BSc Thesis Civil Engineering

Evaluating prEN 1991-1-8 for a mound breakwater from a coastal engineering perspective

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Van Oord
Marine ingenuity

Preface

This report evaluates the draft Eurocode prEN 1991-1-8 - General actions: Actions from waves and currents on coastal structures. The reason to write this report is that this will be the final step towards a Bachelor of Science in Civil Engineering at the University of Twente.

This report aims to describe how the new Eurocode prEN 1991-1-8 will compare to the conventional way of working. Therefore it is aimed at readers who have knowledge of coastal engineering and are interested in how the new Eurocode prEN 1991-1-8 will affect the design of coastal structures.

Gratitude is owed to Ir. W. Molenaar for providing the draft of prEN 1991-1-8.
Furthermore gratitude is owed to Ir. D. van Kester for providing the opportunity to perform my BSc thesis at Van Oord Dredging and Marine contractors. Finally, a lot of gratitude is owed to Ir. P. van Broekhoven & Dr. Ir. B. Borsje for their support and feedback along the way towards graduation.

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Thijmen Verheul

Summary

In Europe, there are design codes for public works called Eurocodes. These codes provide technical regulations on how to design structures. The Eurocodes focus on how loads and uncertainties should be incorporated into the design of a structure. Each member state of the European Union must accept designs based on the Eurocodes. Currently, a new Eurocode (prEN 1991-1-8) is being developed for coastal structures.

The goal of this report is to get an insight into how the new Eurocode prEN 1991-1-8 proposed way of working, compares to the deterministic approach within Van Oord. These insights are currently unknown, as only one other case study has been performed with an older version of prEN 1991-1-8. Meaning that the effect of changes within prEN 1991-1-8 is still unknown.

The main research question is: 'What are the consequences on design steps and parameters of the semi-probabilistic design approach [DA1] in prEN 1991-1-8 in comparison to the deterministic design approach, tested with a physical modelling study for a mound breakwater design case study?'
The sub-questions are:

- In which steps are there differences, in design approach and parameters, between the semi-probabilistic design approach and the deterministic design approach in prEN 1991-1-8?
- In which steps are there similarities, in design approach and parameters, between the semi-probabilistic design approach and the deterministic design approach in prEN 1991-1-8?
- How do the results for the final breakwater from both design approaches compare to a 2D and 3D physical modelling study?

These questions will be answered by the use of a case study.

The results of the semi-probabilistic design approach, proposed by prEN 1991-1-8 are on average 68% bigger in dimensions than the van Oord design. This is due to that prEN 1991-1-8 prescribes different values that are stricter for the acceptable damage parameters and higher for return periods, which leads to the difference in the final design.

All other design steps and parameters are similar between both design approaches, as well as the required data and methods for the wave studies. Due to the bigger final design dimensions, the semi-probabilistic design is not comparable to the outcome of the physical model testing. The deterministic design only differs for the toe dimensions from the outcome of the physical model testing.

So in conclusion the semi-probabilistic design approach does contain a lot of similarities with the deterministic design approach together with a physical modelling study for a mound breakwater design. This means that the way of working within van Oord will only slightly change during the design criteria set-up for the return period and acceptable damage factors but further stay the same.

Contents

1 Introduction

In Europe, there are design codes for public works called Eurocodes. These codes aim to give technical regulations on how to design structures. The Eurocodes focus on how loads and uncertainties should be incorporated into the design of a structure. Each member state of the European Union must accept designs based on the Eurocodes. It may overrule the Eurocode, only if it can demonstrate that the design is technically equivalent to a Eurocode solution. However, in practice, the pressure imposed by international clients and contractors has led to the wide adoption of the Eurocode in both public and private construction [European Commission, 2004].

At the moment of writing, there are ten Eurocodes, each having a different subject [European Commission, 2021]. However, a Eurocode is currently being developed to provide technical regulations for a subject not covered by the Eurocodes. This subject is the action from waves and currents on coastal structures. This new Eurocode is called prEN 1991-1-8 [1]. This new Eurocode should provide uniform guidance on how to deal with these actions. Currently, each country has its own set of guidelines.

Van Oord is keen to know how the changes of the new Eurocode impact the design approach for coastal structures. An example of the changes that the new Eurocode brings are new design approaches. So this report aims to evaluate the difference in design values between a design approach proposed by the new Eurocode and the deterministic design approach that is currently used by van Oord. Therefore a case study is used to compare the design values to see the effects of the different design approaches. The case study is a project that van Oord has performed in the past.

1.1 Problem statement

The problem is that there is very little known about the effects of the new Eurocode. Due to that the new Eurocode in itself is still in a preliminary state and only one publication, a case study for the IJmuiden breakwaters, is found in the literature [van Gemert, 2022]. The case study is also performed with a three-year older version of the new Eurocode. Meaning that claims made during the case study may have become unsubstantiated due to changes in the new Eurocode. This makes it difficult to provide accurate estimations of the effects of the new Eurocode. The current conventional design approach, deterministic, is not allowed anymore in the new Eurocode.

1.2 Research objective

The company that has commissioned this report is the Van Oord Dredging and Marine contractors. As there is little known about the prEN 1991-1-8 van Oord is stepping into unknown territory, when the new Eurocode is published and installed as one of the technical regulations, before research is done into the new Eurocode. Therefore, this report will help provide insight into how the new Eurocode is structured. This will help to determine the consequences the new Eurocode has on the design steps and design parameters of a breakwater for van Oord. Therefore, the objective of this research is: to find what the are the consequences of working based on prEN 1991-1-8 on the design steps and design parameters of a breakwater.

[1] The term prEN 1991-1-8 is meant when talking about the new Eurocode or draft Eurocode.

1.3 Case study

The effects will be researched by a case study. The case study will be provided by van Oord. Van Oord has indicated that the case will be about a mound breakwater, as they have done multiple projects about mound breakwaters. However, no mound breakwater is chosen before the literature study, to make sure that the case applies to the new Eurocode. Resulting in the best possible study of the new Eurocode. The case study is chosen as this provides the same basis for the design approach of the new Eurocode and the conventional design approach of van Oord. Leading to a better comparison.

1.4 Research question

A research question has been formulated to help achieve the research objective. However, for the research question, the chosen new design approach should be determined. This is done in chapter 3, as the design approach should fit the case and available data sources. The chosen design approach is semi-probabilistic [DA1]. Therefore, the research question is:

What are the consequences on design steps and parameters of the semi-probabilistic design approach [DA1] in prEN 1991-1-8 in comparison to the deterministic design approach, tested with a physical modelling study for a breakwater design case study?

1.4.1 Research sub-questions

To answer the research question some sub-questions are formulated to help answer the research question.

1. *In which steps are there differences, in design approach and parameters, between the semi-probabilistic design approach proposed by prEN 1991-1-8 and the deterministic design approach used by van Oord?*

   This sub-question will help with the comparison and determining the consequences of the differences in design steps and parameters when the new Eurocode is installed.

2. *In which steps are there similarities, in design approach and parameters, between the semi-probabilistic design approach and the deterministic design approach proposed by prEN 1991-1-8 and the deterministic design approach used by van Oord*

   This sub-question will help with the comparison and determining the consequences of the similarities in design steps and parameters when the new Eurocode is installed.

3. *How do the results for the final breakwater from both design approaches compare to a 2D and 3D physical modelling study?*

   This sub-question will show if the final dimensions of the breakwater designs are indeed able to cope with the waves. This is done by comparing the results from the 2D and 3D physical modelling studies with the final dimensions of the breakwater designs. Here can be concluded that a design approach is conservative or ambitious, as the physical modelling studies show the point of when a failure happens.

1.5 Research method

Firstly a literature study will be performed to make sure all the information is presented to make a correct design according to the new Eurocode, which is presented in chapter 2. Based on this literature study a suitable case has been picked that will fit both design approaches to make sure a representative comparison can be done, which is shown in chapter 3.

After the case is picked, the design criteria and design parameters of the study area will be determined, see chapter 4. With this information the designing of the breakwater can start, the final breakwater dimensions are then presented in chapter 5. Based on the outcome of the final breakwater design process a comparison can be made between the design process and final design results, see chapter 6. Thereafter a discussion and conclusion can be written, chapter 7 & 8. The final chapter, 9, will provide recommendations for van Oord Dredging and Marine contractors and further research.

1.6 Scope

As stated in the introduction, the new Eurocode aims to fill the knowledge gap that currently exists for actions from waves and currents on coastal structures. However, the scope of this new Eurocode is larger than what can be achieved in the time set for a bachelor thesis. A list is made that will present the scope of the thesis.

- The version of the prEN 1991-1-8 that will used throughout this whole thesis is the version that was published on 30-03-2023.
- One design approach for the new Eurocode will be picked. This makes sure that an in-depth analysis can take place, which leads to a better understanding of the effects of the new Eurocode.
- The Eurocode is analysed from a coastal engineering perspective, meaning that for example no geotechnical aspects will be considered.
- No processes that are relevant during construction are considered. Such as storms during construction.
- The parts that will be designed for the breakwater are the armour layer, underlayer and toe. These parts will be located in a cross-section in the trunk that will endure the biggest loads.
- The deterministic design approach represents the current/conventional design approach that is used.
- The research will limit itself to one case study about a mound breakwater.

2 Literature

This chapter provides the necessary literature to understand the way of working in the new Eurocode. For this thesis, relevant documents are those of the Eurocode, as this is the main subject of this thesis. Next to this, existing literature will be mentioned. Furthermore, the deterministic design approach will be explained.

2.1 Eurocodes

As stated earlier there are 10 different Eurocodes. In figure 1 an overview is presented on what the flowchart is for the use of the Eurocodes. From this chart, it can be seen that the relevant literature is in EN 1990, as EN 1990 forms the basis for all the other Eurocodes. Furthermore, it can also be seen in figure 1 that the new Eurocode will be an addition in the second step of the flow chart, as the new Eurocode is part of EN 1991.

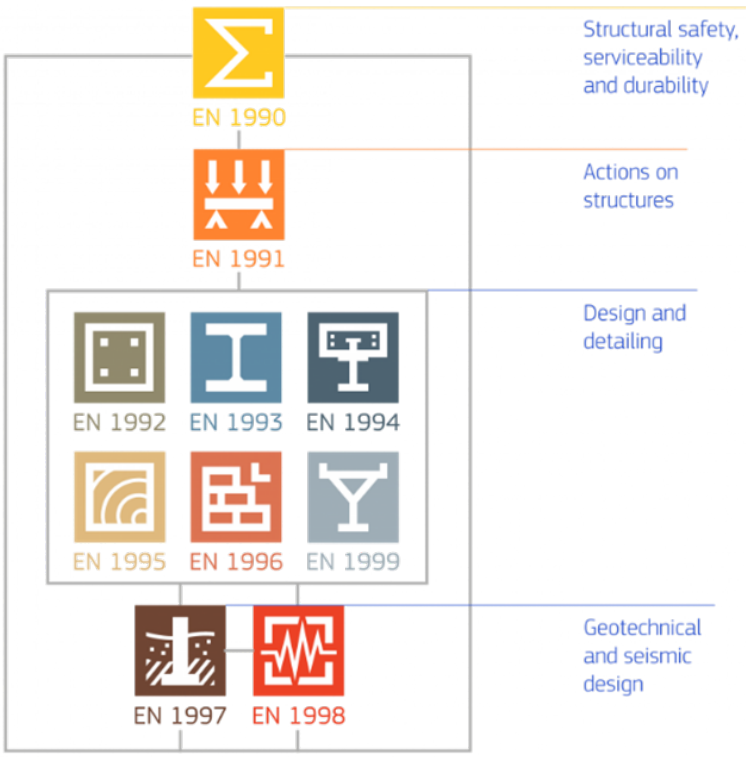


Figure 1: Overview of the Eurocodes [European Commission, 2021].

EN 1991 covers multiple topics which can be seen in table 1. For this thesis, the most relevant topic is prEN 1991-1-8, as the information in this Eurocode is what this thesis is about. Furthermore, other Eurocodes that seem relevant are EN 1991-1-7 and EN 1991-3. However, EN1991-3 falls out of the scope, as it is focused on the structural equilibrium of buildings. The topics in EN 1991-1-7 fall out of the scope with prEN 1991-1-8. All the other Eurocodes are also not relevant as these fall out of the scope of coastal engineering or a part of the construction phase.

Eurocode	Topic
EN 1991-1-1	General actions - Densities, self-weight, imposed loads for buildings
EN 1991-1-2	General actions - Actions on structures exposed to fire
EN 1991-1-3	General actions - Snow loads
EN 1991-1-4	General actions - Wind actions
EN 1991-1-5	General actions - Thermal actions
EN 1991-1-6	General actions - Actions during execution
EN 1991-1-7	General actions - Accidental actions
prEN 1991-1-8	General actions - Actions from waves and currents on coastal structures
EN 1991-2	Traffic loads on bridges
EN 1991-3	Actions induced by cranes and machinery
EN 1991-4	Silos and tanks

Table 1: Overview of topics in EN 1991 [European Commission, 2021].

2.2 EN 1990

EN 1990:2019 makes use of the partial factor method, which is the same as the semi-probabilistic approach. This means that not a global safety factor is used, as would be done in a deterministic approach, but a partial factor.

2.2.1 Design values

Design values are made up of two parts, the characteristic value and a partial factor. The characteristic value is the value of the action (F_k) and the partial factor is a value that will factor in the possibility of deviations that are not wished for. The design value should be bigger than the resistance to be considered as safe. The resistance is the capacity of a structure or parts of a structure to withstand actions without failure.

There are three different types of actions these are: permanent action (G) such as self-weight, variable action (Q) such as wind or snow loads and accidental actions (A) such as explosions or impact from vehicles. However, with just the action itself, the characteristic value is not yet specified enough and differs for different types of action. Permanent actions (G_k) are determined by the following statements in chapter 4.1.2 of EN 1991.

- If the variability of G can be considered as small, one single value Gk may be used.
- If the variability of G cannot be considered as small, two values shall be used: an upper-value $G_{k,sup}$ and a lower-value $G_{k,inf}$.

Variable actions (Q_k) are determined by the following statements in chapter 4.1.2 of EN 1991.

- 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.
- A nominal value, which may be specified in cases where a statistical distribution is not known.

Accidental actions (A_d) should be specified per project. See EN 1991-1-7 for more information.

2.2.2 Limit states

The partial factor method makes use of multiple limit states that should be designed so they do not fail. A limit state is the point at which the design object will just hold, so the failure point will be reached with an increase in load. The EN 1990 uses two limit states, the ultimate limit state (ULS) and the serviceability limit state (SLS). The ULS means the point just before the total failure of the object. SLS means the point where the service of the object is not able to perform anymore. So in short the ULS means that the object will harm safety if exceeded and the SLS means the object will harm comfort/functioning if exceeded.

