A case study of the new breakwater of the Port of Genova

Comparing the PIANC design method with the new EUROCODE

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A case study of the new breakwater of the Port of Genova: Comparing the PIANC design method with the new EUROCODE

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Preface

This report was written as an additional thesis project, being part of the requirements to complete my MSc in Hydraulic Engineering offered by Delft University of Technology. This effort started with great prospects for me to deliver a great result in a short period of time. With the last year being a pretty bumpy ride for me personally, and despite the fact that I needed more time than I expected, I am happy with the result and the lessons learned during this process. I can now confidently say that the design of vertical wall breakwaters is an exciting, yet challenging topic, for which I gained vital knowledge. Therefore, I would like to briefly express my gratitude to the people that helped me in this study.

First of all, my additional thesis supervisor Prof. Alessandro Antonini who gave me this opportunity by providing the idea for a case study in Italy. Secondly, to the other member of my committee Coen Kuiper, I would like to sincerely thank you for our cooperation. Lastly, I would like to say a huge thank you to my family and friends who stood by me this last year and throughout my study years, and who would hopefully be there in the years to come.

Serafeim Bachras, Rotterdam, June 2022

Abstract

Breakwaters used to protect harbours and coastal areas worldwide are one of the most common coastal structures. The complexity of the physical processes associated with the design has led to the development of many empirical formulas while a standardized method for the selection of breakwaters' design parameters in the EU does not yet exist. The most common approach to design such a structure nowadays entails using information and recommendations from design manuals and guidelines such as PIANC and the Eurotop Manual. A new Eurocode 1 which includes specific considerations for coastal structures such as breakwaters is in the development process. This study aims to compare the PIANC method with the method to derive actions/loads included in the Eurocode proposal prEN1991-1-8. To do so the following research question has been formulated:

"What differences between the PIANC method and the method proposed by the new EUROCODE in the design of a vertical wall breakwater can be identified, using the new breakwater at the Port of Genoa as a case study?"

As mentioned in the question, a case study is used. The Port of Genoa one of the biggest ports in Italy plans to construct a new vertical wall breakwater. An initial design is openly available along with wave and water level data. This design is assessed using both methods and is further optimized. The aim is to gain insights into the differences between the PIANC and the new method.

At first, the failure mechanisms of such a structure are defined along with the safety factors and parameters. The data required to perform such an assessment is also an important aspect of the exercise. Most of the data are openly available during the consulting phase for the new breakwater in Genoa. In cases where extra data were necessary, they were based on the literature or on reasonable assumptions.

Based on the failure mechanisms and the retrieved data, the initial breakwater cross-section was assessed. The assessment both with the PIANC method and the method proposed in the new Eurocode proved that this design is sufficient and can be further optimized to decrease its costs. A high-level optimization is also conducted as part of this study in order to better understand the differences between the two methods. It can be concluded that the differences lay more in the method than in the actual result. For example, the proposed Eurocode creates a stable theoretical framework of how to choose a return period. The actual number may be very similar to the one that one would have used either way, but the choice can be argued in a better way.

On the other hand, the use of the new Eurocode revealed some problems and inconsistencies in the document which is confusing in certain parts. In addition, the new Eurocode which among others aims at standardizing the design process, However, parts like the combination of wave and water level actions and the crucial choice of return period for the two main limit state functions are relatively clearer providing a solid base.

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1 Introduction

Around the globe, various types of breakwaters have been designed and constructed with the most common type between them being a rubble mound breakwater. Breakwaters are used to protect vessels, harbours and ports from wave action, but some breakwaters are also used to protect coastal areas from eroding or valuable habitats against the forces of the sea (van den Bos & Verhagen, 2018). In cases where the water depth significantly increases or where environmental concerns of a breakwater with a large footprint exist a vertical face monolithic breakwater is starting to become a more attractive solution.

However, the problem of stability of monolithic breakwater is not yet solved in a way that everyone accepts and follows (van den Bos & Verhagen, 2018). In the last decades, the formulas first proposed by Goda in 1985 are in widespread use regarding the design of vertical face breakwaters.

In Europe, the standardization of design processes of all civil engineering works is something that the ten existing EN Eurocodes try to achieve. They are expected to contribute to the development and proper functioning of the construction market along the continent while leading to uniform levels of safety in Europe and elsewhere (European Commission, n.d.). Breakwater design is about to be part of this standardization and included in Eurocode 1: Actions on structures — Part 1-8: General actions — Actions from waves and currents on coastal structures.

1.1 Research Objective

The main objective of this research is to identify the changes in the cross-section of a vertical wall breakwater that the new Eurocode will bring. At the moment, it is up to the designer to define failure probabilities, return periods, failure mechanisms and other very important criteria. This process is based on the PIANC 196 guidelines and other similar standards and guidelines. Using the new breakwater at the Port of Genoa as a case study the above objective will be achieved. This specific project is still in the design phase, but construction can be expected to start in the next years, so a detailed design of the new breakwater's cross-section would be beneficial.

The breakwater will be designed using both the conventional, PIANC method and the method that the new Eurocode is providing. The study is aiming at identifying the possible differences between the two methods as one of its last objectives, as well as providing insights on the design of the new breakwater of the Port of Genoa.

1.2 Background

The Port of Genoa is a multi-service port located in the northwest of Italy and is part of the Western Ligurian Sea Port Authority (AdSPdMLO). The port is equipped to accommodate all classes of ships and various commodity sectors like containers, general cargo, steel, solid and liquid bulk, petroleum products along with cruise and ferry passengers. In addition, backed up by a group of dedicated companies, the port guarantees a full range of vital complementary services from ship maintenance and repair to customized telecommunications and data processing. In 2019 the port dealt with 4.5 million passengers, 2.7 million TEUs and 15.3 million tonnes of cargo. In the picture below the Port of Genoa is depicted as of 2022.

Figure 1-1: Port of Genoa as of 2022

Based on the above numbers the importance of the Port of Genoa for Italy itself but also for the whole region is evident. Given that vessel sizes continue to increase in the latest years, a new breakwater aiming to enlarge the manoeuvring basin as well as the access channel to the port is needed to allow the port to keep playing a dominant role in national and European markets. At the same time, the current breakwater is close to the breaker zone of the waves and thus is considered unsafe.

A big public debate has taken place regarding the new breakwater of the port. Three different solutions have been proposed, each one with two phases of construction. The solution to be investigated in this study is Solution 3, as shown in the following figure, as this is the one preferred by various stakeholders of the project including the Port Authority.

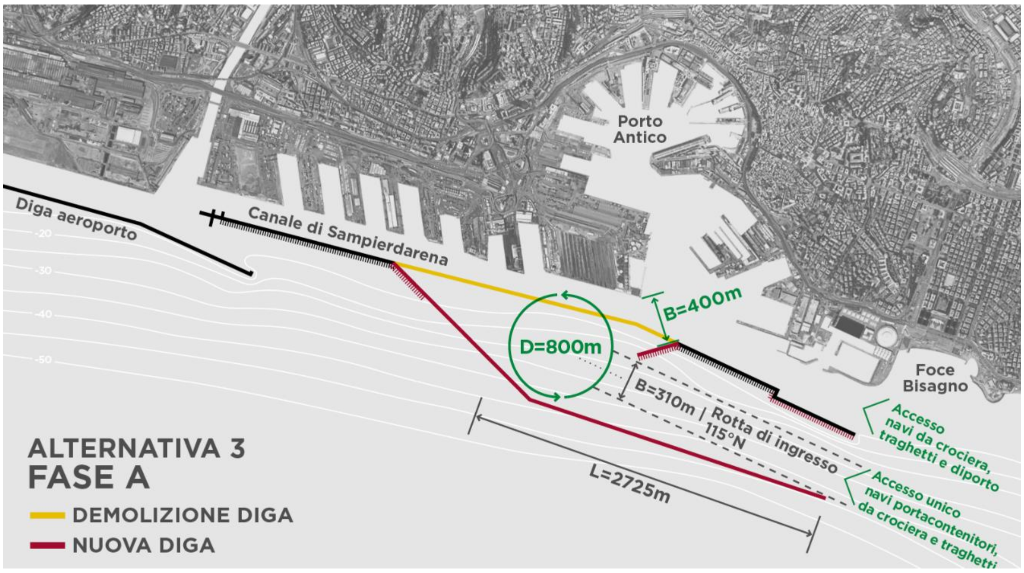

Figure 1-2: Proposed Solution 3 – Phase A (retrieved from Porto di Genova, 2020a)

1.3 Research Question

Based on the introduction, the problem statement and the background related to the breakwater of the Port of Genoa, as explained above, the research question and sub-questions can be stated:

"What differences between the PIANC design method and the method proposed by the new EUROCODE in the design of a vertical wall breakwater can be identified, using the new breakwater at the Port of Genoa as a case study?"

In order to answer this research question, the following sub-questions will be addressed:

A. What are the data required to design the new breakwater and how can this data be acquired?
B. Which are the failure mechanisms to be taken into account?
C. Which are the safety factors of the initial design for each of the two investigated design approaches?
D. How does a cross-section of the breakwater in each of the two cases look like if optimized and what are the differences between the PIANC design method and the new Eurocode 1?

1.4 Research Method

The design method between the two different approaches is expected to be almost identical, in terms of equations and graphs to be used. Therefore, the return periods and the way the boundary conditions are defined will lead to (or may lead to) differences. The 'PIANC design method' will be based on popular reports and guidelines mainly in the PIANC 196 guidelines, along with the Eurotop Manual and the Coastal Engineering Manual (Eurotop, 2018; PIANC, 2016; USACE, 2011) and the design method as explained in the publications of Goda (Goda, 2000). The 'new method' will be based on the prEN1991–1–8: Final Doc (Date: 06-2020) (CEN/TC250, 2020).

As has already been mentioned the new breakwater of the Port of Genoa will be used as a case study. A case study is a research strategy and an empirical inquiry that investigates a phenomenon within its real-life context (Press Academia, 2018). One of the main advantages of using such a big project as a case study -something this study will focus on- is that not much pre-structuring is required compared to other research strategies making the use of a case study much more flexible (Verschuren & Doorewaard, 2010).

