a partial resistance factor. For coastal structures, many of the failure mechanisms are related to hydraulic stability and not to internal strength, which is why it is not immediately clear what part of the structural response is related to resistance and which partial factor belongs to this. In order to fit into the Eurocode framework, the definition of a characteristic resistance value should be described more elaborately in prEN1991-1-8, and indicative values for a partial resistance factor should be provided.

Anyway, the absence of a partial resistance factor means that we are left with a system in which a partial factor is applied to the wave height only. The DA-2 calculations pointed out that especially for armour stability the wave height has a great influence on the design outcome, so from this point of view it does make sense to apply a partial factor on the wave height. However, the same calculations also showed that model uncertainties and the water level (for the crest height and crown wall design) greatly affect the design outcome. This is barely taken into account when solely using the factoring of the wave height as a safety mechanism. In fact, the water level as a hydraulic action is underexposed throughout the entire draft Eurocode, as a partial factor should never be applied to it, and it receives few attention in the clause of prEN1991-1-8 which describes hydrodynamic conditions. This seems strange, as the water level is the underlying sea condition parameter for all hydraulic loads.

Thirdly, factoring the wave height leads to some difficulties in itself. The wave height is often present multiple times in design equations, either directly or indirectly through its influence on the wave period or wavelength. It is confusing for the designer how to handle this. In addition, the influence of the partial factor may vanish entirely in case of depth limitations, for which the wave height has a physical upper limit. It is true that the wave height uncertainty is not relevant any longer in this case, but other uncertainties still exist, such as response model uncertainties or sea climate uncertainty, that are then not covered for.

Finally, the Serviceability Limit State-(Limited Damage) is an important limit state for coastal structures. The partial factor method does not do justice to SLS-(LD), as partial factors are by default set to a value of 1 in EN1990, meaning that safety is predominantly incorporated through return period values in said limit state. This raises even more doubts about the reliability of the proposed partial factor method.

RELEVANCE OF SEMI-PROBABILISTIC APPROACH

The results obtained with the semi-probabilistic approach DA-1 did not resemble the results obtained with the full probabilistic approach DA-2, which can be viewed as the 'most correct' design outcome.

Breakwater element	**Limit state**	**DA-0 Deterministic Result**	**DA-1 Semi-probabilistic Result**	**DA-2 Full probabilistic Result**
Armour layer – rock size	SLS-(LD)	2.21 [m]	1.74 [m]	2.03 [m]
Armour layer – rock size	ULS	1.63 [m]	1.85 [m]	1.52 [m]
Armour layer – artificial units	SLS-(LD)	2.10 [m]	1.25 [m]	1.59 [m]
Armour layer – artificial units	ULS	2.07 [m]	1.73 [m]	1.72 [m]
Crest height	SLS-(LD)	13.27 [m+NAP]	10.35 [m+NAP]	12.42 [m+NAP]
Crown wall – base thickness	ULS	2.25 [m]	1.39 [m]	1.89 [m]

For the case study of IJmuiden, Design approach DA-1 structurally underestimated the required size or height of breakwater elements compared to the DA-2 approach for the Serviceability Limit State-(LD). For the Ultimate Limit State, there is no such similarity for the various breakwater elements when comparing DA-1 and DA-2. The rock armour layer provided an overestimation, the artificial unit size was spot on, and the base thickness of the crown wall gave an underestimation.

Finally, it can be noticed that DA-0 structurally overestimates the required size or height of breakwater elements compared to the DA-2 approach, for SLS-(LD) as well as ULS. Using this approach is thus perhaps conservative, but the structures that are designed with this approach will at least measure up to the target reliability.

It should be noted that full correlation has been assumed in DA-0 and DA-1 (which is a conservative assumption), the actually intended outcome of these design approaches will be somewhat lower. Taking this into account, you might be able to conclude that the deterministic (DA-0) approach gives results closer to the results of the full probabilistic (DA-2) approach than the semi-probabilistic (DA-1) approach does.

It has already been discussed that the difference in these results does not necessarily mean that the entire DA-1 framework is wrong, as the differences may very well originate from wrong assumptions in the calculations, or misinterpretations of what is in the draft Eurocode, which can still be fixed before the definitive version is introduced.

However, the case study did point out that the deterministic approach used in current design practice estimates the full probabilistic approach to a reasonable extent, while it raises less questions than the semi-probabilistic approach. Even more importantly, the effort it took to set up a full probabilistic calculation is comparable to the effort required for the semi-probabilistic calculations, because of the extensive analysis that is required for a semi-probabilistic approach anyway due to varying sea climates. The fact that both the deterministic approach and full probabilistic approach have been embraced into the draft Eurocode has thus been a wise decision. It only raises the question why the semi-probabilistic approach is still so desperately being stuck to as the default approach, as the semi-probabilistic approach is perhaps not very practical to use for coastal structures such as breakwaters.

7.3 RECOMMENDATIONS

GENERAL RECOMMENDATION

Following the conclusions, it is advised to re-evaluate the relevance of a semi-probabilistic approach in the light of coastal structures.

One of the opportunities that is presented by prEN1991-1-8, is the inclusion of the full probabilistic approach into the design framework, which is given as an option in the draft Eurocode. It is recommended to shift (at least part of) the attention towards the full probabilistic approach, and explore the possibilities of making it the default approach for coastal structures, as it raises fewer questions than the semi-probabilistic approach and deals with uncertainties more extensively. For this to work, the description of distributions related to hydrodynamic loads and other model/resistance parameters needs to be more elaborate, and it should be described more clearly how to interpret the safety levels.

SPECIFIC RECOMMENDATIONS RELATED TO DA-1 IN PREN991-1-8

Instead of focusing on the full probabilistic approach, it is of course also possible to improve the semi-probabilistic approach as it is currently proposed by prEN1991-1-8. When this is preferred, it is recommended to include a more systematic explanation of the DA-1 format in EN1991-1-8, describing the characteristic values, partial factors and safety margins to be adopted in design without any ambiguity. More specifically, this comes down to:

- Resolving the inconsistencies within the document, such as the conflicting return periods at different places in the draft Eurocode
- Not only giving characteristic values to be adopted for the sea condition parameters, but also for the forces and for the resistance parameters
- Advising on whether to use the force or the sea condition as an action variable for every element of a structure
- Giving more guidance on the dependence between sea condition parameters, e.g. indicate how ‘moderate correlation’ can quantitatively be determined

- Resolving the room for interpretation related to the description of hydraulic actions, such as suggesting when to treat storm surge and tide in combination or separately with quantitative justification, and stating how the storm duration should be determined
- Describing in more detail how to perform an extreme value analysis, e.g. present rules of thumb for the selection of a threshold value, and set out criteria for the selection of an extreme distribution
- Increasing the amount of attention given to water level as an action variable
- Describing in more detail how to adjust other wave properties as a result of factoring the wave height
- Re-evaluating the magnitude of the characteristic return periods, especially for SLS-(LD)
- Including a proper definition of the Serviceability Limit State-(LD), in particular relative to the other limit states
- Describing in more detail how to set up a joint probability density function, or even providing a way of dealing with correlated variables that is easier to implement.
- Etcetera

As long as (the majority of) these issues are not resolved, it is advised to stick to the use of existing design methods, since both a deterministic approach as well as a probabilistic approach are more practical to design with than the semi-probabilistic approach as described in prEN1991-1-8. Nevertheless, when the issues are resolved, the draft Eurocode could become a valuable tool in the design of coastal structures.

FURTHER STUDIES

Apart from the descriptions being insufficient for the DA-1 method to be properly used, it might also be interesting to investigate the effect of certain adjustments to the semi-probabilistic approach. The scope of this investigation could for example be:

- The incorporation of a safety mechanism that holds for both SLS-(LD) as well as ULS
- The application of a partial load factor onto another parameter than the wave height
- The inclusion of a partial resistance factor for hydraulic stability-type failure mechanisms
- The possibility of working with standardised loads and combinations for hydrodynamic actions

Besides that, it is recommended to work out more test cases with prEN1991-1-8, as it is difficult to assess the quality of the draft Eurocode based on just a single case study. Studies that are probably instructive are:

- A design case in shallow water conditions
- A design case with complex bathymetry
- A design case in which limited data is available
- A design case with a low wave height-water level dependency

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APPENDICES

Appendix A: Relevant tables from prEN1991-1-8 (issued June 2020)

Appendix B: Table A.6.8 from updated Annex A.6 (issued March 2021)

Appendix C: Elaboration of response formulae choice

Appendix D: Additional information Bathymetry

Appendix E: Additional information Correlation analysis

Appendix F: Additional information Hydraulic boundary conditions

Appendix G: Additional information Offshore-nearshore transformation

Appendix H: Sensitivity analysis Armour layer – rock

Appendix I: Sensitivity analysis Armour layer – artificial units

Appendix J: Additional considerations Crest height

Appendix K: Accropode Design Guide Table

Appendix L: Examination of crown wall design

Appendix M: Additional guidance DA-2

Appendix N: PIANC-Calculation

APPENDIX A: RELEVANT TABLES FROM PREN1991-1-8 (ISSUED JUNE 2020)

Table 4.1 - (NDP) – Consequence classes and coastal structure examples

Consequence class	Qualification of consequences		Coastal structure examples
	Loss of human life or personal injury [a]	Economic, social or environmental consequences [a]	
CC4 – Highest	Extreme	Huge	National or regional scale flood protection structures. Structures providing a critical role at a nuclear facility.
CC3 – Higher	High	Very great	Public access structures carrying large numbers of people; residential buildings or supporting properties; or with hazardous sea conditions beneath, e.g. piled jetties/ decks or floating structures. Nationally significant port/ terminal structures, e.g. breakwaters protecting major ports or piled jetties/ decks for import/ export of critical goods (such as energy supplies).
CC2 – Normal	Medium	Considerable	Regional or local port/ terminal structures, e.g. breakwaters protecting small fishing ports/ leisure marinas or floating structures with occasional public access (and limited numbers of people).
CC1 – Lower	Low	Small	Structures with limited access (for trained personnel only) and supporting or protecting non-critical infrastructure or properties, e.g. a single pile supporting a (non-critical) navigation marker.
CC0 – Lowest	Very Low	Insignificant	Structures with no access, not supporting any infrastructure or property, and/ or with no significant environmental protection role.

[a] The consequence class is chosen based on the more severe of these two columns.

Table 4.2 - (NDP) – Design service life for coastal structures

Design service life, T_{life}	Coastal structure examples
years	
100	Common port infrastructure including breakwaters for ports of nationally significant strategic or economic value. Infrastructure for regional flood defence or coast protection.
50	Common port infrastructure for commercial and industrial ports including reclamation, shore protection, breakwaters, quay walls. Infrastructure for local flood defences or coast protection.
25	Structures dedicated to non-renewable natural resources, petrochemicals or similar industrial or commercial applications (such as open-piled jetties, mooring and berthing dolphins).
≤ 10	Temporary structures such as construction material import/ export facilities, temporary works during construction such as cofferdams, other structures with short life such as for a one-off event, or the structure itself during construction.

Table 4.3 - (NDP) – Design Return Period[b]

Limit State Consequence Class[a]	Design service life (T_{life})			
ULS	**< 10 years**	**25 years**	**50 years**	**100 years**
CC3 – Higher	≥ 200 year RP	≥ 500 year RP	≥ 1000 year RP	≥ 2000 year RP
CC2 – Normal	≥ 80 year RP	≥ 200 year RP	≥ 400 year RP	≥ 800 year RP
CC1 – Lower	≥ 30 year RP	≥ 75 year RP	≥ 150 year RP	≥ 300 year RP
SLS-(LD)[c]	**< 10 years**	**25 years**	**50 years**	**100 years**
CC3 – Higher	≥ 100 year RP	≥ 250 year RP	≥ 500 year RP	≥ 1000 year RP
CC2 – Normal	≥ 20 year RP	≥ 50 year RP	≥ 100 year RP	≥ 200 year RP
CC1 - Lower	≥ 10 year RP	≥ 25 year RP	≥ 50 year RP	≥ 100 year RP

[a] Consequence Classes 0 (Lowest) and 4 (Highest) are also described in EN 1990 for use in certain circumstances but are left outside the scope of this document as their use should be occasional and warrants special consideration, i.e. outside the scope of normal application.

[b] Where clear evidence exists that a modest or greater degree of correlation between sea conditions is expected the Return Period used should be that for dependent (joint probability) sea conditions, e.g. of waves, water-levels and/ or currents (or action arising from). Where no correlation is expected, or in some cases where the structure response can be proved to not be sensitive to combined sea actions, the independent sea action may be used, e.g. wave action only.

[c] Limited-Damage serviceability limit state where the structure is only slightly damaged and it is deemed appropriate (in economic, environmental, social or other respects) to undertake repairs; used mainly for the design of engineered mound protection.

Table 4.4 – Limit states for common coastal structures

Coastal structure type	Structure sub-type	Limit state	Example
Fixed cylindrical structures	Cylindrical piles	ULS	See Clause 6
		SLS	See <prEN1990:2020>
	Deck structures	ULS	See Clause 6 and <prEN1990:2020>
		SLS	See Clause 6 and <prEN1990:2020>
	Single erected cylindrical structure	ULS	See Clause 6
		SLS	See <prEN1990:2020>
Mound breakwater	Two-layer armour	SLS-(LD)	SLS-(LD) should relate to a limited damage of armour layer for rock or of artificial units. ULS may also be checked for higher return period sea conditions, normally for the armour layer of rock or of artificial units. See 7.3 for further details.
	Single-layer armour	ULS SLS-(LD)	ULS should relate to damage in the armour layer of rock or of artificial units. SLS-(LD) should relate to a limited damage of the armour layer. See 7.3 for further details.
	Crown-wall or another monolithic concrete element	ULS SLS-(LD)	ULS should apply when the monolithic element is large enough such that it cannot deform without significant fracture. SLS-(LD) should relate to a limited damage of the armour layer. See 7.4.7 for further details
	Combination of above	-	See 7.4.7
Vertical breakwater	Caisson or other vertical wall	ULS	ULS is normally the governing design state, with reserve in sliding resistance and more in stabilising moment. A small amount of sliding can be acceptable under certain conditions. See 8.3 for further details.
	Foundation mound	SLS-(LD)	See mound breakwater in this Table
	Combination of above	-	See 9.3
Coastal embankment	Two-layer armour	SLS-(LD) ULS	SLS-(LD) should relate to a limited damage of the armour layer of rock or of artificial units. ULS may also be checked for higher return period sea conditions, normally for the armour layer of rock or artificial units. ULS should be checked for seawalls or for structural elements of revetments requiring such treatment. See 10.3 for further details.
	Single layer armour (incl blocks)	ULS SLS-(LD)	ULS should relate to a damage of the armour layer of rock or artificial units. SLS-(LD) should relate to a limited damage of the armour layer. See 10.3 for further details.
Floating structure	Single chamber float	SLS	See <prEN1990:2020; Annex A.6>
		ULS	See <prEN1990:2020; Annex A.6>
	Multiple bulkhead float	SLS	See <prEN1990:2020; Annex A.6>
		ULS	See <prEN1990:2020; Annex A.6>

Table 4.5 – (NDP) Design situations and examples of applications

Design situation	Conditions	Examples of applications for coastal structures
Persistent	Normal use and exposure. prEN1990:2020: During everyday use.	For coastal structures this can include typical conditions during everyday use but should also include conditions experienced infrequently, e.g. return period sea conditions with an acceptable probability of occurrence and consequence during the design life. For coastal structures flooding is generally considered in a persistent design situation with an appropriate admissible value (overtopping discharge, rise of water level).
Transient	Temporary use and exposure during a period much shorter than the design life of the structure.	For coastal structures performance during execution and repair shall be considered. For coastal structures temporary environmental influences should normally be considered, e.g. while towing a caisson from dry dock to installation location.
	prEN1990:2020: During execution, repair or temporary environmental influence.	
Accidental	Exceptional conditions or exposure. prEN1990:2020: During flooding, fire, explosion, or impact: or local failure.	Flooding may be considered in an accidental design situation with an appropriate admissible value that will be more tolerant than the one in the persistent design situation. Unplanned ship impact and marine operations can represent an accidental design situation for coastal structures.
Seismic	Exceptional conditions during a seismic event. prEN1990:2020: During an earthquake.	For some coastal structures such as ports/ terminals of national significance, safety under seismic conditions is critical to allow emergency access and supplies for post-earthquake response.
Fatigue	Conditions caused by repeated load cycles. prEN1990:2020: Owing to traffic loads on a bridge, wind induced vibration of chimneys, or machinery induced vibration.	For coastal structures fatigue design situations will typically apply to structures constructed from materials susceptible to fatigue with cyclical loading due to frequent and/ or persistent wave/ current conditions and, e.g. current actions on steel-piled structures.

Table 4.6 - HEA level selection matrix

Consequence class	Hydrodynamic uncertainty [1]		
	LOW[2]	MEDIUM	HIGH[3]
CC0 Lowest[4]	1	1	1
CC1 Lower	1	1	2
CC2 Normal	1	2	2
CC3 Higher	2	2	3
CC4 Highest[4]	2	3	3

[1] Hydrodynamic uncertainty is defined by the level of understanding of environmental sea conditions at the site of interest and will depend on the relative complexity/ severity of physical processes (boundary condition generation and transformation to the site) and also the quality and quantity of available data (in the form of measurements, models, semi-empirical or empirical estimates).

[2] Examples of low hydrodynamic uncertainty may include tidal range < 1m, surge < 0,5m, fetch-limited seas (with fetch < 10 km), uniform currents with spring tide velocities < 1m/s, regular bathymetry or high quality time-series data of relevant sea condition parameters covering several decades at more than one location in the area of interest and high quality (recent) topo-bathymetric data.