EN 1990 states in chapter 6.4.1 the relevant topics for the ULS which should be checked if the structure will uphold, if this is the case the structure is considered safe. These topics are:

- **EQU:** The failure is related to the loss of static equilibrium of the structure or structural members.
- **STR:** Internal failure or excessive deformation of the structure or structural members.
- **GEO:** Failure or excessive deformation of the ground where the strength of soil or rock is significant in providing resistance.
- **FAT:** Failure due to fatigue of the structure or structural members.
- **UPL:** Loss of the structure's or the foundation equilibrium due to upward water pressure or other vertical pressures.
- **HYD:** Hydraulic ground rupture, due to internal erosion due to groundwater flows (piping)

2.2.3 Consequence classes

When designing a structure it is important to know the reliability level of the structure that it should be designed for. When this is known it can be implemented into the design process. There are three different consequence classes (CC). The three consequence classes are:

- CC3: High consequence for loss of life or economic, such as grandstands or concert hall.
- CC2: Moderate consequence for loss of life or economic, such as an office building.
- CC1: Low consequence for loss of life or economic, such as storage buildings.

Each consequence class represents a reliability level. The reliability level is reflected in the value of β and can be used for the ULS and SLS. The β values are shown for a reference period of 50 years and can be found in table 2.

Another way to distinguish between the reliability levels is by a multiplication factor that is applied to the partial factor for actions. This factor is called KFI, the different multiplication factors are: 0.9 (CC1); 1.0 (CC2); 1.1(CC3).

Consequence class	β (ULS)	β (SLS)
CC3	4.3	1.5
CC2	3.8	
CC1	3.3	

Table 2: Overview of reliability values for the consequence classes.

2.2.4 Combined actions

There are 3 different types of design situations. A design situation is a set of physical conditions that represent a real condition for which the limit states of the proposed design are not exceeded. Three different types of design situations are mentioned in EN 1990, these are:

- Persistent design situation; the structure is under normal use and exposed to weather conditions.
- Transient design situation; the structure is under construction or repairs.
- Accidental design situation; the structure is set on fire or experiences an explosion or local failure.

Persistent and Transient design situations are a fundamental combination. For fundamental combinations, equations can be used to determine the design value of the actions (Ed) for the ULS, as multiple actions can happen at the same time. These equations are shown in appendix A.1 [CEN/TC250, 2019]

2.3 prEN 1991-1-8

The layout of the new Eurocode is as follows, firstly the Eurocode provides background information on what data to collect, which design approaches may be used and other relevant information. Hereafter the new Eurocode breaks down the design of different coastal structures.

2.3.1 Scope of prEN 1991-1-8

The new Eurocode will provide rules to determine the wave and current actions for structures in the coastal zone. The coastal zone is defined as the locations where waves and currents are affected by the seabed or shore. Furthermore, the new Eurocode provides principles on how to define design sea conditions. The structure types that are included in the new Eurocode are: fixed cylindrical structures, suspended decks, sub-sea pipelines, rubble mound breakwaters, vertical face breakwaters, composite breakwaters, revetments, seawalls and permanently moored floating structures.

The new Eurocode does not cover everything in the coastal zone the following is out of the scope: flood risk management structures (e.g. dikes or levees), port structures like piers or jetties and installations for mooring and berthing ships. Furthermore the effects of tsunamis, accidental breakdown of retaining structures, waves from passing ships and currents produced by jets or propellers.

The mentioned return periods, importance factors & HEA levels in the draft Eurocode are still open to NDP, which stands for Nationally Determined Parameter. Meaning that the national annexe can prescribe different return periods, importance factors & HEA levels. So the effects of the draft Eurocode can still become larger or smaller on the current way of working, depending on the national annexe.

2.3.2 Design approaches

In the new Eurocode there are 4 different design approaches, these are:

- **DA1: Semi-probabilistic design approach**
  Partial factors are used for loads leading to sensitivity testing of key parameters.
- **DA2: Probabilistic design approach**
  Allowable probabilities of failure or β indexes are used.
- **DA3: Risk informed design approach**
  The use of socio-economic information is used to determine the optimum probability of failure. An example could be determining the target reliability based on a risk assessment that includes the failure consequences, but also all the costs related to building and maintaining the structure.
- **DA4: Design by assisted testing approach**
  Must be in combination with one of the other design approaches. This method is used for physical model tests that are used to validate the assessment of the wave and current actions.

More guidance on the partial factor method is provided in chapter 2.3.8. For DA2 the β values are presented in chapter 2.2.3 and are sourced from EN 1990. No guidance is provided for the target reliability based on a risk assessment in the new Eurocode for DA3.

2.3.3 Actions and Loads

The new Eurocode states that actions from waves and currents should be considered variable-free dynamic actions. Furthermore, the actions on structural parts produced by the global dynamic response on fixed structures exposed to waves and currents, are considered as direct actions. Next to this, actions from waves and currents on fixed coastal structures can be modelled by the equivalent quasi-static actions during the metocean events, if no dynamic analysis is performed. Hydrodynamic loads include the following.

- Pressures and forces of waves or currents
- Forces/moments along structural parts of the structure.
- Mean wave overtopping discharge acting as the indirect load of parts of the coastal structure

Hydrodynamic loads are calculated by the use of the Deep-Sea Extremes Methods (Ds-EM), the subdivisions of Ds-EM are the Marginal Deep-sea Extremes Method (MDs-EM) and the Joint Deep-sea extreme method (JDs-EM). MDs-EM looks into only 1 parameter, and JDs-EM looks into two parameters. Another method is the Full Transfer Approach (FTA). Both methods have certain steps that should be taken.

Deep-Sea Extremes Methods (Ds-EM)

1. Select small sets of offshore metocean parameters based on statistical analysis, using MDs-EM or JDs-EM.
2. Transfer the sets to the coastal structure site.
3. Identify the design values of the hydrodynamic loads as the most unfavourable for the structure.

Full Transfer Approach (FTA)

1. Transfer a large set of the most extreme metocean events in the data to the site of the coastal structure.
2. Identify the design values of the hydrodynamic loads based on an extreme value analysis.

2.3.4 Design situations

In the new Eurocode, there are four design situations these are:

- Persistent: Everyday use, but also the severe conditions that happen irregularly.
- Transient: Construction and maintenance of the coastal structure.
- Fatigue: Repetition of load cycles from wave and current actions
- Seismic: The possibility of tsunamis caused by earthquakes.

What is different in the new Eurocode to the EN 1990:2022 is that there can be multiple specific design situations of the same design situation, e.g. a low water persistent design situation for the toe berm height and a high water level persistent design situation for the crest height.

It is suggested to be helpful to create different design situations for the different components of the structure, but also at the beginning and the end of the life cycle to make sure climate change is incorporated into the design.

2.3.5 HEA

The new Eurocode has a new design concept to determine the consequence class based on metocean parameters and based on this design approach. This new concept is called the Hydrodynamic Estimate Approach. In the table 3 guidance is presented on how to choose which HEA level is appropriate based on the consequence class. (table 4.1 in prEN 1991-1-8)

The hydrodynamic uncertainty refers to the quality of the metocean data, environmental sea conditions at the project site and the complexity of local physical processes. In the chapter **??** guidance on the Hydrodynamic Estimate Approach methodology can be found. Two examples are presented in the new Eurocode to provide some more explanation for hydrodynamic uncertainty.

Consequence class	Hydrodynamic uncertainty		
	Low	**Medium**	**High**
CC1	HEA1	HEA1	HEA2
CC2	HEA1	HEA2	HEA3
CC3	HEA2	HEA3	HEA3

Table 3: HEA level selection matrix.

1. Examples of low hydrodynamic uncertainty can include: tidal range < 1 m, surge $< 0, 5$m, fetch-limited seas (with fetch < 10 km), uniform currents with spring tide velocities < 1 m/s, regular bathymetry.

2. Examples of high hydrodynamic uncertainty can include: tidal range > 5 m, surge > 2 m, ocean seas (swell and wind-waves), non-uniform currents (stratified) and/ or tide or surge current velocities > 3 m/s, irregular bathymetry (e.g. reefs or sub-sea canyons).

2.3.6 Structure design uncertainty

Based on the outcome of table 3 a design approach can be picked by looking at table 4 (table 4.3 in prEN 1991-1-8)

HEA Level	Low to medium structure design/response uncertainty	High structure design /response uncertainty
HEA1	DA1	Not applicable
HEA2	DA1 or DA2	DA1 + DA4 or DA2 + DA4
HEA3	DA1 or DA2 or DA3 or any previous with DA4	DA1 + DA4 or DA2 + DA4 or DA3 + DA4

Table 4: Design approach selection.

Structure design uncertainty is according to the new Eurocode:
'Low structure uncertainty can, for example, apply where the physical processes/ response mechanisms are relatively simple and/ or there is an established and validated structural analysis approach, whereas high structure uncertainty may apply where the physical processes/response mechanisms are complex and/ or there are several analytical methods available giving widely varying results and/or the conditions are significantly outside the application limits of an established structural analysis approach.'

More specific to mound breakwaters the new Eurocode gives the following explanation and examples for low to medium structure design uncertainty in chapter 7.2.1.

Low to medium structure response uncertainty in mound breakwaters is when the limit states are reached after the breakwater is exposed to two or more waves higher than the designed for in that specific limit state. Next to this, enough data is available of structures in similar conditions.

- Breakwaters whose slope protection is made of two armour layers.
- Berm breakwaters
- Reshaping submerged breakwaters

For high structure design uncertainty, it provides the following examples.

- Breakwaters whose slope protection is made of a single layer of armour units.
- Armour units not widely used in practice.
- The crest structure is subject to impulsive loads or heavy wave overtopping.
- The toe of the structure is attacked by breaking waves.
- A breakwater head is under severe wave attack.
- Non-standard build-up of layers is implemented.
- The design of berm breakwaters or submerged ones is not supported by enough data and widely accepted formulae.

2.3.7 Importance factor

The new Eurocode introduces a new term called the importance factor (φ_1). This factor is used for hydrodynamic load verification for the limit states ULS and SLS. The importance factor is multiplied by the characteristic marginal return period and the characteristic joint return period of metocean events to get the needed design value. This design value is then multiplied with the corresponding partial factor.

For the SLS-SDi the design value is the nominal value from a high design period, without partial factor. The importance factor is related to the consequence class of the structure. In table 5. values are provided for the importance factor.

Consequence class	**Value of importance factor φ_1**
CC3	2.0
CC2	1.0
CC1	0.5

Table 5: Importance factors for verification of SLS and ULS.

The importance factors are based on a lifetime of 50 years. The importance factor can be calculated for lifetimes that are longer than 20 years. This is done by dividing the design service life by 50 and multiplying that with the importance factor that corresponds with the consequence class as found in table 5.

2.3.8 Return periods

The target reliability for DA2 is based on the β values mentioned in chapter 2.2.3. For DA3 also a risk analysis should be performed that will determine the target reliability. How the risk analysis should be performed is not specified.

For DA1, the return periods for each design approach are the same for the permanent and transient design situations. For other design situations, no return periods are given. It should also be noted that the new Eurocode mentions that the transient design situations may have shorter return periods. However, it is not mentioned how much shorter is allowed.

Tables 4.5, 7.1 & 7.2 provide the return periods for all the structures and consequence classes. These tables are presented in appendix A.3.

Where tables 4.5 and 7.1 prescribe the return periods for the SLS and table 7.2 the ULS. Table 4.5 presents the return periods for all coastal structures in the scope of the new Eurocode. Table 7.1 presents the SLS values for the 'alternative case-specific' method. The alternative case-specific are specially developed for sloped breakwaters for which it is hard to determine the sensitivity of the breakwater components, such as the toe.
But also for, the level of safety that the design formulae can provide. For a low level of safety, it is advised to use the alternative case-specific values. However, no classification of when the level of safety is high or low is provided.

Table 23 provides an overview of which table should be used per structure.

Type of structure	**Return period table(s) in draft Eurocode**
Mound breakwater	Table 4.5, 7.1 & 7.2
Vertical face breakwater	Overtopping: table 7.1 & 7.2. Other failure types: table 4.5
Composite breakwater	SLS: table 4.5, ULS most conservative between table 4.5 & 7.2
Fixed cylindrical structures & suspended decks	Table 4.5
Floating structures	Table 4.5
Revetments (Coastal embankment)	Table 4.5, 7.1 & 7.2
Seawalls (Coastal embankment)	Overtopping: table 7.1 & 7.2. Other failure types: table 4.5

Table 6: Overview of return period tables.

The return periods are related to the partial factors. As an example, the 2000-year ULS return period is compared to a return period of 100 years with a partial factor of 1.35. The background document to the draft Eurocode mentions that higher return periods keep closer to physics than lower return periods with partial factors. Additionally, the background document states that the partial factor method is 11% more conservative than the nominal return periods [CEN/TC250, 2023].

2.3.9 Data requirements

The data that should be collected for metocean design description are wave height, wave period, water level and current velocity, which are the primary variables. The covariates that also should be included in the metocean design description are wave direction, spectral width, directional spreading, storm (peak) duration, current shear stress, current direction, current turbulence intensity and wave setup. Furthermore, surges and high atmospheric pressures should also be taken into account. It can be that wind is also relevant, if so wind speed, wind direction, wind gust and wind setup should be taken into account. The wave data set should preferably be in the range of 15 to 30 years. Shorter than 15 years is accepted when annual variations are accounted for.

The approach for the determination of storm-representative parameters should fit with the chosen HEA level and design approach. For HEA2 & HEA3 wave data should be obtained from in-situ measured data and numerical data. The transformation of wave data from offshore to nearshore shall be done by one or more of the following methods.

- Numerical method
- Physical model test
- Empirical method

Furthermore, a vertical datum for the water level should also be determined. Based on this datum the mean sea level, the mean high & low water levels, extreme high & low water levels and (spring) tidal range should be expressed. The long-term sea changes due to climate change or geological reasons should be taken into account when determining the above-mentioned parameters.

2.3.10 Extreme value analysis

The extreme value analysis should preferably be performed with the Peaks-Over-Threshold (POT) method. However, the annual maximum method and total sample method may also be applied. The total sample method should be used in situations with limited amounts of data, as this method uses all the data points. POT and the annual maximum method, only use a small set of all the available data points.