1.5 Report Outline

This section is providing an outline of every chapter of this study. To begin with, in the following Chapter, Chapter 2, the two design methods are briefly explained, paying more attention to the new information that Eurocode 1 is providing. At the end of this chapter, a short summary of problems and inconsistencies found in prEN 1991-1-8 is also provided.

Chapter 3, focuses on the case study. First, the data required are given, in addition to how they were collected. Afterwards, the failure mechanisms and the initial cross-section that is used as a basis of design are presented, answering sub-questions A and B.

In the next Chapter, Chapter 4, the information provided in the two previous ones is combined aiming at determining the safety factors (or other relevant parameters) per failure mechanism and per method. This is conducted using the initial design of the new breakwater and in fact, answers sub-question C. The cross-section is optimized for each method, in Chapter 5 providing insights and answering sub-question D.

In Chapter 6, the results and insights gained during this study are discussed and a reflection of this study is provided, and in Chapter 7, which is the last one of this report, the recommendations for further study along with the conclusions are presented by the author.

2 The two design methods

In this chapter, the two investigated methods will be briefly explained. It should be noted that reference will be done only in the aspects related to the design of a vertical wall breakwater. The discussed reports and papers include much more information regarding other types of breakwaters, which are outside of the scope of this research.

2.1 PIANC 196 method

In this section a (very) brief explanation of how vertical wall breakwaters are designed is presented, as shown in the figure below. PIANC Report No. 196 along with the Eurotop manual are the two basic documents used. As also explained in Chapter 1, this method is dependent on the choices of the designer. As the method is widely used it is not elaborated on in this report for the sake of simplicity.

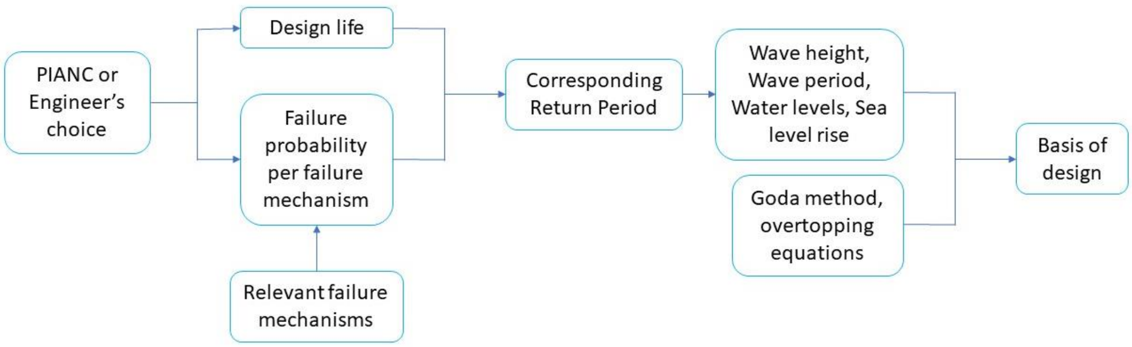

Figure 2-1: PIANC design method schematization

The first design of the case study used for this research, the new breakwater of the Port of Genoa, is based on this method. However, the safety factors of the design are not provided so part of Chapter 4 will be dedicated to calculating them.

2.2 The new EUROCODE

The new 'Eurocode 1: Actions on structures — Part 1-8: General actions — Actions from waves and currents on coastal structures' (prEN 1991-1-8) was first reviewed and analysed in order to understand the procedure that the design of a new vertical wall breakwater requires, which can be summarized as follows:

1. The first step is to pick a consequence class according to Table 4.1 of the report (shown below with custom numbering). This choice is critical as many of the steps that follow are based on this. The table provides a column related to the qualifications of consequences (sub-divided between Loss of human life or personal injury and Economic, social or environmental consequences) as well as coastal structure examples.

Consequence class	Qualification of consequences		Coastal structure examples
	Loss of human life or personal injury [a]	Economic, social or environmental consequences [a]	
CC4 – Highest	Extreme	Huge	National or regional scale flood protection structures. Structures providing a critical role at a nuclear facility.
CC3 – Higher	High	Very great	Public access structures carrying large numbers of people; residential buildings or supporting properties; or with hazardous sea conditions beneath, e.g. piled jetties/ decks or floating structures. Nationally significant port/ terminal structures, e.g. breakwaters protecting major ports or piled jetties/ decks for import/ export of critical goods (such as energy supplies).
CC2 – Normal	Medium	Considerable	Regional or local port/ terminal structures, e.g. breakwaters protecting small fishing ports/ leisure marinas or floating structures with occasional public access (and limited numbers of people).
CC1 – Lower	Low	Small	Structures with limited access (for trained personnel only) and supporting or protecting non-critical infrastructure or properties, e.g. a single pile supporting a (non-critical) navigation marker.
CC0 – Lowest	Very Low	Insignificant	Structures with no access, not supporting any infrastructure or property, and/ or with no significant environmental protection role.
[a] The consequence class is chosen based on the more severe of these two columns.			

Table 2-1: Consequence Classes

2. The next choice that one should make is the design service life (Table 4.2 of prEN 1991-1-8, Table 2-2 below). Again examples of structures are provided to simplify and objectify the choice.

Design service life, T_{life}	Coastal structure examples
years	
100	Common port infrastructure including breakwaters for ports of nationally significant strategic or economic value. Infrastructure for regional flood defence or coast protection.
50	Common port infrastructure for commercial and industrial ports including reclamation, shore protection, breakwaters, quay walls. Infrastructure for local flood defences or coast protection.
25	Structures dedicated to non-renewable natural resources, petrochemicals or similar industrial or commercial applications (such as open-piled jetties, mooring and berthing dolphins).
≤ 10	Temporary structures such as construction material import/ export facilities, temporary works during construction such as cofferdams, other structures with short life such as for a one-off event, or the structure itself during construction.

Table 2-2: Design service life

3. The proposed Eurocode introduces a way to quantify the variety of wave and water level conditions, called Hydrodynamic Estimate Approach (HEA). The two parameters that determine the HEA are the consequence class and the hydrodynamic uncertainty. Therefore, one of the first steps of the design process is to determine the HEA.

Consequence class	Hydrodynamic uncertainty [1]		
	LOW[2]	MEDIUM	HIGH[3]
CC0 Lowest[4]	1	1	1
CC1 Lower	1	1	2
CC2 Normal	1	2	2
CC3 Higher	2	2	3
CC4 Highest[4]	2	3	3

[1] Hydrodynamic uncertainty is defined by the level of understanding of environmental sea conditions at the site of interest and will depend on the relative complexity/ severity of physical processes (boundary condition generation and transformation to the site) and also the quality and quantity of available data (in the form of measurements, models, semi-empirical or empirical estimates).

[2] Examples of low hydrodynamic uncertainty may include tidal range < 1m, surge < 0,5m, fetch-limited seas (with fetch < 10 km), uniform currents with spring tide velocities < 1m/s, regular bathymetry or high quality time-series data of relevant sea condition parameters covering several decades at more than one location in the area of interest and high quality (recent) topo-bathymetric data.

[3] Examples of high hydrodynamic uncertainty may include tidal range > 3 m, surge > 1,0 m, ocean seas (swell and wind-waves), non-uniform currents (stratified) and/ or tide or surge current velocities > 1m/s, irregular bathymetry (e.g. reefs or sub-sea canyons) or limited quality and/ or duration of environmental sea condition parameters or low resolution or historic topo-bathymetric data.

[4] prEN 1990:2020 states that for consequence classes CC0 and CC4 'alternative provisions to those given in the Eurocodes may be used'.

Table 2-3: Hydrodynamic Estimate Approach (HEA)

4. The Design Approach that is advised to be followed, is the next step in the process proposed by prEN 1991-1-8. Five design approaches are included, which are the following:
   - DA-0: The deterministic approach in which the Return Periods together with a global safety factor should provide the required level of safety
   - DA-1: The semi-probabilistic approach that uses characteristic values and partial factors
   - DA-2: The full probabilistic approach for which a limit state function and complete distributions are required
   - DA-3: The approach in which the design is optimised based on risk, social and economic considerations
   - DA-4: The approach that uses physical modelling to assist the design; should be used in combination with one of the other design approaches

   Based on the HEA and the structural response uncertainty, a minimum required design approach can be defined, as shown in the following table. According to the proposed Eurocode "low structure uncertainty can, for example, apply where the physical processes/ response mechanisms are relatively simple and/ or there is an established 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."

HEA Level	Low-to Medium Structure Design/Response Uncertainty[1]	High Structure Design/Response Uncertainty[1]
HEA-1	DA-0	DA-1 or DA-2[2]
HEA-2	DA-1 or DA-0[2]	DA-2 or DA-4[3]
HEA-3	DA-2 or DA-4[3]	DA-2 and DA-4[3]

[1] Where the term uncertainty is defined in NOTE 2 above.
[2] When no partial factor values are readily available.
[3] Where insufficient data is available to support a probabilistic approach, may refer to a part of the structure, e.g. breakwater roundhead or crown wall.
DA-0: Using a deterministic approach with return periods and appropriate sensitivity testing of key parameters, based on application of semi-empirical structure response formulae; global safety factors are applied.
DA-1: Using semi-probabilistic partial load and resistance factors.
DA-2: Using a fully probabilistic approach with allowable probabilities of failure or beta indexes
DA-3: Using a risk-informed method of socio-economic optimisation to determine the optimum probability of failure of the considered structure; this approach is not included here but is recommended by some National Standardisation Bodies.
DA-4: Using a design assisted by testing approach in combination with one or more of DA-0, DA-1, DA-2 (or DA-3).

Table 2-4: Minimum Design Approach level

5. With the above variables chosen, using the corresponding tables of prEN 1991-1-8, the Return Period, that the structure has to be designed for ULS and SLS-(LD), can be found. The tables as provided by prEN 1991-1-8 are shown below. Apart from this Table, however, Chapter 4 of prEN 1991-1-8 provides minimum RP per consequence class. It is not clear to the author which approach is the one proposed by prEN 1991-1-8, given that both are included. For the purpose of this study, Table 2-5 and 2-6 will be used to determine the return period for the two limit states.