[3] Examples of high hydrodynamic uncertainty may include tidal range > 3 m, surge > 1,0 m, ocean seas (swell and wind-waves), non-uniform currents (stratified) and/ or tide or surge current velocities > 1m/s, irregular bathymetry (e.g. reefs or sub-sea canyons) or limited quality and/ or duration of environmental sea condition parameters or low resolution or historic topo-bathymetric data.

[4] prEN 1990:2020 states that for consequence classes CC0 and CC4 'alternative provisions to those given in the Eurocodes may be used'.

Table 4.8 – Minimum Design Approach (DA) level

HEA Level	Low-to Medium Structure Design/Response Uncertainty[1]	High Structure Design/Response Uncertainty[1]
HEA-1	DA-0	DA-1 or DA-2[2]
HEA-2	DA-1 or DA-0[2]	DA-2 or DA-4[3]
HEA-3	DA-2 or DA-4[3]	DA-2 and DA-4[3]

[1] Where the term uncertainty is defined in NOTE 2 above.

[2] When no partial factor values are readily available.

[3] Where insufficient data is available to support a probabilistic approach, may refer to a part of the structure, e.g. breakwater roundhead or crown wall.

DA-0: Using a deterministic approach with return periods and appropriate sensitivity testing of key parameters, based on application of semi-empirical structure response formulae; global safety factors are applied.

DA-1: Using semi-probabilistic partial load and resistance factors.

DA-2: Using a fully probabilistic approach with allowable probabilities of failure or beta indexes

DA-3: Using a risk-informed method of socio-economic optimisation to determine the optimum probability of failure of the considered structure; this approach is not included here but is recommended by some National Standardisation Bodies.

DA-4: Using a design assisted by testing approach in combination with one or more of DA-0, DA-1, DA-2 (or DA-3).

Table 7.1 - (NDP) Values of damage parameters

Sub-system	Unit	Damage parameter (1)	Slope	SLS-(LD) (2)	ULS (3)
Two-layer armour	Rock	D	1:2 – 1:3	0-5 %	20 %
		S_d	1:1.5-1:2	2	8
		S_d	1:3	2	12
		S_d	1:4-1:6	3	17
	Tetrapods	N_d		~0 %	
		N_{od}	1:1.5	0-0.5	1.5
	Cube	N_d	1:1.5-1:2	~0 %	
		N_{od}	1:1.5	0-0,5	2
	Dolosse	N_d	1:1.5	0-2 %	15 %
		N_{od}			2.8 (4)
	Acropode	N_{od}	1:1.33	0	0.5
Single-layer armour	Rock§	S_d	1:1.3-1:1.4	2	8
	Acropode	N_d	1:1.33	0	10%
	Cube	N_{od}		0	0,2
Toe berm (5)	Rock	N_{od}		0.5	4

(1) D: percentage of eroded volume from the middle of the crest down to 1H_s below still water level (SWL)

S_d or S: A_e/D_{n50}^2, where A_e the eroded area around SWL, D_{n50} the median nominal diameter of the armour stones

N_d: percentage of units moved out of place between levels

SWL ± 6D_n for cubes and Dolosses

SWL + 5D_n & SWL -9D_n for Accropodes

where D_n nominal diameter of unit

N_{od}: ratio of number of units displaced out of armour layer to the width of tested section normalized with respect to D_n, or equivalently to D_{n50} for rock.

(2) Few units are displaced.

(3) The underlayer (or filter layer) is exposed to direct wave attack.

(4) For a packing density coefficient φ=0.83 and a waist-to-height ratio r=0.32.

(5) For a typical toe size of about 3-5 stones wide and 2-3 stones high.

§ Typically for revetments

Table 13.1 - (NDP) Safety levels (RL) expressed in β-values for a 50-year reference period

Consequence class	**Limit State**		
	ULS	**SLS-(LD)**[a]	SLS[b]
CC1	3.5	3.2	
CC2	3.8	3.5	1.6
CC3	4.1	3.8	

[a] Defined in Table 4.3

[b] Defined by the discontinuation of the operational and functioning ability of the structure

NOTE 1 For the definition of CCs refer to EN1990 and Table 4.1 of this Standard.

NOTE 2 Link of the probability level with the reliability index β can be found in EN1990, Table C.3.1.

NOTE 3 The reliability requirements can produce in each case the corresponding probabilistic properties of the environmental input parameters. However, the empirical formulae suggested in Annex E to Annex H involve specific representatives of those parameters. For example for the wave height in a sea state Hmo or H1/3 or H1/10 etc. can be involved in a formula; the specific value of any of those input parameters that should be taken into account in a DA-2 application can be calculated through the limit state function $g(\overline{x})=0$ that involves the specific representative actions included in the relevant response formula.

NOTE 4 The β-values of Table C.3.2 EN1990 are compatible with the corresponding values of Table 13.1 (NDP), since the former are associated with the exceedance probability based on a sample representative of extreme values whereas the latter are associated with the exceedance probability based on a sample representative of all values of the relevant parameter over a suitable record length.

NOTE 5 Examples where the SLS target value can be applied include the calculation of a port breakwater crest based on the disruption of port basins due to wave overtopping, similarly in an embankment protecting a coastal road, the direct use of a floating pontoon by humans, etc.

Table A.2 (NDP) Equivalence of safety levels in design approaches

CC	1	2	3
I. Limit State: ULS			
Ia. Reference period (yr)	1		
Return period (yrs) of extreme actions (T_r), Table 4.3 @50yr ref. period DA-0 (1)$	**150**	**400**	**1000**
Annual exceedance probability -extreme events considered (2)	*6.67E-03*	*2.50E-03*	*1.00E-03*
Annual exceedance probability –all events considered (3)*	*4.57E-06*	*1.71E-06*	*6.85E-07*
Reliability index-extreme events considered (β-value) (4)	*2.47*	*2.81*	*3.09*
Reliability index-all events considered (β-value) (5)	*4.44*	*4.64*	*4.83*
Iβ. Reference period (yr)	50		
Exceedance probability -extreme events considered (6)	*2.84E-01*	*1.18E-01*	*4.88E-02*
Exceedance probability -all events considered (7)*	*2.28E-04*	*8.56E-05*	*3.42E-05*
Failure probability -extreme events considered P_f DA-1 (8)	**0.28**	**0.12**	**0.05**
Failure probability -all events considered (9)	*2.28E-04*	*8.56E-05*	*3.42E-05*
Reliability index -extreme events considered β-value (10)	*0.57*	*1.19*	*1.66*
Reliability index -all events considered DA-2β-value (11)	**3.51**	**3.76**	**3.98**
Table 13.1 DA-2	**3.5**	**3.8**	**4.1**
EN1990 Table C.3.2 DA-2	**3.3**	**3.8**	**4.3**

CC	1	2	3
II. Limit State: SLS (LD)			
IIa. Reference period (yr)	1		
Return period (yr) of extreme actions (T_r), Table 4.3 @50yr ref. period DA-0 (1)$	**50**	**100**	**500**
Annual exceedance probability -extreme events considered (2)	*2.00E-02*	*1.00E-02*	*2.00E-03*
Annual exceedance probability –all events considered (3)*	*1.37E-05*	*6.85E-06*	*1.37E-06*
Reliability index-extreme events considered (β-value) (4)	*2.05*	*2.33*	*2.88*
Reliability index-all events considered (β-value) (5)	*4.19*	*4.35*	*4.69*

Continuation of Table A.2 – Equivalence of safety levels in design approaches

IIβ. Reference period (yr)	50		
Exceedance probability -extreme events considered (6)	*6.36E-01*	*3.95E-01*	*9.53E-02*
Exceedance probability -all events considered (7)*	*6.85E-04*	*3.42E-04*	*6.85E-05*
Failure probability -extreme events considered P_f DA-1 (8)	**0.64**	**0.39**	**0.10**
Failure probability-all events considered (9)	*6.85E-04*	*3.42E-04*	*6.85E-05*
Reliability index-extreme events considered β-value (10)	*-0.35*	*0.28*	*1.31*
Reliability index -all events considered β-value (11)	**3.20**	**3.40**	**3.81**
Table 13.1 DA-2	**3.2**	3.5	**3.8**

Key	
(2)	$\frac{1}{T_r}$
(3)	$= (2)\times(\frac{D}{2})\times\frac{1}{24\times365}$, D: mean storm duration, storm history considered as a triangle
(4)	equivalent to (2)
(5)	equivalent to (3)
(6)	$=1\text{-}(1\text{-}\frac{1}{T_r})^{50}$
(7)	$= (6)\times\frac{D}{2}\times\frac{1}{24\times365}$
(8)	equivalent to (6)
(9)	equivalent to (7)
(10)	equivalent to (8)
(11)	equivalent to (9)
*	D=12hr
§	Total safety margin incorporated in actions

Table A.3 - – Principles for the limit state design in DA-1[a]

Limit state	In persistent and transient design situations	In accidental design situations	In seismic design situations	In fatigue design situations
SLS-(LD)	Characteristic combination according to <prEN1990:2020> *or* Frequent combination according to <prEN1990:2020> *or* Quasi-permanent situation according to <prEN1990:2020>	No verification	No verification	No verification
ULS	Fundamental combination according to <prEN1990:2020>	Accidental combination according to <prEN1990:2020>	Seismic combination according to <prEN1990:2020>	Fatigue combination according to <prEN1990:2020>

Table A.4 –(NDP) Return Period of hydraulic sea conditions (design-extreme value) - (Use of independent/marginal distributions)

Consequence Class	Design service life (t_{life})			
	< 10 years	**25 years**	**50 years**	**100 years**
The RP values in this table can be used when combining the sea state components (e.g. wave height, sea level, current velocity) in order to estimate the design (or extreme) value of the sea condition in the fundamental combination of actions. One of the sea state components (the dominant component) is considered with the following RP:				
CC3	[40] y	[100] y	[200] y	[400] y
CC2	[20] y	[50] y	[100] y	[200] y
CC1	[10] y	[25] y	[50] y	[100] y
The other sea state components (the accompanying components) are considered with the following RP[a] :				
CC3	[4] y	[10] y	[20] y	[40] y
CC2	[2] y	[5] y	[10] y	[20] y
CC1	[1] y	[2,5] y	[5] y	[10] y

<These RP values apply to offshore sea conditions, i.e. before propagation to the shoreline>

[a] Fully correlated actions shall be assumed as dominant components

The statistical estimation of the RP value can be based on the upper bound of a given confidence interval, considering the uncertainty due to the limited sampling size.

Table A.5 – (NDP) - Return Period of hydraulic sea conditions (characteristic value) - (Use of independent/marginal distributions)

Consequence Class	Design service life (t_{life})			
	< 10 years	25 years	50 years	100 years
The RP values in this table can be used when combining the sea state components (e.g. wave height, sea level, current velocity) in order to estimate the characteristic value of the sea condition in the characteristic combination of actions. One of the sea state components (the dominant component) is considered with the following RP:				
CC3	[4] y	[10] y	[20] y	[40] y
CC2	[2] y	[5] y	[10] y	[20] y
CC1	[1] y	[2,5] y	[5] y	[10] y
The other sea state components (the accompanying components) are considered with the following RPs :				
CC3	[2] y	[5] y	[10] y	[20] y
CC2	[1] y	[2,5] y	[5] y	[10] y
CC1	[1] y	[1] y	[2,5] y	[5] y
These RP values apply to offshore sea conditions, i.e. before propagation to the shoreline. The statistical estimation of the RP value can be based on the upper bound of a given confidence interval, considering the uncertainty due to the limited sampling size.				

Table A.6 - (NDP) – Return Period hydraulic sea condition (characteristic and design value) – Use of dependent (joint) distributions for moderately to strongly correlated environmental parameters

The heuristic method derived from Turkstra's rule for taking into account dependencies between two variables, for instance "wave height" and "water level", involves the following stages: - using the available statistical data, the joint probability distribution for the wave height and water level is estimated; - curves with equal joint return periods T are determined in the plane (H – wave height, N – water level); - using the marginal laws for H and N, wave height and water level values are determined for the characteristic return periods T related to the dominant sea state component from Table A.4 or Table A.5, denoted (H_T, N_T); - the effect of the action on the structure due to the wave and water level is specified; it may be, for example, the force on a vertical breakwater, the overtopping discharge...; - let's denote (H_T*, N_T*) the point that maximises the effect of the action on the curve with the characteristic return period; - the combinations of leading/accompanying variable actions are formed by calculating the effect of the action with (H_T, N_T*) and (H_T*, N_T). - The most severe of those two combinations is used as an input to check the limit state condition.
The principle of this method may be extended to cases involving more variable parameters simultaneously.

Table A.8 (NDP) — Partial factors on actions and effects of actions for persistent and transient design situations

Action or Effect				**Partial factors**	
Type	Group	Symbol	Resulting effect	For the verification of limit states STR, GEO, EQU, UPL (a)	For the verification of limit states HYD (b)
Variable actions	Hydraulic action	$\gamma_{HYD,\ unfav}$	Unfavourable	1.50 in DC1 and DC 2 1.30 in DC3 1.50/ γ_G in DC4	1.00
		$\gamma_{HYD,\ fav}$	Favourable	1.00	1.00
	Other variable actions	γ_Q		Refer to the Annex giving the partial factor values and consequence factors to be used for the variable action under consideration, according to the limit state and to the design case (DC) under consideration	
Permanent actions	All	γ_G		Refer to the Annex giving the partial factor values and consequence factors to be used for the permanent action under consideration, according to, the limit state and to the design case (DC) under consideration	
(a) Limit states for concrete or steel structures (vertical breakwaters, pipes, fixed or floating structures) refer usually to the categories EQU, STR, UPL and GEO (examples : crack width of concrete, settlement, shear stress failure, overturning sliding, loss of bearing capacity).					

APPENDIX C: TABLE A.6.8 FROM UPDATED ANNEX A.6 (ISSUED MARCH 2021)

Action or effect of action (load)				**Partial factors**				
Type	**Name of action**	**Symbol**	**Resulting effect**					
Design case				**DC1**[a]	**DC2(a)**[b]	**DC2(b)**[b]	**DC3**[c]	**DC4**[d]
Formula				**(8.4)**	**(8.4)**		**(8.4)**	**(8.5)**
Permanent action (G_k)	All	γ_G	unfavourable/ destabilizing	$1{,}35K_F$	$1{,}35K_F$	1,0	1,0	G_k is not factored
	All	$\gamma_{G,stb}$	stabilizing [f]	not used	1,15 [e]	1,0	not used	
	All	$\gamma_{G,fav}$	favourable[g]	1,0	1,0	1,0	1,0	
Variable action (Q_k)	Waves[h]	γ_{Qz}	unfavourable	$1{,}35K_F$[i]	$1{,}35K_F$[i]	$1{,}35K_F$[i]	1,3	$1{,}35K_F$[i] 1,0[k]
	Currents[j]	γ_{Qc}	unfavourable	$1{,}5K_F$	$1{,}5K_F$	$1{,}5K_F$	1,3	$1{,}35K_F$ 1,0[k]
	Water levels	γ_{Qw}	unfavourable	not factored				
	Other variable actions	γ_Q	unfavourable	$1{,}5K_F$	$1{,}5K_F$	$1{,}5K_F$	1,3	$\gamma_{Q,1}/\gamma_{G,1}$[l]
	All	$\gamma_{Q,fav}$	favourable	0				
Effects of actions (E)		γ_E	unfavourable	Effects are not factored				1,5/1,35[m] $1{,}35K_F$[n,o] $1{,}5K_F$[p]
		$\gamma_{E,fav}$	favourable	Effects are not factored				1,0

a Design Case 1 (DC1) is typically used for the structural and geotechnical design. It is used for the hydraulic design of coastal structures loaded by waves and currents, unless the effect of actions of waves and currents are assessed with the Time Series Full Transfer Method.

b Design Case 2 (DC2) is typically used for the combined verification of strength and static equilibrium, when the structure is sensitive to variations in permanent action arising from a single-source. Values of γ_F are taken from columns (a) or (b), whichever gives the less favourable outcome.

c Design Case 3 (DC3) is typically used in some countries for the design of slopes and embankments, spread foundations, and gravity retaining structures. See EN 1997 for details.

d Design Case 4 (DC4) is typically used when it is relevant to apply partial factors on actions together with a partial factor on effects of actions (see EN 1997 for details). It is used for the structural and geotechnical design of coastal structures loaded by waves and currents. It is used for the hydraulic design of coastal structures with the Time Series Full Transfer Method (see EN 1991-1-8 for details).

e The values of $\gamma_{G,stb}$ = 1.15 and 1.0 are based on $\gamma_{G,inf}$ = 1,35 ρ and 1,2 ρ with ρ = 0.85.

f Applied to the stabilizing part of an action originating from a single source.

g Applied to actions whose entire effect is favourable and independent of the unfavourable action.

h The partial factor is applied to the wave height (see EN 1991-1-8 for details).

i This value of γ_{Qz} is given for offshore waves and can be adapted when the waves are assessed nearshore. According to 8.1 (3), the National Annex can alternatively determine directly a design value of the wave height based on a specific design return period.

j The partial factor is applied to the square of the current's velocity (i.e. the force due to the current) (see EN 1991-1-8 for details).

k Those specific values of $\gamma_{Qz}$and γ_{Qc} in DC4 are to be used with the Time Series Full Transfer Method.

l $\gamma_{Q,1}$ = corresponding value of γ_{Q} from DC1 and $\gamma_{G,1}$ = corresponding value of γ_{G} from DC1.

m This value of γ_{E} is used for the structural and geotechnical design of coastal structures loaded by waves and currents (see EN 1991-1-8 for details) unless the effect of actions of waves and currents are assessed with the Time Series Full Transfer Method.

n This value of γ_{E} is used for the design of transversally loaded piles and embedded retaining walls and (in some countries) gravity retaining structures, where live loads are transmitted to the structure by the ground (see EN 1997 for details).

o This value of γ_{E} is used for the hydraulic design of coastal structures when the effect of actions of waves and currents are assessed with the Time Series Full Transfer Method, together with the specific values of of $\gamma_{Qz}$and γ_{Qc} described under note (k) above.

p This value of γ_{E} is used for the structural and geotechnical design of coastal structures when the effect of actions of waves and currents are assessed with the Time Series Full Transfer Method together with the specific values of of $\gamma_{Qz}$and γ_{Qc} described under note (k) above.