[CEN/TC250, 2023]

2.4 Existing literature

There is only one publication on the prEN 1991-1-8, however, this was the 2020 version, where the deterministic design approach was still in the Eurocode. The publication was a case study of a breakwater in IJmuiden, which was constructed in 1960, into the differences between the semi-probabilistic, full probabilistic and deterministic design approaches. The breakwater was located in intermediate water conditions with medium hydrodynamic uncertainty. The project was determined as CC2 and this leads to HEA2.

Van Gemert found that there are indeed differences in the final design dimensions as can be seen in table 7. The author does not state the actual dimensions of the breakwaters, only that DA-2 is the 'most correct' design outcome [van Gemert, 2022].
On average the semi-probabilistic approach yields dimensions that are 21% smaller than the deterministic approach. For full probabilistic, the dimensions are 13% smaller on average than the deterministic approach. It must be noted that for the ULS of rock size, the semi-probabilistic approach does yield a bigger dimension.

Therefore the author suggests that the deterministic design approach is conservative, leading to overestimation of the dimensions. However, this does mean that it makes sure that target reliability will be met.

Furthermore, it was noted that the results of the deterministic approach lay the closest to those of the full probabilistic. This was not expected as the semi-probabilistic design approach is closer related to the deterministic design approach than the full probabilistic design approach. It must be noted that the author claims that the semi-probabilistic

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 7: Overview final design dimensions [van Gemert, 2022].

approach was lacking structure and guidance on assumptions that are needed for the calculations and could have led to wrongful answers [van Gemert, 2022].

The case study was located in intermediate water depth, which is the transition zone from deep water to shallow water, however, it was chosen to design the case according to deep water formulae, as there is more design experience with the deep-water formula.

2.4.1 Knowledge gap

The gap in the existing is the lack of verification. The findings of van Gemert are the only findings of the new Eurocode. Furthermore, it is unknown how the design approaches will differ in results in different water conditions or different bathymetry. Additionally, it is also unknown if in a similar case, the design approaches will provide the same type of results.

Next to this, the new Eurocode has undergone three years of development, which could lead to different design results even for the case of the breakwater in IJmuiden.

2.5 Deterministic design approach

In this chapter, the deterministic design approach of a breakwater case of van Oord will be explained [van Oord, 2013].

It is important to note that the client already had a base design before van Oord started designing. First, the design criteria are determined. The design criteria consist of the following points:

- General specification of the project, e.g. breakwater length.
- Design working life, amount of years the structure is designed to perform.
- Design condition, the probability of exceeding the design conditions due to a meteorological event is specified.
- Breakwater armour stability, the acceptable amount of damage or slope deformation of the breakwater is determined.
- Rock and concrete properties, the density of rock armour and concrete are presented.

- Breakwater wave overtopping, requirements for wave overtopping are set for which the design cannot go over.
- Crest wall stability, a safety factor is introduced for the sliding and overturning of the crest. This is then tested in a 2D physical model test.
- Seismic conditions. Earthquake movement that should be taken into account is mentioned.

After the design criteria are finished the basis of design is formulated. Here parameters and design values are determined that can be used for the design. This consists of two parts, general design parameters and hydraulic design parameters.

General design parameters

- Reference vertical level, the vertical reference level for tidal and bathymetry. E.g. mean sea level.
- Offshore wind conditions, the direction and velocities of the wind are presented.

Hydraulic design parameters

- Tidal levels, tidal harmonic analysis are performed and the spring tidal range is calculated.
- Sea levels, multiple values about the sea level are presented, such as mean sea level, mean monthly highest water level, etc.
- Design conditions at breakwater, extreme wave conditions have been determined by the use of statistical analysis.
- Current speeds, the speed of the omnidirectional current is presented.
- Water density, the density of the water at the breakwater is presented.

With this information, the calculations for the breakwater can be performed. The design approach will look like the following:

1. Verifying the design of the client.
2. Calculating the armour size for perpendicular waves.
3. Factoring in the oblique wave attack.
4. Determining the toe berm armour.
5. Calculating the crest height.
6. Geotechnical stability assessment.
7. Settlement and consolidation assessment.

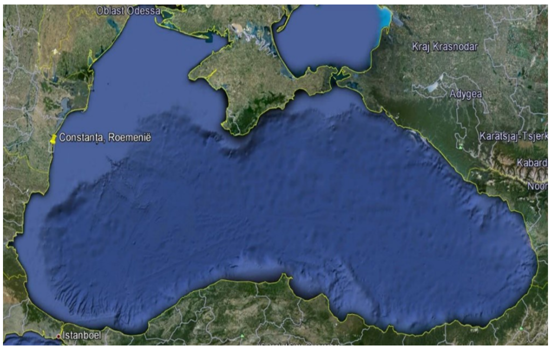
2.6 Conclusion

Furthermore, it is chosen to ignore accidental actions, as these are covered by EN 1991-1-7. This also means that no accidental design situation is applied, only the persistent and transient design situations will be applied to the breakwater.

3 Case

3.1 Costanza Breakwater extension

As stated in chapter 1.5, the mound breakwater case will be chosen based on the literature study. The chosen case is a breakwater extension in the harbour of Constanța, Romania that was constructed in 2014 [van Oord, 2013]. Figure 2 shows that Constanța is located by the black sea and where the breakwater extension is placed. A cross-section of the breakwater is shown in figure 3.



(a) Location of Constanța.

(b) Breakwater extension in yellow.

Figure 2: Locations of Constanța and breakwater extension [Google Maps, 2024].

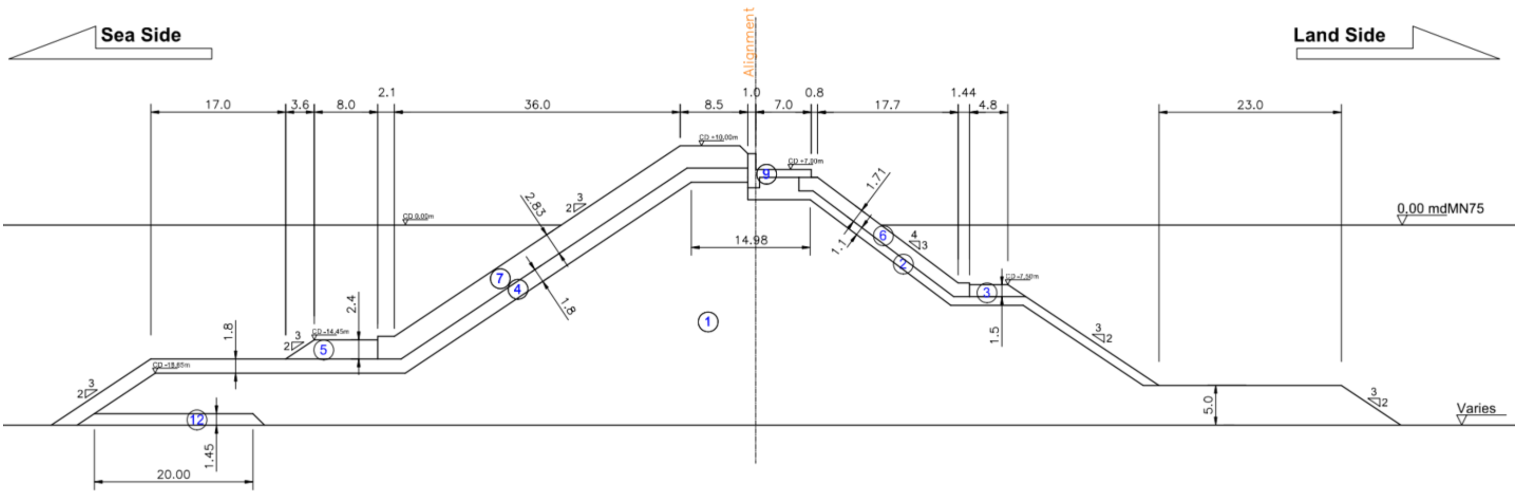


Figure 3: Cross-section of breakwater extension [van Oord, 2013].

3.1.1 Provided documents

Van Oord has provided the following documents for this case.

- Tender documents from the client [Port of Constanta, 2009]
- Wave conditions studies [Arcadis, 2013]
- Design drawings [van Oord, 2013]
- 2D and 3D model test reports [Artelia, 2013]
- Basis of design & calculations [van Oord, 2013]

3.1.2 General classification

Based on the data from the provided documents a starting point can be made. Table 8 provides an overview of the starting parameters for the case study. After table 8 a step-by-step approach from the new Eurocode, as explained in the literature study, is applied to determine the HEA level and design approach.

Parameter	**Value**
Spring tidal range	0.06 m
Tidal range	Negligible
Water depth	22.7 m
Design working life	50 years
Return period storms	100 years
Significant wave height at breakwater	7.5 m
Peak wave period	12 sec
Slope	2:3
Length of extension	1050 m
Rock grading of core of breakwater	100-500 kg
Density of seawater in the Black Sea	1018 kg/m^3

Table 8: Overview of parameters Constanța breakwater extension case [van Oord, 2013] & [Port of Constanta, 2009].

The first step towards a design is determining the hydrodynamic uncertainty and consequence class. For this case, the hydrodynamic uncertainty can be classified as medium, as this best represents the complexity of the local physical processes and metocean data.
The classification of medium is chosen as the case contains examples of low and high uncertainty presented in chapter 2.3.5, e.g. low tidal range, but with open seas as swells and wind waves are present.
It can be noted that no examples of medium uncertainty are presented, this is due to that the new Eurocode does not present examples of medium uncertainty.

The consequence class is determined as CC2, as damage to the breakwater extension will result in moderate consequences for loss of life or economic loss. It could be argued that CC1 and CC3 would even be appropriate. However, as this case is about the extension of the breakwater, it is assumed that the current breakwater will be intact and therefore limit the impact of the damage of the breakwater extension. But it can be noted, that no clear guidance is given on how to determine the consequence class in the draft Eurocode.

Based on the hydrodynamic uncertainty, the HEA level can be determined based on table 3. The outcome of table 3 is HEA2. Now the data should comply with the guidance given in the table provided in appendix A.2.

The provided wave climate and conditions by Van Oord do contain a long-term time series of metocean parameters (16 years), which have undergone a numerical wave transition with the use of a SWAN model. Which fits with HEA2 as shown in appendix A.2. The wave conditions study provides data for return periods up until 200 years. The wave conditions study is done according to the Marginal Deep sea Extremes Method (MDs-EM) and the extreme value analysis is done by the Peaks-Over-Threshold (POT) method.

The available data was in situ and of a time range of 16 years, which is between the 15-30 years the new Eurocode prescribes. The wave study also complies with the mentioned requirements in chapter 2.3.9 for wave studies.

The structure design uncertainty is determined as high, this as the armour layer consists of a single layer of armour units (Accropodes II). A single armour layer is the first example the new Eurocode gives for high structure design uncertainty, see chapter 2.3.6. It must be noted that this is the only example of high structure uncertainty for this case study. It could be argued that the case also fits low/medium structure uncertainty, as van Oord has built similar breakwaters in the same conditions in Constanța. Which is one of the definitions of low to medium structural uncertainty.

Based on the HEA level and structure design uncertainty, table 4 shows that the semi-probabilistic design approach [DA1] in combination with physical testing [DA4] or the probabilistic design approach [DA2] in combination with physical testing [DA4] is appropriate. The chosen design approach is DA1. Due to DA1 being the standard design approach in the new Eurocode. Next to this, the semi-probabilistic design approach fits better with the complexity and given time for a BSc thesis. However, it must be noted that physical testing is not possible for the design produced in this report, as physical testing is expensive and there is no budget for it. But, there have been physical tests performed for the van Oord design.

Furthermore, the water conditions can be calculated for the 100-year return period based on the significant wave height and the water depth at the toe. By dividing the depth by the significant wave height the water conditions can be found. For this case, this leads to a value of 3.03, which is above 3 and therefore deep water conditions [CIRIA et al., 2007].

3.2 Argumentation

This case is chosen based on that the water conditions differ from the case of van Gemert.

The Constanța case is located in deep water instead of intermediate water (3.03 vs 2.53) for the 100-year return period. So the formulae for deep water will indeed be used for a deep water case. Furthermore, for this case, the design documents of the breakwater are available just as a 2D and 3D physical modelling study. With this information, the comparison between the design approaches has more depth. Moreover, The IJmuiden breakwater gets often repairs, which points to that the base design is not equipped for the conditions it has to face. Therefore it is possible that it can lead to a misrepresentation of the results of the new Eurocode. The Constanța breakwater extension is already multiple years in use without problems, therefore it can be concluded that the breakwater can withstand the 1:10 conditions. However, the Constanța breakwater has not yet been exposed to the 100-year return period storm.

Additionally, as the Constanța breakwater was designed in 2013 and the IJmuiden breakwater in 1960 there are differences in the formulae used. Simply due to new research, literature has been published in the meantime. Therefore, a recent case will have a better comparison of the consequences of the new Eurocode. As the Eurocode is also based on the recent design literature.

4 Basis of Design

This chapter has the goal of determining the basis of design. This will consist out of general design rules based on chapter 7 from pr EN 1991-1-8. Hereafter, the data and used method that is needed for each breakwater part is presented.

4.1 General design

4.1.1 Structure characteristics

The structure characteristics are explained in chapter 3.1. So the following list can be made for the structure characteristics:

- Consequence class 2
- HEA level 2
- Semi-probabilistic design approach [DA1]
- Design working lifetime (T_{life}) is 50 years, as required by the Ministry of Transport of Romania [Ministerul Transporturilor, 2003].

4.1.2 Limit states classification

Only three parts of the breakwater will be designed of the breakwater in this report. These are the armour layer, underlayer and toe. The armour layer will be designed twice, one design with rock and one design with artificial concrete units. For each of these parts, except the underlayer, accompanying limit states should be determined.

In table 9 the limit states for each breakwater part are presented. For the armour layers, the limit states are SLS and ULS. This is determined according to chapters 7.2.2 and 7.2.3 in prEN 1991-1-8. For the toe berm, nothing is mentioned. However, the toe berm provides stability to the armour units [van Gent and van der Werf, 2014]. Therefore, the toe berm will also be checked for SLS and ULS conditions.

Breakwater part	**Applicable limit states**
Armour layer - Rock size	SLS & ULS
Armour layer - Concrete size	SLS & ULS
Toe berm	SLS & ULS

Table 9: Limit states for the breakwater parts.

4.1.3 Return periods

The return periods for each consequence class and limit state return periods are determined by tables 4.5, 7.1 and 7.2 (table 20, 21 & 22 in this document) in the new Eurocode. As T_{life} is 50 years, the value of the importance factor will not change. The importance factor for CC2 is 1, so no multiplications will have to be performed that can alter the return period. The return periods are shown in table 10.