Consequence Class	Design service life (t_{life})			
	< 10 years	25 years	50 years	100 years
The RP values in this table can be used when combining the sea state components (e.g. wave height, sea level, current velocity) in order to estimate the design (or extreme) value of the sea condition in the fundamental combination of actions. One of the sea state components (the dominant component) is considered with the following RP:				
CC3	[40] y	[100] y	[200] y	[400] y
CC2	[20] y	[50] y	[100] y	[200] y
CC1	[10] y	[25] y	[50] y	[100] y
The other sea state components (the accompanying components) are considered with the following RP[a] :				
CC3	[4] y	[10] y	[20] y	[40] y
CC2	[2] y	[5] y	[10] y	[20] y
CC1	[1] y	[2,5] y	[5] y	[10] y

<These RP values apply to offshore sea conditions, i.e. before propagation to the shoreline>
[a] Fully correlated actions shall be assumed as dominant components
The statistical estimation of the RP value can be based on the upper bound of a given confidence interval, considering the uncertainty due to the limited sampling size.

Table 2-5: Return period for the dominant and accompanying components based on consequence class and service life (ULS)

Consequence Class	Design service life (t_{life})			
	< 10 years	25 years	50 years	100 years
The RP values in this table can be used when combining the sea state components (e.g. wave height, sea level, current velocity) in order to estimate the characteristic value of the sea condition in the characteristic combination of actions. One of the sea state components (the dominant component) is considered with the following RP:				
CC3	[4] y	[10] y	[20] y	[40] y
CC2	[2] y	[5] y	[10] y	[20] y
CC1	[1] y	[2,5] y	[5] y	[10] y
The other sea state components (the accompanying components) are considered with the following RPs :				
CC3	[2] y	[5] y	[10] y	[20] y
CC2	[1] y	[2,5] y	[5] y	[10] y
CC1	[1] y	[1] y	[2,5] y	[5] y
These RP values apply to offshore sea conditions, i.e. before propagation to the shoreline. The statistical estimation of the RP value can be based on the upper bound of a given confidence interval, considering the uncertainty due to the limited sampling size.				

Table 2-6: Return period for the dominant and accompanying components based on consequence class and service life (SLS-(LD))

6. With a known return period per Limit State (ULS and SLS-(LD)), and taking into account the relevant failure mechanisms the return periods to be used for each one can be determined. The significant wave heights, wave periods and water levels are calculated for those return periods.
7. Regarding the use of Partial factors, Chapter 4 of prEN1991-1-8, refers to Appendix A6 of EN1990. At the same time, Annex A.7 of prEN1991-1-8 contains a table with different values than that of EN1990 and also refers to PIANC196. It is not clear to the author which of the aforementioned options is the one proposed by the Eurocode. Due to the above inconsistency, the use of partial factors will not be taken into account in this report.

2.3 Problems and inconsistencies of the new method

This section provides a summary of different problems and inconsistencies that the author discovered while using prEN1991-1-8. It is meant to act as constructive criticism, aiming at an updated and more easy-to-use version of the report. The unclear or problematic points are summarized below:

- It is not easy to read and to use. The document keeps referring to other sections, appendices and even other documents, without providing the formulas in the document itself.
- The way to choose the consequence class of a structure is confusing (see Table 2.1). There are two criteria included. One is related to the loss of human life or personal injury and the second to economic, social or environmental consequences. At the same time, the table provides examples of structures. The way to determine the consequence class is not clear. One could argue that you can do it based on the examples in the last column of the table. However, if one does that but at the same time the consequences in terms of human life are not consistent with what the table provides, then an inaccuracy comes into place.

- The way to determine the return period is also confusing. Apart from Table 2.3, there are various information in Appendix C of prEN 1991-1-8 and Chapter 4 of the document. Even information related to the failure probabilities, which are directly related to the return periods and therefore can be easily calculated. For this study, Table 2.3 provided above is used.
- The Eurocode introduces partial factors, however, the exact values of those are difficult to determine. The wave action is considered a variable action which is reasonable; however, it is not clear to the author how to treat other wave properties (wave length, wave period) after multiplying the wave height with the corresponding partial factor. This may lead to unrealistic results in terms of physical meaning as in physical terms, wave heights are limited by their steepness. Reference on this subject is made in the last chapter of this report as part of the recommendations.

3 Data Collection and Failure Mechanisms

This chapter is aiming at answering sub-questions A and B. Data collection is a crucial part of every study (Section 3.1), while determining the relevant failure mechanisms for this vertical breakwater project is also of great importance (Section 3.2).

3.1 Data Collection and basis of design

In order to design the new vertical face breakwater of the Port of Genoa certain data are required. These are either acquired from the open-source files available online or are based on other sources and assumptions. In the following table, the data that are used in this study are summarized, and the source of them is indicated along with some additional comments that are considered useful. Appendix A contains most of the information related to wave statistics, while Appendix C contains information related to water level data. It is important to note that for vertical wall breakwaters the design wave height is the maximum wave height (H_{max}). The provided data, specify the significant wave height

Parameter	Source	Extra comments	Value
Significant wave height	(Porto di Genova, 2020b)	-	Check Tables A.1 and A.2 (Appendix A)
Peak wave period		-	
Wave direction		-	
Wave setup and Storm surge		-	Check Table C.1 and C.2 (Appendix C)
Tidal range		-	0.15 cm
Sea Level Rise	(Oppenheimer et al., 2019)	Estimation based on available maps for RCP8.5 scenario and the time slice 2046-2065	0.20 cm
Proposed cross section's geometry	(Porto di Genova, 2020a)	As shown in Figure 3.1 below and Table 3-2	Check Figure 3.1 below
Breakwater's orientation	(Google Earth, 2021)	Estimation from Solution 3 views	110° N
Water depth	(Porto di Genova, 2020a)	As given in page 28	35 m
Bed slope	(NAVIONICS, 2020)	Approximation of a steady slope	1:78

Table 3-1: Required data and sources

A first, preliminary design of the new breakwater was provided by the responsible working group for the stakeholder discussion and the open debate that followed. This design will act as a basis for the following design steps.

The figure below gives a better understanding of this first design. The initial width of this cross-section is 26 m and the freeboard is equal to 7 m. The breakwater consists of six chambers (each chamber is 4 m wide) and the water depth is assumed to be around 35 m. In the next chapter, the safety factors for this first design with both of the investigated methods will be calculated and later on the design will be optimized.

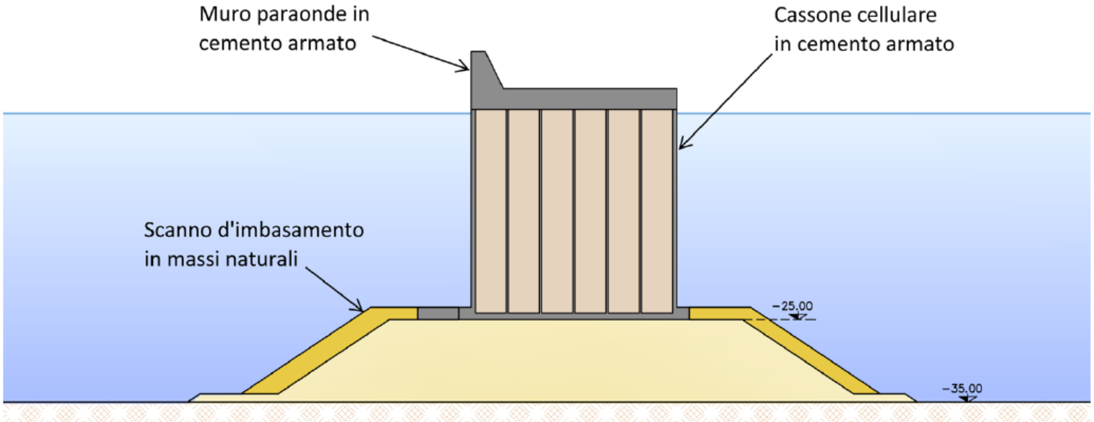

Figure 3-1: Cross-section to be assessed (Porto di Genova, 2020a)

Characteristic	Value or type
Breakwater type	Vertical wall
Freeboard	7 m
Width	26 m
Amount of chambers	6
Chamber dimensions	4 x 4 m^2
External concrete wall thickness	0.5 m
Internal concrete wall thickness	0.2 m
Water depth	35 m
Vertical wall depth	25 m

Table 3-2: First design cross section's details

3.2 Failure mechanisms

For the design, the failure mechanisms that are shown in the following table will be taken into account. These are the major failure modes when designing a vertical wall breakwater. The failure probability -and thus the return period- is related to the method, and will be further explained in the following sections. The way that the structure would fail in any case is shown in the second column.

The relevant parameters and the criteria presented in the last column of the table are based on existing knowledge and literature. For sliding and overturning the typical limit used is 1.2 according to Goda, (2000). In the same publication, Goda recommends designing caisson breakwaters keeping the foundation pressure below 400 to 500 kPa. For the stability of the block toe and the rubble mound, the required dimension of the block (D_{foot}, L_{foot}), along with the rock mass (M_{50}) and rock nominal diameter (D_{n50}) can be quantified. For both the above failure modes the criterion considered is that no damage should occur that may endanger the structural integrity of the caisson breakwater. According to Eurotop, (2018) tolerable overtopping for caisson breakwaters is often not related to structural design (ULS), but more to restrictions from port operations (SLS). In the case of the new breakwater of the port of Genoa, the area on the lee side of the investigated breakwater is part of the access channel and the turning circle. Allowable overtopping is set to 10 l/s per m. Lastly, the maximum allowable transmitted wave height behind the breakwater is assumed to be 1.5 m, also related to the serviceability limit state (SLS-(LD))[1]. This is because there is a second breakwater that is protecting the berths where the allowable wave height is even lower.

