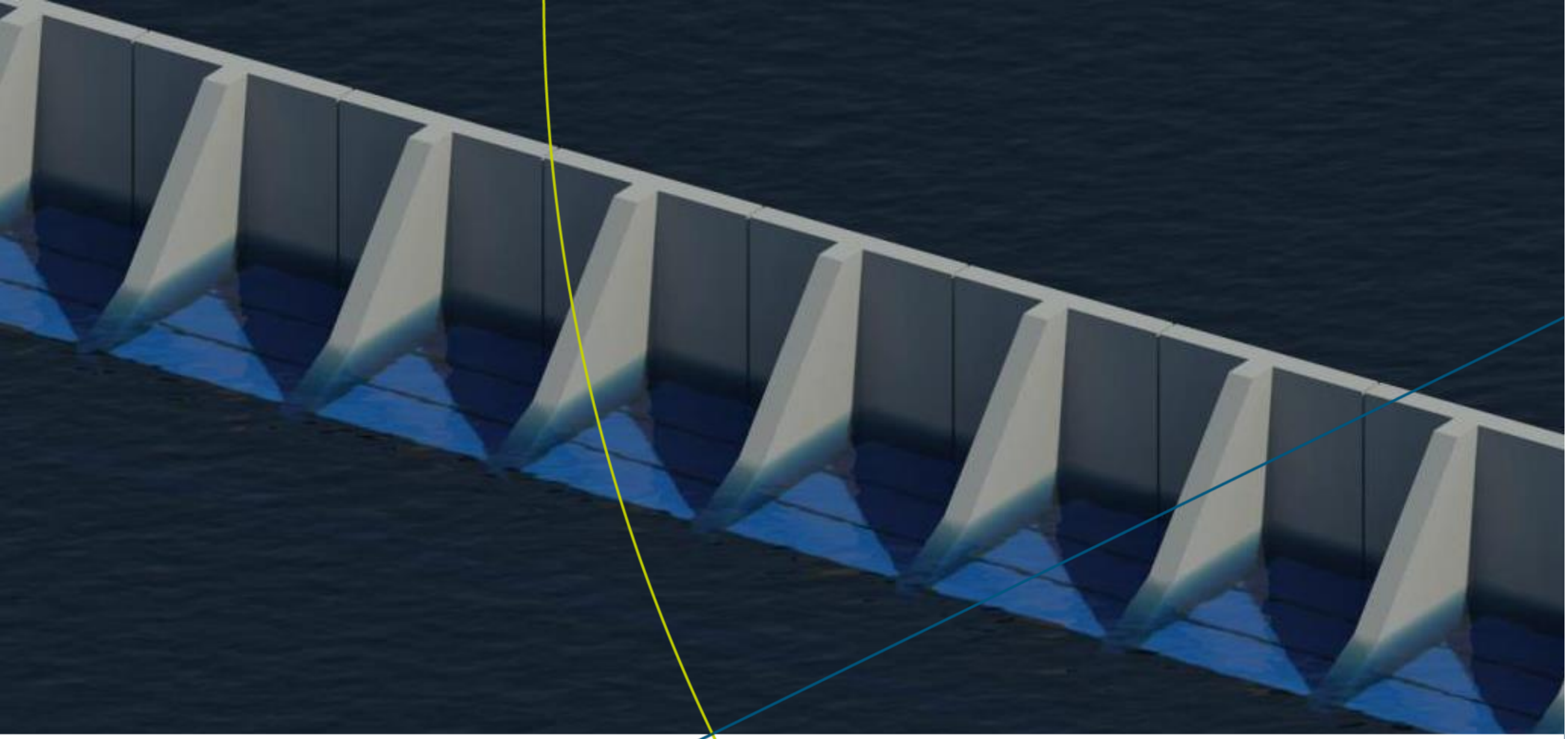

FINAL REPORT

Innovative Breakwater

Low-cost, non-traditional breakwater design

Prepared by:	Jesper van Grieken - 0853476 Danny Janssen - 0853162
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Education:	Civil Engineering (Bsc.)
Revision:	01/Final
Date:	16 June 2015

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Classification
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Foreword

The last phase of the Bachelor study civil engineering at the Rotterdam University of Applied Sciences is a graduation project. For the graduating students, the main purpose of this project is to proof that they have the competences the University expects and that the students are ready for the business community.

The subject of this graduation report is provided by Royal HaskoningDHV. Royal HaskoningDHV is a world leading international service provider in the field of Consultancy and Engineering. They are specialized in Aviation, Buildings, Industry, Energy & Mining, Infrastructure, Maritime & Waterways, Planning & Strategy, Rivers, Delta's & Coasts, Transport & Asset Management and Water Technology. With its headquarters in Amersfoort, The Netherlands, RHDHV is ranked globally in the top 10 of independently owned, non-listed companies and top 40 overall. Their 7,000 staff provides services across the world from more than 100 offices in over 35 countries. With an overarching aim to ***"Enhance Society Together"****, they work closely with clients, stakeholders, industry and academic leaders, to ensure projects are delivered on time and within budget, while providing a better, brighter and sustainable future.*

Since February this year we have been working on this graduation project. We have experienced this graduation period as interesting and very educational. In the beginning we had little knowledge of the design aspects of breakwaters. However, during the project we have learned a lot about these design aspects, especially during the reference design phase. The alternative analysis phase in particular was a challenging phase due to the lack of relevant design formulas and the need to use assumptions. However, we are very pleased with the outcome of this project.

We would like to thank our supervisor from Royal HaskoningDHV, Ronald Stive, and our supervisor from the Rotterdam University of Applied Sciences, Harry Dommershuijzen, for their guidance and advices during the project. We would also like to thank Edwin Schaap, a teacher of the Rotterdam University, for his comments on some of the documents. Finally we would like to thank all the colleagues within Royal HaskoningDHV who have helped and guided us during our graduation project. In particular: Cock van der Lem, Ruud Roelfsema, Kasay Asmerom, Perry Groenewegen and Juan Pablo Lopez Gumucio for their advices and their reviews of documents during the graduation progress.

Rotterdam, 16-06-2015.
Jesper van Grieken & Danny Janssen

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Executive summary

During the last phase of the Bachelor study civil engineering, at the Rotterdam University of Applied Science, students have to prove that they satisfy the competences which the University desires of graduated engineers. The graduation students Jesper van Grieken and Danny Janssen did use this period investigating low-cost, non-traditional breakwater concepts at a set location. Using the assistance and the knowledge of the graduation company, Royal HaskoningDHV, this investigation has been defined and performed as follows:

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater. Without a breakwater, the current estimated yearly downtime is between 80 and 90 percent.

For this reason, financially feasible alternative breakwater concepts have been investigated. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. In order to determine the most favourable alternative, each short listed breakwater concept has been further analysed in more detail. The most promising ones have been finally evaluated on the basis of a multi criteria analysis.

The most favourable breakwater concept, according to the analysis followed, consists of concrete elements which are stabilized by means of a container backfill. The form of the concrete elements is based on the shape of a L-wall, which is strengthened at the backside by a vertical support perpendicular to the front wall. The containers in the backfill have to be completely filled with sand, in order to create enough mass for the container backfill. A schematic overview of the most favourable alternative can be found in figure 1.

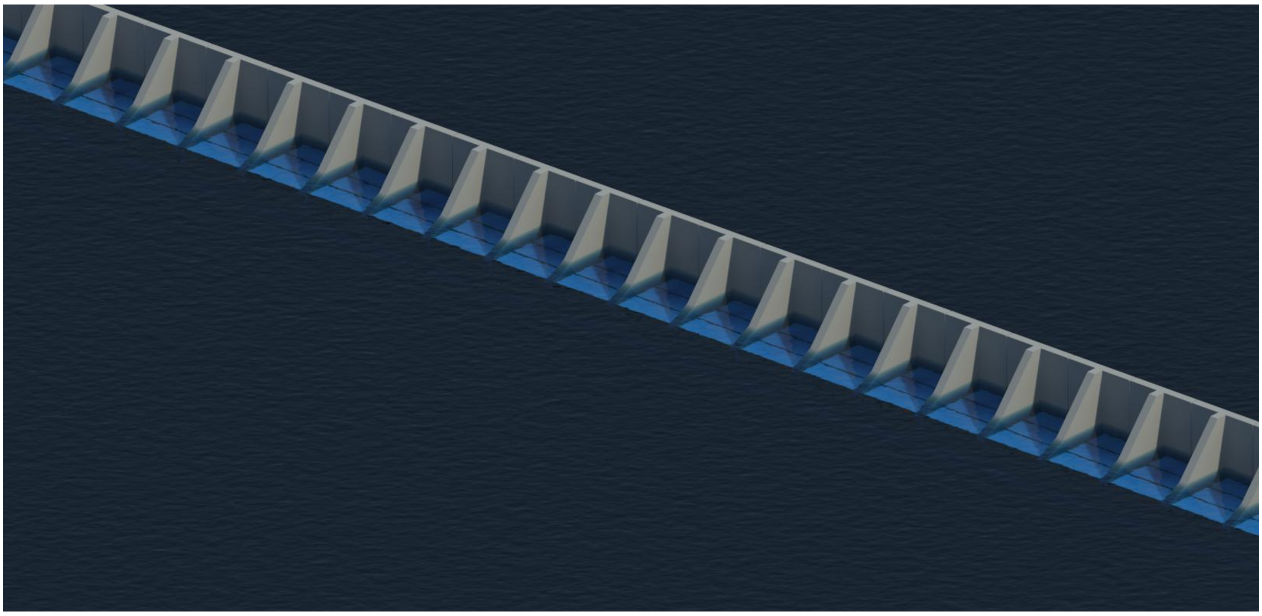

Figure 1: Overview breakwater

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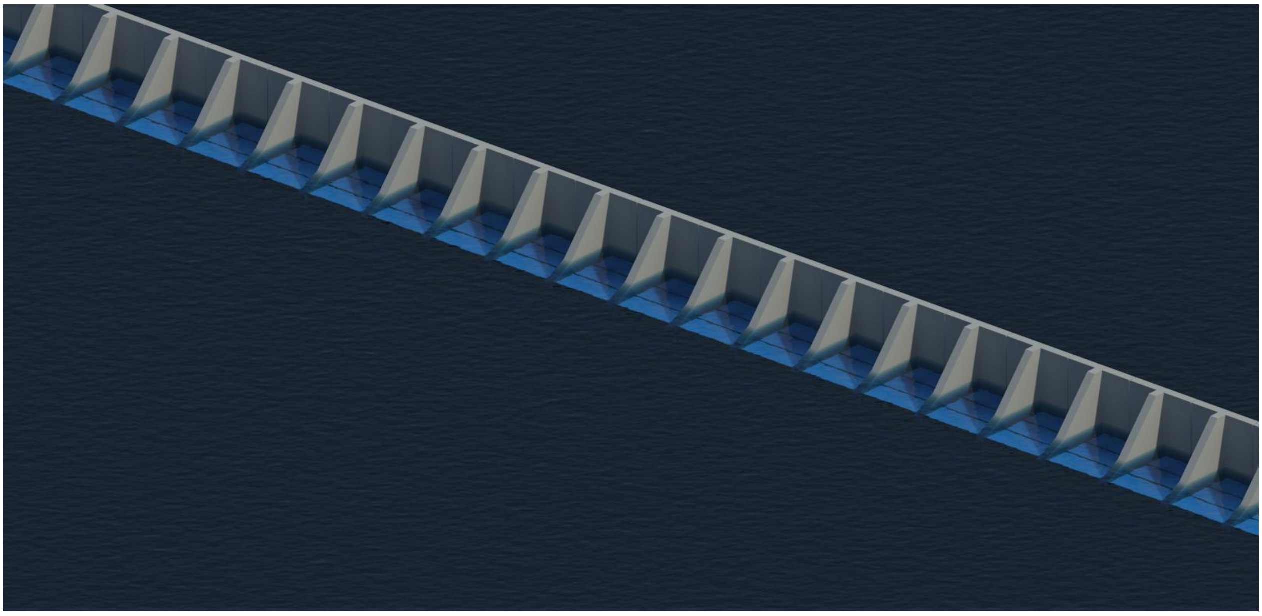
Beknopte samenvatting

In de laatste fase van de bachelor opleiding civiele techniek, te hogeschool Rotterdam, dient er door de studenten aangetoond te worden dat zij voldoen aan competenties die de hogeschool aan afgestudeerde ingenieurs stelt. De studenten Jesper van Grieken en Danny Janssen hebben deze periode ingevuld door een onderzoek te doen naar financieel aantrekkelijke en innovatieve golfbreker concepten op een gestelde locatie. Met de hulp en kennis van het afstudeerbedrijf, Royal HaskoningDHV, is het onderzoek als volgt gedefinieerd en ingevuld:

Voor de west kust van Sumatra is een bulk terminal geconstrueerd voor het laden van zeewaardige duwbakken. Tijdens de ontwerp- en de uitvoeringsfase is onterecht besloten om het bouwen van een golfbreker achterwege te laten. Daarnaast was het project budget te gelimiteerd om een traditionele golfbreker te bouwen. Zonder golfbreker bescherming is de huidige terminal 80 tot 90 procent van de tijd niet te gebruiken.

Vanwege deze rede zijn er financieel aantrekkelijke alternatieve golfbreker varianten ontwikkeld. Deze alternatieve golfbreker varianten dienen de extreme omgevingscondities op de gestelde locatie te kunnen weerstaan. Om te bepalen welk van de golfbreker alternatieven het meest voordelig is, zijn de golfbreker alternatieven in meer detail uitgewerkt. De beste varianten zijn uiteindelijk geëvalueerd op basis van een Multi Criteria Analyse.