Limit state	Return period	New Eurocode table
SLS	100 years	table 4.5
SLS (Alternative case specific)	400 years	table 7.1
ULS	2000 years	table 7.2

Table 10: Marginal return periods for CC2 and medium dependence.

As mentioned in chapter 3.1.1, the data set available only has the return periods till 200 years. However, for the alternative case-specific SLS and ULS, the return periods needed are 400 and 2000 years for CC2. To obtain T_m and H_s for these return periods the data has to be extrapolated. This is done by plotting the data and applying multiple trend-lines, the best-fitting trendline is logarithmic. Based on the logarithmic trendline the data is extrapolated. The wave data is then checked if it is realistic, by checking if the wave steepness and wave breaking of the extrapolated data are valid, see appendix B.1.

4.1.4 Design situations

As mentioned in chapter 2.6 the chosen design situations are persistent design situation and transient design situations. However, the new Eurocode does not present different return periods for a persistent or transient design situation.

4.2 Data collection

In this sub-chapter, the needed data is shown for each component. All the required methodologies and determining of the data are shown in appendix C and sourced from the rock manual, as required by the Eurocode [CIRIA et al., 2007].

4.2.1 Armour layer - Rock

To determine the rock stability of the armour layer the Van der Meer design methodology for deep water is in box 5.13 of the Rock manual will be used, as prescribed in chapter C.3.4. in the new Eurocode. The design methodology consists of seven different steps [CIRIA et al., 2007]. These steps are shown in appendix B.2. Based on the Van der Meer design methodology, the needed parameters are shown in table 11.

Parameter	100 years	400 years	2000 years	Source
H_s (m)	7.50	8.71	10.17	[Arcadis, 2013]
T_m (sec)	10.90	11.94	13.1	[Arcadis, 2013]
Storm duration	12 hours			[van Oord, 2013]
Slope	Seaside: 1:1.5			[Port of Constanza, 2009]
S_d	SLS = 2		ULS = 8	[CEN/TC250, 2023]
c_{pl}	6.2; (σ = 0.4)			[CIRIA et al., 2007]
c_s	1; (σ = 0.08)			[CIRIA et al., 2007]
Permeability	0.4			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			

Table 11: Input for rock armour stability.

4.2.2 Armour layer - Concrete

To calculate the concrete armour layer fewer steps have to be taken than in comparison with the rock armour layer. The concrete armour units use the Hudson formula, as prescribed in chapter C.3.5. in the new Eurocode.

The concrete armour unit that is chosen to be used in the design is Accropode II, as this product is also used in the design by Van Oord. By using the same product the final values are better comparable. The data for the Accropode II armour-layer is presented in table 12, the full determination can be found in appendix B.3.

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.50	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Seabed Slope	0.17%			[van Oord, 2013]
K_D	SLS = 12		ULS = 16	[CIRIA et al., 2007] & [CLI, 2012]
Slope	Seaside: 1:1.5			[Port of Constanza, 2009]
$\rho_{concrete}$	2350 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.31			
N_s	SLS = 2.5	SLS = 2.5	ULS = 2.7	[CIRIA et al., 2007]

Table 12: Input for Accropode II armour stability.

4.2.3 Toe berm

The toe stability is calculated with the use toe stability formula of van der Meer, as stated in chapter C.3.6 in the new Eurocode [CEN/TC250, 2023].

In table 13 the data for the toe berm calculation is presented, the full calculation can be found in appendix B.4.

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.5	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Depth bottom toe (m)	-17.2 MSL			[van Oord, 2013]
h (m)	-24 MSL			[van Oord, 2013]
Design low water level (m)	-0.6 MSL			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			
N_{od}	SLS = 0.5	SLS = 0.5	ULS = 4	[CEN/TC250, 2023]

Table 13: Input for toe stability.

4.2.4 Underlayer of rock component

The underlayer of the armour layer or toe berm protects against erosion of material located below the armour layer or toe berm. Additionally, the underlayer will help the interlocking of the stones in the armourlayer or toe berm. The new Eurocode states in point 7.3.9. (4) that the Coastal Engineering Manual VI.5-3 or the Rock Manual (2007), 5.2.2.10 may be used [CEN/TC250, 2023]. This can be summarized into two design steps, which are shown in appendix B.5.

4.2.5 Underlayer of Accropode II

The design of the underlayer of the Accropode II has a different first step than the underlayer design of a rock component, this different step is shown in appendix B.6.

4.3 Sensitivity analysis

For each designed component a sensitivity analysis will be performed. Every component will also be calculated according to different consequence classes, as no clear guidance is given on how to choose this. As the T_{life} stays 50 years, the importance factor will not change, hence the return periods also do not change for the different consequence classes. For CC1 the return periods are: SLS; 50 years & 200 years, ULS; 1000 years. The return periods for CC3 are: SLS; 200 years & 800 years, ULS; 4000 years.

Additionally, each component is also designed by the use of a partial factor of 1.35 on significant wave height. Which should lead to an 11% bigger design according to background documents of the new Eurocode [CEN/TC250, 2023]. It can already be noted that the 2000-year return period for significant wave height is factor 1.346 bigger than the 100-year return period. This means that a bigger design by the use of a partial factor of 1.35 with a 100-year return period is unlikely.

For the sensitivity analysis of the rock armour layer, the surging and plunging coefficients will increase and decrease by 1 standard deviation. The 1 standard deviation increase is written down as 'c_{pl} and c_s High', low would mean a decrease of 1 standard deviation.

The sensitivity analysis of the Accropode II armour layer is done by using the provided stability numbers (N_s) by the rock manual. As these stability numbers themselves contain a partial factor, it is chosen to increase and decrease these values with +-5%.

For the toe berm the sensitivity analysis is performed by using 3 different low water levels, these water levels have been determined based on the lowest ever recorded water level (-0.3m) and the design low water levels of van Oord (-0.6m). Furthermore, the depth in front of the breakwater varies between -24 and -25 meters. So for each depth, all low water levels will be applied, leading to 6 designs per return period.

For the underlayer, no sensitivity analysis is performed, as the underlayer is directly related to the armour layer. Meaning that the armour layer has the largest influence on the outcome of the underlayer.

5 Results

In this chapter, the results of the calculations that determine the dimensions will be presented for each component of the breakwater. The calculations are presented in appendix C. The conclusions of the sensitivity analysis are also presented. The sensitivity analysis itself can be found in appendix D.

5.1 Armour layer

In this sub-chapter, the armour layer will be calculated. Both rock and Accropode II armour layers are calculated to determine which is a better fit.

5.1.1 Rock armour layer

The first two steps of the design methodology are answered with the data shown in table 11. The results of the other steps are presented in table 14.

Parameter	**Return period**			**Validity Range**
	100 years	***400 years***	***2000 years***	
Number of waves	3963	3392	3564	<7500
S_{om}	0.040	0.037	0.036	0.01-0.06
ξ_m	3.32	3.48	2.20	0.7-7
ξ_{cr}	4.42	4.42	4.42	N/A
Wave type	Plunging	Plunging	Plunging	N/A
N_s	1.45	1.43	2.39	1-4
D_{n50} (m)	3.33	3.69	2.57	N/A
M_{50} (x1000 kg)	96.2	150.1	103.7	N/A

Table 14: Output dimensions of rock armourlayer.

The final rock dimensions found are unrealistically high. Eurocode 13383 indicates that the maximum grading bracket for heavy grading is 10,000 till 15,000 kg [CEN/TC250, 2002], this leads to a D_{n50} of 1.69m [Schiereck and Verhagen, 2019]. But, the designed dimensions prescribe a M_{50} of 150.1 tons of kg and a D_{n50} of 3.69m. This means that a rock armour layer is not suitable for this breakwater.

The sensitivity analysis for the rock armour layer can be found in appendix D.1. Tables 40, 41 and 42 show the results if the standard deviation is taken into account. It can be seen that the difference in D_{n50} and M_{50} for each return period stays the same, for both an increase and decrease of one standard deviation. However, the difference is not equal between an increase and a decrease of one standard deviation.

Finally, the ULS condition is based on a 100-year return period with a partial factor of 1.35, which is shown in table 43. Shows dimensions that are smaller than the 2000-year return period. Meaning that here the 2000 return period is more conservative.

It can also be seen that CC1 would have led to a final design that is 20% smaller for M_{50}. CC3 would have led to a final design that is 23% bigger for M_{50}.

5.1.2 Accropode II armour layer

The results for the Accropode armour layer are presented in table 15.

	Return period		
	100 years	*400 years*	*2000 years*
D_{n50} (m)	2.31	2.68	2.85
Volume (m^3)	12.35	19.32	23.12
Closest Accropode II unit (m^3)	14	20	24

Table 15: Accropode II dimensions for each return period.

Table 15 shows that the final dimensions of the Accropode II are indeed feasible, as standardized Accropode II volumes from the manufacturer can be chosen based on the outcome of the calculations [CLI, 2012]. The leading Accropode II size is 24 m^3, meaning that this size will be used in the design.

The sensitivity analysis for the concrete armour layer in appendix **??**, shows that when the stability numbers, provided in the rock manual, are used the same 2000-year return period Accropode II size should be chosen. However, for other return periods, the results do differ from the Hudson formula, as shown in table 46. Table 47 and 48 show values for the stability numbers with a +-5% difference. Here it can be seen that this can even lead to Accropode II volumes that are not available.

Finally, the ULS condition is based on a 100-year return period with a partial factor of 1.35, which is shown in table 49. Shows dimensions that are smaller than the 2000-year return period. Meaning that here the 2000-year return period is more conservative.

It can also be seen that CC1 would have led to a final design that is 4.1 m^3 smaller in volume. CC3 would have led to a final design that is 4.6 m^3 bigger in volume.

5.2 Toe berm stability

The results for the toe berm dimensions are presented in table 16.

Parameter	Return period		
	100 years	*400 years*	*2000 years*
Toe height (m)	2.91	3.55	3.13
D_{n50} (m)	1.46	1.78	1.57
M_{50} (x1000 kg)	8.0	14.6	10.0
h_t/h	0.58	0.56	0.58
h_t/D_{n50}	9.4	7.35	8.6

Table 16: Output toe-stability calculation.

All values are within the validity limits of h_t/h and h_t/D_{n50}. However, the final rock dimensions found are unrealistically high for the 400-year return period. However, Eurocode 13383 indicates that the maximum grading bracket for heavy grading is 10,000 till 15,000 kg [CEN/TC250, 2002], this leads to a D_{n50} of 1.69m [Schiereck and Verhagen, 2019].

But, the designed dimensions prescribe a M_{50} of 14.6 tons of kg and a D_{n50} of 1.78 m, which would lead to a rock grading of 13-16 tons. Which is outside of the maximum grading. This means other solutions should be pursued to find a suitable toe.

The sensitivity analysis in appendix D.3, shows that the outcome of different depths and low water levels could lead to even bigger required stone dimensions. However, it can be concluded that the sensitivity analysis for 400 years in table 53 also showed that no available scenario is possible for a toe berm made out of rock. This as the lowest D_{n50} (m) is still higher than 1.44 m.

Finally, the ULS condition is based on a 100-year return period with a partial factor of 1.35, which is shown in table 55. Shows dimensions that are smaller than the 2000-year return period. Meaning that here the 2000 return period is more conservative.

It can also be seen that CC1 would have led to a final design that is 38.1% smaller for M_{50}. CC3 would have led to a final design that is 11.3% bigger for M_{50}.

5.3 Underlayer of armourlayer

The underlayers consist of two layers. The top layer is a custom underlayer rock grading of 5-8 tons, which is derived from the Accropode II armour layer. The bottom layer is made of a rock grading of 300-900kg. This means that the total underlayer thickness is 3.94 m.

Van Oord has indicated that the practical limit for rock grading for many quarries is 3-6 tons.

5.4 Reflection on results

The results stay within the validity limits of the formulae of the design methodology. However, it must be noted for all rock components the alternative case-specific SLS provides the highest values. This is something that is not expected as often SLS provides smaller values than ULS. But in this case, it is not strange, as the acceptable damage parameters are low, which is a characteristic of SLS. However, it is now combined with higher return periods than normal.

6 Comparison

In this chapter, the design dimensions of breakwater components will be compared. The comparison will be between the design of van Oord with the design based on the semi-probabilistic design approach. Firstly the design of van Oord is introduced, hereafter the comparison will be done.

6.1 Design of van Oord

In figure 4 the final design of van Oord is presented.

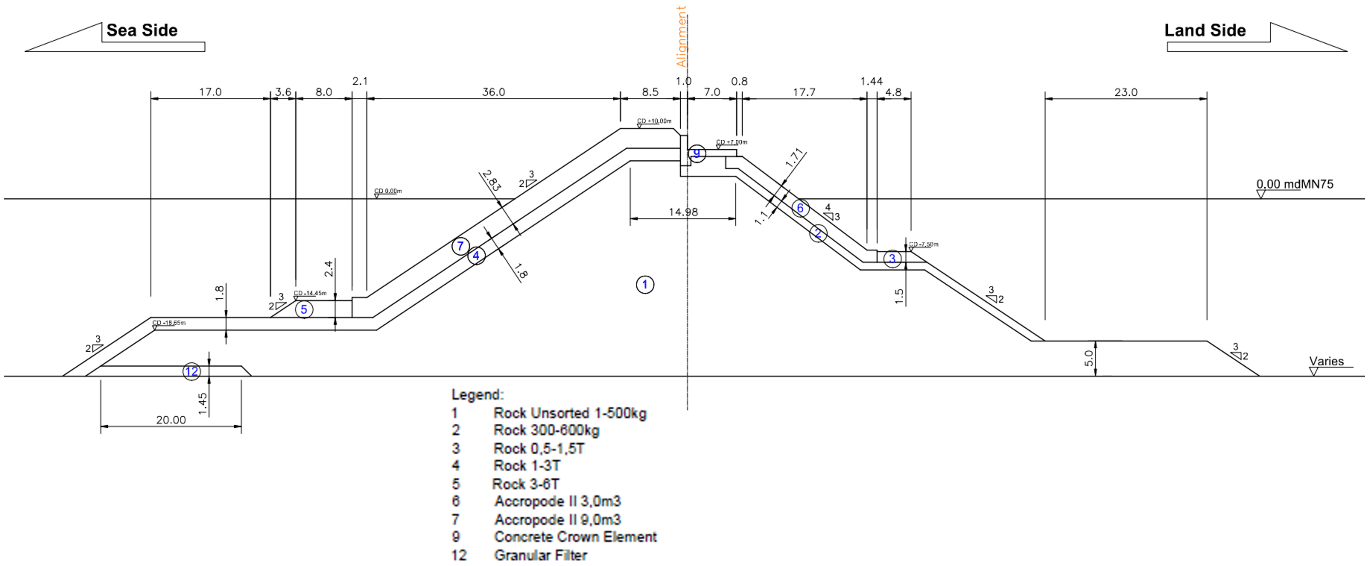


Figure 4: Breakwater design of van Oord [van Oord, 2013].