Failure Mechanism	**Limit State**	**Relevant parameter**
Sliding	ULS	$SF_{sliding} > 1.2$
Overturning	ULS	$SF_{overturning} > 1.2$
Soil bearing capacity	ULS	pe < 400
Block Toe Stability	ULS	D_{foot} , L_{foot}
Berm Rubble Mound Stability	ULS	D_{n50}, M_{50}
Overtopping	SLS-(LD)	q < 10 l/s/m
Transmission	SLS-(LD)	$H_{sT} < 1.5$ m

Table 3-3: Failure mechanisms to be considered, limit states and relevant criteria

[1] It is important to note that prEN1991-1-8 introduces a new limit state called SLS-(LD) where LD means Limited Damage. This new limit state is aimed at structures like breakwaters, where a small damage can be acceptable as repairs are fairly easy to execute.

4 Assessment of the basis of design

After the two methods were briefly explained in Chapter 2, the required data were acquired and the failure mechanisms were defined in Chapter 3, all the information that one needs for the design of the new breakwater is available. This chapter aims at answering sub-questions C and D of this study.

4.1 PIANC design method

The basis of design can be seen in Figure 3-1. The safety factors of this design are calculated with both methods in this section. Based on the previous studies, two main wave directions were identified. The main -dominant- wave direction is included in the scope of this report. First, the PIANC design method is used. For the two limit state functions (ULS and SLS), a corresponding failure probability is defined. This definition is based on PIANC, 2016 and Table 8-9 of the referenced document (Table 4-1 below).

Incipient Damage	Risk to Human life	
Economic Repercussion	Limited	High
Low	0.50	0.30
Medium	0.30	0.20
High	0.25	0.15
Total Destruction	Risk to Human life	
Economic Repercussion	Limited	High
Low	0.20	0.15
Medium	0.15	0.10
High	0.10	0.05

Table 4-1: Maximum probability of admissible damage Pf in the period of working life (PIANC, 2016)

The risk to human life is considered limited, as the investigated structure is a detached breakwater. For incipient damage, the economic repercussion is low leading to a failure probability of 0.50 for SLS. For total destruction, the economic repercussion for a port of significant importance as the one of Genoa is considered high, leading to a failure probability of 0.10. In the table below the failure probability and return period of the PIANC design method are shown. The return period is calculated assuming a design life of 50 years.

Limit State Function	Failure Probability	Return Period (years)
ULS	0.10	475
SLS	0.50	70

Table 4-2: Failure probabilities and return periods per limit state function

In Table 4-3 corresponding significant wave heights and peak periods are determined, for each wave direction. For the determination of the peak period per case, the equations for both the primary and secondary wave directions, are provided in Appendix A. Those data will be used as input in order to determine the safety factors of the primary wave direction.

Return Period (years)	Wave direction (°N)	Significant wave height (m)	Peak period (s)
70	179	7.55	11.00
	203	7.05	10.70
	217	4.90	9.30
475	179	8.90	11.70
	203	8.35	11.45
	217	5.88	10.00

Table 4-3: Significant wave heights and peak periods per return period

Which wave angle to choose?

As can be seen in Table 4.2 three wave angles are identified in both the primary and secondary wave direction. Taking into account the fact that Goda proposed to reduce the incident wave angle by 15° as a safety factor for uncertainties regarding the wave directions, and that the shore normal equals to 200°N, the direction of 203° is assumed to be head-on the breakwater, while the wave coming from 179° has a difference of 21° from the shore normal, and thus the incident wave angle is equal to 6°. Also considering, the difference in significant wave heights between the two wave directions (8.90 vs 8.35 for 475 years of RP), the one with the higher significant wave height is the one to be taken into account.

Combining the information provided in Sections 2.1, 3.1 and 3.2 the safety factor of each failure mechanism is calculated. The significant wave height and peak period related to each return period are used in each case. The output with the safety factor or relevant parameter per failure mechanism, using the above data and the PIANC design method are presented in Table 4-4. The characteristics of the cross-section are those summarized in Table 3-2. The storm surge and wave setup provided in Table A.3 do not cover the return periods that are taken into account in this section (70 and 475 years). Thus, the value of the storm surge wand wave setup for those return periods is calculated based on the existing values (see Appendix C). Sea Level Rise and Tidal Range are assumed to be constant regardless of the return period and are given in Table 3-1. An explanation of which wave angle was chosen is given in the box above.

Primary Wave Direction					
Failure mechanism	Return Period	Significant wave Height (m)	Incident wave angle (degrees)	Storm surge and wave setup (m)	Sea Level Rise and Tidal Range (m)
Sliding	475	8.90	6	0.99	0.35
Overturning	475	8.90	6		
Soil bearing capacity	475	8.90	6		
Block Toe Stability	475	8.90	6		
Berm Rubble Mound Stability	475	8.90	6		
Overtopping	70	7.55	6	0.74	
Transmission	70	7.55	6		

Table 4-4: Primary wave direction (179°N) return periods and inputs

Table 4-5 below provides an overview of the output for each failure mechanism.

Failure mechanism	Limit State	Safety Factor or relevant parameter
Sliding	ULS	$SF_{sliding}$ = 2.21 > 1.2
Overturning	ULS	$SF_{overturning}$ = 2.73 > 1.2
Soil bearing capacity	ULS	pe = 373 < 400
Block Toe Stability	ULS	l x b x t' = 4.0 x 2.5 x 1.2
Berm Rubble Mound Stability	ULS	M_{50} ≥ 1460 kg, Dn50 ≥ 0.84 m
Overtopping	SLS	q= 0.63 < 10 l/s/m
Transmission	SLS	K_T = 0.13 leading to an H_{st} = 0.98 m

Table 4-5: Outputs (primary wave direction, PIANC design method)

The exact data used for the determination of the above are summarized in Appendix B and the equations in Appendix D. It is clear from the above table, that the proposed cross-section is sufficient. Optimization of the cross-section in terms of geometry will be conducted as part of Chapter 5.

4.2 The new Eurocode

In this part, a similar procedure as the one defined in the previous part is conducted. The only difference is the way the return periods are determined based on the method explained in Chapter 2. So firstly those return periods will be determined.

4.2.1 Return periods – Wave and water level data

The first step is to define the consequence class. According to Table 2-1, the failure of a structure classified as CC3 – Higher class, has high consequences in terms of loss of human life or personal injury and very great consequences when it comes to economic, social or environmental ones. The latter is indeed true in the possibility of a failure, however, given the fact that it is a detached breakwater, loss of lives or injuries seem unlikely. It is not clear to the author, if the two should happen at the same time, or if the existence of one of the two criteria is sufficient (see Section 2.3). The provided examples according to the table, include the studied breakwater as they refer among others, to “Nationally significant port / terminal structures, e.g. breakwaters protecting major ports”. The Port of Genoa is the second biggest in Italy and the biggest on the west side, as the country’s biggest is the Port of Trieste (Menon, 2021). Based on the above, for the new breakwater of the Port of Genoa, a consequence class CC3 is chosen.

Similarly, the design life according to prEN1991-1-8 is determined based on Table 2-2. According to it, “Common port infrastructure including breakwaters for ports of nationally significant strategic or economic value”, like the investigated one, requires a design life of 100 years.

The Hydrodynamic Estimate Approach (HEA) is determined based on Table 2-3. As explained above the consequence class chosen is CC3. The hydrodynamic uncertainty is the other parameter relevant for the HEA. According to prEN1991-1-8, “it is defined by the level of understanding of environmental sea conditions at the site of interest and will depend on the relative complexity/ severity of physical processes (boundary condition generation and transformation to the site) and also the quality and quantity of available data (in the form of measurements, models, semi-empirical or empirical estimates)”. The wave and water level data that were used are based on real measurements and detailed modelling. At the same time, the sea has irregular bathymetry and wind waves coming from different directions and a relatively large fetch. Based on the above, a medium hydrodynamic uncertainty is chosen. According to Table 2-3, for a medium HEA and a CC3 consequence class, the Hydrodynamic Estimate Approach gets a value equal to two (2).

The way to choose the Design Approach is discussed in Chapter 2 of this report. The HEA level and the Structure Design/Response Uncertainty are the two factors to be taken into account. In the case of the design formulas for a vertical wall breakwater, the method is widely used for several years but the physical processes are complex. Therefore, a medium structure design uncertainty is chosen. Combining this with the HEA, and using Table 2-4, a Design Approach for DA-1 is proposed. This also matches with the clause of the Eurocode according to which "the semi-probabilistic design approach associated with the partial factors format as proposed in EN1990 shall be the default approach for coastal structures (DA-1)."

Table 2-5 along with the consequence class and the design life, will define the return period of the structure for the dominant component (wave action) and the accompanying components (water level) for the ultimate limit state (ULS), while Table 2-6 provides the same information but for SLS-(LD). For ULS, with a design life of 100 years and a consequence class of CC3, the return period for the dominant component is 400 years, while for the accompanying components it is 40 years. Similarly, for SLS-(LD) the return period of the dominant component is 40 years, and for the accompanying component is 20 years.

Parameter	Correspondent value
Consequence Class	CC3
Design life	100 years
Hydrodynamic Estimate Approach (HEA)	2
Design Approach	DA-1
Return Periods - ULS	Dominant Component: 400 years Accompanying Components: 40 years
Return Periods – SLS-(LD)	Dominant Component: 40 years Accompanying Components: 20 years

Table 4-6: Required parameters based on prEN1991-1-8

For the calculation of the safety factor per failure mechanism, the following data will be used. For wave data (dominant component), two return periods are relevant (400 and 40 years). The same applies to the water levels (accompanying components), with return periods of 40 and 20 years being the relevant ones.

It is important to note, that the use of partial factors would have a very important influence in the assessment as wave height may need to be increased by a factor of 1.35 or even 1.50. However, as explained in Chapter 2 and given that the Goda method already uses a value of the maximum wave height (H_{max}) equal to the significant wave height multiplied by 1.80, the use of a partial factor would lead to non-realistic values of the wave height and is thus not taken into account.