APPENDIX C: ELABORATION OF RESPONSE FORMULAE CHOICE

STABILITY OF ROCK ARMOUR LAYER

The new draft Eurocode mentions three formulae related to rock stability in the armour layer, being the Van Gent formula, the Van der Meer formula and the Hudson formula. The Van der Meer formula has been preferred over the Hudson formula, as it also includes phenomena such as the wave period and the number of incoming waves.

The Van der Meer formula for deep or shallow water can subsequently be chosen.

This consideration has been made based on the ratio h/L, which determines the difference between deep and shallow water. For a wave height of 7.03 m and a depth of 17.81 m (both belonging to a Return Period of 100 years), this ratio equals 0.17 (when assuming that the offshore wavelength can be used for L). This is smaller than $\frac{1}{2}$ [limit deep water] but larger than $\frac{1}{20}$ [limit shallow water], suggesting that we are in water of intermediate depth.

We are thus dealing with a borderline case for the breakwaters of IJmuiden when it comes to selecting the response formula to be used, illustrated by Figure C.1:

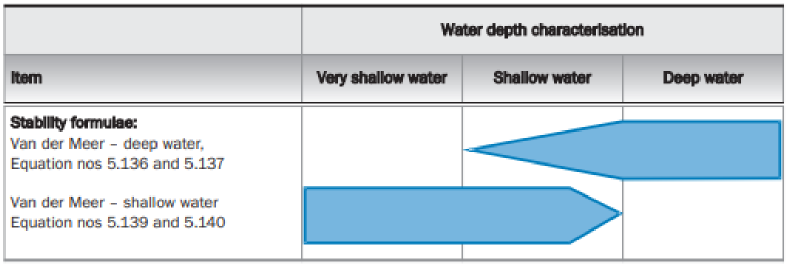


Figure C.1: Specifications on when deep water or shallow water Van der Meer equations should be used according to Table 5.29 in The Rock Manual [Ref. 6]

As the new Eurocode does not provide unambiguity with regards to this topic, the choice has been made to use the **deep-water Van der Meer formulae**, since there is more design experience with this formula.

Most of the parameters used in the Van der Meer-formula fall within the ranges of application, apart from the fictitious wave steepness and the relative water depth. According to prEN1991-1-8, this would actually mean that physical model tests need to be carried out, but this has been left outside the scope of the thesis.

STABILITY OF SLOPE OF ARTIFICIAL UNITS

It has been chosen to work with a fixed value for the stability number. The slope does not explicitly occur in the formula, but damage parameters have only been determined based on a small range of slopes. For Accropodes, stability values have been determined for a slope of 1:1.33, but these values are not expected to change for slightly different slopes, such as the 1:1.5 slope in the design.

According to the Rock Manual [Ref. 6], the behaviour of Accropodes is similar to that of more recently developed single-layer units such as Core-loc and Xbloc, so everything that is discussed relevant to Accropodes will also approximately be true for these types of units.

CREST HEIGHT BASED ON WAVE OVERTOPPING

The wave overtopping formula that has been selected is valid for steep slopes 1:2 to 1:4/3 [Ref. 11]. The breakwater design armoured with concrete artificial units is within this range. Moreover, the roughness factor to be used in the formula has been derived for breaker parameters in a fairly small range, but this has not been taken into consideration.

WAVE FORCING ON CROWN WALL

The draft Eurocode suggests working with either Pedersen's method [Ref. 24] or Martin's method. For Pedersen's method, the configuration of the crown wall that has been come up with, falls outside the range of application of the formula, because H_{m0}/h does not lie in between 0.16 and 0.35. However, main issue lies in the fact that a stable solution can not be arrived at when using Pedersen (see also Appendix L), and not in the fact that a single parameter does not fall into its range of validity. It was found that Martin's method is also not applicable.

In order to still be able to design a crown wall, a non-standard Eurocode response formula has been selected. This response formula is based on wave overtopping [Ref. 22], for which the validity ranges are presented in Table C.1:

	Parameter	**Validity ranges**	**Value in design**	**Within range?**
1	ξ_{op}	1.39-7.77	3.41	Yes
2	$R_c/(\gamma_f H_{m0})$	1.67-6.55	1.93	Yes
3	$\gamma_f R_{u0.1\%}/R_c$	0.36-1.41	1.16	Yes
4	$(R_c - A_c)/C_h$	0.0-0.59	0	Yes
5	$\sqrt{L_m/B}$	2.64-6.54	4.20	Yes
6	F_c/L_{0p}	0.0-0.03	0.004	Yes
7	$\log Q$	-6.0- -2.78	-2.83	Yes

Table C.1: Validity ranges for [Ref. 22] with computed values for eventual crown wall design

APPENDIX D: ADDITIONAL INFORMATION BATHYMETRY

As was explained in Paragraph 3.4.1.1, bathymetric data has been gathered from Waterinfo Extra. This service directs the user to a chart viewer [Ref. 17], that shows on request the bottom level with respect to NAP for the rivers, lakes and coastal system of the Netherlands. See Figure D.1 for an example of what this looks like:

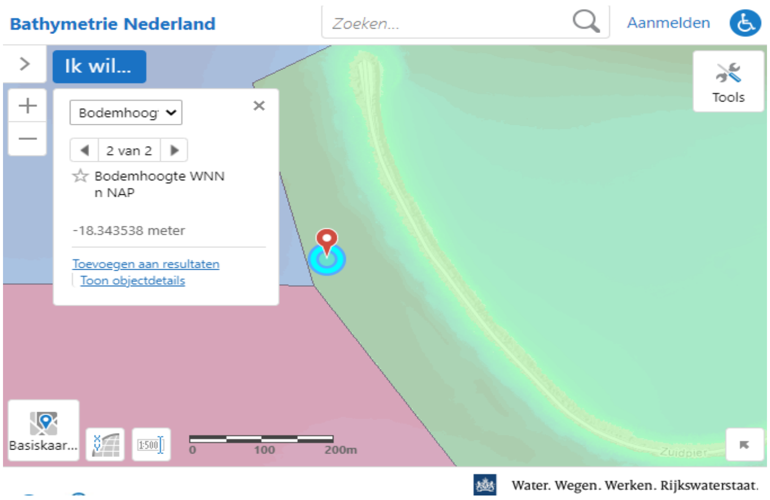


Figure D.1: Bathymetric chart viewer used for determining bottom levels [Ref. 17]

Unfortunately, the service only provides data to about 1.2 km away from the breakwaters, while data was retrieved at about 1.7 km away from the breakwaters. Because of this, the platform Navionics [Ref. 13] has also been used. This platform shows depth contours, with the reference level being equal to Mean Lower Low Water. From Figure F.3, it can be estimated that Mean Lower Low Water is approximately 0.8 m below NAP. Figure D.2 shows the depth contours near the breakwaters of IJmuiden:

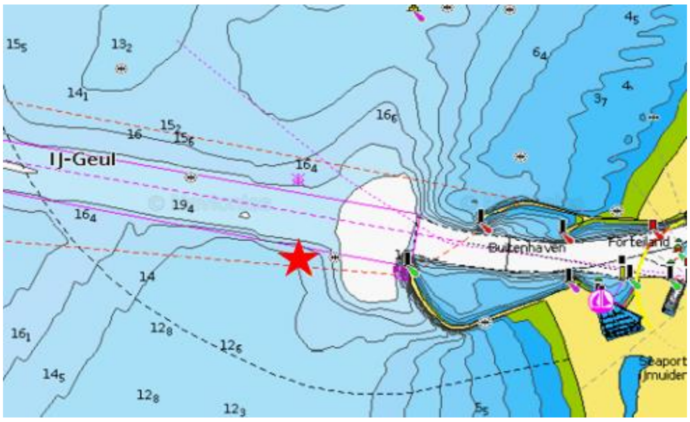


Figure D.2: Depths around the IJmuiden breakwaters, with the star indicating the location of wave and water level measurements [Ref. 13]

It is visible that the location of the measurements lies near the depth contour of 14 m, resulting in a bottom level of about -15 m with respect to NAP. The same bottom level was found at the edge of the measurements from the chart viewer [Ref. 17]. Hence, it has been assumed that for the first 500 m from the measurement location towards the breakwater, the bottom level constantly remains at this level. From the crest of the breakwater towards a certain bottom level, some vertical distance needs to be covered, so it has been assumed that the toe of the structure lies at approximately 50 m from the location of the breakwater crest.

APPENDIX E: ADDITIONAL INFORMATION CORRELATION ANALYSIS

As was mentioned in Section 3.5, the following three aspects are left unanswered by prEN1991-1-8:

- How should the correlation be determined?
- What are the boundaries when speaking of 'less than moderately' and 'moderately or strongly' correlated?
- Should the wave height be correlated to the absolute water level? Or to the storm surge only?

It is thus not specified how the degree of correlation should be determined. In this thesis, the Pearson product-moment correlation coefficient will be used to quantify to the correlation, but different correlation coefficients exist. The Pearson correlation coefficient shows how strong the linear relationship between two parameters is, and can take values between -1 and 1. Values between -1 and 0 indicate negative correlation whereas values between 0 and 1 indicate positive correlation. The higher the absolute value, the stronger the correlation, with 0 meaning no correlation at all and (-)1 denoting fully correlated parameters.

Secondly, it is left open to the interpretation of the user what is meant by 'less than moderately correlated' and 'moderately or strongly correlated'. It is not specified at which value the transition from less than moderately to moderately occurs. One source may tell you that this transition holds for a different value than another source. For instance, the following boundaries should be used according to [Ref. 19]:

Type of correlation	**Lower bound correlation coefficient**	**Upper bound correlation coefficient**
Very weak	0.00	0.19
Weak	0.20	0.39
Moderate	0.40	0.59
Strong	0.60	0.79
Very strong	0.80	1.00

Table E.1: Strength of correlation depending on the absolute value of the Pearson product-moment correlation coefficient according to [Ref. 19]

However, when looking at [Ref. 20], the following values should be used:

Type of correlation	**Lower bound correlation coefficient**	**Upper bound correlation coefficient**
Low	0.00	0.29
Moderate	0.30	0.49
High	0.50	1.00

Table E.2: Strength of correlation depending on the absolute value of the Pearson product-moment correlation coefficient according to [Ref. 20]

The lack of this specification in the new Eurocode may lead to completely different designs, as it determines whether marginal distributions may be used or a joint probability analysis is required. An assumption needs to be made, and in this case the values in Table E.1 are adopted as the proper values.

The third unclarity treated in this Appendix is whether the wave height should be correlated to the water level, or only to the magnitude of the storm surges. The storm surge levels are obtained by subtracting the tide from the water level data.

Two plots have been constructed, both shown in Figure E.1. The left plot shows data points of wave height versus the water level, whereas the right plot shows the data points of wave height versus the storm surge level:

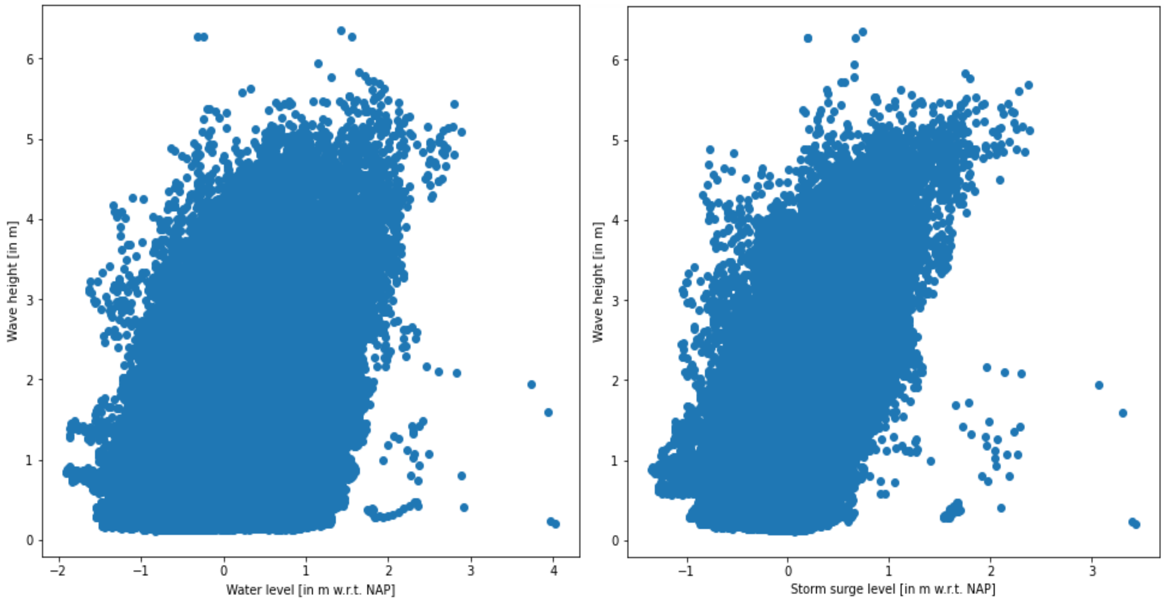


Figure E.1: Scatter plot of wave height versus water level (left) and scatter plot of wave height versus storm surge level (right) for the location of IJmuiden

In both plots, a positive correlation is visible. That is, high waves are more likely to be accompanied by high water levels. From the data in the left plot, a Pearson correlation coefficient equal to 0.30 is calculated, which confirms the positive correlation. However, the magnitude of the coefficient indicates that wave height and water level are less than moderately correlated, according to Table E.1. Nevertheless, a strong correlation would be more logical, as the North Sea has a wave climate dominated by wind, which means that the water level and the waves have the wind as the same driving parameter.

The water level data in the left plot of Figure E.1 include the tidal component, which is independent of the wind and therefore 'pollutes' the data when considering the correlation. The new Eurocode briefly mentions this effect in Clause 13.3.2:

'Also, SL refers mainly to the sea level change due to storm surge that can usually develop a correlation with the wave field through a common driving wind field.'

One could argue that it would therefore make more sense to determine the correlation between the wave height and the storm surge level. The storm surge is the part of the water level affected by the wind, and the correlation coefficient will thus more realistically show how the wind field affects both waves and water level.

From the data in the right plot of Figure E.1, a Pearson correlation coefficient equal to 0.59 is calculated. The magnitude of the coefficient now indicates that wave height and storm surge level are moderately (and almost strongly) correlated. This is also visible in Figure E.1, where the positive correlation seems to be more evident for the plot on the right.

It is assumed that the latter method, correlating the wave height with the storm surge level instead of the water level, is the proper way to determine the dependence between the two parameters. So, for the design of breakwater elements for which a combination of these two sea state parameters is required, the wave height and water level will be treated as being 'moderately or strongly correlated environmental parameters'.

Two remarks should be made about the data that has been used to construct the plots in Figure D.1. Firstly, data has been used for the time period from 2013 to 2021, not for the time period from 2002 to 2021 as was discussed in Paragraph 3.4.1.3. The reason for this is to ensure simultaneous measurements of water level and wave height. Water levels were measured at time intervals of 10 minutes for the entire period of time (2002-2021), whereas wave heights were measured at time intervals of 10 minutes only since 2013. Before that, wave height measurements were taken at a larger time interval. Secondly, the measurements of tidal levels were extracted from a slightly different location than the one specified in Paragraph 3.4.1.2. It is thus not entirely correct to subtract these measurements from the water level measurements. Nevertheless, it is expected that both the aspects mentioned above will not exert significant influence on the degree of correlation.

Finally, a fourth point of discussion is presented, although this point has not been further investigated. The degree of correlation has been determined for all the waves and water levels, but for design purposes we are actually mainly interested in extreme events. Therefore, it could also be the designer’s choice to only consider the correlation of extreme waves.

APPENDIX F: ADDITIONAL INFORMATION HYDRAULIC BOUNDARY CONDITIONS

WAVE HEIGHT

To perform a linear regression analysis on the data set of extreme waves, the following steps need to be followed:

- Arrange the wave heights from lowest to highest and compute the non-exceedance probability P for each data point i with the expression $P_i = \frac{i}{N+1}$; N in this expression is the total number of waves in the data set.
- Translate the non-exceedance probability to an exceedance probability Q: $Q_i = 1 - P_i$.
- For each extremal distribution, a general expression also exists for the exceedance probability. The inverse of this general expression is used, together with a linearised variable specific to that distribution, to fit a straight line with the lowest possible RMSE through the data.
- The intercept A and slope B follow from the fitting procedure. These can be used as estimators for the parameters in the extremal distribution functions. With these parameters known, a wave height can be estimated for every possible exceedance probability, and thus for the return periods of interest.
- For distributions that contain three instead of two function parameters, an assumption first needs to be made for the value of the third parameter before the linear regression can be performed.