Het meest voordelige golfbreker alternatief, volgens de variantenstudie, bestaat uit geprefabriceerde betonnen elementen die zijn verzwaard met (afgeschreven) zeecontainers. De betonnen elementen hebben de vorm van een L-wand, welke aan de binnenzijde is versterkt door een verticale ondersteuning loodrecht op de voormuur. De containers zijn compleet gevuld met zand, om voldoende massa van de container verzwaring te realiseren. In figuur 1 is een schematisch overzicht van het meest voordelige golfbreker alternatief te zien.



Figuur 1: Overzicht golfbreker

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Appendices

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1 Introduction

1.1 Project purpose

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because of underestimating the severe environmental conditions, both during construction and operation, it was wrongly decided to omit a protective solution for the swell waves. Moreover, the project budget was too limited to build a traditional breakwater. This leads to a downtime of 80 to 90 % of the terminal. Due to this downtime, the barge loading terminal is not profitable. For this reason, a solution has to be developed with the purpose to make the project financially feasible. There are different solution directions but RHDHV is especially interested in alternative breakwater concepts to make the project feasible. Summarized, the main purpose of this project is:

Design an alternative, low cost, non-traditional breakwater solution for a specified location at the NW-coast of Sumatra.

1.2 Project location

In 2007, RHDHV investigated the boundary conditions at the project location which is situated at the NW coast of Sumatra facing the Indian Ocean. Due to confidentially reasons, the client does not want the project location to be specified in detail. Therefore, taking the actual RHDHV export terminal project as a starting point, a realistic schematisation of the BW layout/position and the surrounding seabed with actual design boundary conditions has been defined. This realistically schematised situation has been used for all study and design work related to the graduation project. In figure 2, the approximate location of the project is shown.

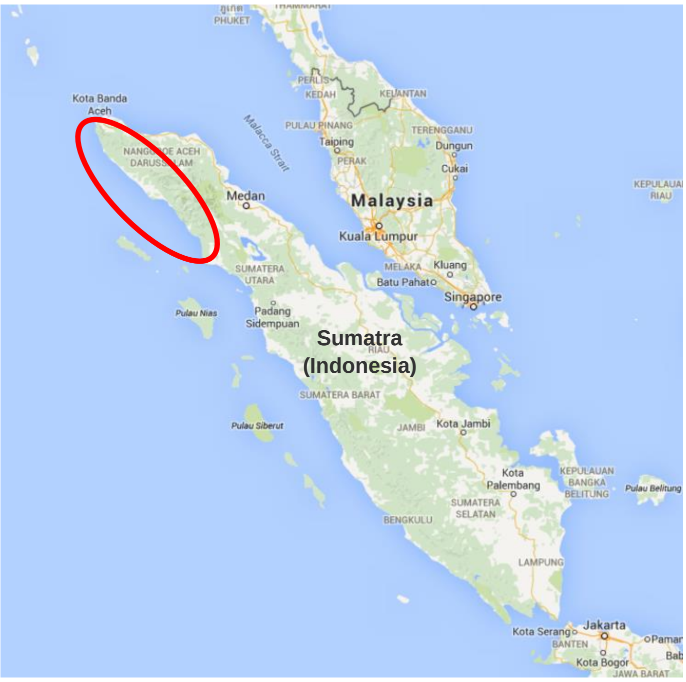


Figure 2: Location of the project (Google)

1.3 Project process

In order to achieve the main purpose of the project, four different phases in the process have been determined: the preparation phase, the reference design phase, the alternative analysis phase and the detailing phase. The results, conclusions and recommendations of each phase serve as input for the next phase. A schematic view of the process can be seen in figure 3.

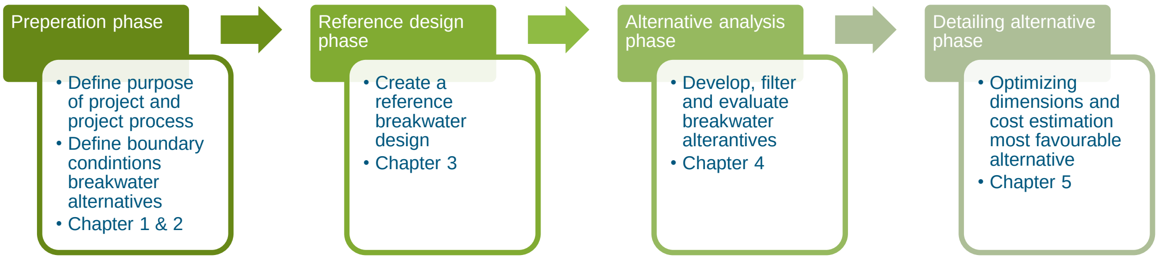


Figure 3: Graduation process

1.3.1 Preparation phase

The preparation phase starts with determining and establishing the project purposes and the required project process for achieving this purpose. This first process step did result in a project plan.
The project starting points, the design requirements and project boundary conditions have been specified and summarized in a basis of design document. In this basis of design, a subdivision between the environmental conditions, the required performances of the breakwater, the used materials and the used codes, standards and guidelines is made. A summary of the results of the preparation phase is found in chapter 1 and 2.

1.3.2 Reference design phase

In order to get a good understanding about the boundary conditions and the problems which can occur during the design of a breakwater, a traditional breakwater design has been made. This rubble mound design is optimized and checked using the most common failure mechanisms. After determining the dimensions of the traditional breakwater, the construction costs of the traditional breakwater have been determined. This reference design is used as a comparison during the next phase of the process. A summary of the results of the reference design phase is found in chapter 3.

1.3.3 Alternative analysis phase

The alternative analysis phase did start with a brainstorm session. During this brainstorm session, alternative breakwater concepts have been developed. The found solutions are summarized in a long list and thereupon reduced to a short list, after which the remaining solutions have been put into a multi criteria analysis (MCA). The output of this MCA is the most favourable breakwater concept. This concept is regarded to be the best alternative breakwater concept for the project location and has been optimized further during the last phase. A summary of the process, the evaluation criteria used during the different steps and the results are found in chapter 4.

1.3.4 Detailing selected alternative

The last phase, in order to achieve the main purpose of the process, is an optimisation of the most favourable alternative breakwater design. The purpose of this phase is to optimise the dimensions of the breakwater as realistic as possible. For the optimized most favourable alternative an as accurate as possible estimate of the costs has been made.

2 Project conditions

2.1 Introduction

In order to determine a complete understanding of the boundary conditions which both the traditional and the alternative breakwater have to adhere to, a basis of design is prepared. In this chapter a summary of the project conditions mentioned in the basis of design can be found. The basis of design itself can be found in appendix 2. The environmental boundary conditions are based on a specific RHDHV export terminal project (DHV, 2011). The breakwater performances/requirements have been determined in consultation with the end client and on the basis of technical guidelines and standards.

2.2 Hydraulic conditions

2.2.1 Bathymetry

The schematized bathymetry of the project location is presented in figure 4. The red lines in the figure show the water depth on that specific location. The displayed water depth is the water depth compared to chart datum (LLWS). The minimum water depth at the breakwater location is 9.5 meters relative to chart datum. The slope of the seabed at the breakwater location is around 1:600.

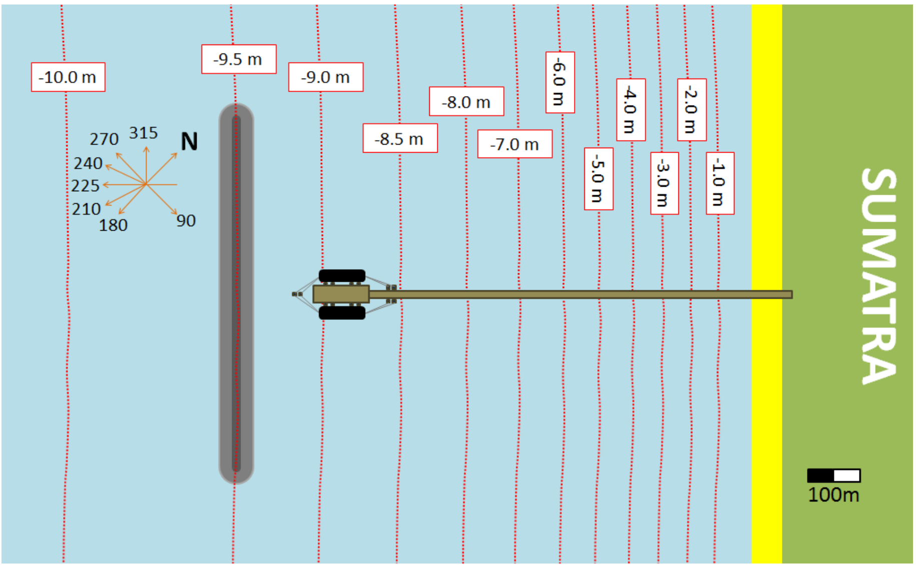


Figure 4: Plan view of bathymetry lines at project location (reference design breakwater)

2.2.2 Water levels

The chart datum is set to LLWS, the water levels of the different tides can be found in table 1.

LLWS	MLWS	MLWN	MHWN	MHWS	HHWS
0.00 m	+0.05 m	+0.20 m	+0.40 m	+0.55 m	+0.75 m

Table 1: Tide levels at project location relative to chart datum

For calculations under extreme environmental conditions a 0.5 meter water level rise caused by wind surge is added to the HHWS. For calculations under both extreme environmental conditions and operational conditions a 0.05 m addition caused by sea level rise is added to HHWS.

2.2.3 Waves

The wave conditions at the project location have been separated in two different types of wave conditions, the normal wave conditions and the extreme storm wave conditions. The normal wave conditions are taken into account when considering the operational conditions at the jetty. The extreme wave conditions are taken into account in order to determine which maximum wave forces the breakwater has to withstand. In table 2 the boundary wave conditions for operational and extreme wave conditions are presented. A more elaborate wave description can be found in appendix 2, the basis of design.

Operational wave conditions			Extreme wave conditions		
H_s (m)	T_p (s)	Dir. as seen from north (°)	H_s (m)	T_p (s)	Dir. as seen from north (°)
1.5	8-18	195-255	3.3	18.6	225

Table 2: Operational and extreme wave conditions

2.2.4 Currents

The limiting operational current velocity is 0.7 m/s in transversal direction and 1.5 m/s in longitude direction for mooring and navigating of the barges (Spanish guideline ROM 3.1-99, 2007). The maximum spring tide current velocity is 0.1m/s, so far below the limiting condition currents and thus this will not lead to additional downtime of the terminal. For this reason the tidal currents will not be taken further into account.

2.2.5 Tsunami

The breakwater designs have not been checked for tsunami wave loading (because the time available for the graduation project did not allow this). However, the normative tsunami wave characteristics at the project location, based on (McKee, 2005), are shown in table 3.

Characteristic	Symbol	Value	Unit
Wave velocity	c	500 – 1000	km/h
Wave height on deep water	H_0	0.5	m
Wave length	L_0	200,000	m
Wave reach (depth)		4000	m
Wave period	T_p	12	min
		720	sec
Wave height on the Sumatra shore	H_s	15	m

Table 3: Tsunami wave characteristics

2.3 Geotechnical conditions

2.3.1 Subsoil

The subsoil at the project location consists of loose, medium to fine grained sand. In the basis of design, a relevant schematisation of the subsoil can be found.

2.3.2 Seismological conditions

The project location is categorized as zone 5-6, with horizontal peak ground accelerations of 0.25-0.30 g. This earthquake has a return period of 500 years. When calculating the earthquake resistance of a breakwater, besides the horizontal earthquake load, a vertical earthquake load has to be applied. According to (Asmerom, 2015), this vertical load is 50% of the horizontal earthquake load and has to be applied in both directions. This means that the value for the vertical earthquake load (0.15g) has to be applied both negative and positive.

2.4 Various

2.4.1 Wind

The limiting operational wind speed for the barges is 22 m/s (Spanish guideline ROM 3.1-99, 2007). The yearly maximum wind speed is 14 m/s. The maximum yearly occurring wind speed is lower than the limiting wind speed, so the local maximum wind will not lead to additional downtime of the terminal. Further it can be assumed that the local maximum wind will not lead to any damage to the breakwater.

2.4.2 Morphology

Due to wave driven long- and cross shore transport, the beach underneath the connection of the jetty with the shore is eroding in the current situation. When a breakwater is constructed, this erosion may likely change from erosion to sedimentation because of the wave sheltering effect of the breakwater. This results in a more stable jetty construction.

2.4.3 Ecological

In this graduation project there are no requirement determined regarding the ecological circumstances. If the breakwater concept will be installed on the project location, the local laws and regulations have to be complied with. The ecological boundary conditions fall outside the scope of this graduation project.

2.5 Breakwater performances

2.5.1 Introduction

For each of the breakwater concepts two wave boundary conditions have been defined: the operational wave conditions and the extreme wave conditions. The operational wave conditions are determined in order to achieve a safe environment for the operations at the jetty and to minimize the downtime of the terminal. The extreme wave conditions describe the wave loads which the breakwater should be able to resist. Next to the wave boundary conditions, additional requirements have been determined for earthquake loading, lifetime and maintenance.

2.5.2 Operational wave conditions

The operational wave conditions are the governing conditions during mooring and navigation of the barges. For the operational wave conditions, three different maximum wave heights have been specified. A further description on the determination of the operational wave limits can be found in appendix 2, the basis of design. In figure 5, a schematic drawing of the operational wave conditions is shown.

- If the waves on the ocean are higher than 1.5 meters, tugboats will stop operating (DHV, 2011)
- The maximum operational wave height in the turning circle is set to 1.0 meters. (PIANC, 2014)
- The maximum operational wave height at the mooring location is set to 0.5 meters (RHDHV, 2014).