Return Period (years)	Wave direction (°N)	Significant Wave height (m)	Peak period (s)
40	179	7.05	10.71
	203	6.58	10.42
	217	4.53	9.01
400	179	8.77	11.66
	203	8.20	11.36
	217	5.74	9.88

Table 4-7: Inputs for wave data

Return Period (years)	Storm surge and wave setup (m)	Sea Level Rise and Tidal Range (m)
20	0.63	0.35
40	0.70	

Table 4-8: Inputs for water level data

4.2.2 Safety factors

The wave heights and periods, along with the water levels are the input required for the first assessment of the initial breakwater design. Similar to Section 4.1, the inputs are summarized in the table below, per failure mechanism.

Primary Wave Direction						
Failure mechanism	Return Period dominant component	Significant wave Height (m)	Incident wave angle (degrees)	Return Period accompanying component	Storm surge and wave setup (m)	Sea Level Rise and Tidal Range (m)
Sliding	400	8.77	6	40	0.70	0.35
Overturning	400	8.77	6	40	0.70	0.35
Soil bearing capacity	400	8.77	6	40	0.70	0.35
Block Toe Stability	400	8.77	6	40	0.70	0.35
Berm Rubble Mound Stability	400	8.77	6	40	0.70	0.35
Overtopping	40	7.05	6	20	0.63	0.35
Transmission	40	7.05	6	20	0.63	0.35

Table 4-9: Primary wave direction (179°N), wave and water level inputs

Failure mechanism	Limit State	Safety Factor or relevant parameter
Sliding	ULS	$SF_{sliding} = 2.23 > 1.2$
Overturning	ULS	$SF_{overturning} = 2.76 > 1.2$
Soil bearing capacity	ULS	pe = 371 < 400
Block Toe Stability	ULS	l x b x t' = 4.0 x 2.5 x 1.2
Berm Rubble Mound Stability	ULS	$M_{50} \geq 1421$ kg, $D_{n50} \geq 0.83$ m
Overtopping	SLS-(LD)	q= 0.50 < 10 l/s/m
Transmission	SLS-(LD)	$K_T = 0.11$ leading to an $H_{si} = 0.78$ m

Table 4-10: Outputs (primary wave direction, prEN1991-1-8 method)

4.3 Summary of assessment

In the two previous sections, the initial design was assessed based on the two investigated methods. The following table provides an overview of the result per failure mechanism per investigated method. It can be summarized that the design is sufficient in both cases and can be further optimized to reduce the dimensions (and therefore the cost) of the cross-section. This will be done in the following Chapter.

Failure mechanism	Safety Factor or relevant parameter	
	PIANC design method	New Eurocode 1
Sliding	$SF_{sliding} = 2.21 > 1.2$	$SF_{sliding} = 2.23 > 1.2$
Overturning	$SF_{overturning} = 2.73 > 1.2$	$SF_{overturning} = 2.76 > 1.2$
Soil bearing capacity	pe = 373 < 400	pe = 371 < 400
Block Toe Stability	l x b x t' = 4.0 x 2.5 x 1.2	l x b x t' = 4.0 x 2.5 x 1.2
Berm Rubble Mound Stability	$M_{50} \geq 1460$ kg, $D_{n50} \geq 0.84$ m	$M_{50} \geq 1421$ kg, $D_{n50} \geq 0.83$ m
Overtopping	q= 0.63 < 10 l/s/m	q= 0.50 < 10 l/s/m
Transmission	$K_T = 0.13$ leading to an $H_{si} = 0.98$ m	$K_T = 0.11$ leading to an $H_{si} = 0.78$ m

Table 4-11: Assessment overview

5 Optimization

The safety factors using both design methods and the initial design as a basis are now known. This chapter will lead to an updated version of the cross-section, in other words, optimize it, again using both methods. In order to do so, the same equations are used but this time without taking the initial design into account, but the criteria and parameters provided in Table 3-3. This way a sufficient but more economical design will be determined, while the width and freeboard of the breakwater will be re-calculated. This will be done first using the PIANC design method, and then using the method of prEN1991-1-8. In the last section of this Chapter, the two cross-sections will be drawn and possible differences between the two will be summarized.

5.1 Optimization using the PIANC design method

The wave and water level data provided as inputs are the same as those used when assessing the initial design, as given in Table 4-4. Using those data, a new geometry of the breakwater cross-section is determined. The geometry of the optimized cross-section is given in Table 5-1, along with the relevant safety factors and parameters.

Characteristic	New Value	Initial value
Freeboard	5.2 m	7 m
Width	20 m	26 m
Amount of chambers	5	6
Chamber dimensions	3.65 x 3.65 m^2	4 x 4 m^2
External concrete wall thickness	0.5 m	0.5 m
Internal concrete wall thickness	0.2 m	0.2 m

Table 5-1: Output of the optimization using the PIANC design method

The above geometry leads to a design that is sufficient. For example, pe = 399 > 400, with the soil bearing capacity being the limiting failure mechanism in terms of width, with transmission being the limiting one in terms of freeboard. The following figure shows the new cross-section, out of scale with five chambers instead of six, a total width of 20 m, and a freeboard of 5.2 m.

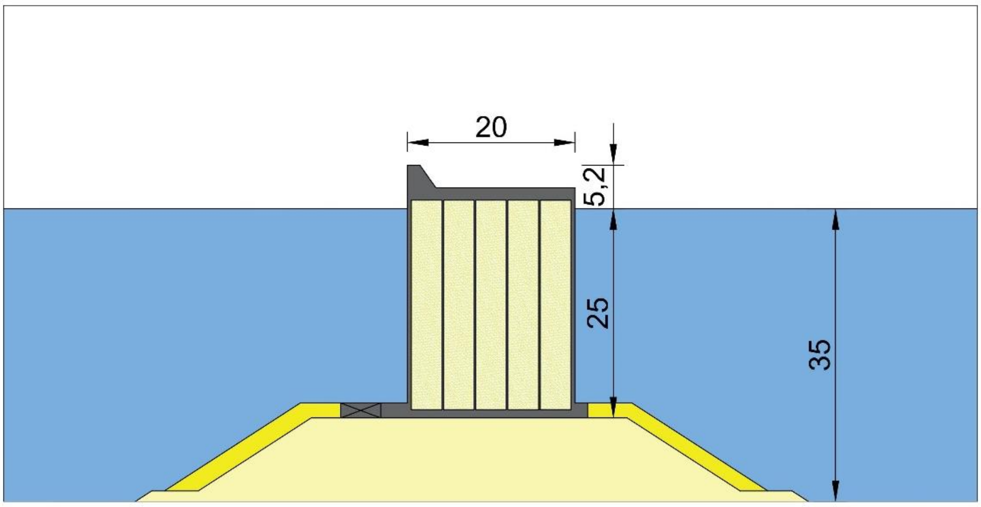

Figure 5-1: Optimized cross-section, based on the PIANC design method

5.2 Optimization using the new Eurocode 1

Same as in the previous section wave and water level data remain those explained before. The output of the optimization using the new Eurocode 1, is summarized in Table 5-2 below.

Characteristic	New Value	Initial value
Freeboard	4.5 m	7 m
Width	19 m	26 m
Amount of chambers	5	6
Chamber dimensions	3.48 x 3.48 m^2	4 x 4 m^2
External concrete wall thickness	0.5 m	0.5 m
Internal concrete wall thickness	0.2 m	0.2 m

Table 5-2: Output of the optimization using the new Eurocode

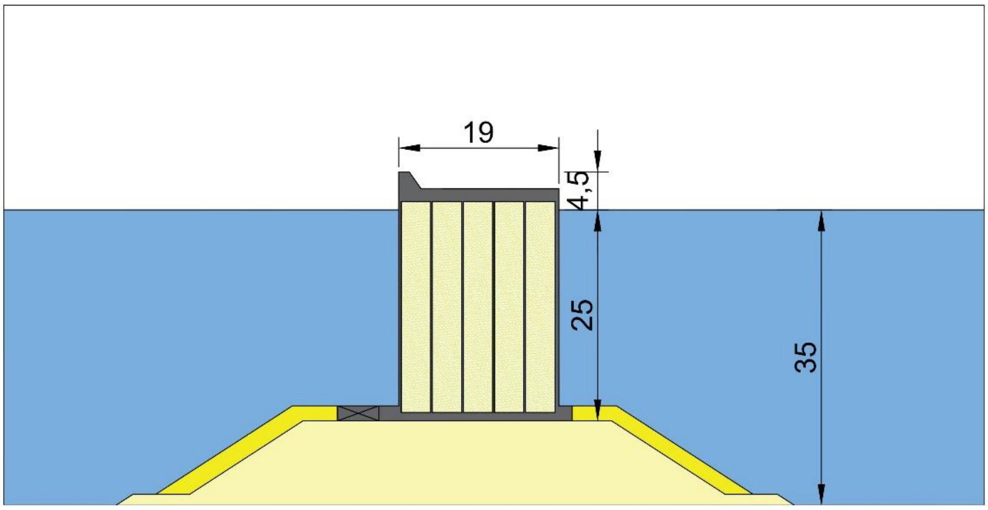

Figure 5-2: Optimized cross-section, based on the new Eurocode 1

5.3 The two cross-sections

The cross-section optimized using prEN1991-1-8 is very similar to the optimized version that is sufficient according to the PIANC design method, with a slightly reduced width (19 m instead of 20 m) and a slightly reduced freeboard (4.5 m instead of 5.2 m). This implies that the differences between the two methods when similar Return Periods are used are insignificant. Given that only small differences can be identified in the optimized cross-sections of both methods, a detailed comparison is not required. Figures 5-1 and 5-2 provide an overview of the two cross-sections.

6 Reflection

In this chapter, the results of this research are discussed and new insights are briefly explained. Precious insights gained from the Eurocode 1 proposal and the comparison with the PIANC design method are elaborated.

This study aimed at comparing an existing method of design with a new method proposed to be part of Eurocode 1. A new vertical wall breakwater, which is in the development phase is used as a case study. The initial design was first assessed, using both methods. Afterwards, the cross-section is optimized and the methods are compared. The primary wave direction (180°-240°) was taken into account. Regarding the secondary wave direction (105°-180°), it was decided to not investigate it, as it entails milder wave conditions. This wave direction leads to a wave angle which at the same time is more favourable for the breakwater to be designed.