The extreme distributions are set out below [Ref. 7]:

Exponential distribution

$$Exceedance\ probability: \quad Q = exp\left(-\left(\frac{H_s - \gamma}{\beta}\right)\right)$$

$$Inverse: \quad H_s = \gamma - \beta * \ln Q$$

$$Linearised\ variable: \quad X_{E,i} = -\ln Q_i$$

$$Linearised\ expression: \quad H_{s,i} = A + B * X_{E,i}$$

$$Estimator\ (slope): \quad \hat{\beta} = B$$

$$Estimator\ (intercept): \quad \hat{\gamma} = A$$

Gumbel distribution

$$Exceedance\ probability: \quad Q = 1 - exp\left[-exp\left(-\left(\frac{H_s - \gamma}{\beta}\right)\right)\right]$$

$$Inverse: \quad H_s = \gamma - \beta * \ln(-\ln(1 - Q))$$

$$Linearised\ variable: \quad X_{G,i} = \ln(-\ln(1 - Q_i))$$

$$Linearised\ expression: \quad H_{s,i} = A + B * X_{G,i}$$

$$Estimator\ (slope): \quad \hat{\beta} = B$$

$$Estimator\ (intercept): \quad \hat{\gamma} = A$$

Weibull distribution

$$Exceedance\ probability: \quad Q = exp\left(-\left(\frac{H_s - \gamma}{\beta}\right)^{\alpha}\right)$$

$$Inverse: \quad H_s = \gamma + \beta * [-\ln(Q)]^{1/\alpha}$$

$$Linearised\ variable: \quad X_{W,i} = [\ln(1/Q_i)]^{1/\hat{\alpha}}$$

$$Linearised\ expression: \quad H_{s,i} = A + B * X_{W,i}$$

$$Estimator\ (slope): \quad \hat{\beta} = B$$

$$Estimator\ (intercept): \quad \hat{\gamma} = A$$

Generalised Pareto distribution

$$Exceedance\ probability: \quad Q = \left(1 + \alpha\frac{H_s - \gamma}{\beta}\right)^{-1/\alpha}$$

$$Inverse: \quad H_s = \gamma + \beta * \left(\frac{Q^{-\alpha} - 1}{\alpha}\right)$$

$$Linearised\ variable: \quad X_{P,i} = \frac{Q_i{}^{-\hat{\alpha}} - 1}{\hat{\alpha}}$$

$$Linearised\ expression: \quad H_{s,i} = A + B * X_{P,i}$$

$$Estimator\ (slope): \quad \hat{\beta} = B$$

$$Estimator\ (intercept): \quad \hat{\gamma} = A$$

The results of the linear regression analysis are shown in Figure F.1 and Table F.1:

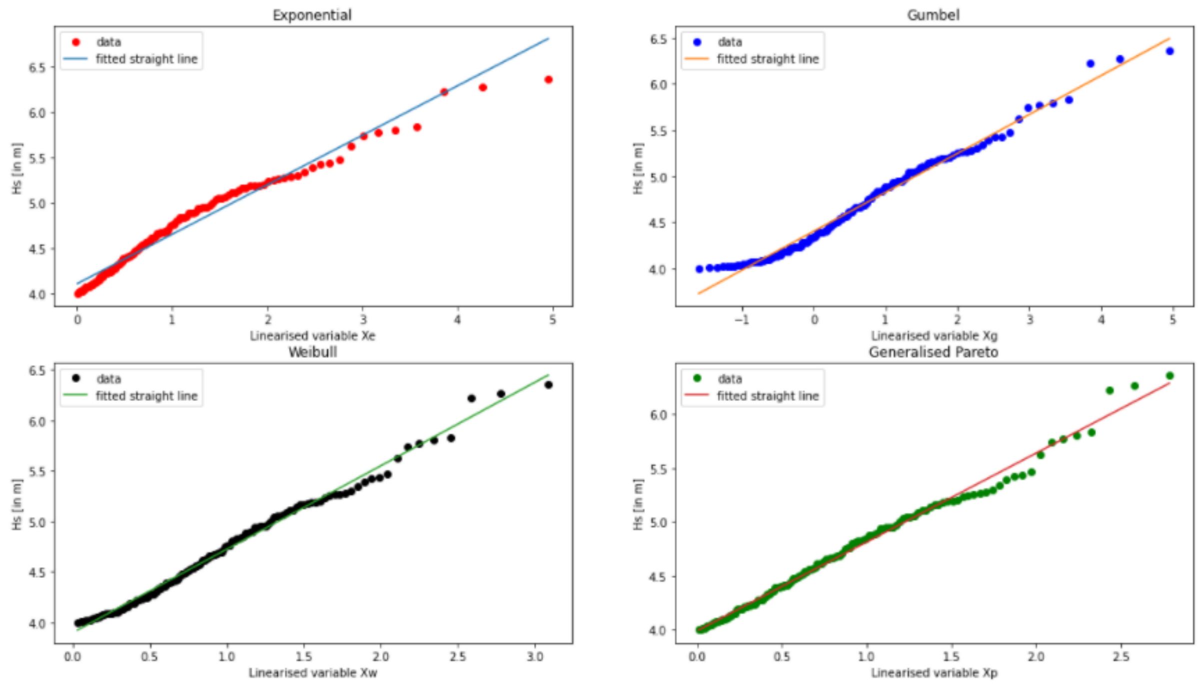


Figure F.1:Fitting of straight line through data set of extreme waves for four different distributions

Distribution	Intercept A [m]	Slope B [-]	Alpha [-]
Exponential	4.107	0.544	-
Gumbel	4.404	0.421	-
Weibull	3.896	0.827	1.42
Generalised Pareto	3.993	0.822	-0.26

Table F.1: Estimated values for parameters that describe the extremal distribution functions

The distributions can be used to compute the return period that belongs to a certain wave height, or vice versa, with the relationship $R = \frac{1}{Q*N_S}$. In this expression, N_s equals the number of storms per year. The result is displayed in Table F.2:

	Distribution of Wave Height [m]			
Return Period [y]	**Exponential**	**Gumbel**	**Weibull**	**Generalised Pareto**
1	5.21	5.23	5.26	5.29
2	5.59	5.54	5.57	5.60
5	6.09	5.93	5.95	5.93
10	6.47	6.23	6.22	6.13
20	6.84	6.52	6.47	6.30
50	7.34	6.91	6.80	6.48
100	7.72	7.20	7.03	6.59
200	8.09	7.49	7.26	6.69
500	8.59	7.88	7.55	6.79
1000	8.97	8.17	7.76	6.85

Table F.2: Numerical results of Extreme Value Analysis for various distributions (belongs to Figure 3.9)

For the Weibull distribution, the statistical uncertainty accompanying the distribution and the values belonging to the upper limit of the 68% and 95% confidence intervals have been determined. This has been done by means of bootstrapping, for which the following steps should be undertaken:

- Random data points are drawn (with replacement) from the set of extreme events, until the new data set is equally large as the original.
- Linear regression is performed for the new data set. The estimated function parameters are stored in separate arrays.
- This process is repeated a lot of times (e.g. 1000 runs), after which percentile values are computed for the array of every function parameter to determine the confidence intervals.

	Weibull Distribution		
Return Period [y]	**CSE [m]**	**UL 68%-CI [H_s in m]**	**UL 95%-CI [H_s in m]**
1	5.26	5.38	5.53
2	5.57	5.71	5.88
5	5.95	6.11	6.31
10	6.22	6.40	6.62
20	6.47	6.67	6.91
50	6.80	7.01	7.27
100	7.03	7.25	7.54
200	7.26	7.49	7.79
500	7.55	7.80	8.12
1000	7.76	8.03	8.37

Table F.3: Tabular display of uncertainties in Weibull distribution (belongs to Figure 3.10)

WAVE PERIOD

The first aspect of interest is to explore whether the mean relationship between wave period and wave height cited in Clause C.2.4.6 could be applied to this test case. The mean relationship reads $T_{1/3} \cong 3.3 * {H_{1/3}}^{0.63}$. As the wave period data that has been gathered is the mean wave period and the relationship describes the significant wave period, the relationship has been adjusted using the expression $T_{1/3} \cong 1.2T_m$ (see Clause C.2.1.4) to become $T_m \cong 2.75 * {H_{1/3}}^{0.63}$. In Figure F.2, the available wave period data has been plotted together with the relationship described above:

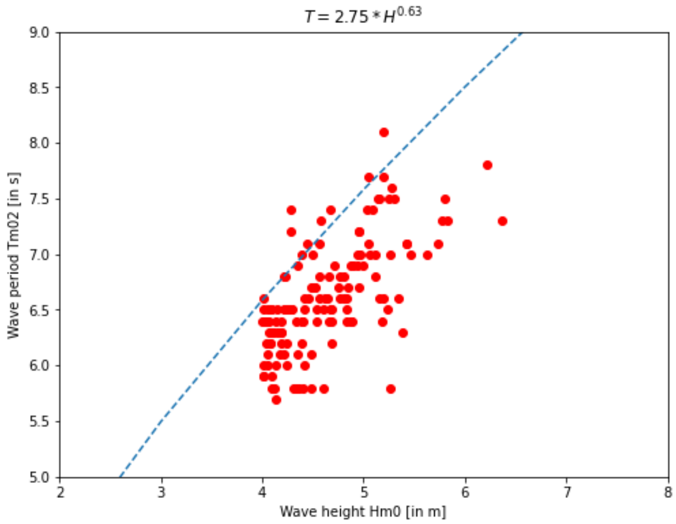


Figure F.2: Plot of relationship described in prEN1991-1-8 relative to wave data

From the figure it becomes evident that the relationship quoted by the new Eurocode is not suitable for the wave climate near IJmuiden.

WATER LEVEL

Relative magnitude

In order to evaluate the relative magnitude of tides and surges, two plots have been constructed. The first plot (Figure F.3) shows the magnitude of the astronomical tide, along with three well-known tide levels:

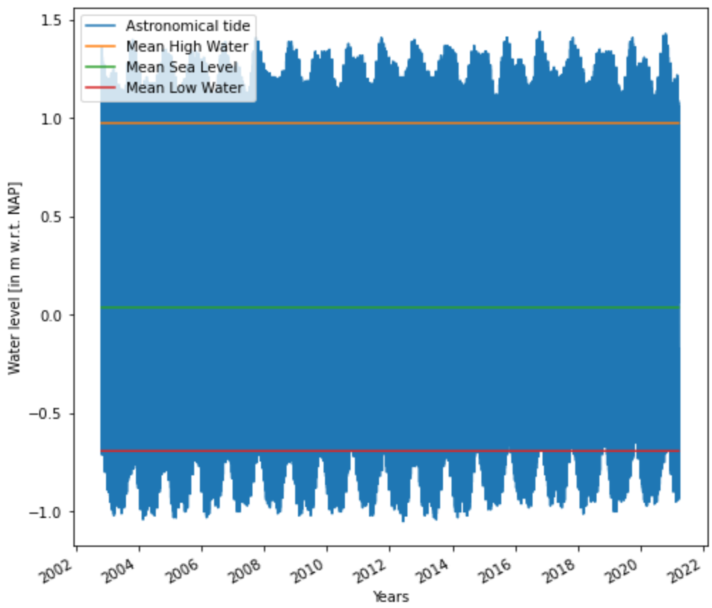


Figure F.3: Time series of tidal component near IJmuiden

The second plot (Figure F.4) shows the magnitude of the storm surge, which has been constructed by subtracting the astronomical tide from the absolute water level (see Figure 3.4):

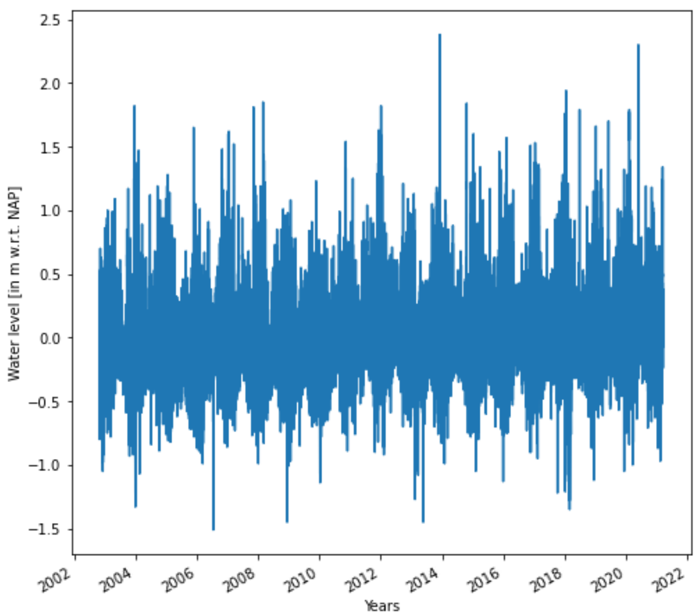


Figure F.4: Time series of storm surge component near IJmuiden

From the data in Figure F.3, it can be computed that the maximum tidal range in this time period was equal to 2.28 m. From the data in Figure F.4, a maximum storm surge for this time period of 2.38 m was found.

Extreme Value Analysis

The return periods of interest should firstly be selected. The water level will primarily function as the combination value. However, as full correlation will be assumed (see Section 3.5), the return periods of interest are the same as those of the wave height. This means a return period of 100 years for ULS and a return period of 10 years for SLS-(LD).

Next, a suitable threshold level and event separation time should be chosen. The same event separation time as in the wave analysis of 36 h has been used. For the selection of the threshold level, additional information from Waterinfo [Ref. 15] has been used. This platform namely mentions that water levels >1.80 m+NAP count as ‘slightly elevated water levels’, and water levels >2.00 m+NAP count as ‘elevated water levels’ for the location of IJmuiden. Having this latter value as the threshold would result in a small data set for the Extreme Value Analysis, which is why a threshold value of 1.80 m+NAP has eventually been selected.

The data should then be fit to a distribution and extrapolated towards the return periods of interest:

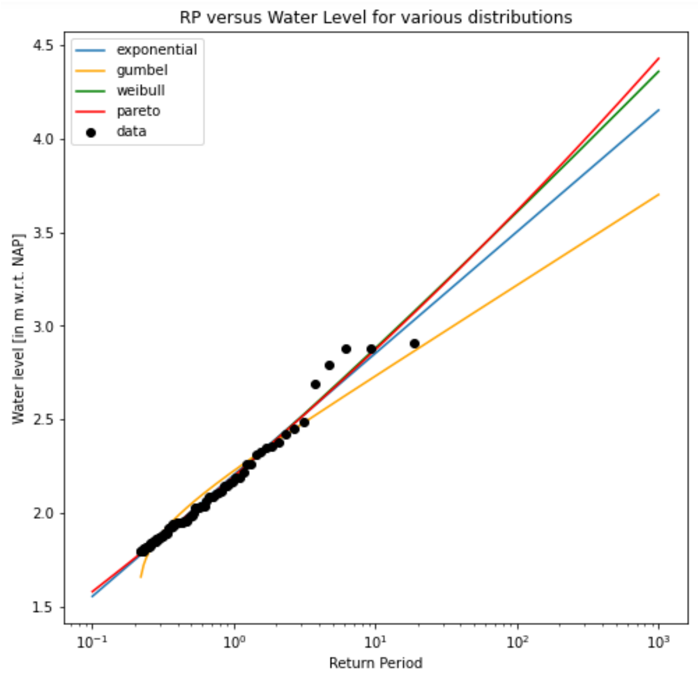


Figure F.5: Fitting the data set of extreme water levels to several distribution types

Choosing the proper distribution was not straightforward. Clause C.2.4.2 in the new Eurocode gives examples of extremal distribution functions for storm wave heights, but does not specify whether these could also be used for water levels. Several distribution types have been tried which, apart from the Gumbel distribution, turned out to lie very closely together. The Weibull distribution has eventually been chosen, as its RMSE was lowest and this distribution is also used in the extreme wave analysis.

Extreme Value Analysis on surge only

It is also possible to treat surge separately from the tide. The Extreme Value Analysis is then performed for the storm surge component displayed in Figure F.4, with a threshold value of 1.00 m and an event separation time of 36 hours, resulting in 7.3 storms per year. The results are displayed in Figure F.6 and Table F.4:

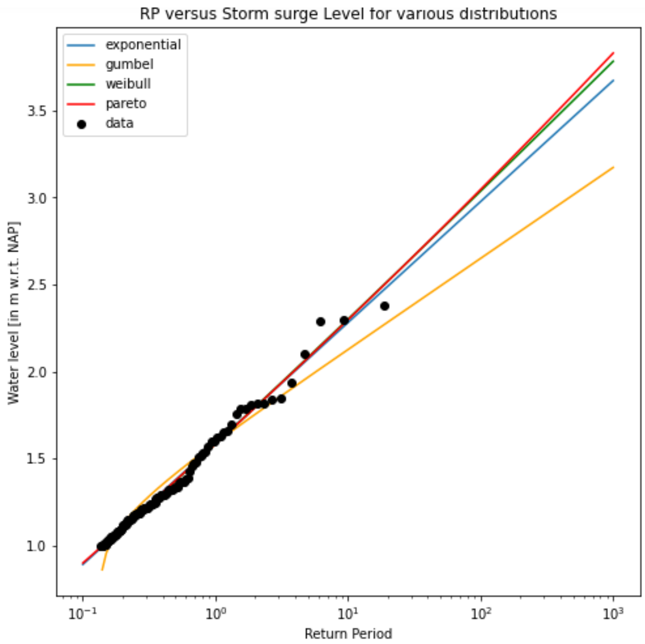


Figure F.6: Fitting the data set of extreme storm surges to several distribution types

Distribution	Intercept A [m]	Slope B [-]	Alpha [-]	RMSE [-]
Exponential	0.987	0.302	-	0.0267
Gumbel	1.155	0.227	-	0.0639
Weibull	0.998	0.286	0.96	0.0262
Generalised Pareto	0.991	0.291	0.02	0.0265

Table F.4: Estimated values for parameters that describe the extremal distribution functions (for storm surge events)

The Weibull distribution has the lowest RMSE, so the return periods are calculated for this distribution function, see the numerical values in Table F.5:

Return Period [y]	1	2	5	10	20	50	100	200
Water level [m+NAP]	1.58	1.80	2.08	2.30	2.52	2.81	3.04	3.26

Table F.5: Water level for various return periods, based on EVA of surge only

When the Extreme Value Analysis is performed for the surge only, a tidal datum should be added to the surge values to arrive at the total water level.