Starting with the armour layer it can be seen that the dimensions of the Accropode II are $9m^3$ on the seaside. The dimensions of $9m^3$ is a standardized unit, the calculated volume was 8.83 m^3. The toe berm uses rocks of 3-6 tons of kg, which translates to D_{n50} of 1.19m and M_{50} of 4933 kg. The underlayer uses rocks of the grading bracket 1-3 tons of kg for both the armour layer and the toe berm. The grading bracket 1-3 tons of kg, which has a D_{n50} of 0.9m.

In table 17 an overview of the calculated values and the chosen values if applicable.

Component	**Parameter**	**Calculated value**	**Design value**
Armourlayer	D_{n50} (m)	2.07	2.08
	Volume (m^3)	8.83	9
Toe	D_{n50} (m)	1.21	1.22
	M_{50} (kg)	4564	4773
Underlayer	D_{n50} (m)	-	0.89
	M_{50} (kg)	-	1833

Table 17: Calculated values in comparison with chosen dimensions.

6.2 Comparison of dimensions

In table 18 the difference per component is shown.

Component	**Parameter**	**Van Oord**	**Draft Eurocode**	**Difference**
Armourlayer	D_{n50} (m)	2.07	2.85	+37 %
	Volume (m^3)	8.83	23.12	+162%
Toe	D_{n50} (m)	1.21	1.78	+47%
	M_{50} (kg)	4564	14 567	+219%
Underlayer of armourlayer	Layer thickness (m)	1.8	3.94	+119%

Table 18: Comparison in dimensions of both designs.

As shown in table 18 the difference between the calculated values is significant. The smallest difference is 36% till and growing to a difference of 219%. To find the explanations for these findings it is necessary to dive into the design steps and design parameters that are used.

One of the major differences is the return period. Van Oord uses a 100-year return period for the ULS, without a partial factor. The draft Eurocode however uses 2000 years as a return period for the ULS. Meaning that a different significant wave height and period will be used. Due to that the return periods are also quite far apart, so the difference will be bigger. The draft Eurocode uses a significant wave height that is 36% bigger for ULS, for the mean wave period it is 20% bigger. Therefore it is logical to conclude that the outcome of the draft Eurocode will also be bigger.

Another difference in the design approach is the damage parameter that is used to set the level of acceptable damage. For the toe berm design, the Van Oord design uses a N_{od} of 2 for the 100-year return period. The draft Eurocode uses for the same return period a N_{od} of 0.5. This means that less damage is accepted for the same return period, leading to a design that is bigger in the final dimensions, as it is built to withstand more force.

Methode			**Van Oord**	**Draft Eurocode**
Acceptable Damage parameter (N_{od})	Toe	SLS	2	0.5
		ULS		4
	Armour layer	SLS	0	0
		ULS	0.5	0.5
Return period	SLS		100	100 & 400
	ULS (only used for physical testing)		$100 * 120\%$	2000

Table 19: Difference in N_{od} and return periods.

Apart from these two points, the design steps in the design approaches are similar. This is due to that the draft Eurocode makes references to the rock manual (2007), coastal engineering manual (2003-2011) & Eur0top manual (2018), which is already incorporated into the design approach of Van Oord.

Furthermore, it is important to note that the equipment required to install the Accropodes II and the rocks should also increase in size. Instead of installing Accropodes II of 20.2 tons, the crane should install Accropodes II of 56.4 tons over a radius of 50.6 meters[2]. This means that the 200-ton crane which was used by van Oord during the project, should be replaced by a 400-ton crane [Eurogruas, nd].
Additionally, the concrete consumption increases from 1.295 $\frac{m^3}{m^2}$ to 1.760 $\frac{m^3}{m^2}$, an increase of 36%. Meaning that costs will increase for the construction company and the client. But, also extra CO_2 emissions, which enlarges the effect of the construction on the environment.

6.3 Physical model tests

The design presented in chapter 6.1 is tested by 2D and 3D physical model tests. Here the design is tested with an overload of 120% in comparison with the design conditions The 120% overload means that the physical testing takes into account a 20% higher uncertainty. So 120% overload condition can also be seen as a partial factor of 1.2 on all metocean parameters, so the significant wave height being 9m instead of 7.5m, but the T_m also increases. The overload factor of 1.2 is lower than factor 1.35 the new Eurocode introduces as a partial factor in the semi-probabilistic design approach. So the new Eurocode assigns more uncertainty in the metocean parameters than the physical testing does. This can be seen as the significant wave height of 9m for ULS is higher than the significant wave height of the 400-year return period, but lower than the 10.2m for the 2000-year return period.

During the physical testing, it was found that all components of the breakwater, except the seaside toe, were within acceptable limits. The seaside toe eroded and the rocks out of the seaside toe were displaced towards the armour layer, where they could potentially break the Accropodes II. Therefore it was advised to redesign the toe [Artelia, 2013]. In the new design, the toe uses a rock grading of 3-9 tons in the toe, now the toe is within the acceptable limits.

The design based on the draft Eurocode has no breakwater component with the same dimensions as that of the physical model. The physical model testing does not meet the same significant wave height (10.2m versus 9.0m) as required by the draft Eurocode for this case study. This outcome could have been expected as the new Eurocode allocates more uncertainty to the metocean parameters than the physical testing does.

[2]This is the distance of the crown wall to the bottom of the armour layer.

7 Discussion

The first discussion point is the availability of data. For the significant wave height, wave period, design low water levels and storm duration, the data was not available for the return periods of 400 and 2000 years. This has led to that the data for significant wave height and wave period have been extrapolated. Even though the average difference is equal to zero, does not necessarily mean that the uncertainty in the extrapolated values is little. This means that the dimensions can differ quite a lot or very little. It should be noted that the extrapolated values do not produce unrealistic waves, as this was checked in appendix B.1.
However, it must be noted that the extrapolation, always has an uncertainty, no matter which method is used. This is due that the datasets are not sufficiently large to validate or calibrate the extrapolated data [Kearns et al., 2019]. Meaning it is impossible to reach 100% certainty for the values of the mentioned parameters.

Additionally, only 1 design low water level was available, so no data extrapolation is possible. As the dimensions of the toe are sensitive to the water depth, which is affected by the water level, it can be assumed that the uncertainty in the toe dimensions is relatively high. This could have been solved by incorporating the design low water level in the design wave conditions report. As here the high design water level is computed, meaning the low design water level could also have been computed.

Furthermore, the formulae used for the rock armour layer are meant for deep-water conditions. However, for return periods higher than 100 years, it becomes intermediate water depth conditions. However, it is chosen to stick with the deep-water formulae, as the shallow water formulae required other data parameters that were not yet computed. It does mean that the outcome is less accurate, but, the expectation is that the final result would have been only slightly smaller. As the shallow water formulae put a limit on the wave height. Nevertheless, for the outcome of this case study, the result is minimal, as the required rock size is not realistically feasible.

Due to budgetary restrictions, it is not possible to do physical testing for the designed components in this report. Meaning the design has followed the design approach proposed that fits with low structural design uncertainty, which does not fit with a single armour layer. This means that the consequences of physical testing on the final design cannot be determined.

Another discussion point is the statement of TC250 that the nominal return periods are less conservative (11%) than the return periods with a partial factor. This conclusion by TC250 was made after multiple case studies [CEN/TC250, 2023]. However, in this case study this is not the case. For both armour-layer designs and toe berm design the partial factor method is less conservative than the nominal return period. On average the partial factor method provides a D_{n50} which is 2% smaller and for M_{50} 7% smaller.
Which begs the question if this case study is an outlier. But, this can also be the result of the uncertainty in the significant wave height and wave period. As the partial factor method provides a significant wave height which is 0.05m lower than the nominal return period. Meaning it is logical that the outcome of the partial factor method is also smaller, so less conservative.

Furthermore, classifying cases by the use of consequence class, HEA level and structure design uncertainty is not written down clearly in the new Eurocode. Leading possibly to wrong choices, which means that the design process can differ. Which then can lead to wrongfully designed structures, not being able to withstand the environmental conditions. As the sensitivity analysis showed for different consequence classes the final result indeed differs. Meaning wrong choices can put extra costs or liability on van Oord.

Lastly, due to the scope of this research, the findings provide only a specific view of the draft Eurocode. If other perspectives are taken into account, such as a geotechnical perspective, it could provide new views on how the new Eurocode provides guidance.
Furthermore, only one case is studied, meaning that the findings might not be general.
Also only one of three design approaches is used, meaning that the other two design approaches could lead to different design steps, design parameters and final designs, hence the consequences for van Oord can therefore also differ.
Next to this, not all breakwaters components are designed, the rearward armour layer for example. Meaning that the guidance of the new Eurocode and results for these parts is unknown. The consequences of the new Eurocode on different structure types remain also unknown.

8 Conclusion

The research question that this thesis tries to answer is: '*What are the consequences on design steps and parameters of the semi-probabilistic design approach [DA1] in prEN 1991-1-8 in comparison to the deterministic design approach, tested with a physical modelling study for a breakwater design case study?*' In this chapter, the sub-research questions will be answered to provide a final answer to the main research question.

8.1 Research sub-question 1

'*In which steps are there differences, in design steps and parameters, between the semi-probabilistic design approach proposed by prEN 1991-1-8 and the deterministic design approach used by van Oord?*'

The only differences are found during the steps in which the design criteria and design parameters are set up, so before the calculations. The new Eurocode requires that the consequence class, hydrodynamic uncertainty and structure design uncertainty are determined as design steps. Based on these steps, the return periods are determined. Furthermore, the new Eurocode differs in the acceptable damage parameters. The new Eurocode prescribes specific values for SLS and ULS, currently clients themself can determine the acceptable damage parameters for SLS and ULS.

8.2 Research sub-question 2

'*In which steps are there similarities, in design steps and parameters, between the semi-probabilistic design approach and the deterministic design approach proposed by prEN 1991-1-8 and the deterministic design approach used by van Oord?*'

The design steps in the deterministic design approach of Van Oord during calculations are similar to those of the draft Eurocode. This is due to that the draft Eurocode makes references to the same literature Van Oord uses. Meaning that the calculations are done by the same methods and manuals. Furthermore, the way wave studies are performed also conforms to the way the draft Eurocode prescribes. Furthermore, van Oord has performed a physical test for this case, which the new Eurocode also prescribes.

8.3 Research sub-question 3

'*How do the results for the final breakwater from both design approaches compare to a 2D and 3D physical modelling study?*

The physical modelling study does provide a design that is closer to that of the draft Eurocode than the van Oord design is to the draft Eurocode design. Due to that a higher significant wave height is used in the physical modelling study, but still not high enough for the new Eurocode. Additionally, the physical model study design does not have to comply with the lower damage numbers of the draft Eurocode, meaning that the outcome dimensions are allowed to be smaller for the physical model test. So the dimensions are indeed still far off compared to the draft Eurocode design. This is expected due to the high sensitivity of the final design dimensions for damage numbers.

Meaning that the van Oord and physical modelling design are the most closely related to each other.

8.4 Research question

So in conclusion the semi-probabilistic design approach does contain a lot of similarities with the deterministic design approach together with a physical modelling study for a breakwater design. The consequences are only felt during the design criteria when determining the return periods and acceptable damage factors. This means that the way of working within van Oord will slightly change during the design criteria set-up for the return period and acceptable damage factors, but further stay the same.

9 Recommendations

9.1 Recommendations

As the Dutch national annex is not yet determined, as the new Eurocode is not yet installed, it provides to opportunity to voice the wishes or concerns Van Oord has on a national level. Which can lead to a national annex that differs from the draft Eurocode. It must be noted, that this means that in theory, every country can have different national annexes, leading to different final designs per country under the same metocean circumstances.

Assuming the national annexes will be equal to the draft Eurocode. It is recommended to redesign more previous projects done by van Oord according to the new Eurocode.
It is advised to do this for all the new design approaches proposed by the new Eurocode. This will provide a more complete picture of the consequences of the chosen design approaches in different circumstances.

A possible long-term recommendation can be to investigate how the bigger dimensions of final designs will impact the construction itself. For example, the need for bigger equipment, the availability of materials in larger sizes, costs or environmental impacts.

9.2 Further research

Building on the discussion points mentioned in chapter 7 it is recommended to:

- Perform another case study in which all the metocean data is provided for all return periods.
- Perform another case study for which the water conditions are fully deep water or fully shallow water for all return periods
- Research what the consequences are of the other design approaches on design steps, design parameters and final design.
- Due to the minimal amount of publications, it is hard to draw valid conclusions about the draft Eurocode in general. Therefore it is recommended to research multiple case studies, which will boost the amount of case study data, so more valid conclusions can be drawn.

9.3 prEN 1991-1-8

The recommendation for the new Eurocode itself is:

- Clarify and provide more arguments on how to choose a certain consequence class, hydrodynamic uncertainty and structure design uncertainty.

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A Appendix: Literature

A.1 Combined actions EN 1990

Persistent and Transient design situations are a fundamental combination. For fundamental combinations, equations 1 can be used to determine the design value of the actions (Ed) for the ULS, as multiple actions can happen at the same time.

$$E_{\mathrm{d}} = E(\sum j \geq 1(\gamma_{\mathrm{G,\ j}} \cdot G_{\mathrm{k,\ j}}) + \gamma_{\mathrm{p}} \cdot P + \gamma_{\mathrm{Q,\ 1}} \cdot Q_{\mathrm{k,\ 1}} + \sum i \geq 1(\gamma_{\mathrm{Q,\ i}} \cdot \Psi_{0,\ \mathrm{i}} \cdot Q_{\mathrm{k,\ i}})) \quad (1)$$

$$j \geq 1;\ i \geq 1$$

Within the brackets, four terms need explaining. The first term is the sum of all the characteristic values of permanent actions multiplied by the partial factors. The second term is the representative value of the pre-stressing load multiplied by the partial factor for prestressing. The third term is the characteristic value for variable action multiplied by the partial factor. The fourth term is the sum of all the remaining variable actions. The sum consists of the partial factor for a remaining variable action multiplied by a reduction factor for that force multiplied by the characteristic value for the remaining variable actions.