The results of the assessment showed that the initial cross-section is sufficient regardless of the method of use. For example, the safety factor for sliding is 2.27 and 2.29 for the PIANC design method and the new Eurocode respectively. Reflecting on the results and the use of the methods, the PIANC design method gives a lot of flexibility to the engineer that uses it. The method proposed by prEN1991-1-8 tries to standardize the process, however certain steps and decisions remain subjective and up to the engineering judgement of the user and not entirely clear. Examples of such decisions are:

- The choice of consequence class and design life
- The determination of the return periods, as two different tables are provided
- The choice of the Hydrodynamic Estimate Approach
- The use of partial factors, and their application in other wave parameters other than the design wave height (peak period, wavelength etc)

Lastly, making a comparison between the optimized vertical wall breakwater cross-section of each method led to similar results while it has both “advantages” like the combination of actions and “disadvantages” like the lack of clarity regarding the choice partial factors, that may lead to unrealistic results. If partial factors were used the new method would lead to more conservative results as the design wave heights would be 35-50% higher. In other words, it is challenging to get to a final conclusion before further updating the existing Eurocode 1 proposal (prEN1991-1-8). All the aforementioned comments are based on the results of the investigated case study only. The wave climate and the water level fluctuations are typical for a port in the Mediterranean but very different wave and water level loads can be expected in other geographical areas (in Japan or in the North Sea for example). The importance of storm surge and the tide is limited in the case of this case study which may affect the results.

Reflecting on the work conducted during this research the author is still sceptical about the use of regulatory texts and documents. The document used (prEN1991-1-8) proved to be more complicated than expected. At the same time, it was made once again clear, that certain civil engineering applications continue to rely on engineering judgement and choices despite the significant standardization of certain processes that has happened in the past years.

7 Conclusions and Recommendations

In this final chapter, the main research question and sub-questions are answered. At the same time, the lessons learned during this process provide recommendations for further study.

7.1 Conclusions

This section aims at summarizing the conclusions of this study, by answering the sub-questions and the main research question.

7.1.1 Sub-Questions

The first sub-question that the author tried to answer is the following:

"What are the data required to design the new breakwater and how can they be acquired?"

In order to design the new breakwater of the Port of Genova wave data and water level data was the first requirement. Due to the importance of the project those data were available online for the open discussion on the project. The offshore – onshore wave transformation was also done so the onshore significant wave heights were directly used. Water depth and bottom slope were acquired using Navionics while some data related to the geometry of the initial design of the breakwater were given in the open source project files. Reasonable assumptions were done for any missing data.

A second important step in every breakwater design case is the definition of the relevant failure mechanisms, which is a topic discussed via the second sub-question which is as follows:

"Which are the failure mechanisms to be taken into account?"

The breakwater of the Port of Genova is a vertical wall breakwater. Based on existing knowledge of the author and the relevant literature the following failure mechanisms were chosen:

- Sliding
- Overturning
- Soil bearing capacity
- Block toe stability
- Berm Rubble mound stability
- Overtopping
- Transmission

Based on the two above answers, the third sub-question can be answered:

"Which are the safety factors of the initial design for each of the two investigated design approaches?"

This question was the main focus of this study. Based on the initial design that was available and using the two methods (the PIANC design method and the method proposed on the new Eurocode 1), to assess the structure. Table 4-11 summarizes the safety factor and relevant parameters per failure mechanism for both methods. The result is that this design is sufficient for both methods.

The last sub-question that was addressed in this study is related to the optimization of the cross-section of the new breakwater of the Port of Genoa:

“How does a cross-section of the breakwater in each of the two cases look like if optimized and what are the differences between the PIANC design method and the new Eurocode 1?”

To answer this question a vertical wall breakwater was re-designed aiming at the minimum required width and freeboard. The optimized design -given its smaller dimensions- is a more economical option. For the PIANC design method, a cross-section with a width of 20 m and a freeboard of 5.2 m would be sufficient. Using prEN1991-1-8 the optimized cross-section has a width of 19 m and a freeboard of 4.5 m.

7.1.2 Main research question

In this part, the answer to the main research question of this study will be given. This is done on the basis of and by combining the answers to the sub-questions explained in part 7.1.2.

“What differences between the PIANC design method and the method proposed by the new EUROCODE in the design of a vertical wall breakwater can be identified, using the new breakwater at the Port of Genoa as a case study?“

To answer this question the findings of the sub-questions are taken into account along with other insights gained during the study. In terms of a sufficient cross-section, the two methods as applied using the new breakwater at the Port of Genoas as a case study, appear to be very similar. The two optimized cross-sections have similar width and freeboard.

The new Eurocode introduces a combination of actions (wave actions and water levels) with a dominant and an accompanying component, and for each component a corresponding return period. At the same time, the concept of the Hydrodynamic Estimate Approach (HEA) and various design approaches are introduced. All the above lead to an answer to the main research question: the two methods led to comparable results even if the approach to defining return periods is quite different. The above conclusion is based on the application of both methods in the new breakwater of the Port of Genoa, in the Mediterranean where storm surge is limited for example.

7.2 Recommendations

This research focused on the new vertical breakwater to be constructed at the Port of Genoa, and the differences between the PIANC design method and a new method proposed by the new Eurocode. It is evident that not all questions that arise can be answered as part of an additional thesis and thus certain topics that are outside of the scope of this study but should be further studied according to the author are proposed below:

- This report touches upon the new Eurocode 1 using only one case study. Further research, that applies the new method to more cases is necessary to get a more convincing picture of it.
- There are not many breakwaters of this type in Europe. Thus, more research during the design and construction of this project would be definitely beneficial as it could provide valuable data to researchers and engineers.

- In this study, it was decided to disregard the secondary wave direction as both the significant wave height and wave direction are more favourable. Nonetheless, the possibility of waves coming from both directions at the same time is not investigated. A more detailed, probabilistic analysis could provide a better understanding of the situation.
- The new Eurocode 1 does not appear to be clear and consistent. Some inconsistencies found were discussed in Chapter 2. The author is not an expert on this kind of texts, but judging from the point of view of a young engineer, the report does not have enough clarity when making important engineering decisions (e.g. consequence class, HEA, partial factors). Further explanations on how to decide on those matters would substantially improve the report.
- Inconsistencies appeared in the choice of partial factors for the wave actions as well. The relevant chapter of the prEN1991-1-8 should be improved and made more explicit.
- Based on the optimization of the specific breakwater case, a substantial decrease in the dimensions of the cross sections can be achieved. This is true for both the PIANC design method and the new Eurocode 1. It is recommended that the conducted analysis and probabilities of failure used, should be verified by the design team of the new breakwater of the port of Genoa and/or design the breakwater with a detailed model in order to include parameters that may have been neglected in this report.

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Appendix A – Wave data

In this Appendix, the wave data acquired are presented. It should be noted, that for the wave data the offshore to onshore transformation was not conducted by the author of this study, but was part of the preliminary design that was proposed during the public debate of the project.

The wave data follow a Weibull distribution (with α = 1.00) as can be seen from the data provided by Porto di Genova. Using the shape and scale factors and the equations provided in van den Bos & Verhagen, (2018) the wave heights for every return period can be calculated.

SETTORE A 180 - 240 °N

Totale eventi: 697 **Altezza di soglia: 3.75 m** **Numero di eventi sopra la soglia N: 169**

Elenco delle 5 mareggiate più intense

Data	H_s [m]	T_p [s]	Dir [°N]	TR
21/03/2008	7.1	11.0	224	75.7
29/10/2018	6.9	10.5	217	26.2
28/12/1999	6.5	10.8	226	15.9
16/12/2011	6.4	10.3	221	11.4
20/11/2015	6.0	10.0	227	8.9

T_R	GUMBEL		WEI(.75)		WEI(1.0)		WEI(1.4)		WEI(2.0)	
(anni)	H_s (m)	c.i.(m)	H_s (m)	c.i.(m)	H_s (m)	c.i.(m)	H_s (m)	c.i.(m)	H_s (m)	c.i.(m)
2	5.3	0.3	5.2	0.2	5.3	0.3	5.4	0.3	5.4	0.3
5	5.9	0.4	5.8	0.4	5.9	0.5	6.0	0.4	6.0	0.4
10	6.4	0.6	6.3	0.5	6.4	0.6	6.4	0.6	6.4	0.5
15	6.6	0.6	6.6	0.6	6.7	0.7	6.7	0.6	6.7	0.5
20	6.8	0.7	6.8	0.7	6.9	0.8	6.9	0.7	6.8	0.6
25	7.0	0.7	7.0	0.8	7.1	0.8	7.0	0.8	6.9	0.6
50	7.4	0.9	7.5	1.0	7.5	1.0	7.5	0.9	7.3	0.7
100	7.9	1.1	8.0	1.2	8.0	1.3	7.9	1.1	7.7	0.9
150	8.1	1.2	8.3	1.3	8.3	1.4	8.1	1.2	7.9	1.0
250	8.5	1.3	8.7	1.5	8.7	1.6	8.5	1.3	8.2	1.1
500	8.9	1.5	9.2	1.7	9.2	1.8	8.9	1.5	8.5	1.2
A	0.67		0.34		0.69		1.33		2.39	
B	2.94		3.35		2.84		2.04		0.85	
r	0.993		0.977		0.991		0.997		0.997	
MIR	NO		NO		SI		NO		NO	

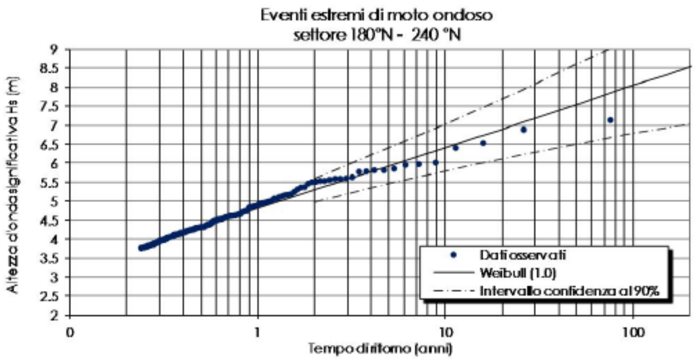


FIGURA 8-6 - GENOVA LARGO – SETTORE A: 180 - 240°N. VALORI DELL'ALTEZZA D'ONDA SIGNIFICATIVA IN FUNZIONE DEL TEMPO DI RITORNO SECONDO LE DIVERSE DISTRIBUZIONI STATISTICHE - METODO P.O.T. (PERIODO:1 GENNAIO 1979 - 31 DICEMBRE 2018)