Clause C.1.1 in prEN1991-1-8 suggests addition of Mean High Water Springs, which has a value of 1.15 m+NAP near IJmuiden [Ref. 27], though other tidal levels are also allowed.

This would result in the following values:

Return Period [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Water level [m+NAP]	2.73	2.95	3.23	3.45	3.67	3.96	4.19	4.41

Table F.6: Water level for various return periods, based on EVA of surge only and adding tidal level MHWS

It has been chosen to continue working with the results obtained when performing the EVA on the combination of surge and tide.

Sea level-rise

The KNMI [Ref. 31] has drawn up four different climate scenarios, for the sight years 2050 and 2085. As the design lifetime of the breakwaters will be 50 years, the 'end of life' of the structure will be around 2075, which is closer to 2085 than it is to 2050. Therefore, it is assumed that the numbers published for 2085 are most relevant. For the purpose of this research, it is not required to obtain a thorough understanding of the different climate scenarios. Figure F.7 shows the absolute expected sea-level rise and the expected rate of change of sea-level rise. A value of 50 cm will be adopted for the sea-level rise, as this falls within the boundaries of each of the climate scenarios.

Scenario veranderingen voor het klimaat rond 2085 [c] (2071-2100)				Natuurlijke variaties gemiddeld over 30 jaar [D]
G_L	G_H	W_L	W_H	
+1,5 °C	+1,5 °C	+3,5 °C	+3,5 °C	
Lage waarde	Hoge waarde	Lage waarde	Hoge waarde	
+25 tot +60 cm	+25 tot +60 cm	+45 tot +80 cm	+45 tot +80 cm	± 1,4 cm
+1 tot +7,5 mm/jaar	+1 tot +7,5 mm/jaar	+4 tot +10,5 mm/jaar	+4 tot +10,5 mm/jaar	± 1,4 mm/jaar

Figure F.7: Expected absolute sea-level rise and rate of change for the different climate scenarios [Ref. 25]

APPENDIX G: ADDITIONAL INFORMATION OFFSHORE-NEARSHORE TRANSFORMATION

The following input has been provided to SwanOne to perform the offshore nearshore transformation:

- No currents.
- The bottom profile as described in Subsection 3.4.3, see Figure G.1.
- The wave direction assumed to be perpendicular to the breakwater, see Figure G.2.
- Other parameters as presented in Figure G.3. The wind velocity is an estimation, the peak period has been derived from the mean period using empirical relations.

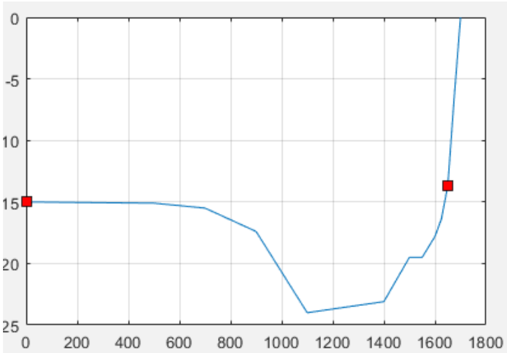


Figure G.1: Bottom levels SwanOne

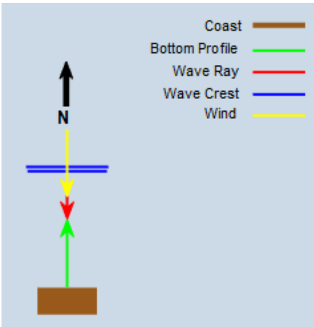


Figure G.2: Orientations SwanOne

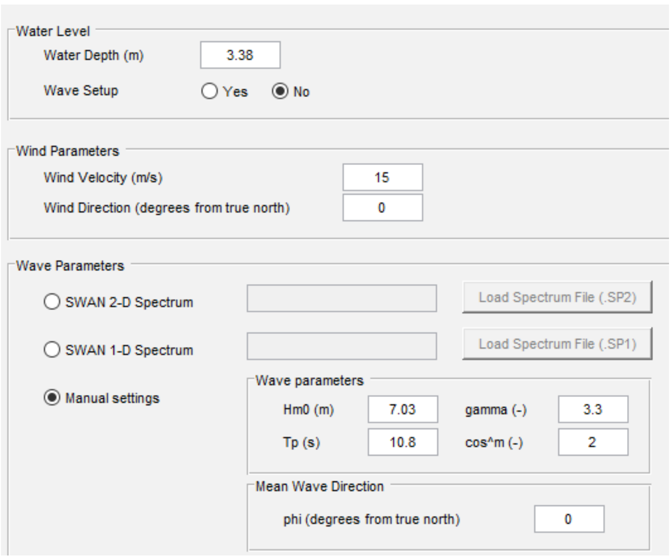


Figure G.3: Action parameters SwanOne

APPENDIX H: SENSITIVITY ANALYSIS ARMOUR LAYER - ROCK

Several design choices have been made to arrive at a nominal rock diameter for the armour layer. Nevertheless, there are certain aspects that could have been interpreted differently, which would have resulted in alternative design outcomes. The most important of these aspects are:

- The selection of return periods as a function of consequence class and T_{life}
- The choice of the extreme wave distribution function
- The use of a certain non-exceedance value for the extrapolation of wave heights
- The storm duration that has been opted to calculate with
- The wave period that has been reasoned to accompany the wave height
- The coefficient in the response formula that can be altered as a way of incorporating safety on the resistance side
- The application of a partial factor to the wave height in other expressions apart from the stability number
- The way the discrepancies in Tables 4.3/A.4&A.5 and Tables A.6.8/A.7 are interpreted

These aspects are elaborated upon in this Appendix.

Selection of return periods

Since the appropriate consequence class and design service life is still open to interpretation in prEN1991-1-8, which was explained in Chapter 3, various return periods (as displayed in Table 3.3) can be selected.

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	-	-
20	Weibull	50	6.47	7.80	1.0	1:3	2	12	5538	C_{pl} = 6.2	1.81	15.7	4.0	12.1
40	Weibull	50	6.72	7.95	1.0	1:3	2	12	5433	C_{pl} = 6.2	1.87	17.5	7.5	25.0
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	-	-
200	Weibull	50	7.26	8.26	1.35	1:3	12	12	5230	C_{pl} = 6.2	1.90	18.3	2.7	9.6
400	Weibull	50	7.48	8.38	1.35	1:3	12	12	5155	C_{pl} = 6.2	1.96	19.9	5.9	19.2

Gumbel vs. Weibull vs. Generalised Pareto

Since the way of fitting the appropriate distribution function is still open to interpretation in prEN1991-1-8, which was explained in Chapter 3, various distributions can be chosen from.

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Generalised Pareto	50	6.13	7.59	1.0	1:3	2	12	5692	C_{pl} = 6.2	1.72	13.4	-	-
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.741	14.0	1.2	4.5
10	Gumbel	50	6.23	7.65	1.0	1:3	2	12	5647	C_{pl} = 6.2	1.744	14.1	1.4	5.2
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Generalised Pareto	50	6.59	7.87	1.35	1:3	12	12	5489	C_{pl} = 6.2	1.74	13.9	-	-
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	6.3	20.1
100	Gumbel	50	7.20	8.22	1.35	1:3	12	12	5255	C_{pl} = 6.2	1.89	17.8	8.6	28.1

50% non-exceedance value vs. 84% non-exceedance value vs. 97.5% non-exceedance value for wave height

Since it is not conclusively described in prEN1991-1-8 whether the central statistical estimate should be used when determining the design wave height, or whether the upper limits of a certain confidence interval should be taken, different non-exceedance values can be opted for.

Return Period	Distribution	%non-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	-	-
10	Weibull	84	6.40	7.75	1.0	1:3	2	12	5574	C_{pl} = 6.2	1.79	15.2	2.9	8.6
10	Weibull	97.5	6.59	7.87	1.0	1:3	2	12	5489	C_{pl} = 6.2	1.84	16.5	5.7	17.9
Return Period	**Distribution**	**%non-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	-	-
100	Weibull	84	7.26	8.26	1.35	1:3	12	12	5230	C_{pl} = 6.2	1.90	18.3	2.7	9.6
100	Weibull	97.5	7.50	8.39	1.35	1:3	12	12	5149	C_{pl} = 6.2	1.96	20.0	5.9	19.8

Effect of storm duration choice

It remains unclear which value should be taken for the duration of the storm. You could determine the average storm duration from the data and use this in the design, but it is not said that the average storm duration is representative for the storm you are designing for. The Rock Manual, in Box 5.13 [Ref. 6], uses a storm duration of 6 hours in a calculation example. The Breakwater Design Lecture Notes [Ref. 7] speaks of storm durations between 3 to 12 hours. The new Eurocode itself mentions a storm duration of 12 hours in the legend of Table A.2, but it is not specified whether this value should be used in the above expression. The effect of this choice on the design is shown here.

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	6	2827	C_{pl} = 6.2	1.62	11.4	-	-
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	7.4	22.8
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.35	1:3	12	6	2657	C_{pl} = 6.2	1.72	13.5	-	-
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	7.6	23.7

Influence wave steepness

In Clause 7.4.1 in the new Eurocode the following statement is made:

'The validity range of semi-empirical design formulae shall not be exceeded, unless the proposed design can be proven to be conservative, or has been verified by previous model tests, or in type (a) breakwaters by documented full-scale experience. For cases that do not meet above criteria, hydraulic model testing should be undertaken.'

According to the above, hydraulic model tests should be performed as the fictitious wave steepness is outside the range of validity defined by Van der Meer which should be between 0.01-0.06. Important to note is that the individually counted mean period T_m should be employed in the expression for the fictitious wave steepness, but instead we have used the spectrally estimated mean period T_{m02} in the calculations, as data of the former parameter were not available.

We have already established that the spectrally estimated mean period is generally smaller than the individually counted mean period, see Subsection 3.4.1.4. Actually doing hydraulic model tests is outside the scope of this research, but it can be investigated how the design changes when the wave period is larger and therefore the wave steepness lower. For this, the wave period has been increased by 5 and 10 percent respectively:

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	-	-
10	Weibull	50	6.22	8.02 (5%)	1.0	1:3	2	12	5387	C_{pl} = 6.2	1.78	14.8	2.3	5.7
10	Weibull	50	6.22	8.40 (10%)	1.0	1:3	2	12	5142	C_{pl} = 6.2	1.81	15.7	4.0	12.1
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	-	-
100	Weibull	50	7.03	8.54 (5%)	1.35	1:3	12	12	5059	C_{pl} = 6.2	1.88	17.7	1.6	6.0
100	Weibull	50	7.03	8.94 (10%)	1.35	1:3	12	12	4832	C_{pl} = 6.2	1.92	18.7	3.8	12.0

Mean value vs. upper limit 68%-confidence interval vs. upper limit 90%-confidence interval for coefficient c_{pl}

For breakwaters, there is no such thing as a partial resistance factor, but the uncertainty can be dealt with by choosing 'safe' values for the coefficient in the Van der Meer-formula. In the introduction to Chapter 4, it was established that this should only be done in DA-0 and not in DA-1, as the partial factor deals with the uncertainty. However, it is still useful to investigate how it would influence the design outcome, as a partial resistance factor lacks. In the Van der Meer-formula for plunging waves, the average value of the coefficient equals c_{pl} = 6.2 with a standard deviation of 0.4. A value of 5.8 thus corresponds to the upper limit of the 68% confidence interval. In current design practice, it is also common to use the upper limit of the 90%-confidence interval, with a value of c_{pl} = 5.5.

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	-	-
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 5.8	1.86	17.1	6.9	22.1
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 5.5	1.96	20.0	12.6	42.9
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	-	-
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 5.8	1.97	20.4	6.5	22.2
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 5.5	2.08	23.9	12.4	43.1

Impact of partial factor use

Although the value of the partial factor that should be used is known, it is poorly described in the new Eurocode how to deal with it. Factoring a regular load is straightforward, but some questions arise when factoring the wave height. The wave height occurs in the stability number, but it also emerges in the wave steepness. Should the partial factor also be applied in the wave steepness? And if so, should the wave period then be adjusted to ensure that the wave steepness still has a physically realistic value?

The following 3 situations will be investigated:

1) Wave height factored in both stability number and wave steepness; wave period not adjusted.
2) Wave height factored in both stability number and wave steepness; wave period adjusted to guarantee constant steepness.
3) Wave height only factored in the stability number.

Situation	**Return Period**	**Distribution**	**%-EV**	**H_s**	**T_m**	**Stability Number**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
1	100	Weibull	50	7.03	8.13	3.50	1:3	12	12	5314	C_{pl} = 6.2	1.71	13.3	-	-
2	100	Weibull	50	7.03	9.44	3.29	1:3	12	12	4576	C_{pl} = 6.2	1.82	15.9	6.4	19.5
3	100	Weibull	50	7.03	8.13	3.24	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	8.1	25.6

It has been assumed that only the wave height occurring in the stability number should be factored with a certain partial factor, not the wave height occurring in the fictitious wave steepness. Nevertheless, the new draft Eurocode does not mention this explicitly.

Treatment of discrepancies in Tables 4.3/A.4&A.5 and Tables A.7/A.6.8

The mismatch in the values presented in Table 4.3 compared to Tables A.4&A.5, and the values presented in Tables A.7 and Table A.6.8 (EN1990), have been thoroughly discussed. The room for interpretation makes that the user can 'choose' between:

- Return period of either 10 or 100 years in SLS-(LD)
- Return period of either 100 or 400 years in ULS
- Partial factor of either 1 or 1.35 in ULS

Return Period	Distribution	%-EV	H_s	T_m	Partial Factor	Slope	S_d	Storm Duration	N	Coefficient	D_{n50}	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	7.64	1.0	1:3	2	12	5654	C_{pl} = 6.2	1.74	14.0	-	-
100	Weibull	50	7.03	8.13	1.0	1:3	2	12	5314	C_{pl} = 6.2	1.96	19.9	12.6	42.1
Return Period	**Distribution**	**%-EV**	**H_s**	**T_m**	**Partial Factor**	**Slope**	**S_d**	**Storm Duration**	**N**	**Coefficient**	**D_{n50}**	**M_{50}**	**%ΔD**	**%ΔM**
100	Weibull	50	7.03	8.13	1.0	1:3	12	12	5314	C_{pl} = 6.2	1.37	6.8	-	-
400	Weibull	50	7.48	8.38	1.0	1:3	12	12	5155	C_{pl} = 6.2	1.45	8.1	5.8	19.1
100	Weibull	50	7.03	8.13	1.35	1:3	12	12	5314	C_{pl} = 6.2	1.85	16.7	35.0	145.6
400	Weibull	50	7.48	8.38	1.35	1:3	12	12	5155	C_{pl} = 6.2	1.96	19.9	43.1	192.6

APPENDIX I: SENSITIVITY ANALYSIS ARMOUR LAYER – ARTIFICIAL UNITS

Accropode stability number

The Rock Manual states the following with regards to this:

'As start of damage and failure for Accropodes are very close, although at very high stability numbers, it is recommended that a safety factor for design is used of about 1.5 on the Hs/(ΔDn)-values.'

If this safety factor is incorporated into the aforementioned values, this would result in stability numbers of 2.5 for SLS-(LD) and 2.7 for ULS. Below it is investigated how the different alternatives, i.e. incorporating a safety factor into the formula, applying it to the wave height or both, lead to different designs:

Return Period	Distribution	%-EV	H_s	Partial Factor	Delta	Slope	N_{od}	Safety Factor	Stability Number	D_n	M_{50}	%ΔD	%ΔM
10	Weibull	50	6.22	1.0	1.34	1:1.5	0	No	3.7	1.25	4.7	-	-
10	Weibull	50	6.22	1.0	1.34	1:1.5	0	Yes	2.5	1.85	15.3	48.0	225.5

Start of damage

Return Period	Distribution	%-EV	H_s	Partial Factor	Delta	Slope	N_{od}	Safety Factor	Stability Number	D_n	M_{50}	%ΔD	%ΔM
100	Weibull	50	7.03	1.35	1.34	1:1.5	>0.5	No	4.1	1.73	12.3	-	-
100	Weibull	50	7.03	1.35	1.34	1:1.5	>0.5	Yes	2.7	2.62	43.2	51.4	243.1

Failure

Even though the Rock Manual clearly states that a safety factor in the formula should be used for design purposes, prEN1991-1-8 seems to claim that this is covered by the partial load factor.