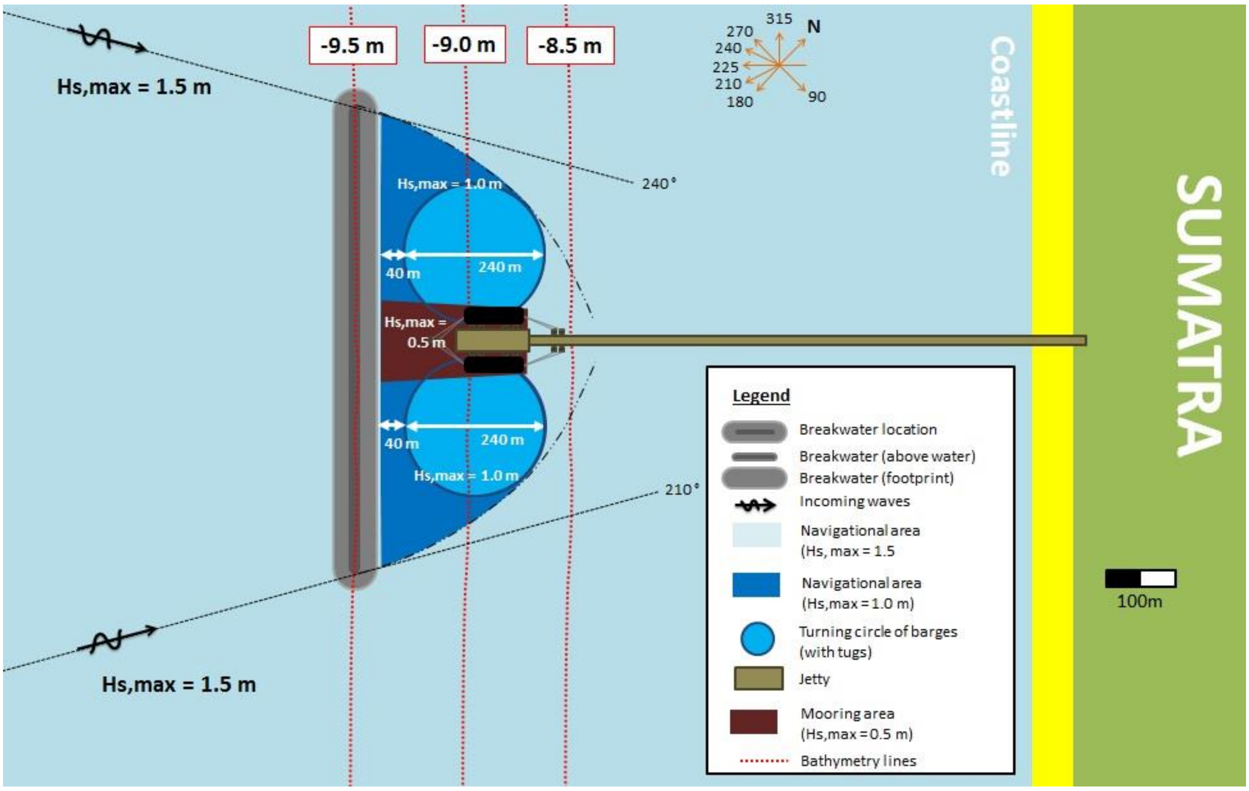


Figure 5: Operational wave conditions

2.5.3 Extreme wave conditions

Each of the developed breakwater concepts should be able to withstand the extreme wave conditions, in order to create realistic and comparable breakwater dimensions and herewith breakwater costs. Damage caused by the extreme wave conditions is acceptable, as long as the breakwater still can satisfy its operational performance. This is a requirement set by the client.

2.5.4 Earthquake loading

All of the developed breakwater concepts should be able to withstand the extreme environmental conditions, like earthquakes and waves. Due to a lack of time during the graduation project, the seismic stability of the alternative breakwater has not been checked.

2.5.5 Lifetime and Maintenance

The minimum required lifetime of a breakwater alternative is set to 10 years. This required lifetime is shorter than the lifetime of the reference breakwater. The main purpose of the project is to invent a low-cost breakwater alternative, even if the lifetime of the breakwater will be reduced by this costs saving. The breakwater with a reduced lifetime can be applied on project locations which require a temporary breakwater protection. During the lifetime of the innovative breakwater, the client is able to save enough money for an possible rebuild. Besides, the lifetime of the barge loading terminal is not known. To save extra money during the lifetime of the breakwater, no maintenance is allowed.

2.5.6 Breakwater costs

The assessment of the total breakwater costs is based on the material costs and three multiplication factors. The total material costs are estimated by multiplying the total required amount of material with the corresponding unit price of the material. The three multiplication factors are: $f_{construction}$, $f_{complexity}$ and $f_{contractor}$.

The factor $f_{construction}$ is a multiplier for the costs for the used equipment, the costs for the installation of construction parts, the transportation of construction parts, the costs for preparation work onshore and other construction related costs.

The factor $f_{complexity}$ is a multiplier for the complexity of the construction and construction parts for which the costs are difficult to assess.

The factor $f_{contractor}$ is a multiplier for the indirect costs of the contractor (survey costs, engineering costs, the construction site and site office costs, some unexpected expenses, profits and other contractor related costs).

The total construction costs are calculated by multiplying the total material costs with these three factors:

$$€_{total} = \left(\left(€_{material} * f_{construction}\right) * f_{complexity}\right) * f_{contractor}$$

There is no cost limit determined for the traditional breakwater design because the purpose of the traditional breakwater design is to create a realistic reference price to compare the alternative breakwater designs with. The limit for the costs of the alternative breakwater designs is set to €25,000,000.- because a reasonable reduction compared to the reference design is strived for.

3 Reference design

3.1 Introduction

The reference design is a rubble mound breakwater. Using the design criteria mentioned in the rock manual (CIRIA; CUR; CETMEF, 2007), a realistic definition of the breakwater dimensions has been determined. Both a primary rock armour layer and a primary concrete armour layer have been studied, in order to optimize the reference design. The rock armour layer appears to be the most competitive breakwater design. In appendix 3, reference design, the design calculations can be found.

3.2 Breakwater dimensions

The rubble mound reference design breakwater consists of a primary rock armour layer, a secondary rock filter layer and a core of quarry run. The front slope is constructed 1:3 and the rear slope 1:5. The dimensions of the reference design breakwater can be found in figure 6. The length of the breakwater is 610 meters.

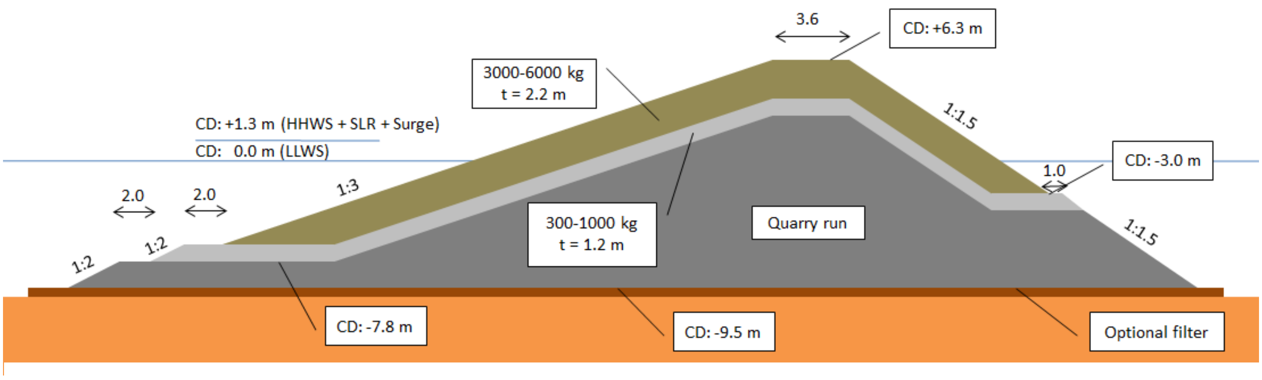


Figure 6: Dimensions reference breakwater

3.3 Lifetime

If the rubble mound material which is applied in the breakwater passes all of the tests, mentioned in the rock manual (CIRIA; CUR; CETMEF, 2007), a minimal durability of 50 years can be guaranteed. Thereby, the breakwater has to be constructed within the placing tolerances mentioned in the rock manual.

3.4 Construction method

There are two methods for constructing a breakwater, a land-based construction method and a waterborne construction method. The preferred construction method for the reference design breakwater is the land-based construction method because of the lower expected downtime during construction. The only downside of this construction method is that an auxiliary construction is needed to start building the breakwater in the dry. Figure 7 shows the location of the required auxiliary construction.

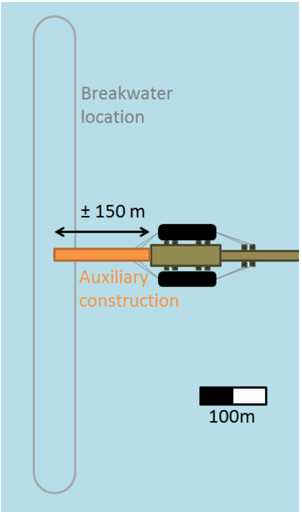


Figure 7: Location of the auxiliary construction

3.5 Costs

The calculated material usage and material costs of the reference design breakwater can be found in table 4. The unit prices for the different rubble mound gradations are based on the advice of experts of RHDHV.

	Material usage (m^3)	Unit price (€/m^3)	Material costs
Quarry run	262,820	40	€ 10,512,804.-
Rock 300 – 1000 kg	48,182	60	€ 2,890,930.-
Rock 3000 – 6000 kg	75,351	80	€ 6,028,069.-
		Total	**€ 19,431,803.-**

Table 4: Reference design material costs

In order to determine the total construction costs of the breakwater, three multiplication factors have been applied:

$$f_{construction} = 1.3$$
$$f_{complexity} = 1.1$$
$$f_{contractor} = 1.35$$

The construction factor is set to 1.3 because of the large amount of labour required and the transportation required for the materials. The complexity factor is set to 1.1 because of the unknown costs for the required auxiliary construction. And the indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), because: a quarry has to be opened to provide the required materials, the whole construction process has to be supervised, a site office has to be build and the profit made by the contractor has to be taken into account.

When multiplying the material costs with these multiplication factors, the total construction costs are approximately **€ 37,500,000.-**. The total construction costs have an uncertainty bandwidth of -25% to +30%, based on the uncertainty of the multiplication factors. The detailed cost estimation can be found in appendix 3, reference design.

4 Alternative breakwater analysis

4.1 Introduction

This chapter describes the process which starts at the breakwater design requirements and ends with the most favourable alternative breakwater concept. The reference design is used in order to make a good comparison of the advantages and disadvantages of the alternative designs. The alternative design process is schematized in figure 8, process for determining best alternative.

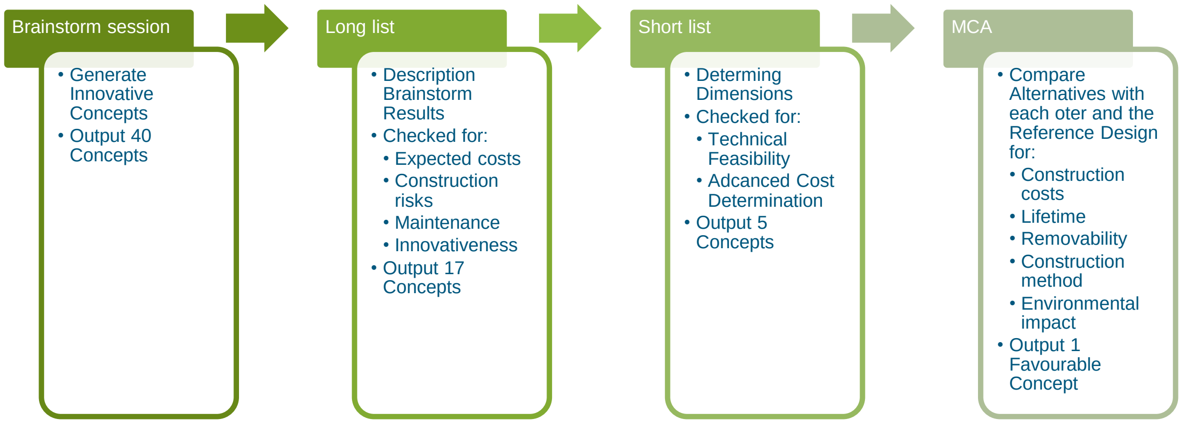


Figure 8: Process for determining best alternative

4.2 Brainstorm session

The main goal of the brainstorm session was to generate and visualise as many breakwater concepts as possible. With the help of RHDHV colleagues, alternative breakwater concepts have been developed. The generated breakwater concepts are described in the long list report. The introductory document and the presentation used for the brainstorm session can be found in appendix 4.

4.3 Long list

The input of the long list analysis is the breakwater concepts which are developed during the brainstorm session, complemented with breakwater concepts which have been developed during the course of the graduation project. The main purpose of the long list exercise was to reduce the number of alternatives which will be investigated further in the short list. The critical examination in the long list is based on the material usage, the complexity of the construction, the required maintenance and the rate of innovation. The criteria material usage and construction complexity directly influence the total construction costs of the alternative breakwater; the breakwater has to be low-cost. The maintenance criterion is set by the client; he specifically requested a maintenance free breakwater solution. The criterion of innovation is set, since the breakwater has to be non-traditional. Table 5 shows an overview of all the concepts from the long list, as resulted from the brainstorm session.

The 17 concepts that proceed to the short list are: nr 3 geo-tubes or –containers, nr 4 refraction islands, nr 6 vertical piles, nr 7 pile pyramid, nr 12 jacket breakwater, nr 16 slab with support, nr 17 bellows structure, nr 19 car wreckages, nr 21 old tire, nr 22 container pyramid, nr 24 container wall with struts, nr 25 old ships, nr 28 vertical tubes with band, nr 34 sandwich breakwater, nr 35 enclosed waste, nr 36 replacing core and nr 39 piles and tires The 40 different concepts and the reason of omitting some of them from the long list are described in appendix 5, long list.