Figure A.1: Peak over threshold method and wave data distribution

Primary Wave Direction						
Return Period [yrs]	Offshore			Onshore at point P1 (-50 m)		
	Hs [m]	Tp [s]	Direction [°N]	Hs [m]	Tp [s]	Direction [°N]
2	5.3	9.6	180	5.2	9.8	179
	5.3	9.6	210	4.8	9.8	203
	5.3	9.6	240	3.3	9.8	216
5	5.9	10.0	180	5.8	10.6	179
	5.9	10.0	210	5.4	10.6	203
	5.9	10.0	240	3.7	9.8	216
10	6.4	10.3	180	6.3	10.6	179
	6.4	10.3	210	5.9	10.6	203
	6.4	10.3	240	4.0	9.8	216
50	7.5	11.0	180	7.3	11.4	179
	7.5	11.0	210	6.8	11.4	203
	7.5	11.0	240	4.7	11.4	217
100	8.0	11.3	180	7.8	11.4	179
	8.0	11.3	210	7.3	12.3	203
	8.0	11.3	240	5.1	12.3	217
250	8.7	11.6	180	8.5	12.3	179
	8.7	11.6	210	7.9	12.3	203
	8.7	11.6	240	5.5	13.2	217
500	9.2	11.9	180	8.95	12.3	179
	9.2	11.9	210	8.4	12.3	203
	9.2	11.9	240	5.9	13.2	217

Table A.1: Significant wave height, peak period, and wave direction for the primary wave direction

Secondary wave direction						
Return Period [yrs]	Offshore			Onshore at point P1 (-50 m)		
	Hs [m]	Tp [s]	Direction [°N]	Hs [m]	Tp [s]	Direction [°N]
2	3.6	7.9	120	3.0	9.1	132
	3.6	7.9	150	3.5	8.5	153
	3.6	7.9	180	3.5	8.5	179
5	4.1	8.4	120	3.5	9.1	132
	4.1	8.4	150	4.0	9.1	152
	4.1	8.4	180	4.0	8.5	179
10	4.6	8.8	120	4.0	9.1	131
	4.6	8.8	150	4.5	9.1	152
	4.6	8.8	180	4.5	9.1	179
50	5.7	9.5	120	5.0	9.1	131
	5.7	9.5	150	5.6	9.8	152
	5.7	9.5	180	5.6	9.8	179
100	6.1	9.8	120	5.4	9.1	131
	6.1	9.8	150	5.9	10.6	152
	6.1	9.8	180	6.0	10.6	179
250	6.7	10.2	120	5.9	10.6	131
	6.7	10.2	150	6.5	10.6	152
	6.7	10.2	180	6.6	10.6	179
500	7.2	10.5	120	6.40	11.4	131
	7.2	10.5	150	7.0	10.6	152
	7.2	10.5	180	7.1	10.6	179

Table A.2: Significant wave height, peak period, and wave direction for the secondary wave directions

Peak period

A relation between H_s and T_p is also given and is the following:

$$T_p = a * H_s^b, with\ a = 5\ and\ b = 0.39$$

This equation will be used to calculate the peak period for every return period required, based on the significant wave height calculated from the Weibull distribution, and stands for the **dominant/primary wave direction.**

For the **secondary wave direction** the equation is as follows:

$$T_p = a * H_s^b, with\ a = 4.84\ and\ b = 0.39$$

Appendix B – Strom Surge and Wave Setup

Retrun period [yrs]	Storm Surge and wave setup [m MSL]
1	0.37
2	0.43
4	0.51
10	0.57
25	0.66
50	0.72
100	0.78
250	0.86

Table B.1: Wave setup and storm surge

Using the values of this table as a basis, a relevant equation was defined, to be able to acquire other values as well. This equation is the following: $y = 0.3927 * x^{0.1506}$ also shown in the figure below. Based on this equation values for various return periods can be acquired.

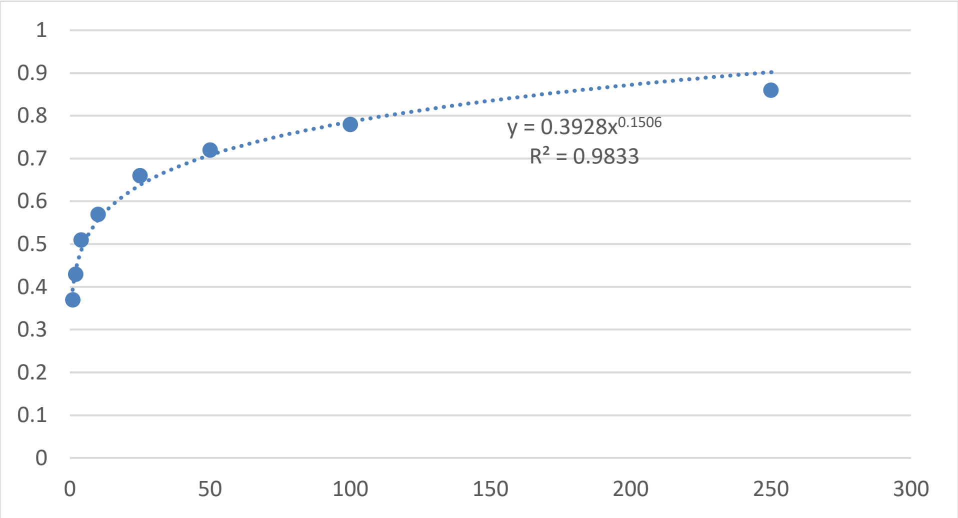


Figure B.1: Storm surge and wave setup approximation

Return Period	Storm surge and wave setup (m)
20	0.63
40	0.70
70	0.74
475	0.99

Table B.2: Storm surge and wave setup for the relevant return periods

Appendix C – Design Calculation example

This appendix includes the design equations used in a Python script that the author of this research developed. The design equations are the same for the two methods investigated in this research. All equations used are based on Takahashi, 2002 unless stated otherwise. The example in this appendix focuses in the assessment of the initial design of the new breakwater of the Port of Genoa using the return periods and input data as defined from prEN1991-1-8. The input data of this example (geometric parameters, wave and water level data) are summarized below.

Parameter	Symbol	Value		Unit
Return period (ULS)	RP	Dominant component	400	years
	RP	Accompanying component	40	years
Return period (SLS)	RP	Dominant component	40	years
	RP	Accompanying component	20	years
Significant wave height (ULS)	H_s	8.77		m
Peak period (ULS)	T_p	11.66		sec
Storm surge and wave setup (ULS)	s_{surge}	0.70		m
Significant wave height (SLS)	H_s	7.05		m
Peak period (SLS)	T_p	10.71		sec
Storm surge and wave setup (SLS)	s_{surge}	0.63		m
Tidal range	HAT	0.15		m
Sea Level Rise	SLR	0.20		m
Water depth	h	35		m
Vertical wall depth	d	25		m
Cross section width	B	26		m
Freeboard	R_c	7		m
Amount of chambers	-	6		-
Chamber dimensions	-	4 x 4		m^2
External concrete wall thickness	t_{ext}	0.5		m
Internal concrete wall thickness	t_{int}	0.2		m
Bottom slope	$\tan\theta$	1/80		-
Wave angle	β	24		degrees

Table C.1 – Input data

The various constants used, and their respective values and units are summarized in the following Table.

Parameter	Symbol	Value	Unit
Specific weight of concrete	$\gamma_{concrete}$	24	KN/m^3
Specific weight of sand	γ_{sand}	20	KN/m^3
Specific weight of water	γ_{water}	10.25	KN/m^3
Water density	ρ_{water}	1.025	t/m^3
Gravitational acceleration	g	9.806	m/s^2
Friction coefficient	μ	0.60	-

Table C.2 – Constants

Design water level

The design water level will be calculated based on the following equation, and the input values provided in Table D.1.:

$$DWL = Storm\ Surge + HAT + SLR = \ +0.98\ m\ MSL$$
$$h_c = R_c - DWL = \ +6.02\ m\ MSL$$

Design wave height

The onshore significant wave height at the toe of the structure is provided in the open source data related to the design of the new breakwater at the port of Genoa. The design wave height is the maximum wave height according to the equation:

$$H_{max} = 1.8\ * H_{s,onshore}$$

The above equation leads to an $H_{max,ULS}$ = 15.79 m and $H_{max,\ SLS}$ = 12.69 m

Breakwater weight calculation

The weight of the breakwater for the dry and in situ conditions is based on the dimensions of the cross-section and the specific weight of each part, as shown in the below calculations:

$$W_a = \ \gamma_{sand} * (B - 2 * t_{ext} - 5 * t_{int}) * d + \gamma_{concrete} * \big((d * (2 * 2 * t_{ext} + 5 * t_{int})\big) + 2.5 * B + (h_c - 1.5) * 4)$$

$$W = \ W_a - (\gamma_{water} * h_c * B)\ /\ g$$

The in situ weight to be used later in the stability calculations is equal to 897.3 tf/m.

Dispersion equation and other equations

Using the dispersion equation the wave number is determined: k = 0.0348, and the wavelength L = 180.34 m. The deep water wavelength is equal to 212.18 m.