APPENDIX J: ADDITIONAL CONSIDERATIONS CREST HEIGHT

TABLES FROM EUROTOP [REF. 11]

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; H_{m0} > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; H_{m0} > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; maintained and closed grass cover; H_{m0} = 1 – 3 m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Figure J.1: Tolerable overtopping discharges for structural design, from EurOtop section 3.3.3

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Significant damage or sinking of larger yachts; H_{m0} > 5 m	>10	>5,000 – 30,000
Significant damage or sinking of larger yachts; H_{m0} = 3-5 m	>20	>5,000 – 30,000
Sinking small boats set 5-10 m from wall; H_{m0} = 3-5 m Damage to larger yachts	>5	>3,000-5,000
Safe for larger yachts; H_{m0} > 5 m	<5	<5,000
Safe for smaller boats set 5-10 m from wall; H_{m0} = 3-5 m	<1	<2,000
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000

Figure J.2: Tolerable overtopping discharges for property, from EurOtop section 3.3.4

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 2000 2000 2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Figure J.3: Tolerable overtopping discharges for people and vehicles, from EurOtop section 3.3.5

SENSITIVITY ANALYSIS CREST HEIGHT

Mean value approach vs. Design assessment approach
In the formula used in Subsection 4.3.1, the mean values of the coefficients c_1 (=0.09) and c_2 (=1.5) are used. This is why the formula is also referred to as the 'mean value approach'. However, as the formula is of empirical nature, the EurOtop Manual suggests adding one standard deviation to these coefficients to account for the scatter in the formula. This is referred to as the 'design assessment approach', and the coefficients would then become 0.1035 and 1.35, respectively. Nevertheless, as was explained in the introduction to Chapter 4, the draft Eurocode prescribes to not include standard deviations in empirical formulae for DA-1, as a safety margin is provided by the use of partial factors. On the other hand, as partial factors are not used for SLS-(LD), it would make sense to include safety in the formula.

q_{tol}	0.020 m^3/s per m	q_{tol}	0.020 m^3/s per m
C_1	0.09	C_1	0.1035
C_2	1.5	C_2	1.35
RP H_s	10 y	**RP H_s**	10 y
$Hm0_s$	6.22 m	**H_s**	6.22 m
Correlation	Full	**Correlation**	Full
RP η	10 y	**RP η**	10 y
η	3.38 m+NAP	**η**	3.38 m+NAP
Y_f	0.46	Y_f	0.46
Y_β	1	Y_β	1
R_c	6.97 m	R_c	7.90 m
A	10.35 m+NAP	**A**	11.28 m+NAP

Despite these considerations, it has been chosen to do the calculations with the 'mean value approach' formula.

Tolerable overtopping discharges

PrEN1991-1-8 refers to the EurOtop Manual in Clause 7.4.3 for overtopping threshold values, but it is not evident what tolerable overtopping discharge should be designed for, especially not in combination with the different limit state definitions. Several options have been explored. The other parameters in the calculations were equal to those presented in the table above.

Parameter	Unit	Value			
q_{tol}	m^3/s per m	0.001	0.005	0.020	0.100
R_c	m	9.79	8.31	6.97	5.31
A	m+NAP	13.17	11.69	10.35	8.69

Correlation

Full correlation has been assumed, but this was a conservative design choice. Below, the sensitivity in the design outcome is shown when working with weak correlation.

q_{tol}	0.020 m^3/s per m	**q_{tol}**	0.020 m^3/s per m	**q_{tol}**	0.020 m^3/s per m
C_1	0.09	**C_1**	0.09	C_1	0.09
C_2	1.5	**C_2**	1.5	C_2	1.5
RP H_s	10 y	**RP H_s**	10 y	RP H_s	5 y
H_s	6.22 m	**H_s**	6.22 m	H_s	5.95 m
Correlation	Full	**Correlation**	Weak	Correlation	Weak
RP η	10 y	**RP η**	5 y	RP η	10 y
η	3.38 m+NAP	**η**	3.17 m+NAP	η	3.38 m+NAP
Y_f	0.46	**Y_f**	0.46	Y_f	0.46
Y_β	1	**Y_β**	1	Y_β	1
R_c	6.97 m	**R_c**	6.97 m	R_c	6.61 m
A	10.35 m+NAP	**A**	10.14 m+NAP	A	9.99 m+NAP

If you would construct a joint distribution, this would then probably result in a crest height between 10.14 and 10.35 m+NAP.

APPENDIX K: ACCROPODE DESIGN GUIDE TABLE

ACCROPODE™ Design Guide Table

Unit Volume (m³)	V = 0,34H³		1.0	2.0	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	28.0
Unit Height (m)	$H = (V/0.34)^{(1/3)}$		1.43	1.81	2.07	2.27	2.45	2.60	2.87	3.09	3.28	3.45	3.61	3.75	3.89	4.01	4.13	4.35
Equivalent Cube Size (m)	$Dn = V^{1/3}$		1.00	1.26	1.44	1.59	1.71	1.82	2.00	2.15	2.29	2.41	2.52	2.62	2.71	2.80	2.88	3.04
Armour Thickness (m)	T = 1,29 Dn		1.29	1.63	1.86	2.05	2.21	2.34	2.58	2.78	2.95	3.11	3.25	3.38	3.50	3.61	3.72	3.92
Armour concrete consumption and coverage	Packing density Φ (-)		0.645	0.645	0.645	0.643	0.642	0.640	0.637	0.634	0.631	0.628	0.625	0.625	0.625	0.625	0.625	0.625
	Consumption (m³/m²)		0.645	0.813	0.930	1.021	1.098	1.164	1.275	1.366	1.445	1.514	1.575	1.638	1.697	1.751	1.803	1.898
	Number of units (u/m²)		0.645	0.406	0.310	0.255	0.220	0.194	0.159	0.137	0.120	0.108	0.098	0.091	0.085	0.080	0.075	0.068
	Porosity (%)		50.00	50.00	50.00	50.12	50.24	50.36	50.60	50.83	51.07	51.31	51.55	51.55	51.55	51.55	51.55	51.55
Filter stone underlayer - to meet the following requirement NUL/NLL < 3.0	NLL (tons)	Standard	0.17	0.34	0.50	0.67	0.84	1.01	1.34	1.68	2.02	2.35	2.69	3.02	3.36	3.70	4.03	4.70
		Min/Max*	0.1 0.2	0.2 0.4	0.4 0.7	0.5 0.9	0.6 1.1	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.6 3.1	1.9 3.5	2.1 3.9	2.4 4.4	2.6 4.8	2.8 5.2	3.3 6.1
	NUL (tons)	Standard	0.34	0.67	1.01	1.34	1.68	2.02	2.69	3.36	4.03	4.70	5.38	6.05	6.72	7.39	8.06	9.41
		Min/Max*	0.2 0.4	0.5 0.9	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.9 3.5	2.4 4.4	2.8 5.2	3.3 6.1	3.8 7.0	4.2 7.9	4.7 8.7	5.2 9.6	5.6 10.5	6.6 12.2
	Thickness (m) for standard NLL&NUL Specific density 2,6 t/m3	Kt=1,15	1.06	1.33	1.52	1.68	1.81	1.92	2.11	2.28	2.42	2.55	2.66	2.77	2.87	2.96	3.05	3.21
		Kt=0.9*	0.83	1.04	1.19	1.31	1.41	1.50	1.65	1.78	1.89	1.99	2.08	2.17	2.24	2.32	2.38	2.51

APPENDIX L: EXAMINATION OF CROWN WALL DESIGN

OPTIMISATION OF CROWN WALL CONFIGURATION

For the first design proposal, the height of the breakwater will be set at 10.35 m+NAP. This means that the top of the crown wall extends to this height. The base level at which the crown wall rests has been chosen to equal 4.85 m+NAP, just above the design water level. The crown wall then provides for the rest of the height and equals 5.5 m, with the upper part being unprotected. The choice of leaving part of the wall unprotected leads to a reduction of the required material, but it should be explored whether this design proposal can result in a stable solution.

The first design proposal is sketched in Figure L.1. Apart from the description above, explanations of other variables in the figure can be found in Sections 4.4 and 4.5.

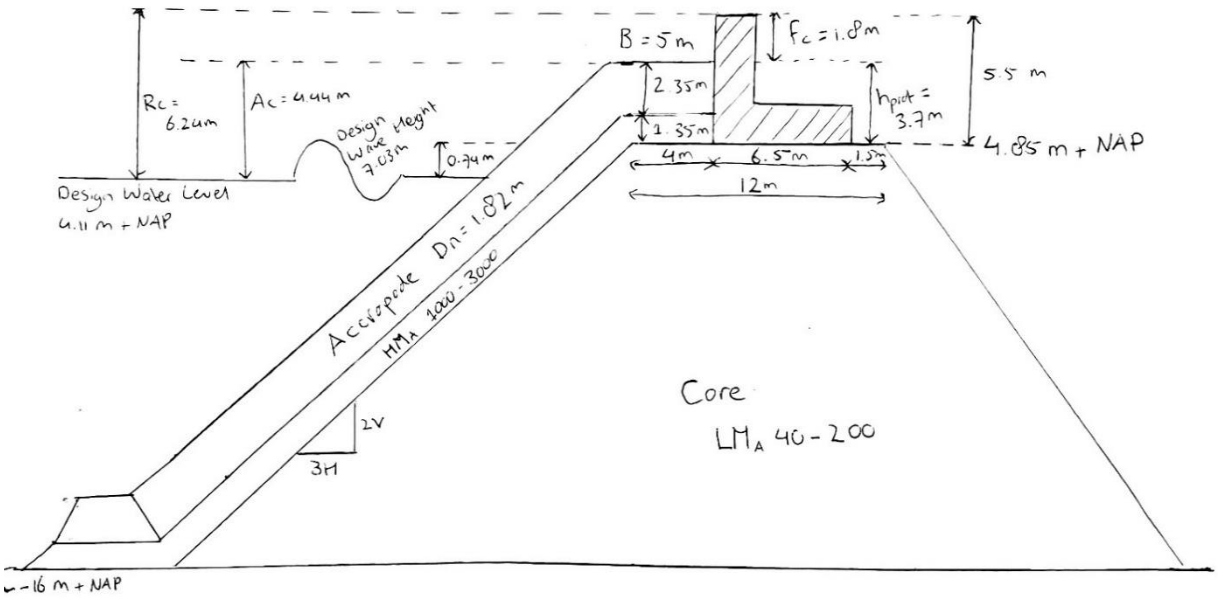


Figure L.1: Cross-sectional design breakwater IJmuiden (first design proposal)

The forces on the crown wall will be estimated based on wave overtopping rates [Ref. 22].

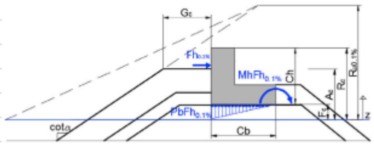
Design calculations for configuration as shown in Figure L.1

Required parameters	
zeta_0p	3,405688557
(Rc-Ac)/Cl	0,327
Q	0,006076228
log Q	-2,216365948
Fc/L0p	0,004033518
g	9,81
rho_w	1025
rho_c	2400
Ch	5,5

Paramete	Description	Value	Unit
Fh	Dimensionless horizontal force	2,014409202	-
Fh0.1%	Horizontal force	306,3627454	kN
ys	Partial factor on horizontal force	1	-
ysFh0.1%	Factored horizontal force	306,3627454	kN
PbF	Dimensionless uplift pressure corresponding to same wave that caused horizontal force	0,314911113	-
PbFh0.1%	Up-lift pressure	8,707902428	kN/m
Cb	Width of the crown wall	6,5	m
Fu	Vertical force assuming triangular pressure distribution	28,30068289	kN
ys	Partial factor on vertical force	1	-
ysFu	Factored vertical force	28,30068289	kN
MhF	Dimensionless horizontal moment	0,681054129	-
MhFh0.1%	Horizontal moment	1139,364206	kNm
ys	Partial factor on horizontal moment	1	-
ysMhFh0.:	Factored horizontal moment	1139,364206	

$$Fh = \frac{Fh_{0.1\%}}{(0.5\rho g C_h^2)} = \left((0.27 \cdot \ln(\xi_{0p}) + 0.1)(logQ + 6) + 0.23\right)\left(0.5 \cdot \frac{(R_c - A_c)}{C_h} + 1\right) - 0.15$$

$$PbF = \frac{PbFh_{0.1\%}}{(0.5\rho g C_h)} = 0.02 \cdot \left(\frac{F_c}{L_{0p}}\right)^{-1/2}$$



$$MhF = \frac{Mh_{(Fh0.1\%)}}{(\rho g C_h^3)} = 1.08 + 0.18 \cdot logQ$$

Sliding

t1	Thickness of bottom slab	3,2	m
t2	Thickness of vertical wall face	1	m
Vcw	Volume of crown wall	23,1	m^3
Mcw	Mass of crown wall	55440	kg
Fg	Weight of crown wall	543,8664	kN
f	Friction coefficient	0,6	-
f(Fg-Fu)	Left-hand side of the equation	309,3394303	kN
Fh	Right-hand side of the equation	306,3627454	kN
LHS exceeds RHS?		Yes	

$$f(F_G - F_U) \geq F_H \qquad \textbf{for stability against sliding} \qquad (5.199)$$

where:

F_G = (buoyancy-reduced) weight of the crown wall element (N), = $(M_{cw} - V_{cw}\,\rho_w)g$, where M_{cw} and V_{cw} are the mass and the volume of the crown wall

F_U = wave-induced uplift force (N)

F_H = wave-induced horizontal force (N)

f = friction coefficient (-).

Overturning

Vcw1	Volume of crown wall face	2,3	m^3
Fcw1	Weight of crown wall face	54,1512	kN
d1	Distance between centre of gravity Vcw1 and right base corner	6	m
Mg1	Stabilising moment due to mass of crown wall face	324,9072	kNm
Vcw2	Volume of crown wall base	20,8	m^3
Fcw2	Weight of crown wall base	489,7152	kN
d2	Distance between centre of gravity Vcw2 and right base corner	3,25	m
Mg2	Stabilising moment due to mass of crown wall base	1591,5744	kNm
Mg	Stabilising moment due to mass of crown wall	1916,4816	kNm
du	Distance between point of application Fu and right base corner	4,333333333	m
Mu	Moment due to uplift force	122,6362925	kNm
Mg-Mu	Left-hand side of the equation	1793,845307	kNm
Mh	Right-hand side of the equation	1139,364206	kNm
LHS exceeds RHS?		Yes	

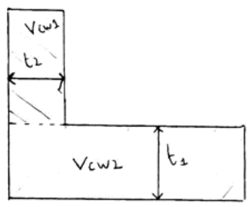
$$M_G - M_U \geq M_H \qquad \textbf{for stability against overturning} \qquad (5.200)$$

where:

M_G = stabilising moment due to mass of the crown wall element (Nm)

M_U = wave-generated moment due to uplift force (Nm)

M_H = wave-generated moment due to horizontal force (Nm).



It can be noticed that the partial factor on the forces equals 1. This is the case since the wave height has already been factored, resulting in a less negative value for 'log Q' and hence a larger horizontal force.

The design outcome shows that a stable solution is possible for the configuration in Figure L.1, but that the crown wall will be very sturdy in this case. That is, the thickness of the bottom slab will be very large and its width is also of a considerable size. There are four hydraulic variables in the equations that exert influence on the forcing:

- ξ_{0p}: The designer cannot change this value, under the assumption that the slope is a given and will not be altered.
- $\log Q$: The designer can change this value by varying the level of the crown wall top. If this level is raised, the freeboard increases, which results in less overtopping and a more negative value of log Q yielding a smaller horizontal force. Nevertheless, if you take a closer look at the formula, you also see that the force may not decrease by that much as the larger crown wall height has an increasing effect on the forcing.
- $\frac{R_c - A_c}{C_h}$: The designer can change this value by raising the base level on which the crown wall rests. If this is done, a larger part of the armour layer will protect the underlying vertical wall face. The smaller the part of the crown wall that is unprotected, the smaller the resulting force will be.
- $\frac{F_c}{L_{0p}}$: The designer can change this value by raising the base level on which the crown wall rests. if this parameter increases, which can only be achieved by an increase of F_c, the uplift pressure decreases.

A better solution can thus be achieved by raising the height of the breakwater. This can either be done by having a higher base level of the crown wall in combination with a smaller crown wall height C_h, or by having a lower base level of the crown wall in combination with a larger crown wall height C_h. The latter solution means that a smaller amount of rock is necessary in the core, but it also means that a larger portion of the crown wall is unprotected and that pressure is exerted on a larger surface.

If you consider these possible choices, it looks most promising to ensure that the vertical face of the crown wall is entirely protected to reduce the forcing on the crown wall. The geometrical parameter C_b can also be played with, but a compromise will always have to be made: a wider crown wall means larger self-weight and hence stabilising force, but also a larger surface on which the uplift pressure acts.

Two new alternatives are proposed to examine the effect of changes in the hydraulic parameters. In the first alternative, the base level is left unaltered but the crown wall is extended by half a metre. In the second alternative, the top level is left unaltered but the base level is raised to a height such that no part of the vertical wall face is unprotected. The width of the crown wall remains equal.