1 Island	2 Natural coast	3 Geo-tubes or -containers	4 Refraction islands	5 Refraction gully	6 Vertical piles
7 Pile pyramid	8 Combined wall	9 Mikado piles	10 Pipes with angle	11 Tube with curve	12 Jacket breakwater
13 Caisson with sheet piles	14 Slab with support	15 Hollow breakwater	16 Slab with support	17 Bellows structure	18 Floating constructions
19 Car wreckages	20 Old airplane pieces	21 Old tires	22 Container pyramid	23 Container core	24 Container wall with struts
25 Old ships	26 Mangrove	27 Lava hole	28 Vertical tubes with band	29 Natural sedimentation	30 Offshore platform
31 Iceberg without ice	32 Removable breakwater No picture available	33 Rent a breakwater No picture available	34 Sandwich breakwater	35 Enclosed waste	36 Replacing core
37 Replacing armour layer	38 Berm breakwater	39 Piles and tires	40 Container armour layer with tire core		

Table 5: Long list alternatives

4.4 Short list

The short list report gives a further description of the 17 breakwater concepts which were determined to be feasible according to the long list phase. The main purpose of the short list is to estimate the breakwater costs and the lifetime. In order to determine the costs of the breakwater concepts, the dimensions of each breakwater concept have been determined roughly. The total construction costs of the breakwater concepts are calculated in the same way as the reference design, in other words, the construction cost of the breakwater is calculated by multiplying the total material costs with three factors. In order to give a good estimation of the lifetime of the breakwater concept, some material properties have been assessed. If the total construction costs of the breakwater are higher than €25,000,000.- or the lifetime of the breakwater is less than 10 years, the breakwater concept will not be investigated further. The estimated construction costs have an uncertainty from -23% to +36%, based on the uncertainty of the multiplication factors. The uncertainty of the costs is comparable to the uncertainty of the reference design breakwater costs. Table 6 shows a summary of the short list alternatives, of which only 5 variants could proceed to the MCA phase. See appendix 6, short list, for the assessment of the costs and lifetime determination.

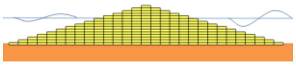
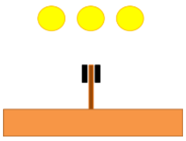
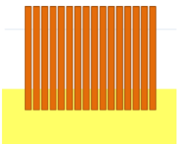
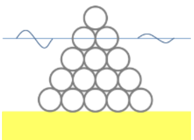
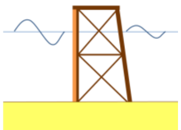
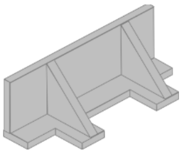
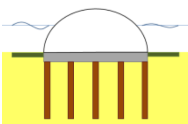
Pictogram			Proceed to MCA
	Name	**Geo-tubes or –containers**	NO
	Costs	€ 30,000,000.-	
	Lifetime	At least 40 years	
	Name	**Refraction islands**	NO
	Costs	€ 39,400,000.-	
	Lifetime	At least 50 years	
	Name	**Vertical piles**	NO
	Costs	€ 30,500,000.-	
	Lifetime	Approximately 20 years	
	Name	**Pyramid of piles**	NO
	Costs	€ 35,400,000.-	
	Lifetime	At least 50 years	
	Name	**Jacket breakwater**	NO
	Costs	€ 34,000,000.-	
	Lifetime	Approximately 50 years	
	Name	**Slab with support**	**YES**
	Costs	€ 18,000,000.-	
	Lifetime	At least 25 years	
	Name	**Bellows structure**	NO
	Costs	€ 54,800,000.-	
	Lifetime	Approximately 50 years	

Table 6: Short list alternatives

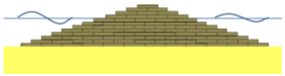
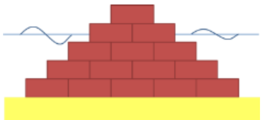
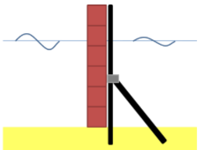
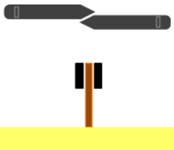
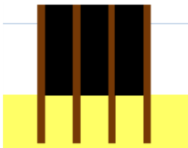
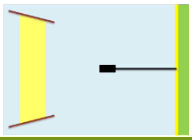
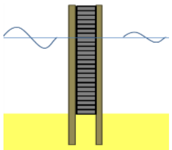
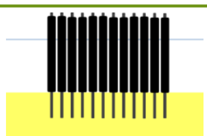
Pictogram			Proceed to MCA
	Name	**Car wreckages**	NO
	Costs	€ 65,300,000.-	
	Lifetime	Approximately 20 years	
	Name	**Old Tires**	NO
	Costs	€ 37,300,000.-	
	Lifetime	At least 50 years	
	Name	**Container pyramid**	NO
	Costs	€ 28,400,000.-	
	Lifetime	At least 20 years	
	Name	**Container wall with struts**	NO
	Costs	€ 45,100,000.-	
	Lifetime	At least 10 years	
	Name	**Old ships**	**YES**
	Costs	€ 22,000,000.-	
	Lifetime	Approximately 48½ years	
	Name	**Vertical tubes with band**	**YES**
	Costs	€ 20,000,000.-	
	Lifetime	Approximately 50 years	
	Name	**Sandwich breakwater**	NO
	Costs	€ 110,700,000.-	
	Lifetime	Not determined / variable	
	Name	**Enclosed waste**	**YES**
	Costs	€ 20,500,000.-	
	Lifetime	At least 10 years	
	Name	**Replacing core**	NO
	Costs	€ 33,000,000.-	
	Lifetime	At least 50 years	
	Name	**Piles and tires**	**YES**
	Costs	€ 25,000,000.-	
	Lifetime	Approximately 50 years	

Table 6: Short list alternatives (continuous)

4.5 Multi criteria analysis

The multi criteria analysis (MCA) critically evaluates the reference design and the 5 breakwater concepts that proceeded from the short list. Each breakwater concept is rated for the criteria: construction costs, lifetime, removability, construction risks and environmental impact. Table 7 shows the results of the MCA. Each criterion consists of a score varying from 1 (very poor) to 5 (very good). The maximum score which can be achieved is 30 points. The criterion construction costs weight double, since the main purpose of the project is to develop a low-cost breakwater.

Design	Construction costs	Lifetime	Construction Risks	Removability	Environmental Impact	Total score
Reference design	0	5	3	3	3	14
Slab with support	5	3	5	3	2	23
Old Ships	2	3	5	1	3	16
Vertical Tubes with band	4	5	1	2	2	18
Enclosed waste	3	3	2	2	2	15
Piles and Tires	1	5	1	2	4	14
Weight factor	2	1	1	1	1	

Table 7: Results of the MCA

As can been seen in table 7, the most favourable design according to the MCA is the slab with support design. Therefore, the slab with support design will be detailed further in the last step of the process, see figure 9. A more detailed description of the score per criterion and per alternative, an argumentation and sensitivity analysis of the weight factor and a sensitivity analysis of the criterion construction costs can be found in appendix 7, MCA.

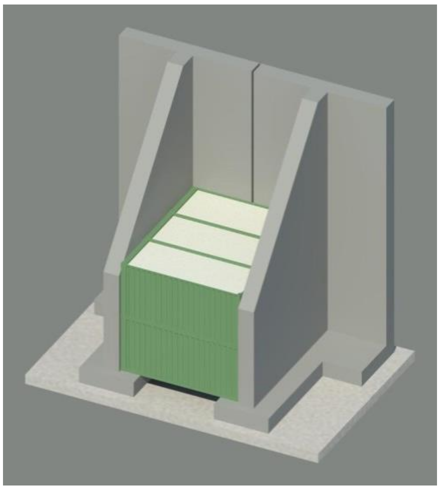


Figure 9: 3D-view slab with container backfill

5 Most favourable alternative

5.1 Introduction

The most favourable breakwater alternative, according to the alternative breakwater analysis, is the “slab with support breakwater” with a container backfill. The breakwater alternative consists of a concrete L-wall which is supported by vertical supports at the backside perpendicular to the front wall. In order to reduce the amount of concrete, a container backfill is applied, which consists of 20 feet containers. The containers in the backfill will be completely filled with sand, which will increase the gravitational forces of the breakwater. This chapter will give a summary of the assessed dimensions. The supporting breakwater calculations can be found in appendix 8, breakwater optimization.

5.2 Breakwater dimensions

5.2.1 Main dimensions

There are two kinds of stabilities when looking at the breakwater, the internal and the external stability. In order to guarantee the internal stability of the concrete structure, some strength calculations have to be performed. Due to limited time, the concrete element dimensions have been determined indicatively by an experienced engineer of RHDHV. The external stability has been checked for two different failure mechanisms, overturning and sliding, by the graduation students themselves. Based on these calculations, the breakwater dimensions have been assessed, see figure 10. It is noted that the dimensions mentioned in this figure can be optimized further.

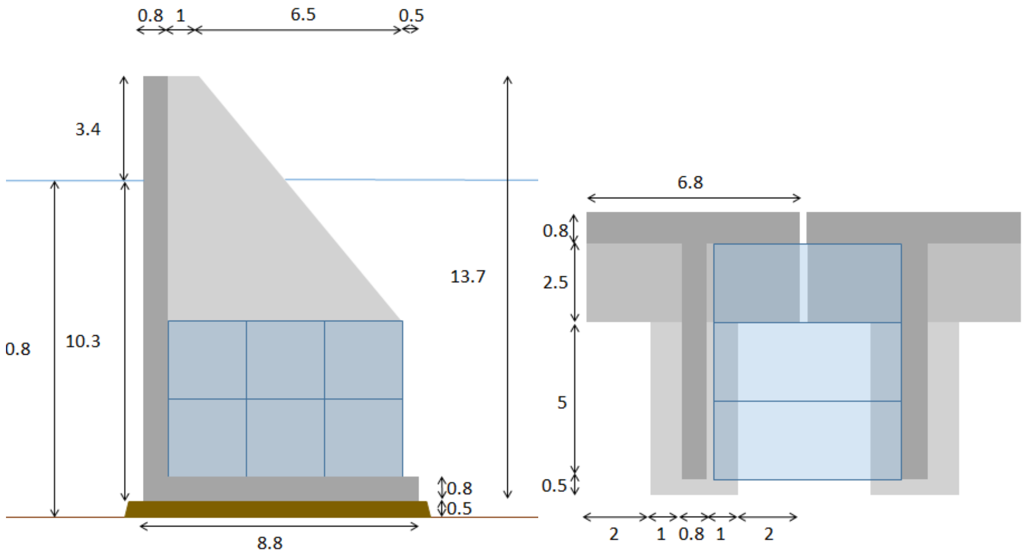


Figure 10: Dimensions slab with support breakwater

The minimum required breakwater length is 595 meters. This length is based on the expected transmission through the breakwater, the overtopping over the breakwater (the influence of overtopping on the wave height at the lee side of the breakwater is nihil) and the diffraction around the breakwater. When dividing the minimal required breakwater length by the width of a concrete element, a minimum of 88 elements is required, in order to achieve the required length and thus provide acceptable operational wave conditions.

5.2.2 Container protection

The purpose of the containers is twofold. One is to “connect” the concrete slabs ant two is to keep the sand mass at the right place. A few years after installation of the containers, gaps in the container walls could occur, due to corrosion caused by salt water. Moreover, the gaps between the concrete elements, caused by the placement tolerances, will result in some wave forces on the container walls as well. Due to these wave forces, gaps in the container walls could develop earlier.

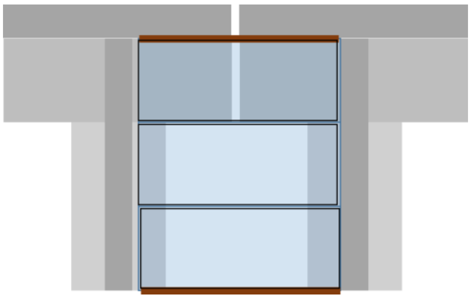


Figure 11: Container protection

In order to prevent the containers from sand losses, each container will be protected in two different ways, the first protection is a steel plate which protects the outside facing container walls, this plate will be welded to the external container walls. A geotextile will be applied inside each container, in order to prevent the other container walls and the container floor from sand flowing out of possible occurring gaps. During further research, it may be concluded that only one of these protections is required. For now, both will be taken into account, in order to be sure that no sand will escape out of the containers. The locations of the required steel plate container protection can be seen in figure 11.

5.3 Lifetime

When applying a proper concrete mixture which is able to withstand the impacts caused by the salt ocean water, the lifetime of the concrete element can be set to at least 50 years. In order to achieve this lifetime, the concrete mixture, reinforcement and the required concrete cover have to be designed during a further research.

It is assumed that the container frame will, in the best case, lasts for at least 25 years. However this 25 years lifetime is based on a container which is placed into salt water without any fill added. If a sand fill is added, the corrosion process of the container frame may be slowed down, due to the sand protection. The exact lifetime of the container frame has to be determined in a further design phase. This may result in a lifetime of more than 25 years.

5.4 Construction method

The concrete elements have to be pre-fabricated at an on-shore site. The steel plates and geo-textiles have to be installed in the container and the containers will be partly filled with sand, in order to increase the weight of the container. After these on-shore preparations, the breakwater elements and the fortified containers will be transported to the construction site with barges.

The construction on site is divided into six different construction steps.