The design value of the actions (Ed) for the SLS is the same formula, but only the partial factors are retracted, as shown in equation 2.

$$E_{\mathrm{d}} = E(\sum j \geq 1(G_{\mathrm{k,\ j}}) + P + Q_{\mathrm{k,\ 1}} + \sum i \geq 1(\Psi_{0,\ \mathrm{i}} \cdot Q_{\mathrm{k,\ i}})) \quad (2)$$

$$j \geq 1;\ i \geq 1$$

For the combination actions of accidental design situations the same formulas apply, only then is the accidental action (Ad) also added as a term.

A.2 HEA methodology guidance

Table 4.2 (NDP) — Hydrodynamic Estimate Approach methodology guidance

HEA level	Boundary condition data	Pathway assessment	Other considerations
HEA1	Locally or nationally determined metocean parameters data, mainly from published sources, with limited probabilistic definition or limited reliability information attached.	Wind-wave[a] hindcast using empirical calculations taking into account fetch/ duration limitations. Extrapolation of sea-level or current values from published values for nearby sites.	Data unlikely to be calibrated or validated for developed statistical treatments. Expertise is required.
HEA2	Individual (marginal) long-term[b] time-series data (or statistical distributions/ summaries thereof) of metocean parameters. Measured data, or, if not available, calibrated and validated model data, e.g. regional or global hindcast models, or synthetic data.	Numerical wave transformation[c] 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.	Data likely to be used for the statistical assessment of the representative values of the actions from waves and currents (hydrodynamic loads), including the combination of metocean parameters, based on marginal statistics.
HEA3	Joint long-term[b] time-series data (or statistical distributions/ summaries thereof) of metocean parameters. Measured data, or, if not available, calibrated and validated model data, e.g. regional or global hindcast models, or synthetic data.	Calibrated numerical wave/ water level/ current transformation[c] model representing (with reasonable accuracy) all key physical processes expected.	Data likely to be used for the statistical assessment of the representative values of the actions from waves and currents (hydrodynamic loads), including the combination of metocean parameters, based on joint statistics.

[a] When the predominant wave energy system only is analysed and can be expected to be wind-sea dominated.

[b] «Long-term» is to be understood in relation to the design service life and to the return periods of the metocean events, also considering the quality of data available (filtering/ removal of erroneous records).

[c] This is based on the typical situation where long-term boundary condition data is not available in close proximity to the structure's site and must be transformed from a remote location.

A.3 Return periods

The return periods for each design approach are the same for the permanent and transient design situations. For other design situations, no return periods are given. It should also be noted that the new Eurocode mentions that the transient design situations may have shorter return periods. However, it is not mentioned how much shorter is allowed. Table 20 (table 4.5 in prEN 1991-1-8) shows what the return periods are for each design approach.

Conse-quence class	Importance factor (φ_I)[a]	Characteristic marginal return period (leading metocean parameter)[b]	Combination marginal return period (accompanying metocean parameter)[c]		Characteristic joint return period[d]
CC3	2,0	200 y	Very High dependence:	140 y	700 y
			High dependence:	40 y	
			Medium dependence:	13 y	
			Low dependence:	6 y	
CC2	1,0	100 y	Very High dependence:	70 y	350 y
			High dependence:	25 y	
			Medium dependence:	10 y	
			Low dependence:	5 y	
CC1	0,5	50 y	Very High dependence:	40 y	175 y
			High dependence:	15 y	
			Medium dependence:	8 y	
			Low dependence:	4 y	

[a] The importance factor is given in Table 4.4 (NDP).
[b] The characteristic marginal return period is equal to the value given in 4.7.2(1) multiplied by the importance factor.
[c] When the degree of dependence is not known, the "medium dependence" figure can be used.
[d] The characteristic joint return period is equal to the value given in 4.7.2(2) multiplied by the importance factor. Figures are valid for a two-variable joint analysis only.

Table 20: Return periods for the hydrodynamic loads.

However, not all structures covered by the new Eurocode fit in the return periods of table 20. The new Eurocode introduces two more tables with return periods. Table 21 (7.1 in new Eurocode) presents the return periods for SLS and table 22 (7.2 in new Eurocode) presents the return periods for ULS.

Conse-quence class	Importance factor (φ_I)[a]	Alternative case-specific characteristic marginal return period (leading metocean parameter)[b]	Alternative case-specific combination marginal return period (accompanying metocean parameter)[c]		Alternative case-specific characteristic joint return period[d]
CC3	2,0	800 y	Very High dependence:	530 y	2 800 y
			High dependence:	140 y	
			Medium dependence:	30 y	
			Low dependence:	7 y	
CC2	1,0	400 y	Very High dependence:	270 y	1 400 y
			High dependence:	75 y	
			Medium dependence:	20 y	
			Low dependence:	6 y	
CC1	0,5	200 y	Very High dependence:	140 y	700 y
			High dependence:	40 y	
			Medium dependence:	13 y	
			Low dependence:	6 y	

[a] The importance factor φ_I is given in Table 4.4 (NDP).
[b] The alternative case-specific characteristic marginal return period is equal to the value given in 7.2.2(2), Note 1, multiplied by the importance factor φ_I.
[c] When the degree of dependence is not known, the "medium dependence" figure can be used
[d] The alternative case-specific characteristic joint return period is equal to the value given in 7.2.2(2) Note 2, multiplied by the importance factor φ_I. Figures are valid for a two-variable joint analysis only.

Table 21: SLS return periods for the hydrodynamic loads.

Consequence class	Importance factor (φ_I)[a]	Design marginal return period (leading metocean parameter)[b]	Design combination marginal return period (accompanying metocean parameter)[c]		Design joint return period[d]
CC3	2,0	4 000 y	Very High dependence:	1 800 y	12 000 y
			High dependence:	370 y	
			Medium dependence:	45 y	
			Low dependence:	6 y	
CC2	1,0	2 000 y	Very High dependence:	900 y	6 000 y
			High dependence:	190 y	
			Medium dependence:	30 y	
			Low dependence:	6 y	
CC1	0,5	1 000 y	Very High dependence:	450 y	3 000 y
			High dependence:	100 y	
			Medium dependence:	18 y	
			Low dependence:	5 y	

[a] The importance factor φ_I is given in Table 4.4 (NDP).

[b] The design marginal return period is equal to the value given in 7.2.3(1), Note 1 multiplied by the importance factor φ_I.

[c] When the degree of dependence is not known, the "medium dependence" figure can be used

[d] The design joint return period is equal to the value given in 7.2.2(1), Note 2 multiplied by the importance factor φ_I. Figures are valid for a two-variable joint analysis only.

Table 22: ULS return periods for the hydrodynamic loads.

Table 23 provides an overview of which return periods should be picked per structure.

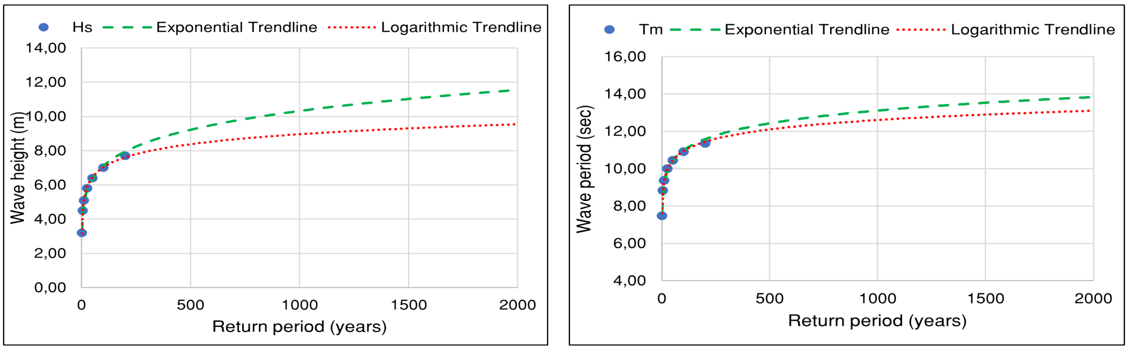
Type of structure	**Return period table(s)**
Mound breakwater	Table 20, 21 & 22
Vertical face breakwater	Overtopping: table 21 & 22. Other failure types: table 20,
Composite breakwater	SLS: table 21, ULS most conservative between table 20 & 22
Fixed cylindrical structures & suspended decks	Table 20
Floating structures	Table 20
Revetments (Coastal embankment)	Table 20, 21 & 22
Seawalls (Coastal embankment)	Overtopping: table 21 & 22. Other failure types: table 20

Table 23: Overview of return period tables.

B Appendix: Methodology

B.1 Data extrapolation

The logarithmic and exponential are the trendlines that best fit the data. These trendlines are shown in figure 5.

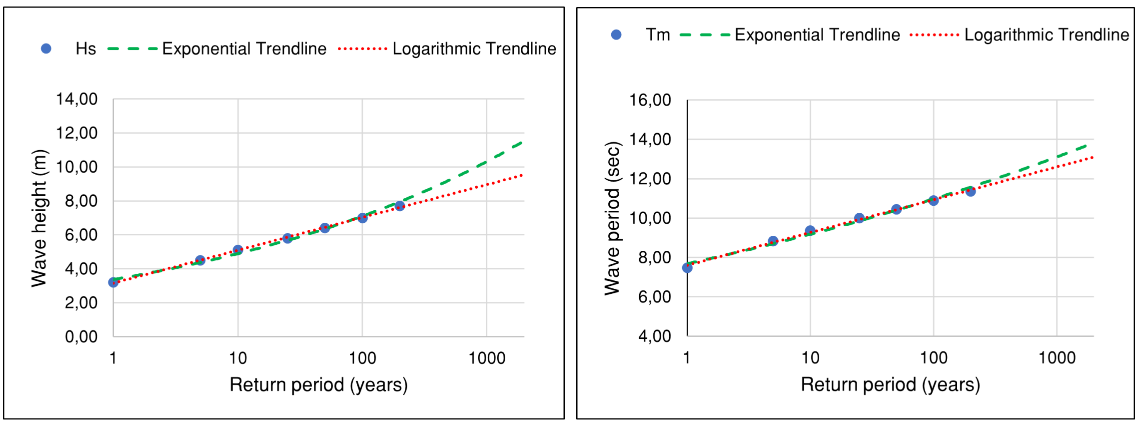


(a) Significant wave height (b) Mean wave period

Figure 5: Plotted return periods with the trendlines.

The formulas that are used in the trendlines are used to reverse predict the known return periods, so it is possible to see the difference. The average difference between the logarithmic trendline and the known data set is for both T_m and H_s 0.000 meters. For the exponential trendline, the difference is bigger, for T_m the difference is 0.73 seconds and for H_s 0.000 meters. This is shown in table 24 and 25.

As a final check, the known data is plotted on a logarithmic scale for the return periods, in combination with both trend lines. The trendline that will follow the line of the data points the best is the most representative one. Based on both these checks the logarithmic trend line is chosen as the most representative trendline, as can be seen in figure 6.



(a) Significant wave height (b) Mean wave period

Figure 6: Plotted return periods with the trendlines on a logarithmic scale.

In table 24 and 25 the average difference is shown between the trendlines and the available data.

Return period (years)	$\mathbf{T_m}$ **(seconds)**	**Exponential** $\mathbf{T_m}$ **(seconds)**	**Log** $\mathbf{T_m}$ **(seconds)**	**Difference Exponential** $\mathbf{T_m}$ **(seconds)**	**Difference Log** $\mathbf{T_m}$ **(seconds)**
1	7,48	7,11	7,60	-0,37	0,13
5	8,83	8,05	8,77	-0,78	-0,06
10	9,37	8,49	9,27	-0,88	-0,10
25	10,00	9,12	9,93	-0,88	-0,07
50	10,45	9,62	10,43	-0,83	-0,02
100	10,90	10,15	10,94	-0,75	0,03
200	11,35	10,71	11,44	-0,64	0,09
Average				**-0,73**	**0,00**

Table 24: Difference between the two trendlines for T_m.

Return period (years)	$\mathbf{H_s}$ **(meter)**	**Exponential** $\mathbf{H_s}$ **(meter)**	**Log** $\mathbf{H_s}$ **(meter)**	**Difference Exponential** $\mathbf{H_s}$ **(meter)**	**Difference Log** $\mathbf{H_s}$ **(meter)**
1	3,3	3,48	3,24	0,18	-0,06
5	4,7	4,56	4,71	-0,14	0,01
10	5,3	5,12	5,34	-0,18	0,04
25	6,1	5,97	6,18	-0,13	0,08
50	6,8	6,71	6,81	-0,09	0,01
100	7,5	7,53	7,44	0,03	-0,06
200	8,1	8,46	8,07	0,36	-0,03
Average				**0,00**	**0,00**

Table 25: Difference between the two trendlines for H_s.

The values for the significant wave height and mean wave period for all the return periods are presented in table 26. Furthermore, the logarithmic data is also checked to stay within the boundaries for wave steepness ($< \frac{1}{7}$) and wave breaking ($< 0.5 *$ water depth) [Schiereck and Verhagen, 2019]. Meaning that the wave data will not result in unrealistic values.

Return period (years)	**T_m (seconds)**	**H_s (meter)**	**Log T_m (seconds)**	**Log H_s (meter)**	**Valid for Wave steepness**	**Valid for breaking waves**
1	7,48	3,3	7,60	3,24	True	True
5	8,83	4,7	8,77	4,71	True	True
10	9,37	5,3	9,27	5,34	True	True
25	10,00	6,1	9,93	6,18	True	True
50	10,45	6,8	10,43	6,81	True	True
100	10,90	7,5	10,94	7,44	True	True
200	11,35	8,1	11,44	8,07	True	True
400			11,94	8,71	True	True
500			12,10	8,91	True	True
800			12,44	9,34	True	True
1000			12,60	9,54	True	True
2000			13,10	10,17	True	True
4000			13,60	10,81	True	True

Table 26: Values for T_m and H_s for the return periods.

B.2 Rock armour layer

The data that is needed for each step is mentioned, however, when the same data is needed in multiple steps it will only be mentioned in the first step to prevent repetitiveness.

Step 1: T_m and H_s at toe

The values that are needed are the local[3] $\mathrm{T_m}$ and the local $\mathrm{H_s}$ for each return period. $\mathrm{T_m}$ can be calculated by dividing $\mathrm{T_p}$ by 1.11 [van Oord, 2013].

Step 2: Damage parameter

The new Eurocode shows in table C.1 damage parameters for different parts of the breakwater and the unit that is used. A part of table C.1 is presented in table 27. The start of failure is an equal state to that of the SLS and the failure state of the ULS [CEN/TC250, 2023]. The values for the slope determines the damage parameters.