Design calculations for alternative configurations

Alternative 1		**Alternative 2**	
h_cw	10,85	h_cw	10,35
Rc	6,74	Rc	6,24
h_base	4,85	h_base	6,65
Fc	0,74	Fc	2,54
Ac	4,44	Ac	6,24
Wat lev	4,11	Wat lev	4,11

Required parameters	
zeta_0p	3,405688557
(Rc-Ac)/Cl	0,383333333
Q	0,004573583
log Q	-2,339743427
Fc/L0p	0,004033518
g	9,81
rho_w	1025
rho_c	2400
Ch	6

Required parameters	
zeta_0p	3,405688557
(Rc-Ac)/Cl	0
Q	0,006076228
log Q	-2,216365948
Fc/L0p	0,013844779
g	9,81
rho_w	1025
rho_c	2400
Ch	3,7

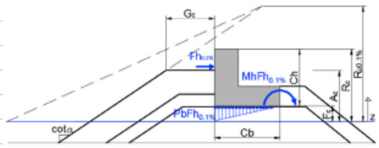
Table L.1: Set up and required design parameters for alternative 1 (raising crown wall height) and alternative 2 (raising crown wall base)

Forces in alternative 1:

Paramete	Description	Value	Unit
Fh	Dimensionless horizontal force	2,00345775	-
Fh0.1%	Horizontal force	362,6148336	kN
ys	Partial factor on horizontal force	1	-
ysFh0.1%	Factored horizontal force	362,6148336	kN
PbF	Dimensionless uplift pressure corresponding to same wave that caused horizontal force	0,314911113	-
PbFh0.1%	Up-lift pressure	9,499529921	kN/m
Cb	Width of the crown wall	6,5	m
Fu	Vertical force assuming triangular pressure distribution	30,87347224	kN
ys	Partial factor on vertical force	1	-
ysFu	Factored vertical force	30,87347224	kN
MhF	Dimensionless horizontal moment	0,658846183	-
MhFh0.1%	Horizontal moment	1430,970426	kNm
ys	Partial factor on horizontal moment	1	-
ysMhFh0.:	Factored horizontal moment	1430,970426	

$$Fh = \frac{Fh_{0.1\%}}{(0.5\rho g C_h^2)} = \left((0.27 \cdot \ln(\xi_{0p}) + 0.1)(logQ + 6) + 0.23\right)\left(0.5 \cdot \frac{(R_c - A_c)}{C_h} + 1\right) - 0.15$$

$$PbF = \frac{PbFh_{0.1\%}}{(0.5\rho g C_h)} = 0.02 \cdot \left(\frac{F_c}{L_{0p}}\right)^{-1/2}$$



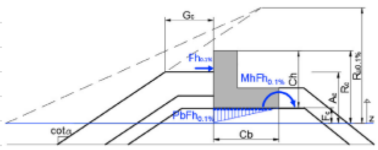
$$MhF = \frac{Mh_{(Fh0.1\%)}}{(\rho g C_h^3)} = 1.08 + 0.18 \cdot logQ$$

Forces in alternative 2:

Paramete	Description	Value	Unit
Fh	Dimensionless horizontal force	1,710257156	-
Fh0.1%	Horizontal force	117,7138981	kN
ys	Partial factor on horizontal force	1	-
ysFh0.1%	Factored horizontal force	117,7138981	kN
PbF	Dimensionless uplift pressure corresponding to same wave that caused horizontal force	0,169975759	-
PbFh0.1%	Up-lift pressure	3,161925188	kN/m
Cb	Width of the crown wall	6,5	m
Fu	Vertical force assuming triangular pressure distribution	10,27625686	kN
ys	Partial factor on vertical force	1	-
ysFu	Factored vertical force	10,27625686	kN
MhF	Dimensionless horizontal moment	0,681054129	-
MhFh0.1%	Horizontal moment	346,8803315	kNm
ys	Partial factor on horizontal moment	1	-
ysMhFh0.:	Factored horizontal moment	346,8803315	

$$Fh = \frac{Fh_{0.1\%}}{(0.5\rho g C_h^2)} = \left((0.27 \cdot \ln(\xi_{0p}) + 0.1)(logQ + 6) + 0.23\right)\left(0.5 \cdot \frac{(R_c - A_c)}{C_h} + 1\right) - 0.15$$

$$PbF = \frac{PbFh_{0.1\%}}{(0.5\rho g C_h)} = 0.02 \cdot \left(\frac{F_c}{L_{0p}}\right)^{-1/2}$$



$$MhF = \frac{Mh_{(Fh0.1\%)}}{(\rho g C_h^3)} = 1.08 + 0.18 \cdot logQ$$

In alternative 1, the forces increase with respect to the initially proposed design. Apparently, the change of 'log Q' does not make up for the increase in C_h and $(R_c - A_c)/C_h$. In alternative 2, the forces are drastically reduced. The downside of this alternative is that more material is needed as the base level needs to be raised by almost 2 m. On the other hand, the required concrete quantity is significantly less, and the higher base level can be mitigated by opting for a smaller crown wall width.

Based on the considerations above, the final configuration has a crown wall that is completely protected by the armour layer and therefore has a height of 3.7 m, but with a reduced width of 4.5 m. The crest height is set to 10.5 m+NAP, so as to ensure that the value of 'log Q' also lies within its range of validity. See Section 4.4 for a detailed cross-sectional drawing of this configuration. In Section 4.5 and in the remainder of this Appendix, the design of the crown wall for this configuration is elaborated upon.

COMPARISON OF CROWN WALL DESIGN METHODS

Different crown wall design formulae can be implemented. The draft Eurocode itself mentions Pedersen and Martin, but the site conditions lie outside the specified ranges of validity. Still, it could be informative to consider the design result with Pedersen [Ref. 24]. The second design formula that is considered is Molines [Ref. 23]. The design parameters are listed in Table L.2:

Pedersen		**Molines**	
Parameter	**Value**	**Parameter**	**Value**
$R_{u0.1\%}$	21.3 m	$R_{u0.1\%}$	15.8 m
$P_{b0.1\%}$ (=P_m/2)	74 kN/m	$\gamma_f R_{u0.1\%}/R_c$	1.14
$F_{h0.1\%}$	264.6 kN	$(R_c-A_c)/C_h$	0
$M_{h0.1\%}$	538.5 kNm	$\sqrt{(L_m/B)}$	4.20
F_U	169 kN	F_c/C_h	0.73
t_1	3.7 m (maximum)	$F_{h0.1\%}$	210.0 kN
F_G	392 kN	$P_{b(Fh0.1\%)}$	0.15 kN/m
$f^*(F_G - F_U) > F_{h0.1\%}$?	No	$M_{h0.1\%}$	427 kNm
		F_U	0.22 kN
		t_1	3.20 m
		F_G	350.8 kN
		$f^*(F_G - F_U) > F_{h0.1\%}$?	Yes

Table L.2: Crown wall design following the method of Pedersen or Molines, respectively

NOTE: The values presented in Table 4.2 include multiplication with a partial factor. For Pedersen, the wave height was factored, whereas for Molines the forces were factored.

It is noticed that Pedersen [Ref. 24] does not yield a stable design solution, which can be explained by the fact that the maximum up-lift force is considered, regardless of whether this force will act on the structure simultaneously with the maximum horizontal force, as opposed to the other 2 crown wall design methods considered in this thesis.

Molines [Ref. 23] provides a design solution that is significantly larger than the design solution obtained with [Ref. 22]. In addition, a stable design solution could only be reached when applying the partial factor to the force instead of the wave height. Apparently, the wave height has much more influence on the outcome than was the case for the crown wall design based on overtopping rates [Ref. 22].

ALTERNATIVE APPLICATION OF PARTIAL FACTOR IN CROWN WALL DESIGN

Full calculation option 2 in Table 4.26 (wave height not factored, forces factored by 1.5)

Required parameters	
zeta_0p	3,405688557
(Rc-Ac)/Ch	0
Q	0,001482286
log Q	-2,829067963
Fc/L0p	0,014662384
g	9,81
rho_w	1025
rho_c	2400
Ch	3,7

Paramete	Description	Value	Unit
Fh	Dimensionless horizontal force	1,446261793	-
Fh0.1%	Horizontal force	99,54357608	kN
ys	Partial factor on horizontal force	1,5	-
ysFh0.1%	Factored horizontal force	149,3153641	kN
PbF	Dimensionless uplift pressure corresponding to same wave that caused horizontal force	0,165168684	-
PbFh0.1%	Up-lift pressure	3,07250295	kN/m
Cb	Width of the crown wall	4,5	m
Fu	Vertical force assuming triangular pressure distribution	6,913131638	kN
ys	Partial factor on vertical force	1,5	-
ysFu	Factored vertical force	10,36969746	kN
MhF	Dimensionless horizontal moment	0,570767767	-
MhFh0.1%	Horizontal moment	290,7083351	kNm
ys	Partial factor on horizontal moment	1,5	-
ysMhFh0.1	Factored horizontal moment	436,0625027	

Sliding			
t1	Thickness of bottom slab	2,09	m
t2	Thickness of vertical wall face	1	m
Vcw	Volume of crown wall	11,015	m^3
Mcw	Mass of crown wall	26436	kg
Fg	Weight of crown wall	259,33716	kN
f	Friction coefficient	0,6	-
f(Fg-Fu)	Left-hand side of the equation	149,3804775	kN
Fh	Right-hand side of the equation	149,3153641	kN
LHS exceeds RHS?		Yes	

Overturning			
Vcw1	Volume of crown wall face	1,61	m^3
Fcw1	Weight of crown wall face	37,90584	kN
d1	Distance between centre of gravity Vcw1 and right base corner	4	m
Mg1	Stabilising moment due to mass of crown wall face	151,62336	kNm
Vcw2	Volume of crown wall base	9,405	m^3
Fcw2	Weight of crown wall base	221,43132	kN
d2	Distance between centre of gravity Vcw2 and right base corner	2,25	m
Mg2	Stabilising moment due to mass of crown wall base	498,22047	kNm
Mg	Stabilising moment due to mass of crown wall	649,84383	kNm
du	Distance between point of application Fu and right base corner	3	m
Mu	Moment due to uplift force	31,10909237	kNm
Mg-Mu	Left-hand side of the equation	618,7347376	kNm
Mh	Right-hand side of the equation	436,0625027	kNm
LHS exceeds RHS?		Yes	

Result is a thickness of the bottom slab of 2.09 m instead of 1.39 m.

Full calculation option 3 in Table 4.26 (wave height factored by 1.35, forces factored by 1.5/1.35)

Required parameters	
zeta_0p	3,405688557
(Rc-Ac)/Cl	0
Q	0,005583691
log Q	-2,253078634
Fc/L0p	0,014662384
g	9,81
rho_w	1025
rho_c	2400
Ch	3,7

Paramete	Description	Value	Unit
Fh	Dimensionless horizontal force	1,694438735	-
Fh0.1%	Horizontal force	116,6251448	kN
ys	Partial factor on horizontal force	1,111111111	-
ysFh0.1%	Factored horizontal force	129,5834942	kN
PbF	Dimensionless uplift pressure corresponding	0,165168684	-
	to same wave that caused horizontal force		
PbFh0.1%	Up-lift pressure	3,07250295	kN/m
Cb	Width of the crown wall	4,5	m
Fu	Vertical force assuming triangular	6,913131638	kN
	pressure distribution		
ys	Partial factor on vertical force	1,111111111	-
ysFu	Factored vertical force	7,681257375	kN
MhF	Dimensionless horizontal moment	0,674445846	-
MhFh0.1%	Horizontal moment	343,5145438	kNm
ys	Partial factor on horizontal moment	1,111111111	-
ysMhFh0.:	Factored horizontal moment	381,6828264	

Sliding			
t1	Thickness of bottom slab	1,66	m
t2	Thickness of vertical wall face	1	m
Vcw	Volume of crown wall	9,51	m^3
Mcw	Mass of crown wall	22824	kg
Fg	Weight of crown wall	223,90344	kN
f	Friction coefficient	0,6	-
f(Fg-Fu)	Left-hand side of the equation	129,7333096	kN
Fh	Right-hand side of the equation	129,5834942	kN
LHS exceeds RHS?		Yes	

Overturning			
Vcw1	Volume of crown wall face	2,04	m^3
Fcw1	Weight of crown wall face	48,02976	kN
d1	Distance between centre of gravity Vcw1 and	4	m
	right base corner		
Mg1	Stabilising moment due to mass of crown wall face	192,11904	kNm
Vcw2	Volume of crown wall base	7,47	m^3
Fcw2	Weight of crown wall base	175,87368	kN
d2	Distance between centre of gravity Vcw2 and	2,25	m
	right base corner		
Mg2	Stabilising moment due to mass of crown wall base	395,71578	kNm
Mg	Stabilising moment due to mass of crown wall	587,83482	kNm
du	Distance between point of application Fu and	3	m
	right base corner		
Mu	Moment due to uplift force	23,04377213	kNm
Mg-Mu	Left-hand side of the equation	564,7910479	kNm
Mh	Right-hand side of the equation	381,6828264	kNm
LHS exceeds RHS?		Yes	

Result is a thickness of the bottom slab of 1.66 m instead of 1.39 m.

CORRELATION

Full correlation has been assumed, but this was a conservative design choice. Below, the sensitivity in the design outcome is shown when working with weak correlation.

Description	Notation	Value (H_{m0} dominant)	Value (η dominant)	Unit
Characteristic value wave height	H_{m0}	7.03	6.22	m
Partial factor on wave height	γ_{Qz}	1.35	1.35	-
Design value wave height	$H_{s,d}$	9.49	8.40	m
Design value water level	η_d	3.38	4.11	m+NAP
Horizontal force	$F_{h,0.1\%}$	111.2	110.5	kN
Up-lift force	F_u	6.13	6.91	kN
Overturning moment	$Mh_{(Fh0.1\%)}$	326.8	324.4	kNm
Partial factors on forces and moments	γ_E	1.0	1.0	-
Required bottom slab thickness to achieve stability	t_1	1.27	1.27	m

Table L.3: Crown wall design outcome for less than moderately correlation design variables

If you would construct a joint distribution, this would then probably result in a thickness of the crown wall between 1.27 and 1.39 m.

APPENDIX M: ADDITIONAL GUIDANCE DA-2

EXPRESSIONS

$$N = \frac{D * 3600}{T_m}$$

$$s_{0m} = \frac{2\pi(\mu_1 H_s)}{gT_m^2}$$

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \text{ ; } in\ case\ of\ rock\ armour\ layer$$

$$\Delta = \frac{\rho_c - \rho_w}{\rho_w}; in\ case\ of\ concrete\ armour\ layer$$

$$T_m = 3.065 * \sqrt{\mu_1 H_s}$$

$$R_c = A - \mu_2 \eta$$

$$\xi_{0p} = \frac{\tan\alpha}{\sqrt{s_{0p}}}$$

$$s_{0p} = \frac{\mu_1 H_{m0}}{L_{0p}}$$

$$L_{0p} = \frac{g * T_p^2}{2\pi}$$

$$T_p = \frac{1.2}{0.9} * 3.065 * \sqrt{\mu_1 H_{m0}}$$

$$Q = c_1\, exp\left[-\left(c_2 \frac{R_c}{(\mu_1 H_{m0})\gamma_f \gamma_\beta}\right)^{1.3}\right]$$

$$A_c = R_c$$

$$P_b = 0.02 * \left(\frac{F_c}{L_{0p}}\right)^{-\frac{1}{2}} * (0.5\rho_w g C_h)$$

$$F_c = A - C_h - \mu_2 \eta$$

$$M_{cw} = V_{cw} * \rho_c$$

$$V_{cw} = t_1 * C_b + (C_h - t_1) * t_2$$

DISTRIBUTIONS

The distributions will be categorised according to the type of uncertainty it deals with. This distinction is explained in Section 2.1. A useful concept to explain here is the coefficient of variation. The coefficient of variation is a measure that expresses the standard deviation as a percentage of the mean. When for a certain parameter little information is available about the extent of uncertainty, an assumption can be made for the magnitude of the coefficient of variation. The standard deviation can then be estimated on the basis of this assumption.

Physical uncertainties

The vast majority of the uncertainties in breakwater design is of physical nature. They are uncertainties related to the design load, to material properties and to geometrical parameters. The distribution of the wave height is one such uncertainty, as it is unknown what the wave height will be that the breakwaters should be able to resist during its lifetime. The same goes for the design water level. In addition, the storm duration relates to uncertainty in the sea climate.
Material properties to which a distribution should be assigned are the densities of water, rock and concrete.
A geometrical parameter that is accompanied by uncertainty is the slope, as a result of construction inaccuracies.

- Wave Height H_s / H_{m0}:

The distribution of the wave height has already been determined in Subsection 3.6.1, being a Weibull distribution. The distribution was fitted to the data by means of linear regression, in which an intercept of 3.90 m, a slope of 8.3 and a shape parameter of 1.42 was found. In Prob2B, the cumulative function of the Weibull distribution is parameterised as follows:

$$F_X(x) = 1 - e^{-\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^k}$$

For this function, k is the shape parameter, ε is the intercept and the term (u- ε) should equal the slope. In Figure M.1, the proper input of parameters to Prob2B along with a graph of the distribution is shown:

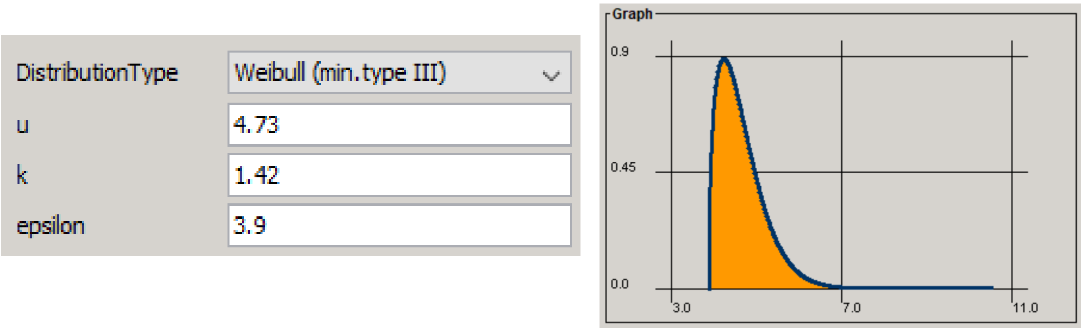


Figure M.1: The Weibull distribution for the wave height in Prob2B

- Water level η:

In Paragraph 3.6.3.1, it was described that a Weibull distribution is also adopted for the combined influence of tide and surge. This distribution will also be applied in the DA-2 calculation, even though a Gumbel distribution would perhaps be more logical. To the Weibull distribution belong the following function parameters: Intercept A equals 1.79 m+NAP, Slope B is 0.25 and alpha = 0.92. The sea-level rise should also be taken into account, which can be achieved by shifting the distribution 50 cm upwards. This means that the value of the intercept should be 0.5 m higher. In Figure M.2, the proper input of parameters (following the Prob2B Weibull parameterisation) along with a graph of the distribution is shown.