1) Installation of the filter layer
2) Installation of the concrete elements with a jack up barge
3) Installation of the first layer of containers
4) Further filling of the first layer of containers
5) Installation of the second layer of containers
6) Further filling of the second layer of containers

5.5 Costs

The quantified material usage and material costs of the slab with support breakwater can be found in table 8. The determination of the unit prices can be found in appendix 8, breakwater optimization.

	Material usage	Unit price	Material costs per element	Material costs breakwater
Concrete	157.28 m^3	€ 600,-/m^3	€ 94,368.-	€ 8,304,384.-
Containers	6 pcs	€ 1,000.-/pcs	€ 6,000.-	€ 528,000.-
Container fill	231.3 m^3	€ 5.-/m^3	€ 1,157.-	€ 101,772.-
Container protection	4 pcs	€ 835.-/pcs	€ 3,340.-	€ 293,920.-
Geotextile in container	6 pcs	€ 400.-/pcs	€ 2,400.-	€ 211,200.-
Filter layer	-	-	-	€ 150,000.-
			Total	**€ 9,590,753.-**

Table 8: Slab with support breakwater material costs

In order to determine the total construction costs of the breakwater, three multiplication factors have been applied:

$$f_{construction} = 1.4$$
$$f_{complexity} = 1.0$$
$$f_{contractor} = 1.35$$

The factor $f_{construction}$ is set to 1.4. This factor includes the costs for the jack up barge (including mobilisation and demobilisations), the barges, the ship that installs the filter layer and all the other construction equipment for among others the sand filling. Thereby, the costs for the construction workers who install the geotextile, weld the container protection, prepare and cast the concrete elements and man the machinery is included in the factor $f_{construction}$.

The factor $f_{complexity}$ is set to 1.0 for the slab with support breakwater because it is a relative simple construction and there are no construction parts of which the costs are unknown.

The factor $f_{contractor}$ is set to 1.35 because a special location has to be established for the prefabrication of the concrete elements and there are a couple of supervisors needed to survey the different construction processes (like the construction of the concrete elements, the preparation of the containers and the building steps itself).

$$€_{total} = €_{material} * f_{construction} * f_{complexity} * f_{contractor}$$

$$€_{total} = € \, 9{,}590{,}753 * 1.4 * 1.0 * 1.35 \approx € \, 18{,}100{,}000.-$$

The total construction costs of the slab with support breakwater are estimated to be:

$$€ \, \mathbf{18{,}100{,}000.-}$$

The total construction costs of the slab with support breakwater have an uncertainty from -14% to +27%, based on the uncertainty of the multiplication factors. The detailed cost estimate can be found in appendix 8, breakwater optimisation.

6 Conclusion

6.1 Conclusion project

The most favourable alternative for the project location is the slab with support breakwater with a container backfill. The total construction costs of this breakwater are approximately €18,100,000.-. The total estimated costs reduction, compared to the reference design is approximately € 19,400,000.-. In reality, the total cost reduction may be less than estimated, because the design is not checked for all the occurring failure mechanisms. However, the dimensions of the slab with support breakwater have not been optimized in detail yet which may result in a reduction of material costs. The failure mechanisms that have to be investigated and the possible optimizations are described in chapter 7, recommendations.

6.2 Conclusion process

The main question of this graduation report is:

Design an alternative, low cost, non-traditional breakwater solution for a specified location at the NW-coast of Sumatra.

The slab with support breakwater can definitely be classified as low cost, since the breakwater alternative is almost €20 million less expensive than the reference design breakwater. The breakwater is non-traditional, since the breakwater concept has not been constructed anywhere in the world yet.

In the project plan is mentioned that Royal HaskoningDHV's main purpose of the project was to find a technically sound and effective breakwater concept, at considerably lower costs than is needed for a traditional breakwater. The slab with support breakwater might be developed by RHDHV in the future into a feasible alternative for situations where the business case makes this applicable. Therefore, it can be concluded that the main purpose of RHDHV is achieved.

7 Recommendations

7.1 Introduction

The most favourable design resulting from the alternative breakwater analysis is the slab with support breakwater design. However, this design is not yet ready to be constructed. It has to be checked for several design aspects that have not been taken into account in this graduation project. Moreover, the design could be further optimised by investigating, (re)calculating or modelling some parts of the construction. Both the design aspects that have to be checked and the possible further optimisations are described in this chapter.

7.2 Design aspects

7.2.1 Container backfill

An advanced assessment of the corrosion rate and container protection methods has to be performed, in order to obtain a more accurate forecast of the lifetime of the container backfill.

7.2.2 Filter layer

In order to define if a filter layer beneath the concrete elements is required; a pore pressure calculation has to be performed. Based on this pore pressure calculation, it can be determined whether a filter layer is required or not.

7.2.3 Scour

If scour will occur, in front of the breakwater construction, the breakwater may tip forward. In order to prevent this failure mechanism, an advanced scour calculation has to be performed. If scour occurs, the subsoil has to be protected with a rock armour layer or another solution.

7.2.4 Breakwater heads

In this graduation project, only the dimension of the typical breakwater cross section has been determined. The dimensions of the breakwater heads have not been determined yet. Possible solutions for the breakwater heads are to apply extra containers at both heads or create some special concrete head elements. The stability of the breakwater heads has to be investigated during a further research.

7.2.5 Settlements subsoil

The reference design will cause settlements of more than a meter, see appendix 3, reference design. Therefore, it is assumed that the subsoil will settle when loaded by the concrete elements. In order to verify this assumption, a settlement calculation has to be performed. The height of the concrete elements has to be compensated for this settlement to guarantee the required crest height.

7.2.6 Earthquake resistance

The project location is situated in a seismologic active area. In this current design, the stability of the concrete elements and the stability of the subsoil due to seismological impact have not been investigated. In order to guarantee a stable construction during this load combination, a seismological impact calculation has to be performed.

7.2.7 Tsunami

Another environmental impact in seismologic active areas is a tsunami. This extreme environmental force has not been taken into account when calculating the stability of the concrete element. When designing the concrete slab breakwater in more detail, the wave load caused by a tsunami has to be taken into account.

7.2.8 Element connection

If the placing tolerances are determined to be unacceptable, a guiding construction may be applied. The main purpose of this guiding construction is to increase the placement accuracy. During an eventual further design, the dimensions of this element connection have to be detailed if the placing tolerances appear to be unacceptable.

7.2.9 Stability element

The stability of the elements is only checked for sliding and overturning towards the lee side of the breakwater. However, the overturning stability towards the ocean side of the breakwater caused by a wave trough is not checked. To guarantee the stability of the element, this has to be checked (for both the construction period and after construction is finished) in the future.

7.2.10 Wave conditions during construction period

The maximum wave conditions during construction are not yet investigated. Based on our calculations the concrete elements will be stable for sliding and overturning without backfill when placed during wave conditions with a maximum significant wave height of 1.5 meter. However, to determine the boundary conditions for the construction period, the wave conditions must be investigated further.

7.2.11 Morphology

In the basis of design is mentioned that the shoreline is eroding where the jetty reaches the coast. The breakwater could have a positive effect on preventing this erosion. In order to validate this assumption, an advanced morphological model has to be developed.

7.2.12 Concrete mixture and reinforcements

The concrete mixture, the required concrete cover and the concrete reinforcements are not designed yet. To guarantee the lifetime of the elements of at least 50 years, these designs have to be performed.

7.2.13 Span of the containers

When installing the container backfill, the second and the third container row will be installed with a span. This span may cause deformations and possible failure of the container floors, since the maximum fill of the containers is already exceeded. For the calculations of the container backfill is assumed that the span of the containers will not lead into container floor failure. This assumption has to be investigated during a further research.

7.3 Optimisations

7.3.1 Length of the breakwater

The length of the breakwater is based on the transmission-length diagram given in the basis of design. However, this diagram is based on a breakwater location in the centreline of the crest of a rubble mound breakwater. The slab with support design does not have a rear slope. Therefore, the slab with support breakwater construction can be placed closer to the jetty than a rubble mound breakwater. When the slab with support breakwater is placed closer to the jetty, the required breakwater length will possibly be less than estimated with the transmission-length diagram in the basis of design. This may be so because the diffraction shelter of the breakwater moves closer to the jetty when the structure is placed closer to the jetty. Therefore, the length of the breakwater regarding the diffraction, wave transmission and overtopping must be investigated further using wave modelling software.

7.3.2 Concrete elements

The internal stability has been determined using rough estimations based on the judgement of experienced structural engineers. The dimensions of the concrete elements can be optimized, which may result in less concrete. This optimisation has to be performed using advanced modelling software, in order to create a good analysis of the internal forces and required reinforcement. Also the possibility of pre-stressed concrete can be investigated to reduce the required amount of concrete.

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Appendix 1: Project Plan

Innovative Breakwater

16 June 2015

Final Report

Rotterdam University

University of Applied Sciences

Royal HaskoningDHV

Enhancing Society Together

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Document title	Appendix 1: Project Plan
Document short title	Project Plan
Status	Final Report
Date	16 June 2015
Project name	Innovative Breakwater

Drafted by	Jesper van Grieken, Danny Janssen	
Checked by	Ronald Stive	
Date/initials check	………………….	………………….
Approved by	Harry Dommershuijzen	
Date/initials approval	………………….	………………….

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SUMMARY

This plan of action will describe the approach of the graduation project innovative breakwater. This project will be performed by two students of the Rotterdam University of applied sciences, Jesper van Grieken and Danny Janssen, both graduating in their bachelor civil engineering.

The graduating company is Royal HaskoningDHV, which will support the graduating students with sharing a part of their knowledge, offering a workplace during the graduation period and giving feedback on the delivered work.

The project will investigate the feasibility of applying an alternative and low cost breakwater at the coast of Sumatra, facing the Indian Ocean. This “innovative” breakwater is envisaged to protect a recently constructed export jetty for bulk material. Because of underestimating the severe environmental conditions (swell waves), both during construction and operation, it was initially decided to omit the application of a dedicated breakwater. This study will consider the feasibility of a low cost, non-traditional solution.

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CONTENTS

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1 PROJECT BACKGROUND

1.1 Project Definition

During the last phase of the bachelor period at the University of Rotterdam, a graduation project has to be done. This project will be done at the engineering consultancy Royal HaskoningDHV. The project definition has been set as follows:

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because of underestimating the severe environmental conditions, both during construction and operation, it was wrongly decided to omit a solution for the swell waves. Moreover, the project budget was too limited to build a traditional breakwater. This leaded to a downtime of 80 to 90 %.

For this reason, a solution for the swell waves is needed. There are a lot of different solution directions but RHDHV is especially interested in alternative breakwater concepts to make the project financially feasible. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference purpose also a traditional breakwater solution will be developed, i.e. the reference design.

The main question of this project is:
Design an alternative, low cost, non-traditional breakwater solution for of a specified location at the NW-coast of Sumatra.

Figure 1 is a schematic drawing of the situation on the project location. The breakwater is not build yet at the project location.

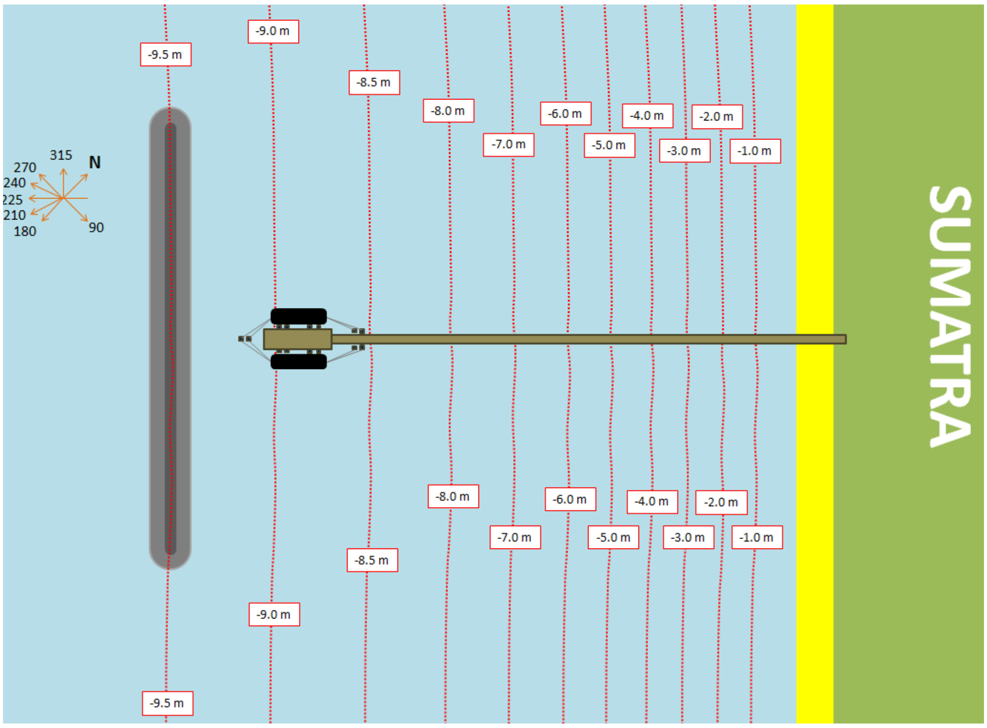

Figure 1: Schematic location breakwater

1.2 RHDHV

Royal HaskoningDHV (RHDHV) is a world leading international service provider in the field of Consultancy and Engineering.
They are specialized in Aviation, Buildings, Industry, Energy & Mining, Infrastructure, Maritime & Waterways, Planning & Strategy, Rivers, Delta's & Coasts, Transport & Asset Management and Water Technology.
With its headquarters in Amersfoort, The Netherlands, RHDHV is ranked globally in the top 10 of independently owned, non-listed companies and top 40 overall. Their 7,000 staff provides services across the world from more than 100 offices in over 35 countries.