Sub-system	**Unit**	**Damage parameter**	**Slope**	**Start of failure**	**Failure state**
Two-layer armour	Rock	D_{er}	1-2:1:3	0-5 percent	20 percent
		S_d	1:1.5-1:2	2	8
		S_d	1:3	2	12
		S_d	1:4-1:6	3	17

Table 27: Part of table C.1 of the new Eurocode.

Step 3: Number of waves

The number of waves can be calculated by equation 3.

$$N_\mathrm{w} = \frac{\text{Storm duration(hours)}}{T_\mathrm{m}} \times 3600 \tag{3}$$

Data on storm duration in hours is needed to calculate the number of waves.

Step 4: Calculating the surf similarity parameter ξ_m

The surf similarity parameter can be calculated by using the slope of the breakwater and the fictitious wave steepness, as shown in equation 4.

$$\xi_\mathrm{m} = \frac{\tan(\alpha)}{\sqrt{s_\mathrm{om}}} \tag{4}$$

Where α is equal to the slope of the breakwater and $\mathrm{s_{om}}$ is the fictitious wave steepness. The equation for the fictitious wave steepness is presented in equation 5.

$$s_\mathrm{om} = \frac{2\pi \cdot H_\mathrm{s}}{g \cdot {T_\mathrm{m}}^2} \tag{5}$$

Where the local significant height and the local mean wave period for each return period are used.

[3]Local refers to the location of the toe of a breakwater.

Step 5: Determining type of waves

To determine if waves are plunging or surging the critical surf similarity parameter ξ_{cr} should be calculated. If $\xi_m < \xi_{cr}$ the waves are plunging. When they are equal or $\xi_m > \xi_{cr}$, the waves are surging. ξ_{cr} is calculated by equation 6.

$$\xi_{\mathrm{cr}} = [\frac{c_{\mathrm{pl}}}{c_{\mathrm{s}}} \cdot P^{0.31} \cdot \sqrt{\tan\alpha}]^{\frac{1}{P+0.5}} \tag{6}$$

The values for c_{pl}, c_s and the notional permeability (P) are required. c_{pl} and c_s are the coefficients for plunging and surging waves. The notional permeability refers to how permeable the whole breakwater structure is. So the size of the rock armour, filter layers and core influences the notional permeability.

Step 6: Stability number(N_s)

Based on the outcome of Step 5, the stability number (N_s) can be calculated for plunging or surging waves. The equation for plunging waves is equation 7 [van der Meer, 1988].

$$\frac{H_{\mathrm{s}}}{\Delta \cdot D_{\mathrm{n50}}} = c_{\mathrm{pl}} \cdot P^{0.18} \cdot (\frac{S_{\mathrm{d}}}{\sqrt{N}})^{0.2} \cdot \xi_{\mathrm{m}}{}^{-0.5} \tag{7}$$

S_d is the damage parameter from step 2 and ξ_m is calculated in step 4. Δ is the relative buoyant density (ρ_{rock}/ρ_{water} - 1). The equation for surging waves is shown in equation 8.

$$\frac{H_{\mathrm{s}}}{\Delta \cdot D_{\mathrm{n50}}} = c_{\mathrm{s}} \cdot P^{-0.13} \cdot (\frac{S_{\mathrm{d}}}{\sqrt{N}})^{0.2} \cdot \sqrt{\cot\alpha} \cdot \xi_{\mathrm{m}}{}^{P} \tag{8}$$

The stability number (N_s) is equal to the right side of equations 7 and 8, as shown in equation 9.

$$\frac{H_{\mathrm{s}}}{\Delta \cdot D_{\mathrm{n50}}} = N_{\mathrm{s}} \tag{9}$$

This means that equation 7 can be rewritten in a different form with the use of equation 9 as shown in equation 10.

$$N_{\mathrm{s}} = c_{\mathrm{pl}} \cdot P^{0.18} \cdot (\frac{S_{\mathrm{d}}}{\sqrt{N}})^{0.2} \cdot \xi_{\mathrm{m}}{}^{-0.5} \tag{10}$$

For surging waves, it means that equation 8 can be rewritten in a different form with the use of equation 9 as shown in equation 11.

$$N_{\mathrm{s}} = c_{\mathrm{s}} \cdot P^{-0.13} \cdot (\frac{S_{\mathrm{d}}}{\sqrt{N}})^{0.2} \cdot \sqrt{\cot\alpha} \cdot \xi_{\mathrm{m}}{}^{P} \tag{11}$$

With equation 10 or 11 the stability number for each return period can be calculated.

Step 7: Armourstone size(D_{n50})

Using equation 9 D_{n50} can be calculated. Based on D_{n50}, the median mass (M_{50}) of the needed stone can be calculated. This is done by using equation 12 [CIRIA et al., 2007].

$$M_{50} = D_{\mathrm{n50}}{}^{3} \cdot \rho_{\mathrm{rock}} \tag{12}$$

All the data that is needed for the rock armour layer is presented in table 28.

Parameter	100 years	400 years	2000 years	Source
H_s (m)	7.50	8.71	10.17	[Arcadis, 2013]
T_m (sec)	10.90	11.94	13.1	[Arcadis, 2013]
Storm duration	12 hours			[van Oord, 2013]
Slope	Seaside: 1:1.5 & Portside: 1.5:2			[van Oord, 2013]
S_d	SLS = 2	SLS = 2	ULS = 8	[CEN/TC250, 2023]
c_{pl}	6.2; 5% lower limit = 5.5			[CIRIA et al., 2007]
c_s	1; 5% lower limit = 0.87			[CIRIA et al., 2007]
Permeability	0.4			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			

Table 28: Input for rock armour stability.

B.3 Concrete armour layer

To calculate the concrete armour layer less steps have to be taken in comparison with the rock armour layer. The concrete armour units use the Hudson formula as expressed in equation 13 [CIRIA et al., 2007]. Where K_D represents the stability coefficient.

$$\frac{H_s}{\Delta \cdot D_{n50}} = N_s = (K_D * \cot\alpha)^{\frac{1}{3}} \tag{13}$$

The concrete armour unit that is chosen to be used in the design is Accropode II, as this product is also used in the design by Van Oord. By using the same product the final values are better comparable. When the breakwater slope is known, all parameters are known and D_{n50} can be calculated with the use of equation 13.

However, the volume of the Accropodes II should also be determined. The volume is expressed as D_{n50} to the power 3, according to the manufacturer [CLI, 2012].

The K_D of Accropode II is however dependent on the seabed slope according to the manufacturer, which can be seen in figure 7. The manufacturer only provides the K_D values for the ULS, the K_D for the SLS is found in the rock manual [CIRIA et al., 2007].

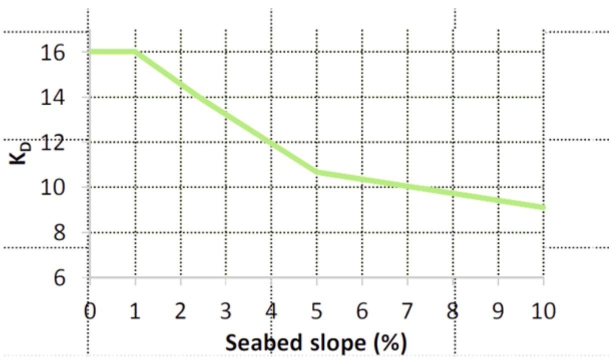


Figure 7: Relationship between K_D and the seabed slope for ULS [CLI, 2012].

Another method to calculate the stability number (N_s) can also be used to calculate D_{n50}. The Rock Manual has provided stability numbers for SLS and ULS for the Accropode. The stability numbers (N_s) are 2.5 for SLS and 2.7 for ULS [CIRIA et al., 2007].

However, these values have a larger uncertainty, as the values are based on other case studies and the values of these case studies are then divided by a partial factor of 1.5. This method will be used to perform a sensitivity analysis.

The data for the Accropode II armour layer is presented in table 29

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.50	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Seabed Slope	0.17%			[van Oord, 2013]
K_D	SLS = 12		ULS = 16	[CIRIA et al., 2007] & [CLI, 2012]
Slope	Seaside: 1:1.5 & Portside: 1.5:2			[van Oord, 2013]
$\rho_{concrete}$	2350 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.31			
N_s	SLS = 2.5	SLS = 2.5	ULS = 2.7	[CIRIA et al., 2007]

Table 29: Input for Accropode II armour stability.

B.4 Toe berm

The toe stability is calculated with the use of equation 14, as stated in chapter C.3.6 in the new Eurocode [CEN/TC250, 2023].

$$\frac{H_s}{\Delta \cdot D_{n50}} = (2+6.2\cdot(\frac{h_t}{h})^{2.7})\cdot {N_{od}}^{0.15} \text{ within range of: } 0.4 < \frac{h_t}{h} < 0.9 \text{ and } 3 < \frac{h_t}{D_{n50}} < 25 \tag{14}$$

Where the parameters are the following:

- Δ is the relative buoyancy.
- h_t is the water depth above the toe of the breakwater.
- h is the water depth before the toe of the breakwater.
- N_{od} is the damage number, which is the number of armour units that are displaced within a width of D_{n50} across the breakwater.

Equation 14 has another boundary, the water depth cannot be more than two times the significant wave height. Furthermore, The damage number N_{od} can be found in table 30, which is a part of table C.1 in the new Eurocode.

Sub-system	**Unit**	**Damage Parameter**	**Start of failure**	**Failure state**
Toe berm	Rock	N_{od}	0.5	4

Table 30: Damage parameter for the toe.

The new Eurocode prescribes the toe berm dimensions as two or three stones high and three to five stones wide [CEN/TC250, 2023], which is equal to the rock manual. In the case study, the height is determined as two times D_{n50}.

In equation 14 h_t can be calculated by subtracting the height of the toe from the depth where the toe is located. However, the height of the toe is unknown as D_{n50} is unknown. This means that for the first time filling in equation 14 the answer is unlikely to be accurate, meaning that multiple iterations are needed to find the value for which the height of the toe is equal to two or three times D_{n50}.

For this case study, the iterations will be run till the first 2 numbers after the decimal point are equal. This is assumed to be accurate and feasible to achieve in construction, as a higher accuracy is not realistically feasible, as stones this size are not measured by the millimetre.

Furthermore, only one design low water level was found, meaning that for each return period, the same design low water level is used. This is not realistic, so therefore a sensitivity analysis is performed with different low water levels.
In table 31 the data for the toe berm calculation is presented.

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.5	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Depth bottom toe (m)	-17.2 MSL			[van Oord, 2013]
h (m)	-24 MSL			[van Oord, 2013]
Design low water level (m)	-0.6 MSL			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			
N_{od}	SLS = 0.5	SLS = 0.5	ULS = 4	[CEN/TC250, 2023]

Table 31: Input for toe stability.

B.5 Underlayer of rock component

The underlayer of the armour layer or toe berm protects against erosion of material located below the armour layer or toe berm. Additionally, the underlayer will help the interlocking of the stones in the armourlayer or toe berm. The new Eurocode states in point 7.3.9. (4) that the Coastal Engineering Manual VI.5-3 or the Rock Manual (2007), 5.2.2.10 may be used [CEN/TC250, 2023]. This can be summarized into two steps.

Step 1

The rock manual prescribes equation 15 to determine the M_{50} of the underlayer [CIRIA et al., 2007].

$$\frac{M_{50,\text{ underlayer}}}{M_{50,\text{ armourlayer}}} = \frac{1}{10} \text{ to } \frac{1}{15} \tag{15}$$

No indication is given in the new Eurocode to use the minimum, maximum or mean value in this formula. It is decided that the mean value is used for this formula.

Step 2
Furthermore, should the underlayer also comply with the granular filter criteria to ensure stability within the layer. The filter criteria check for three types of filter instability. The filter layer should pass all the filter criteria. The criteria are presented here below [Burcharth and Hughes, 2002].

- Retention criteria

$$\frac{D_{15,\text{ upper layer}}}{D_{85,\text{ lower layer}}} < \text{ 4 to 5} \tag{16}$$

$$\frac{W_{50,\text{ upper layer}}}{W_{50,\text{ lower layer}}} < \text{ 15 to 20} \tag{17}$$

- Permeability criteria

$$\frac{D_{15,\text{ upper layer}}}{D_{15,\text{ lower layer}}} > 1 \tag{18}$$

- Internal stability criteria

$$\frac{D_{60}}{D_{10}} < 10 \tag{19}$$

- Filter layer thickness can never be less than 30 cm for rock. However, if two times the diameter of the larger stones is bigger than 30 cm this criteria should be used to determine the layer thickness.

B.6 Underlayer of Accropode II

The design of the underlayer of the Accropode II has a different first step. The manufacturer provides the value of Nominal Lower Limit (<10%) and Nominal Upper Limit (>70%) for the weight of the rock for the underlayer, based on these values an appropriate underlayer grading can be chosen. Which is then checked by the same filter criteria as for the underlayer of a rock component, as shown in step 2 of chapter B.5.

NLL and NUL of the rock underlayer can be calculated with M_{50} as shown in equation 20 [CIRIA et al., 2007].

$$NLL = 0.156 \cdot {M_{50}}^{1.16} \qquad NUL = 2.52 \cdot {M_{50}}^{0.92} \tag{20}$$

C Appendix: Calculations

C.1 Rock armour layer

All the data that is needed for the rock armour layer is presented in table 32.

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	7.50	8.71	10.17	[Arcadis, 2013]
T_m (sec)	10.90	11.94	13.1	[Arcadis, 2013]
Storm duration	12 hours			[van Oord, 2013]
Slope	Seaside: 1:1.5 & Portside: 1.5:2			[van Oord, 2013]
S_d	SLS = 2	SLS = 2	ULS = 8	[CEN/TC250, 2023]
c_{pl}	6.2; 5% lower limit = 5.5			[CIRIA et al., 2007]
c_s	1; 5% lower limit = 0.87			[CIRIA et al., 2007]
Permeability	0.4			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			

Table 32: Input for rock armour stability.

By filling in the equations presented in appendix B.2. The table 33 can be made.

Parameter	**Return period**			**Validity Range**
	100 years	***400 years***	***2000 years***	
Number of waves	3963	3392	3564	<7500
S_{om}	0.040	0.037	0.036	0.01-0.06
ξ_m	3.32	3.48	2.20	0.7-7
ξ_{cr}	4.42	4.42	4.42	N/A
Wave type	Plunging	Plunging	Plunging	N/A
N_s	1.45	1.43	2.39	1-4
D_{n50} (m)	3.33	3.69	2.57	N/A
M_{50} (x1000 kg)	96.2	150.1	103.7	N/A

Table 33: Output dimensions of rock armour layer.

As the results are within the validity limits no physical testing has to be done for the rock armour layer according to chapter 7.3.2 of the new Eurocode.