Goda method parameters

According to Goda, the intensities of wave and uplift pressure can be calculated using the following equations:

$$p_1 = \frac{1}{2} * (1 + cos\beta)(\alpha_1 + \alpha_2 * cos^2\beta) * \rho * H_{max}$$

$$p_2 = \frac{p_1}{coshkh}$$

$$p_3 = a_3 * p_1$$

$$p_4 = p_1 * \left(1 - \frac{h_c}{\eta^*}\right)\ \ if\ \eta^* > h_c \quad or \quad p_4 = \ 0 \ \ if\ \eta^* \le h_c$$

In the above equation η^* is the theoretical maximum level at which pressure is exerted, and is calculated as below:

$$\eta^* = 0.75 * (1 + cos\beta) * H_{max}$$

The uplift pressure follows the equation:

$$p_u = \frac{1}{2} * (1 + cos\beta) * \alpha_1 * a_3 * \rho * H_{max}$$

The model coefficients of the Goda method as calculated as below:

$$a_1 = 0.6 + \frac{1}{2} * (\frac{2kh}{sinh2kh})^2$$
$$a_2 = \min\left\{\frac{h_b - d}{3 * h_b}\left(\frac{H_{max}}{d}\right)^2, \frac{2d}{H_{max}}\right\}$$
$$a_3 = 1 + \frac{h'}{h} * (1 - \frac{1}{coshkh})$$

Using the pressures p_1, p_2, p_3 and p_4 along with the uplift pressure p_u, the stability and overturning calculations can be executed.

Stability

The horizontal force is calculated as follows:

$$F_H = \frac{1}{2}(p_1 + p_3) * h' + \frac{1}{2} * (p_1 + p_4) * h_c^*$$

In addition, the uplift force is determined using the following equation:

$$F_U = \frac{1}{2} * p_u * B$$

with B being the caisson width and $h_c^*=\min(\eta^*,h_c)$. The horizontal and uplift forces are determined equal to F_H = 215.8 tf/m and F_U = 94.4 tf/m.

The safety factor against stability can now be calculated as below:

$$SF_{Sliding} = \frac{\mu * (W - F_U)}{F_H} = \mathbf{2.23} > 1.2$$

Therefore, the cross-section is sufficient against sliding.

Overturning

The two moments at the toe of the structure are calculated as follows:

$$M_H = \frac{1}{6}(2p_1 + p_3) * h^2 + \frac{1}{2}(p_1 + p_4) * h' * h_c^* + \frac{1}{6}(p_1 + 2p_4) * h_c^{*2}$$
$$M_U = \frac{2}{3} * F_U * B$$

The two moments at the toe of the structure are: M_H = 3631 tf*m/m and M_U = 1637 tf*m/m.

The horizontal distance between the centre of gravity of the caisson and its heel is assumed to be t = B/2.

Based on those the safety factor against overturning can now be calculated:

$$SF_{Overturning} = \frac{(W * t - M_U)}{M_H} = \mathbf{2.76} > 1.2$$

Therefore, the cross-section is sufficient against overturning.

Soil bearing capacity

According to Goda, the foundation should be at most 400 kPa. The load is dependent on the nett vertical force that is exerted on the soil (W_e), the width of the caisson (B) and the eccentricity of the reaction force (t_e). The load is calculated based on the following equation:

$$pe = \frac{2 * W_e}{3 * t_e} \; if \; t_e \leq \frac{B}{3}$$

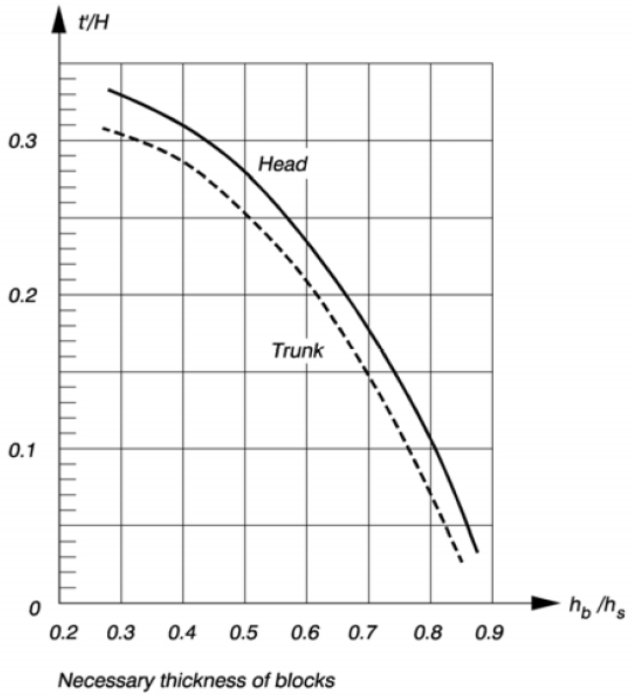
$$pe = \frac{2 * W_e}{B * (2 - 3 * (t_e/B))} \; if \; t_e > \frac{B}{3}$$

The net vertical force is calculated as below:

$M_e = M_{gt} - M_H - M_U$, while the eccentricity t_e is equal to M_e/W_e The load is determined equal to **371 kPa** and given that the limit for the foundation load is 400 kPa, the bearing capacity of the soil is sufficient.

Block toe dimensions

For the dimensions of the concrete block toe to be placed in front of the vertical wall breakwater, the width, length and thickness of the block need to be calculated. The above is based on Takahashi, (2002) and the following graph:



l	x	b	x	t'	app. mass t
2.5		1.5		0.8	6
3.0		2.5		1.0	15
4.0		2.5		1.2	25
5.0		2.5		1.4	37
5.0		2.5		1.6	42
5.0		2.5		1.8	48
5.0		2.5		2.0	53
5.0		2.5		2.2	58

Specification of outer block geometries in meters. Hole volume should be app. 10% of total volume.

Assuming that $h_b/h_s = 0.72$ and H = 8.77 m and using the "trunk" option in the above graph, t' is determined to be equal to 1.14 m. As a result, the following dimensions are chosen based on the table on the right: **l x b x t' = 4.0 x 2.5 x 1.2**

Minimum rock mass and rock dimension for berm stability

In order to determine the required rock mass for a stable berm the following equation is used:

$$M_{50,berm} = \frac{\rho_c}{{N_s}^3 * (Sr-1)^3} * {H_s}^3$$

In the above equation Sr is as follows:

$$\mathrm{Sr} = \rho_c/\rho_{\mathrm{water}}$$

while N_s is calculated as below:

$$N_s = max\left\{1.8, \left(1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1.8 * exp\left[-1.5\frac{(1-\kappa)^2}{\kappa^{\frac{1}{3}}}\frac{h'}{H_{1/3}}\right]\right)\right\}$$

and κ is calculated based on the following:

$$\kappa = \frac{2 * k' * h'}{\sinh(2 * k' * h')} sin^2(k' * B_M)$$

In the last equation, it is assumed that k' = 2π/L' where L' is the wavelength at the water depth h'. Using the dispersion equation, k' = 0.0398 and the mass of the blocks at the berm should be at least 1421 kg. In other words **$M_{50,berm}$ ≥ 1421 kg**.

For the calculation of the nominal diameter of the blocks to be placed at the berm the following equation will be used:

$$D_{n50,berm} = \frac{H_s}{\Delta * N_s}$$

In the above equation the relative density Δ is as follows:

$$\Delta = (\rho_c - \rho_{water})/\rho_{water}$$

While Ns is calculated in the same way as before. The minimum nominal diameter of the blocks is calculated equal to 0.83 m. In other words **$D_{n50,berm}$ ≥ 0.83 m**

Overtopping

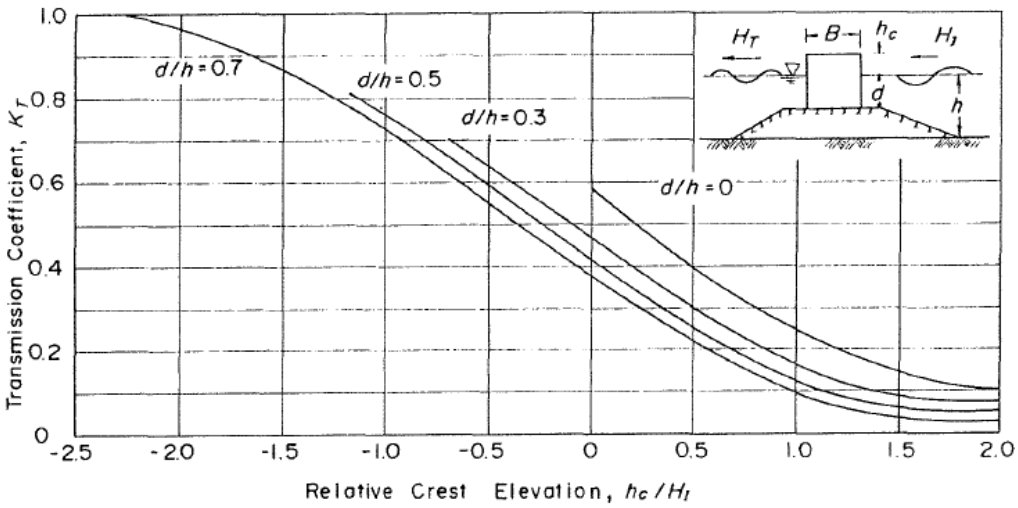
The acceptable overtopping discharge is assumed to be: $q_{acceptable}$ = 10 l/m/s. In order to calculate the wave impact the following equation will be used:

$$\frac{q}{\sqrt{g * H_{m0}^3}} = a * \exp\left(-b\frac{R_C}{H_{m0}}\right) for\ 0.1 < \frac{R_C}{H_{m0}} < 3.5$$

Where α = 0.04 and b = 1.8. Using the above equation the overtopping discharge is determined equal to **q = 0.504 l/m/s < $q_{acceptable}$**

Transmission

The transmission coefficient is calculated using the following graph from Takahashi, (2002).



For this example, d = 25.98 m, h = 35.98 m, as a result d/h = 0.72

Also taking into account that h_c = 6.02 m and H_i = 7.05 m, h_c/H_i = 0.84

Using the above graph, the transmission coefficient $K_T = 0.11$ and **thus the transmitted wave height $H_T = 0.78$ m < 1.5 m.**

Appendix E – Jupyter notebook

For the purpose of this study, a Jupyter notebook was developed by the author to perform all relevant calculations easily and efficiently. The QR code below leads to a Git Hub page where the relevant notebook is uploaded.