With an overarching aim to ***Enhance Society Together***, they work closely with clients, stakeholders, industry and academic leaders, to ensure projects are delivered on time and within budget, while providing a better, brighter and sustainable future.

1.3 Project setting

In 2007, RHDHV investigated the boundary conditions at the project location which is situated at the NW coast of Sumatra facing the Indian Ocean. Due to confidentially reasons, the client does not want the location to be specified in detail. Therefore, taking the actual RHDHV export terminal project as a starting point, a realistic schematisation of the BW layout/position and the surrounding seabed with actual design boundary conditions have been defined. This realistically schematised situation will be used for all study and design work related to the graduation project.

1.4 Location of Graduating

The graduating location is for most of the time at the regional RHDHV office in Rotterdam. The Business Line, under which guidance the graduating investigation takes place, is the business line Maritime and waterways. This Business Line is involved in national and international projects like the so called "Room for the River" projects in the Netherlands and the "World Islands" in Dubai.

1.5 Project Purpose RHDHV

Royal HaskoningDHV's main purpose of the project is finding a technically sound and effective breakwater concept, at considerably lower cost than is needed for traditional breakwaters. The project succeeds if RHDHV can develop later on, in successive steps, this innovative breakwater concept into a feasible alternative for situations where the business case makes this applicable.

1.6 Success of the project

The project will succeed if:

- The graduating students pass their bachelor.
- The university is satisfied about the delivered products.
- The project results can be used for further investing by RHDHV.

2 PROJECT ACTIVITIES

2.1 The phases

The project is divided in 6 different phases:

1. Preparation phase
2. Reference design
3. Alternative designs
4. Detailing selected alternative
5. Final report
6. Preparation presentation

2.2 Activities per phase

Each different phase can be divided in several activities:

1. Preparation phase
    - 1.1. Project plan
    - 1.2. Time schedule
    - 1.3. Basis of Design (BoD)

2. Reference design
    - 2.1. Length, orientation and location
    - 2.2. Breakwater concepts (2 or 3 concepts)
    - 2.3. Design selected concept
        - 2.3.a.1. Hydraulic design
        - 2.3.a.2. Seismic design
        - 2.3.a.3. Material specification
    - 2.4. Lay-out, typical cross section and breakwater heads
    - 2.5. Costs
    - 2.6. Construction method and programme

3. Alternative designs
    - 3.1. Brainstorm session
    - 3.2. Generate alternatives
    - 3.3. Develop feasible options (short list)
    - 3.4. Design concepts
    - 3.5. Multi-Criteria Analysis (MCA)

4. Detailing selected alternative
    - 4.1. Hydraulic design
    - 4.2. Structural design
    - 4.3. Seismic design
    - 4.4. Material specification
    - 4.5. Construction method
    - 4.6. Costs
    - 4.7. Detailed drawings
    - 4.8. *Optional: Numerical modelling*

5. Final report
    5.1. Write advisory report
    5.2. Combining documents (appendixes for advisory report)

6. Preparation of presentation and defence of final report

The results of these phases are described in chapter 3.

3 PROJECT RESULTS

3.1 Purpose of the Project

The purpose of the project is to select and develop an alternative, low cost, non-traditional breakwater solution for a specified location. The desirable result is a recommendation for the technically and financially most attractive design concept. This recommended design will be detailed and optimized within the timeframe available.

3.2 Intermediate results

The following products will be intermediate results:

- Project plan (end of phase 1)
- Basis of Design (end of phase 1 / during phase 2)
- Reference design (end of phase 2)
- Brainstorm session on alternative concepts (start phase 3)
- Midterm presentation (during phase 3)
- Alternative designs including different concepts and MCA (end of phase 3)
- Detailed design of the selected alternative (end of phase 4)
- Concept final report (3 weeks before deadline final report)
- Results of the “Go/No-Go meeting” (First week of June)
- Final report (Tuesday 16 June 2015)
- Presentation and defence of final report (between 1 and 3 July)

3.2.1 Basis of Design

In order to define the project starting points, the design and project boundary conditions will be specified and summarized in a basis of design (BoD) document. In this basis of design, a subdivision between the environmental conditions, the required performances of the breakwater, the project requirements, the used materials and the used codes, standards and guidelines is made. The basis of design is delivered as a separate document.

The project requirements (part of the Basis of Design) are included in appendix I.

3.2.2 Reference design

In order to get a good impression about the boundary conditions and the problems which can occur during the design of a breakwater, a traditional breakwater design will be determined. This design will be optimized and checked using the most common failure mechanisms. After determining the dimensions of the breakwater, the costs of the traditional breakwater will be calculated.

Besides the rubble mount breakwater, some other conventional breakwater designs will be considered. These conventional breakwaters will be compared on feasibility at the project location and construction costs.

3.2.3 Alternative designs

The brainstorm session's purpose is to invent several different alternatives. The alternative designs will be compered on costs, feasibility and sustainability. This will be done using a Multi-Criteria Analysis (MCA). It is not expected that multi-purpose solutions will be feasible, because multi-purpose solutions usually are more expensive and the budget is limited.

The costs of the alternatives are determent based on the following aspects:

- Materials
- Construction of the breakwater
- Durability
- Maintenance

3.2.4 Detailed design of the selected alternative

The most favourable alternative design will be optimised further. This design will be checked at the most common failure mechanisms. The level of detail of the detailed design of the selected breakwater is dependent on the spare time available. If there is some spare time left after the calculations, the detailed design of the selected alternative will be optimized by putting it into a numerical model (time dependant).

3.2.5 Final Report

The final report consists of a recommendation for an alternative, low cost, non-traditional breakwater solution. This report describes the benefits and cons of the innovative breakwater. The following appendixes are contained in the final report.

- Project plan
- Basis of Design
- Reference design report
- Alternative design report (including a MCA)
- Further design of the innovative breakwater

4 PLANNING AND ORGANISATION

4.1 Time schedule

The total duration of this graduation project is 22 weeks. Time is reserved for every phase:

1. Preparation phase (2,5 weeks)
2. Reference design (5 weeks)
3. Alternative designs (8 weeks)
4. Detailing selected alternative (4 weeks)
5. Final report (2 weeks)
6. Preparation presentation (2,5 weeks)

Phase 2 and 3 are partly parallel. A detailed time schedule can be found in appendix II.

4.2 Organisation

4.2.1 Students

This bachelor thesis will be performed by two students:

Name:	Danny Janssen	Gender:	Male
Address:	Tonnnemanplein 8 3151 PR Hoek van Holland		
Phone number:	+31 6 15 38 57 13		
E-mail:	Danny.janssen@rhdhv.com		
Function:	Graduating student		
Powers:	None		
Availability:	All workdays unless otherwise mentioned in the planning.		

Name:	Jesper van Grieken	Gender:	Male
Address:	Lamoen 41 3232 RE Brielle		
Phone number:	+31 6 27 34 55 50		
E-mail:	Jesper.van.grieken@rhdhv.com		
Function:	Graduating student		
Powers:	None		
Availability:	All workdays unless otherwise mentioned in the planning.		

4.2.2 Supervision

This bachelor thesis will be supervised by two people:

Name:	Ronald Stive	Gender:	Male
Address (RHDHV):	George Hintzenweg 85 Postbus 8520, 3009 AM Rotterdam		
Phone number:	+31 88 348 92 25	+31 6 22 60 25 48	
E-mail:	Ronald.stive@rhdhv.com		
Function:	Company supervisor		
Powers:	Guiding students, advising graduation teacher, company evaluation		
Availability:	Monday till Friday		

Name:	Harry Dommershuijzen	Gender:	Male
Address (University of Rotterdam):	G.J. de Jonghweg 4 - 6 3015 GG Rotterdam		
Phone number:	+31 10 794 58 72	+31 6 13 49 08 23	
E-mail:	h.j.dommershuijzen@hr.nl		
Function:	Graduation teacher		
Powers:	Guiding students, review final report and graduation presentation.		
Availability:	Monday till Friday		

4.3 Project meetings

Every day the students start and end the day with discussing what they have done that day / will be doing that day and will divide the different tasks.

During the project there will be several meetings with Mr. Stive and/or Mr. Dommershuijzen. The students are responsible for these meetings.

Meetings with Mr. Stive will take place at least once every week. When Mr. Stive is not available, the meeting will be with one of the colleagues.

Meetings with Mr. Dommershuijzen will take place approximately every three weeks, and/or when an intermediate result is completed.

4.4 Archiving

The products will be saved on one of the two computers of the students or in the shared dropbox folder, depending on the product. All intermediate results will be saved in the shared dropbox folder.

5 QUALITY

5.1 Maintaining quality

To maintain the quality of the completed products, each product will be reviewed by the other graduating student. Every intermediate result and the final report will be checked by the company supervisor, Mr. Ronald Stive, and discussed with the company supervisor and the graduation teacher, Mr. Ronald Stive and Mr. Harry Dommershuijzen. The feedback of Mr. Stive and Mr. Dommershuijzen will be processed in the product before it is made final.

5.2 Layout

Every product will have the layout provided by Royal HaskoningDHV and/or the layout learned during the education period on the University of Rotterdam, depending on the product.

5.3 References

There will be referred to the references according to the APA-method.

5.4 Used software

Software used during the process will be used as learned on the University of Rotterdam or as explained by one of the employees of Royal HaskoningDHV.

5.5 Advice

When advice is needed, the students ask one of the colleagues of Royal HaskoningDHV and/or one of the teachers of the University of Rotterdam.

6 APPENDIX I: THE PROJECT REQUIREMENTS

6.1 Requirements

6.1.1 General Requirements

Bathymetry: The water depth is 9.5 meter (Traditional) and 9.5 – 10 meter (Alternative, width dependent).

Tidal Conditions: See table below:

LLWS	MLWS	MLWN	MHWN	MHWS	HHWS
-0.30 m	-0.25 m	-0.10 m	0.10 m	0.25 m	0.45 m

Available are: The available area for the breakwater is defined as unlimited to the ocean side.

6.1.2 Ultimate Limit Boundary Conditions

Waves: The breakwater construction has to withstand a storm with a return period of 1/100 years, consisting of the following wave conditions:

H_s 3.3 m
T 18.6 s
Direction 228.2° (as seen from the north)

Earthquake: The breakwater has to withstand an earthquake of 9 Mw with a return period of 1/250 years.

Damage: Maintainable damage at the breakwater is acceptable, but failure is not.

6.1.3 Operational Boundary Conditions

- Maximum occurring Hs for tugboats is 1.5 m
- Maximum wave height at turning circle is 1.0 m
- Maximum wave height at moored barges is 0.5 m

The terminal will stop operating when waves > 1.5 meter will occur, the wave conditions are set as follows:

H_s 1.5 m
T 8 - 18 s
Direction 210 – 240° (as seen from the north)

6.1.4 Design life

Traditional Breakwater: The design life of the traditional breakwater design is set to 50 years. During these 50 years maintenance is accepted.

Alternative Breakwater: The design life of the alternative breakwater designs has to be investigated. The alternative breakwater designs have to be maintenance free.

7 APPENDIX II: TIME SCHEDULE

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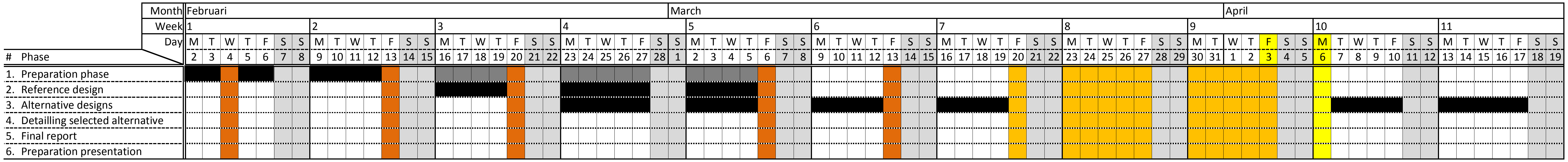
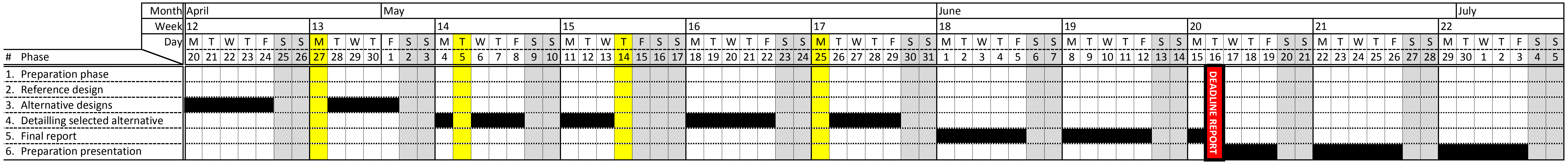
Time Schedule Project Innovative Breakwater
16-6-2015



midterm presentation in week 7 or week 10



Legend

and	work in progress
	Weekend, not available at the office
	Journey to Asia
	Dutch Hollidays, not available at the office
	School related abcense, not available at the office, project will proceed. (No info after 27-4)
	Deadline report

Danny Janssen
Jesper van Grieken

Appendix 2: Basis of Design

Innovative Breakwater

16 June 2015

Final Report

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Document title	Appendix 2: Basis of Design
Document short title	Basis of Design
Status	Final Report
Date	16 June 2015
Project name	Innovative Breakwater

Drafted by	Jesper van Grieken, Danny Janssen	
Checked by	Ronald Stive	
Date/initials check	………………….	………………….
Approved by	Harry Dommershuijzen	
Date/initials approval	………………….	………………….

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SUMMARY

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater.