C.2 Accropode II armour layer

The data for the Accropode II armour layer is presented in table 34

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.50	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Seabed Slope	0.17%			[van Oord, 2013]
K_D	SLS = 12		ULS = 16	[CIRIA et al., 2007] & [CLI, 2012]
Slope	Seaside: 1:1.5 & Portside: 1.5:2			[van Oord, 2013]
$\rho_{concrete}$	2350 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.31			
N_s	SLS = 2.5	SLS = 2.5	ULS = 2.7	[CIRIA et al., 2007]

Table 34: Input for Accropode II armour stability.

By filling in the equation presented in appendix B.3. The table 35 can be made, no validity range is available for the Hudson formula.

	Return period		
	100 years	***400 years***	***2000 years***
D_{n50} (m)	2.31	2.68	2.85
Volume (m^3)	12.35	19.32	23.12
Closest Accropode II unit (m^3)	14	20	24

Table 35: Accropode II dimensions for each return period

As the Hudson formula does not have a validity limit, physical testing should be done for the rock armour layer according to chapter 7.3.2 of the new Eurocode. This would mean that the Accropode II armour layer is exposed to the offshore height waves multiplied by 1.35, where the offshore wave steepness remains the same. It is unclear for which return period the 1.35 multiplication factor applies.

C.3 Toe berm

In table 36 the data for the toe berm calculation is presented.

Parameter	**100 years**	**400 years**	**2000 years**	**Source**
H_s (m)	SLS = 7.5	SLS = 8.71	ULS = 10.17	[Arcadis, 2013]
Depth bottom toe (m)	-17.2 MSL			[van Oord, 2013]
h (m)	-24 MSL			[van Oord, 2013]
Design low water level (m)	-0.6 MSL			[van Oord, 2013]
ρ_{rock}	2600 kg/m^3			[van Oord, 2013]
ρ_{water}	1018 kg/m^3			[van Oord, 2013]
Δ	1.55			
N_{od}	SLS = 0.5	SLS = 0.5	ULS = 4	[CEN/TC250, 2023]

Table 36: Input for toe stability.

By filling in the equations presented in appendix B.4. The table 37can be made.

Parameter	**Return period**		
	100 years	***400 years***	***2000 years***
Toe height (m)	2.91	3.55	3.13
$\mathbf{D_{n50}}$ **(m)**	1.46	1.78	1.57
$\mathbf{M_{50}}$ **(x1000 kg)**	8.0	14.6	10.0
$\mathbf{h_t/h}$	0.58	0.56	0.58
$\mathbf{h_t/D_{n50}}$	9.4	7.35	8.6

Table 37: Output toe-stability calculation.

$\frac{h_t}{h}$ & $\frac{h_t}{D_{n50}}$ are for all values within the validity limits of 0.4-0.9 for $\frac{h_t}{h}$ and 3-25 for $\frac{h_t}{D_{n50}}$. This means no physical testing has to be done for the rock armour layer according to chapter 7.3.6 of the new Eurocode.

C.4 Underlayer

The underlayer is based on the Accropode armour layer. As the 24 m^3 Accropodes are used the underlayer should fit in the provided NLL and NUL range by the manufacturer. For 24 m^2 the values are: NLL = 4.03 tons and NUL = 8.06 tons. Based on these values a custom grading of 5-8 tons is chosen. This grading results in an NLL of 4.12 tons and NUL of 8.09 tons, with a M_{50} of 6.5 tons. This grading is chosen based on trial and error to find the right NLL and NUL values. Based on this grading the values of D_{xx} can be calculated. Table 38 shows the needed input and output data for the filter rules.

Parameter	**Core (100-500 kg)**	**Underlayer (5-8 tons)**
D_{10}	0.42	1.49
D_{15}	0.43	1.51
D_{60}	0.58	1.65
D_{85}	0.69	1.73
D_{n50}	0.46	1.36
M_{50}	247	6513

Filter Rules	**Output**	**(Within) range**
Equation 16	2.20	**(Yes)** < 5
Equation 17	26.34	**(No)** < 15 to 20
Equation 18	3.51	**(Yes)** >1
Equation 19	1.10	**(Yes)** <10

Table 38: Accropode underlayer grading.

As the second retention criteria is not met, another underlayer is needed that will be placed in between the core and the top underlayer. As the M_{50} of the top underlayer is known, the first step is to calculate the M_{50} of the underlayer in between the underlayers. When taking the average of 1/10 and 1/15 and multiplying this with the M_{50} of the top underlayer. This leads to a M_{50} of 543 kg. Which fits with the custom grading of 300-900 kg, with M_{50} of 557 kg. Based on this grading the values of D_{xx} can be calculated. Table 39 shows the needed input and output data for the filter rules.

Parameter	**Core (100-500 kg)**	**Underlayer (300-900 kg)**
D_{10}	0.42	0.59
D_{15}	0.43	0.61
D_{60}	0.58	0.75
D_{85}	0.69	0.84
D_{n50}	0.46	0.60
M_{50}	247	557

Filter Rules	**Output**	**(Within) range**
Equation 16	0.88	**(Yes)** < 5
Equation 17	2.25	**(Yes)** < 15 to 20
Equation 18	1.46	**(Yes)** >1
Equation 19	1.26	**(Yes)** <10

Table 39: Accropode underlayer grading.

With the second underlayer, all the criteria are met. So the Accropode II armour-layer needs two underlayers, each underlayer has a thickness of 2*D_{n50}, which means that the combined layer thickness is $2 \cdot 1.36 + 2 \cdot 0.6 = 3.94m$.

Important to note, that the new Eurocode does not mention the underlayer has to undergo physical testing.

D Appendix: Sensitivity analysis

D.1 Senstivity Analysis Rock armourlayer

Parameter	**C_{pl} and C_s High**		**C_{pl} and C_s low**	
Number of Waves	3963			
S_{om}	0.04			
ξ_m	3.32			
ξ_{cr}	4.35		4.50	
Wave type	Plunging		Plunging	
N_s	1.54	*+6.5%*	1.35	*-6.5%*
D_{n50}(m)	3.13	-6.1%	3.56	6.9%
M_{50}(kg)	79 747	-17.1%	117 507	22.2%

Table 40: Rock armour sensitivity on C_{pl} and C_s for 100 years return period.

Parameter	**C_{pl} and C_s High**		**C_{pl} and C_s low**	
Number of Waves	3619			
S_{om}	0.039			
ξ_m	3.37			
ξ_{cr}	4.35		4.50	
Wave type	Plunging		Plunging	
N_s	1.54	*+6.5%*	1.36	*-6.5%*
D_{n50}(m)	3.63	-6.1%	4.13	+6.9%
M_{50}(kg)	124 395	-17.1%	183 296	+22.2%

Table 41: Rock armour sensitivity on C_{pl} and C_s for 400 years return period.

Parameter	**C_{pl} and C_s High**		**C_{pl} and C_s low**	
Number of Waves	3297			
S_{om}	0.038			
ξ_m	3.42			
ξ_{cr}	4.35		4.5	
Wave type	Plunging		Plunging	
N_s	2.04	*+6.5%*	1.79	*-6.5%*
D_{n50}(m)	3.21	-6.1%	3.65	+6.9%
M_{50}(kg)	85 960	-17.1%	126 661	+22.2%

Table 42: Rock armour sensitivity on C_{pl} and C_s for 2000 years return period.

Parameter	Value	
Number of Waves	3963	
S_{om}	0.55	
ξ_m	2.85	
ξ_{cr}	4.42	
Wave type	Plunging	
N_s	2.06	+7.5%
D_{n50}(m)	3.16	-7.4%
M_{50}(kg)	82 260	-20.6%

Table 43: Rock armour sensitivity for ULS 100 years with partial factor 1.35 on waves.

Parameter	50 years		200 years		1000 years	
Number of Waves	4134		3806		3429	
S_{om}	0.040		0.040		0.038	
ξ_m	3.34		3.32		3.40	
ξ_{cr}	4.42		4.42		4.42	
Wave type	Plunging		Plunging		Plunging	
N_s	1.44	*-0.8%*	1.45	*+0.2%*	1.92	*-0.1%*
D_{n50}(m)	3.04	*-8.6%*	3.59	*-7.2%*	3.21	*-6.2%*
M_{50}(kg)	73 356	*-23.8%*	120 082	*-20.0%*	85 652	*-17.4%*

Table 44: Rock armour sensitivity CC1 for all return periods.

Parameter	200 years		800 years		4000 years	
Number of Waves	3806		3473		3176	
S_{om}	0.040		0.039		0.038	
ξ_m	3.32		3.39		3.45	
ξ_{cr}	4.42		4.42		4.42	
Wave type	Plunging		Plunging		Plunging	
N_s	1.45	*+0.3%*	1.45	*+0.2%*	1.92	*0.0%*
D_{n50}(m)	3.59	*+7.7%*	4.14	*+7.2%*	3.63	*+6.2%*
M_{50}(kg)	120 082	*+24.8%*	184 582	*+23.0%*	124 228	*+19.8%*

Table 45: Rock armour sensitivity CC3 for all return periods.

D.2 Sensitivity Analysis Concrete Armour unit

	Return period		
	100 years	***400 years***	***2000 years***
D_{n50} (m)	2.32	2.70	2.84
Volume (m^3)	12.55	19.63	23.02
Closest Accropode II unit (m^3)	14	20	24

Table 46: Accropode II dimensions for each return periods, N_s.

	Return period		
	100 years	***400 years***	***2000 years***
D_{n50} (m)	2.21	2.57	2.71
Volume (m^3)	10.84	16.95	19.88
Closest Accropode II unit (m^3)	12	18	20

Table 47: Accropode II dimensions for each return periods, +5 % N_s.

	Return period		
	100 years	***400 years***	***2000 years***
D_{n50} (m)	2.45	3.31	2.99
Volume (m^3)	14.64	36.26	26.85
Closest Accropode II unit (m^3)	16	Requires custom template	28

Table 48: Accropode II dimensions for each return periods, -5 % N_s.

Parameter	Value	% difference
D_{n50} (m)	2.83	-0.5%
Volume (m^3)	22.69	-1.4%

Table 49: Accropode II dimensions based on partial factor 1.35 and 100-year return period.

	Return period		
	50 years	***200 years***	***1000 years***
D_{n50} (m)	2.10	2.50	2.67
Volume (m^3)	9.2	15.56	19.07
Closest Accropode II unit (m^3)	10	16	20

Table 50: Accropode II dimensions for each CC1 return periods.

	Return period		
	200 years	***800 years***	***4000 years***
D_{n50} (m)	2.50	2.88	3.03
Volume (m^3)	15.56	23.83	27.70
Closest Accropode II unit (m^3)	16	24	28

Table 51: Accropode II dimensions for each CC3 return periods.

D.3 Toe berm sensitivity analysis

Water level (m)	-0.30	-0.60	-0.90	-0,30	-0.60	-0.90
Bottom level (m)	-24	-24	-24	-25	-25	-25
Water depth on top of toe (h_t)	-14.02	-13.69	-13.35	-13.86	-13.52	-13.18
Water depth in front of toe (h)	-23.70	-23.40	-23.10	-24.70	-24.40	-24.10
h_t/h	0.59	0.58	0.58	0.56	0.55	0.55
D_{n50} of armour (m)	1.44	1.46	1.48	1.52	1.54	1.56
M_{50} of armour (kg)	7741	8036	8350	9142	9486	9850
Thickness toe (m)	2.88	2.91	2.95	3.04	3.08	3.12

Table 52: Toe sensitivity for 100 years return period.

Water level (m)	-0.30	-0.60	-0.90	-0,30	-0.60	-0.90
Bottom level (m)	-24	-24	-24	-25	-25	-25
Water depth on top of toe (h_t)	-13.40	-13.05	-12.70	-13.20	-12.85	-12.50
Water depth in front of toe (h)	-23.70	-23.40	-23.10	-24.70	-24.40	-24.10
h_t/h	0.57	0.56	0.55	0.53	0.53	0.52
D_{n50} of armour (m)	1.75	1.78	1.80	1.85	1.87	1.90
M_{50} of armour (kg)	13 986	14 567	15 189	16 441	17 106	17 813
Thickness toe (m)	3.50	3.55	3.60	3.70	3.75	3.80

Table 53: Toe sensitivity for 400 years return period.

Water level (m)	-0.30	-0.60	-0.90	-0,30	-0.60	-0.90
Bottom level (m)	-24	-24	-24	-25	-25	-25
Water depth on top of toe (h_t)	-13.21	-13.47	-13.12	-13.62	-13.28	-12.93
Water depth in front of toe (h)	-23.70	-23.40	-23.10	-24.70	-24.40	-24.10
h_t/h	0.56	0.58	0.57	0.55	0.54	0.54
D_{n50} of armour (m)	1.54	1.57	1.59	1.64	1.66	1.69
M_{50} of armour (kg)	9581	9990	10 429	11 463	11 947	12 465
Thickness toe (m)	3.09	3.13	3.18	3.28	3.32	3.37

Table 54: Toe sensitivity for 2000 years return period.

Parameter	100 jaar	% difference
D_{n50} (m)	1.56	-0.65 %
M_{50} (kg)	9798	-1.96 %
Thickness toe (m)	3.11	-0.65 %

Table 55: Toe dimensions based on partial factor 1.35 and 100 year return period.

Parameter	**Return period**					
	50 years		***200 years***		***1000 years***	
Toe height (m)	2.41	*-17.2%*	3.03	*-14.8%*	2.68	*-14.3%*
$\mathbf{D_{n50}}$ **(m)**	1.21	*-17.3%*	1.51	*-14.7%*	1.34	*-14.4%*
$\mathbf{M_{50}}$ **(kg)**	4558	*-43.3%*	9014	*-38.1%*	6285	*-37.1%*
$\mathbf{h_t/h}$	0.63		0.61		0.62	
$\mathbf{h_t/D_{n50}}$	11.77		8.97		10.37	

Table 56: Toe-stability for CC1.

Parameter	**Return period**					
	200 years		***800 years***		***4000 years***	
Toe height (m)	3.03	*+3.9%*	3.68	*+3.6%*	3.17	*+1.4%*
$\mathbf{D_{n50}}$ **(m)**	1.51	*+4.0%*	1.84	*+3.6%*	1.59	*+1.2%*
$\mathbf{M_{50}}$ **(kg)**	9014	*+12.2%*	16 213	*+11.3%*	10 401	*+4.1%*
$\mathbf{h_t/h}$	0.61		0.58		0.60	
$\mathbf{h_t/D_{n50}}$	8.97		7.02		8.46	

Table 57: Toe-stability for CC3.