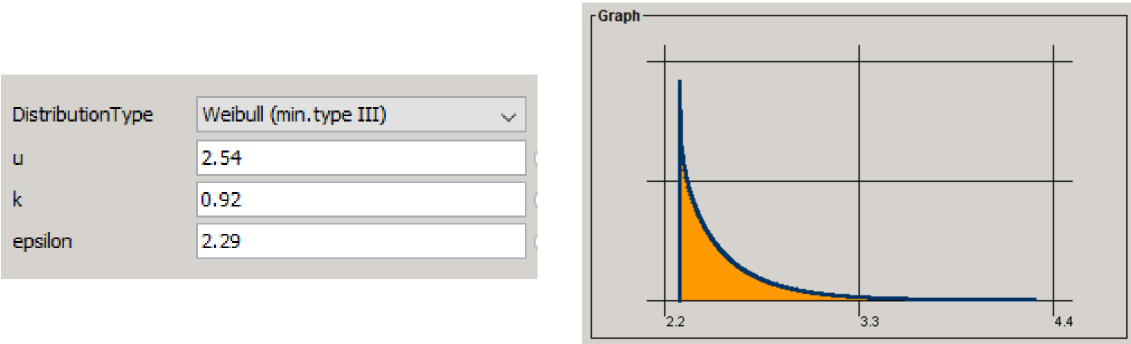


Figure M.2: The Weibull distribution for the water level in Prob2B

- Storm duration D:

In Section 4.1, it was established that the exact value of the storm duration is usually unknown. Therefore, it is assumed that the storm duration is normally distributed. Typical values in the semi-probabilistic calculations ranged between 6 and 12 hours, which is why an average value of 9 hours has been opted for. However, more extreme values such as 3 hours and 18 hours are also allowed, but with a low probability of occurrence. A standard deviation of 2 hours has been assumed, to ensure that the entire range of possible storm durations is accounted for, resulting in the normal distribution shown in Figure M.3:

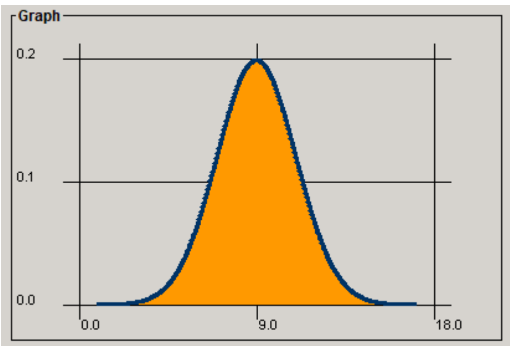


Figure M.3: The normal distribution for the storm duration in Prob2B

- Slope tan α:

The slope of the breakwater has been chosen to equal 1:3 for an armour layer consisting of rock, and 1:1.5 for an armour layer consisting of artificial units. Nevertheless, in practice it may occur that the constructed slope deviates slightly from the slope that was designed for. A coefficient of variation of 4% has been assumed, resulting in a standard deviation of 0.013 and 0.0267, respectively.

- Density of water ρ_w:

The density of water is a material property that is not constant. As the water near the breakwaters of IJmuiden is sea water, hence salt water, an average value for the density of 1025 kg/m^3 has been adopted. The density varies with temperature and salinity, but at the surface these changes are small, ranging between approximately 1023 and 1027 kg/m^3. A value of 2 kg/m^3 is thus appropriate for the standard deviation.

- Density of rock ρ_s:

The density of rock is a material property that is not known with great precision. A theoretical value for it is 2650 kg/m^3. The practical value in the field may differ from this, depending on the quarry that the rock originates from. Nevertheless, the variations will fall within certain boundaries, because if it differs by too much the rock will simply not be accepted for construction purposes. A higher density is advantageous, which is why the upper limit has been put further away from the value of 2650 kg/m^3 than the lower limit, as it will still be accepted in practice. A uniform distribution has been adopted, displayed in Figure M.4. This is an assumption, as no further specifications are given in the draft Eurocode.

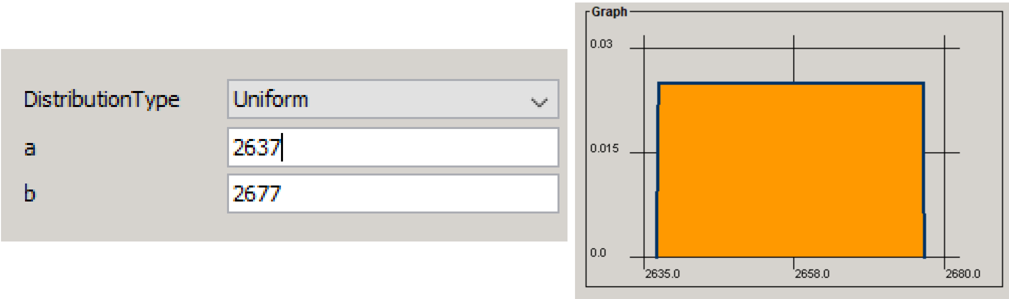


Figure M.4: The uniform distribution for the density of rock in Prob2B

- Density of concrete ρ_c:

A theoretical value for the density of concrete is 2400 kg/m^3. Compared to rock, the concrete is produced in a more controlled environment, so the density is known with quite some accuracy. Because of this, a small standard deviation of 10 kg/m^3 has been assumed.

- Roughness factor γ_f:

A distribution has been assumed by consulting Table 6.2 in the EurOtop Manual [Ref. 11]. Here it is found that for Accropodes the roughness factor equals 0.46, but for other artificial elements that show similar behaviour to Accropodes the roughness factor is slightly higher or lower (e.g. 0.44 for Xbloc). Hence, the factor of 0.46 will probably be prone to some uncertainty, but not by that much and also within certain limits. A uniform distribution with boundaries 0.43 and 0.46 has been adopted.

- Friction factor f:

A normal distribution with a mean of 0.6 and standard deviation of 0.05 has been adopted, based on the Rock Manual [Ref. 6] and based on the expertise of Dr. Ir. B. Hofland.

- Nominal rock diameter D_{n50}

For the nominal rock diameter, a triangular distribution has been assumed. If the D_{n50} is exceeded than the grading is not sufficient, so the boundaries are very fixed and not normal distributed. For the magnitude of the spread around the mean, Table A-2 in Bed, Bank and Shore Protection [Ref. 9] has been consulted. Here you can for example find that the class HM_A 6000-10000 (the largest class) has a d_{n50} of 1.44 m with a range that spans of 1.41 m to 1.47 m.

If these values are interpreted as a mean and spread of 0.03, the coefficient of variation is about 2 percent. This CV will be applied to all the rock diameter distributions in this subsection. So for the mean value of 1.85 that followed from DA-1 for ULS, the spread around the mean equals 0.04, and hence the limits become 1.81 and 1.89, see Figure M.5

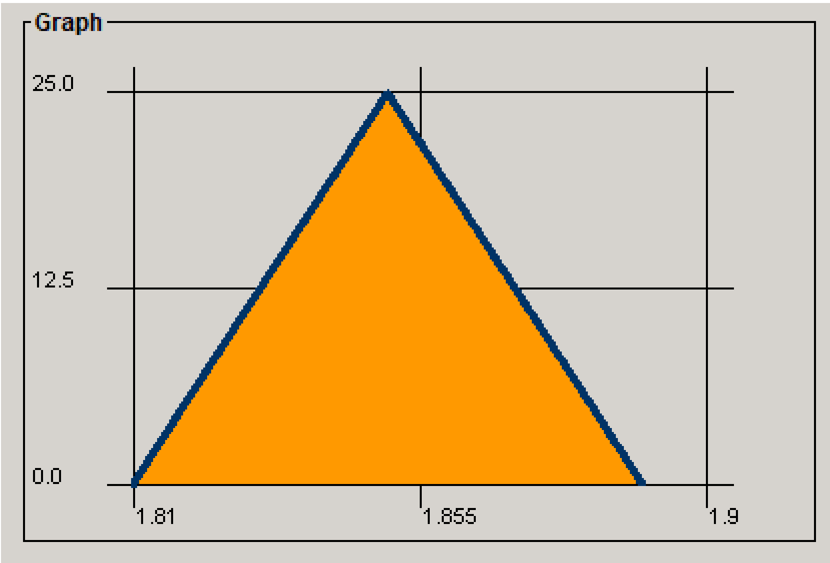


Figure M.5: Example of triangular distribution for nominal rock diameter in Prob2B

Statistical uncertainties

For the design of the IJmuiden breakwaters, there is uncertainty related to the estimation of wave height and water levels, caused by the limited amount of data.

- Uncertainty wave height μ_1:

The exact value of the wave height is prone to uncertainty due to extrapolation (see Figure 3.10). Because of this, the parameter μ_1 is introduced that will be multiplied with the wave height. In order to determine the magnitude of this uncertainty, Table M.1 has been constructed. In this table, the ratio between the 84%-exceedance value and the central statistical estimate of the wave height is presented for various return periods. The same is done for the ratio between the 84%-non-exceedance value and the central statistical estimate. The 84%-exceedance value and 84%-non-exceedance value lie one standard deviation below and above the average value, respectively.

Ratio	RP [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1000**
$H_{84\%\text{-}EV}$ / H_{CSE}		0.973	0.969	0.966	0.964	0.964	0.960	0.960	0.959	0.956	0.956
$H_{84\%\text{-non-}EV}$ / H_{CSE}		1.027	1.029	1.030	1.031	1.034	1.034	1.036	1.036	1.036	1.037

Table M.1: Ratio between 84%-(non-)exceedance value and central statistical estimate of wave height for various return periods

It is assumed that μ_1 is normally distributed with an average of 1.0 and a certain standard deviation. From Table M.1 it follows that the standard deviation should either be 0.03 or 0.04. Some additional uncertainty of 1 percentage point (this is an assumed value) has been added because there is also uncertainty related to the selection of the appropriate extreme distribution function. It has been decided to use a standard deviation of 0.04 in SLS-(LD) and a standard deviation of 0.05 in ULS, as the higher return periods are more relevant for this limit state which are accompanied by more uncertainty.

- Uncertainty water level μ_2:

The parameter μ_2 has been introduced that will be multiplied with the water level, to account for its statistical uncertainty. In order to determine the magnitude of this uncertainty, Table M.2 has been constructed. The idea behind this table is similar to that of Table M.1. The ratios in Table M.2 were determined based on the water levels in which the sea-level rise has not yet been incorporated. Nevertheless, this will not have a significant influence on the overall uncertainty, as the magnitude of sea-level rise is also a prediction.

Ratio	RP [y]	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**	**1000**
$\eta_{84\%\text{-}EV}$ / η_{CSE}		0.967	0.959	0.950	0.944	0.937	0.930	0.927	0.924	0.919	0.916
$\eta_{84\%\text{-non-}EV}$ / η_{CSE}		1.024	1.030	1.038	1.043	1.044	1.048	1.051	1.054	1.057	1.058

Table M.2: Ratio between 84%-(non-)exceedance value and central statistical estimate of water level for various return periods

It is assumed that μ_1 is normally distributed with an average of 1.0 and a certain standard deviation. From Table M.2 it follows that the standard deviation should lie between 0.02 and 0.08. The uncertainty is greater for the ratios below 1, but failure will most likely occur for values above 1. Some additional uncertainty of 1 percentage point (this is an assumed value) has been added because there is also uncertainty related to the selection of the appropriate extreme distribution function. It has been decided to use a standard deviation of 0.06 in SLS-(LD) and a standard deviation of 0.07 in ULS, as the higher return periods are more relevant for this limit state which are accompanied by more uncertainty.

Model uncertainties
Model uncertainties when designing a breakwater exist on the side of the load as well as on the side of the resistance. The strenght-related model uncertainties are tackled by the empirical coefficients in the design formulae. They represent the scatter in the formula and therefore the uncertainty related to the real-life response in comparison with the calculated model response. These empirical coefficients are the Van der Meer coefficient, the acceptable stability number, the overtopping coefficients and the crown wall model uncertainty.

- Van der Meer coefficient c_{pl}:

Van der Meer set the average value of this parameter to equal 6.2, with a standard deviation of 0.4 [Ref. 6].

- Acceptable stability number $N_{s,d}$:

In Subgroup A of the PIANC report [Ref. 12] on the analysis of rubble-mound breakwaters, on page 11, the following statement can be found:

'For the Accropode, the values of 3.7 and 4.1 may be considered to be stochastic variables with a standard deviation around 0.2, giving coefficients of variation around 5%, but later results suggests that a coefficient of variation of 10% may be more appropriate.'

Using this information, the following distributions apply to the acceptable stability number $N_{s,d}$. In SLS-(LD), it is assumed that $N_{s,d}$ is normally distributed with a mean of 3.7 and a standard deviation of 0.35. In ULS, it is assumed that $N_{s,d}$ is normally distributed with a mean of 4.1 and a standard deviation of 0.40.

- Overtopping coefficients c_1 and c_2:

According to EurOtop [Ref. 11], the average values of c_1 and c_2 are 0.09 and 1.5, with standard deviations of 0.0135 and 0.15, respectively.

- Crown wall model uncertainty CMU:

Contrary to the other response formulae, there is no model coefficient in the formulae for the crown wall coefficient that accounts for the deviations between the measurements and the fit of the formula, which it why it has been introduced. Only the spread in the horizontal wave force is accounted for. In Subsection 5.1.4, it was established that the standard deviation for the dimensionless horizontal force Fh equals 0.35. Filling in the mean values in the expression for the dimensionless horizontal force results in an average of 0.47. This can thus be accounted for in Prob2B by setting the average of CMU to 1 and the standard deviation to 0.7.This is quite a large uncertainty, and some of the uncertainty will probably already be covered by the scatter in the overtopping formula. A less extreme, though quite arbitrary, standard deviation of 0.2 has been assumed.

Deterministic variables

- Notional permeability: Some uncertainty related to the parameter P probably exists, but it has been assumed that this uncertainty is already covered for by c_{pl}
- Obliqueness factor: The variations in the wave direction will not be taken into account, as there was no data available on the direction. Therefore, the obliqueness factor γ_β is taken to be deterministic and is set to 1, which translates to perpendicular wave attack
- Gravitational acceleration: Deterministic because it is a physical constant, equal to 9.81 m/s^2
- Damage parameter: The value of the damage parameter S is taken to be deterministic, as this is set as a minimum criterion
- Tolerable overtopping discharge: See explanation ‘Damage parameter’
- Nominal diameter concrete armour: The diameter of the artificial units is deterministic, because they are prefrabricated with very accurate moulds
- The crest height is assumed to be deterministic
- The parameters that describe the geometry of the crown wall (crown wall height/width, thickness vertical face/bottom slab) are all assumed to be deterministic

APPENDIX N: PIANC-CALCULATION

It might be useful to see how PIANC [Ref. 12] calibrated their partial factors. This consideration is only possible for the rock-armour layer, and as the calibration was not based on a large amount of data points, there is also a lot of uncertainty that comes with this method.

The expression for the limit state and the partial factors are presented below:

$$\frac{1}{\gamma_z} 6.2 S^{0.2} P^{0.18} \Delta D_{n50} \cot\alpha^{0.5}\, s_m^{0.25} N^{-0.1} \geq \gamma_H H_{ss}^{tL}$$

$$\gamma_H = \frac{H_{ss}^{tpf}}{H_{ss}^{tL}} + {\sigma'}_{F_{Hs}}^{\left(1+\left(\frac{H_{ss}^{3tL}}{H_{ss}^{tL}}-1\right)k_\beta p_f\right)} + \frac{0.05}{\sqrt{p_f N}}$$

$$\gamma_z = 1 - (k_\alpha p_f)$$

An explanation of the required parameters for the calculation, along with their values, is shown in Table N.1:

Description	Symbol	SLS-(LD)	ULS	Unit
Lifetime of the structure	t_L	50	50	years
Time period belonging to target probability of failure	t_{pf}	100	400	years
3 times the lifetime of the structure	t_{3tL}	150	150	years
Representative wave height, corresponding to lifetime	H_{ss}^{tL}	6.80	6.80	m
Wave height corresponding to target failure probability	H_{ss}^{tpf}	7.03	7.48	m
Wave height corresponding to 3 times the lifetime	H_{ss}^{3tL}	7.16	7.16	m
Target failure probability	p_f	0.39	0.12	-
Factor representing quality of data	$\sigma'_{F_{Hs}}$	0.1	0.1	-
Number of data points	N	141	141	-
Fit coefficient	k_β	38	38	-
Fit coefficient	k_α	0.027	0.027	-
Partial safety factor load	γ_H	1.057	1.170	-
Partial safety factor resistance	γ_z	1.025	1.057	-
Damage parameter	S	2	12	-
Rock diameter	D_{n50}	2.07	1.64	m

Table N.1: Parameters required for the calculation of the rock diameter using the semi-probabilistic approach as proposed by PIANC

NOTE 1: Other parameters (Δ, P, s_{0m}, N, tan α) are the same as in DA-1 calculation.

NOTE 2: $\sigma'_{F_{Hs}}$ typically has values of in between 0.05 and 0.15. That's why a median value of 0.1 has been adopted. The nominal rock diameter is not very sensitive to using a smaller uncertainty ($\sigma'_{F_{Hs}}$= 0.05; γ_H=1.045/1.136; D_{n50}=2.05/1.59) or a larger uncertainty ($\sigma'_{F_{Hs}}$= 0.15; γ_H=1.074/1.207; D_{n50}=2.10/1.69).