For this reason, financially feasible alternative breakwater concepts have to be developed. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference, a traditional breakwater solution will be developed as well, i.e. the Reference Design.

The purpose of this document is to determine the different boundary conditions which could occur at the set location. Thereby, the minimal required performances of the reference breakwater and the alternative breakwater concepts have been determined. After describing the required performances, some material properties will be described. The basis of design will be completed with an enumeration of the used design codes, standards and guidelines.

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CONTENTS Page

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1 INTRODUCTION

1.1 Background

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because of underestimating the severe environmental conditions, both during construction and operation, it was wrongly decided to omit a solution for the swell waves. Moreover, the project budget was too limited to build a traditional breakwater. The current estimated yearly downtime is between 80 and 90%, which makes the business case of the terminal not lucrative.

For this reason, a solution for the swell waves is needed. There are a lot of different solution directions but RHDHV is especially interested in alternative breakwater concepts to make the project financially feasible.

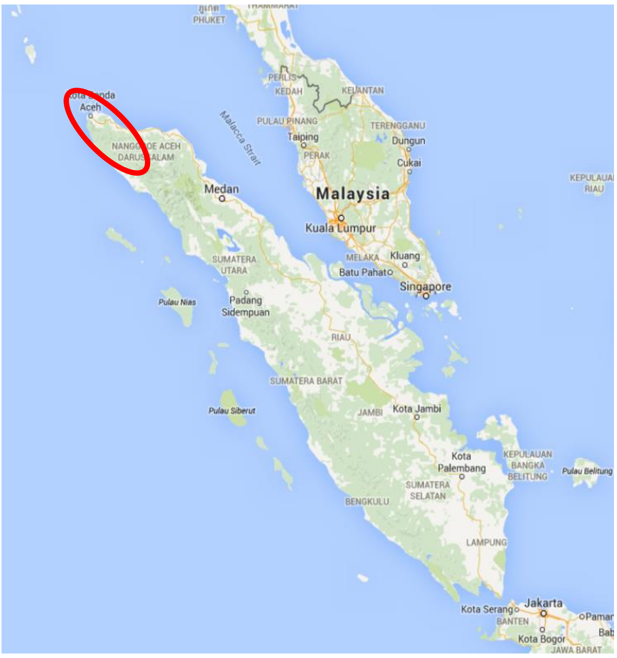

Figure 1: Location of the project.

In 2007, RHDHV investigated the boundary conditions of the project location which is situated at the NW coast of Sumatra facing the Indian Ocean. Due to political reasons, the client does not want the location to be specified in detail. Therefore, taking the actual RHDHV export terminal project as a starting point, a realistic schematisation of the BW layout/position and the surrounding seabed with actual design boundary conditions have been defined. This realistically schematised situation will be used for all study and design work related to the graduation project.

1.2 The project

The purpose of the project is to select and develop an alternative, low cost, non-traditional breakwater solution for a specified location. The desirable result is a recommendation for the technically and financially most attractive design concept. This recommended design will be detailed and optimized within the timeframe available.

2 PROJECT DESCRIPTION

2.1 Project setting and breakwater situation

The main purpose of the breakwater is to reduce the incoming waves approaching the terminal. The breakwater will be situated in front of the barge loading terminal. Within the feasibility study of RHDHV, the dimensions of the jetty and the loading terminal are specified. In figure 2 a schematic drawing of the terminal, including the area where the traditional breakwater will be located, can be seen. The entire jetty, including the terminal has a length of 850 meters, according to (DHV, 2011).

The simulated breakwater at figure 2 indicates the dimensions of the traditional breakwater. The alternative breakwater may be several times larger or smaller, depending on the alternative breakwater its setup. There is no limit determined for the dimensions of the breakwater, as long as the terminal is accessible for the barges.

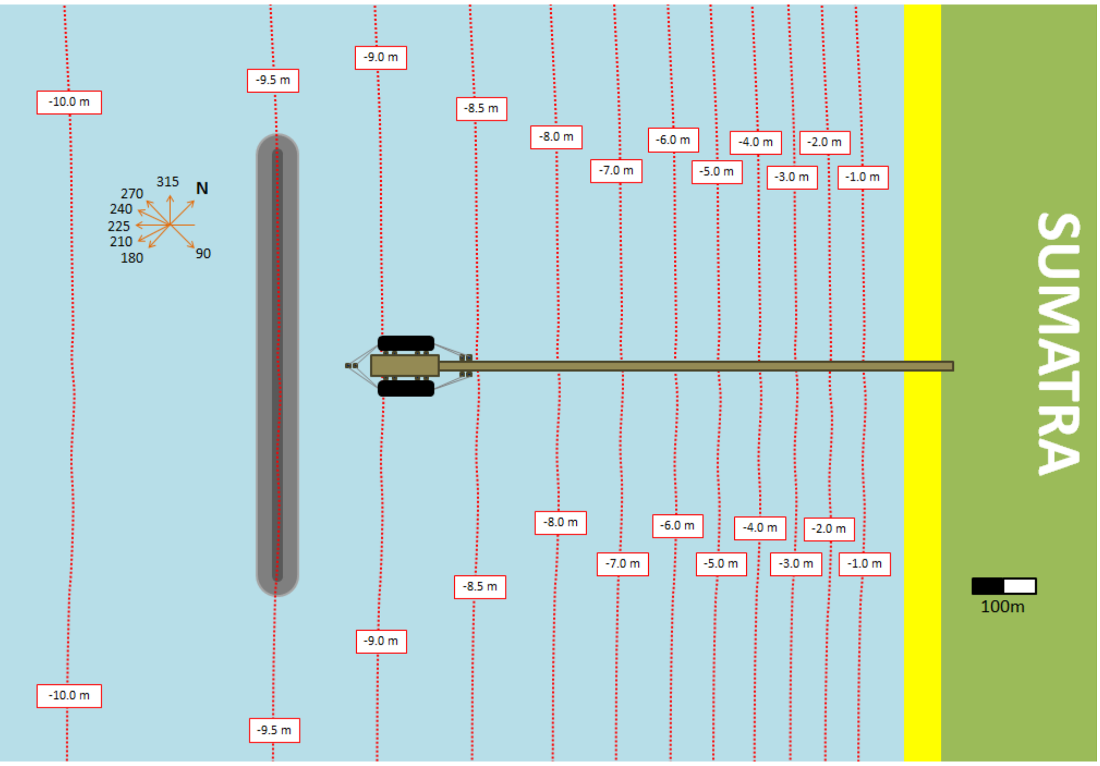

Figure 2: Schematic drawing of the project setting and breakwater situation

2.2 Vessel Characteristics

The terminal has enough capacity for two 8,000 DWT barges, one on each side of the terminal. The barges are ocean going and are propelled by tugs. The characteristics of the design barge can be seen in Table 1.

Barge characteristics	
Loading Capacity (ton) = DWT	8,000
Loaded Draft (m)	4.5
LOA (m)	91.44
Breadth (m)	24.38
Maximum lowering (m)	3.0
Under keel clearance (m)	1.0
Minimum water level (m)	8.5

Table 1: Vessel Dimensions (PT. Mitra Lingkungan Dutaconsult, 2013)

In order to convey the barges to the terminal, tugboats are required. The ocean going tugboats are approximately 30 meters long. Tugboats will stop operating, when the significant wave height appears to be 1.5 meters or higher (DHV, 2011). In order to safely convoy one ocean going barges, two tugboats are required.

2.3 Limiting operational conditions

2.3.1 Introduction

There are three limiting operational wave heights (H_s) specified for the terminal, these operational limits have been determined using several guidelines and assumption reports.

- If the waves at the ocean are higher than 1.5 meters, tugboats will stop operating (DHV, 2011).
- The maximum operational wave height in the turning circle is set to 1.0 meters. (PIANC, 2014).
- The maximum operational wave height at the mooring location is set to 0.5 meters (RHDHV, 2014).

2.3.2 Winds and Currents

The limiting operational wind speed for a terminal is equal to 22 m/s (Spanish guideline ROM 3.1-99, 2007). The yearly maximum wind speed is 14 m/s (paragraph 3.3), the maximum yearly occurring wind speed is lower than the limiting wind speed, so the limiting condition wind will not lead into additional downtime of the terminal.

The limiting operational current velocity is equal to 0.7 m/s in transversal direction and 1.5 m/s in longitude direction, applied for mooring and navigating of the barges (Spanish guideline ROM 3.1-99, 2007). The maximum spring tide current velocity is 0.1m/s (paragraph 3.7), so the limiting condition currents will not lead into additional downtime of the terminal.

2.4 Navigational Assumptions

The mooring process of the barges assisted by the tugs is described with the figures 3 until 9. These figures have been developed in order to give a good impression of the required navigational area for safe mooring. These figures have been developed based on an interview with (Stive, 2015).

In Figure the barge with the tugs is approaching the port on full speed, with a small angle compared to the direction of the coastline and breakwater.

When the barge is inside the protective area of the breakwater, the back tug starts to reduce the speed of the combination. As seen in Figure 4.

As soon as the speed of the combination allows it, the tugs reduce the length of the towing lines with their winches, and bring the whole combination to a stop. See figure 5.

The tugs pull the barge next to the terminal so it can be moored. See figure 6.

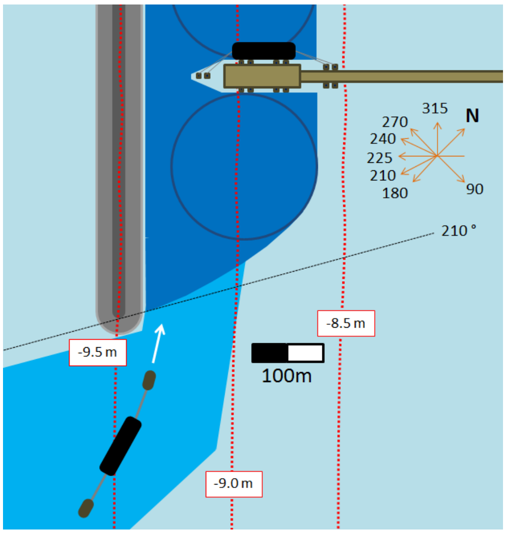

Figure 3: Approach of the barge, assisted by two tugs.

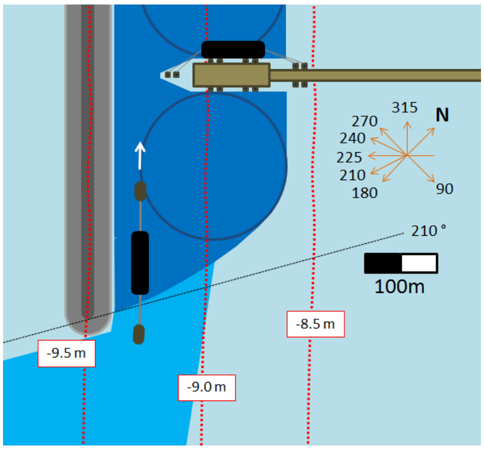

Figure 4: The back tug reduces the speed of the combination.

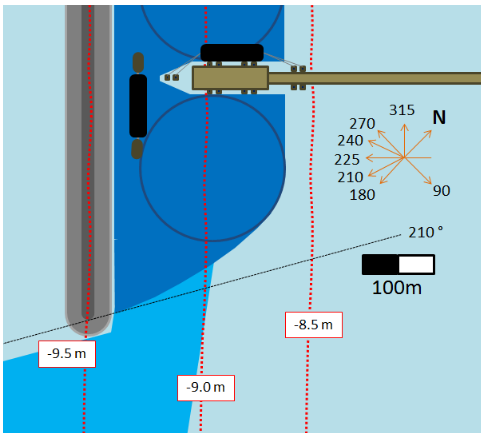

Figure 5: The combination has stopped.

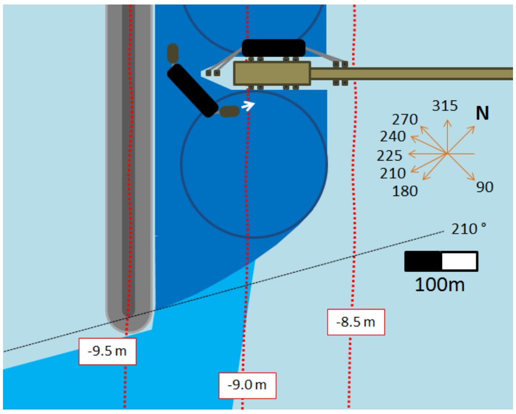

Figure 6: The barge is pulled next to the terminal.

When the barge is loaded, the tugs pull the barge away from the terminal, and they start turning in the navigation area. See figure 7. When the combination has turned towards the ocean, the combination accelerates to full speed and heads towards the open water see Figure 8 and Figure 9.

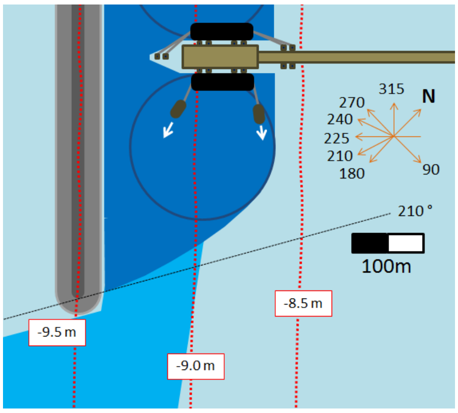

Figure 7: The tugs pull the barge away from the terminal

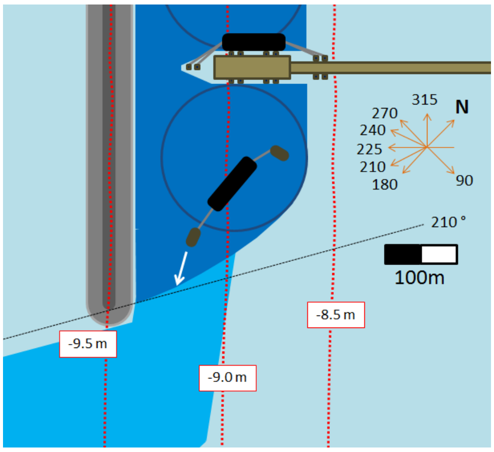

Figure 8: Barge leaving the mooring area

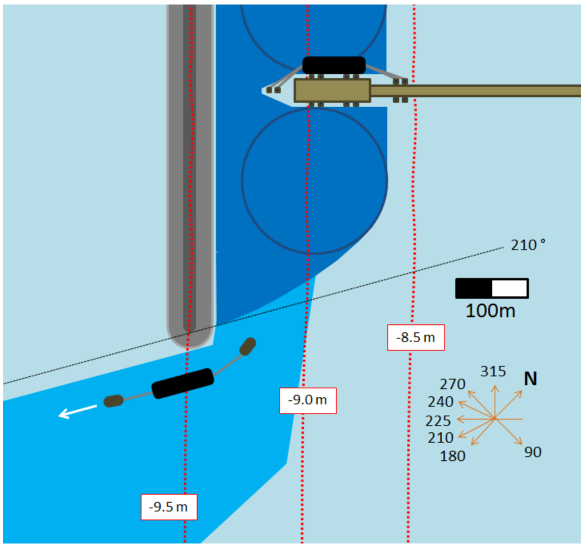

Figure 9: Barge leaving mooring area

3 ENVIROMENTAL CONDITIONS

3.1 Introduction

This chapter describes the environmental conditions which may occur at the specified project location. These environmental conditions have been defined using the feasibility study of port x (DHV, 2011) and by applying some internet searching for the conditions which have not been mentioned in the feasibility study.

3.2 General Climate

The climate on the project location is known as a tropical monsoon climate. This climate is characterized by heavy rainfall, high humidity, high temperature and low winds. These winds vary considerably depending on the season.

A monsoon is a seasonal prevailing wind which last for several months. The climate on the project location can be divided in two seasons: an East monsoon season and a West monsoon season. The East monsoon season lasts from October till April; the West monsoon season lasts from April till October.

There are some small differences between the two monsoon seasons. According to (Soberón, 2008), the differences regarding the rainfall, the humidity, the temperature and the winds, can be seen in Table 2.

	East monsoon season	West monsoon season
Average air temperature	27.1 °C	27.3 °C
Average minimum air temperature	20.1 °C	19.6 °C
Average maximum air temperature	31.9 °C	32.4 °C
Average humidity	82 %	81 %
Average monthly rainfall	80 mm/month	87 mm/month
Average number of days of rain per month	7 days/month	6 days/month
Average maximum rainfall	35 mm/day	39 mm/day
Average visibility during bad weather	6.3 to 11.1 km	6.1 to 10.3 km

Table 2: The differences between East monsoon season and West monsoon season on the project location (Soberón, 2008)

3.3 Wind

3.3.1 Introduction

Wind data from a wind station located approximately 230 km off the coast of the project location is available. On basis of this wind station the wind conditions for the project location are characterized as (DHV, 2011):

- Moderate winds for 50% of the time with wind speeds lower than 4 m/s.
- Wind speeds below 6 m/s for about 80% to 90% of the time.
- Wind speeds below 8 m/s for about 95% of the time.
- The maximum wind speed per year is 12 to 14 m/s.
- The winds are related to the monsoon system.

3.3.2 North East Monsoon

Table 3 and table 4 present the joint-probability tables of the wind station for respectively the North-East monsoon and the South-West monsoon:

Wind Speed [m/s]	Direction [°N]												Total
	-15 15	15 45	45 75	75 105	105 135	135 165	165 195	195 225	225 255	255 285	285 315	315 345	
< 2.0	2.1	1.6	1.1	2.1	2.2	1.9	2.2	1.9	2.5	3.9	2.6	2.7	**26.9**
2.0 : 4.0	3.0	2.1	1.5	3.1	3.3	2.8	3.1	2.6	3.6	5.9	3.8	3.8	**38.5**
4.0 : 6.0	3.4	1.2	0.8	0.8	1.0	1.3	2.0	0.7	2.0	3.9	2.5	4.8	**24.4**
6.0 : 8.0	0.8	0.5	0.1	0.1	0.1	0.3	0.5	0.4	0.9	2.0	0.9	1.0	**7.6**
8.0 : 10.0	0.1	0.1	-	-	-	-	0.1	-	0.4	0.5	0.2	0.2	**1.5**
10.0 : 12.0	-	-	-	-	-	-	-	-	0.1	0.3	0.2	0.3	**0.9**
12.0 : 14.0	-	-	-	-	-	-	-	-	-	-	-	0.1	**0.1**
> 14.0	-	-	-	-	-	-	-	-	-	-	-	-	**-**
Total	**9.3**	**5.5**	**3.5**	**6.2**	**6.6**	**6.4**	**7.8**	**5.5**	**9.5**	**16.4**	**10.2**	**12.9**	**100.0**

Table 3: Probability of occurrence of wind speed and direction (in %) at wind station for North-East monsoon (DHV, 2011)

3.3.3 South East Monsoon

Wind Speed [m/s]	Direction [°N]												Total
	-15 15	15 45	45 75	75 105	105 135	135 165	165 195	195 225	225 255	255 285	285 315	315 345	
< 2.0	0.7	0.3	0.5	1.0	1.5	2.0	1.8	1.9	3.0	2.8	1.4	1.3	**18.1**
2.0 : 4.0	1.2	0.5	0.9	2.1	2.9	4.0	3.5	3.8	5.8	5.7	2.8	2.6	**35.7**
4.0 : 6.0	1.0	0.2	0.2	1.2	2.0	4.4	4.1	2.4	4.8	4.7	1.3	1.7	**27.9**
6.0 : 8.0	0.1	0.1	0.3	0.2	0.7	2.3	1.2	1.2	2.4	2.8	1.0	0.3	**12.5**
8.0 : 10.0	-	-	-	0.2	0.1	0.1	0.2	0.4	0.8	1.6	0.5	0.2	**3.9**
10.0 : 12.0	-	-	-	0.1	-	-	-	0.2	0.2	0.8	0.2	-	**1.4**
12.0 : 14.0	-	-	-	-	-	-	-	-	0.1	0.1	-	-	**0.2**
> 14.0	-	-	-	-	-	-	-	-	0.1	-	-	-	**0.1**
Total	**3.0**	**1.0**	**1.8**	**4.8**	**7.1**	**12.7**	**10.8**	**9.7**	**17.0**	**18.5**	**7.2**	**6.1**	**100.0**

Table 4: Probability of occurrence of wind speed and direction (in %) at wind station for South-West monsoon (DHV, 2011)

3.4 Bathymetry

In 2012, DHV made a bathymetric map of the project location; this data has been implemented into a schematic side and top view, showing the different bathymetric lines. The water depth on the traditional breakwater location is approximately 9.5 meters relative to Chart Datum (CD). The chart datum is equal to the lowest water level. The water depth of the alternative breakwater location is dependent on the width of the breakwater. Figures 10 and 11 show the schematisations of the bathymetric map provided by DHV.

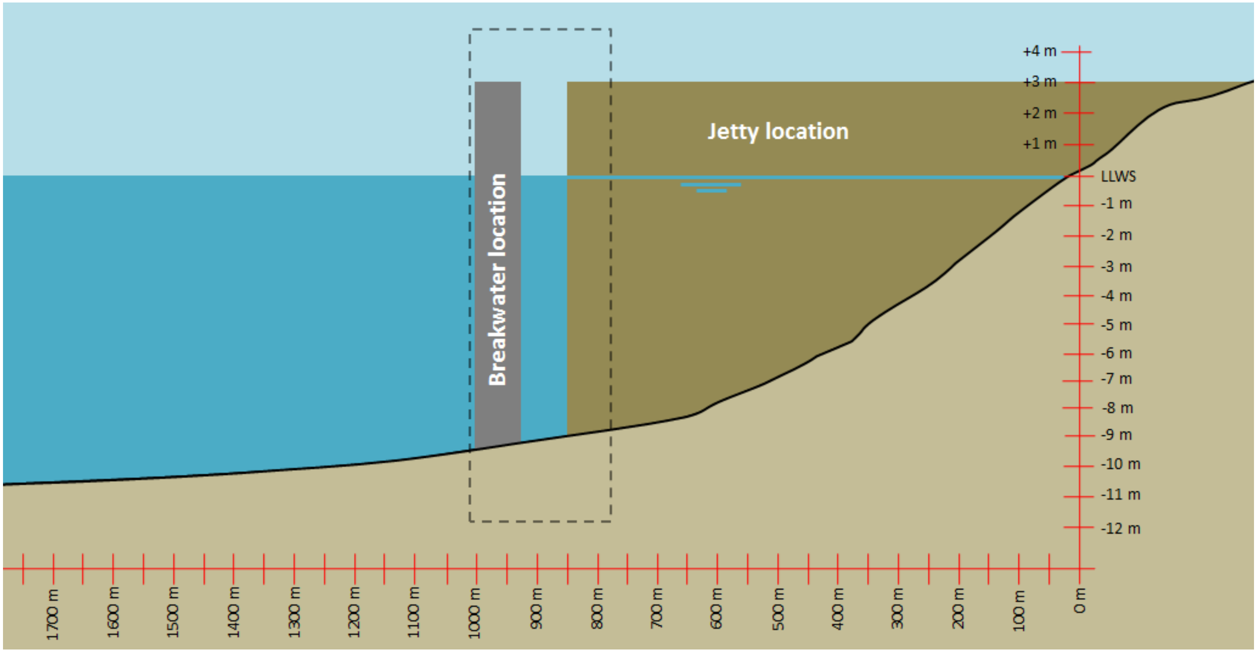

Figure 10: Bathymetric map, cross section of the jetty

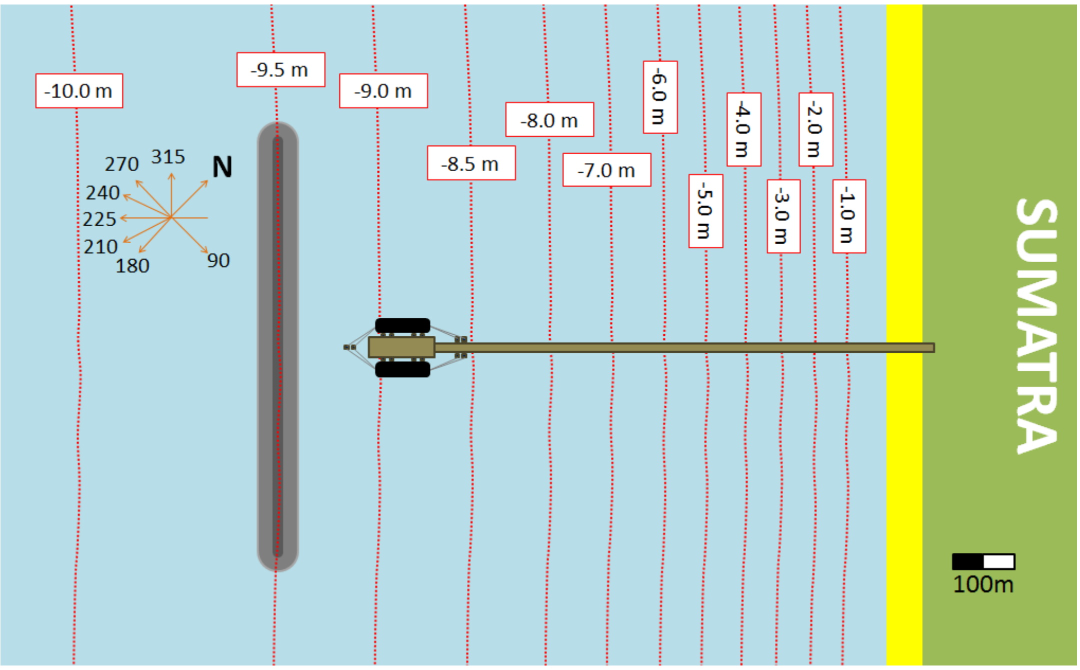

Figure 11: Plan view of bathymetry lines at project location

Figure 12 shows a schematisation of the cross section of the head of the terminal and the location of the breakwater (the dashed grey rectangular in figure 10).

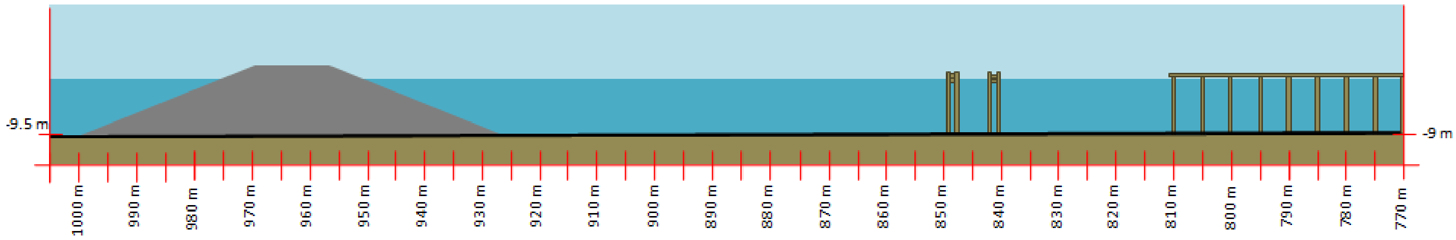

Figure 12: Cross section of the head of the terminal and the breakwater (dashed grey rectangular in figure 10)

According to the feasibility study (DHV, 2011), the seabed slope starting from the breakwater location (central depth -9.5 meters) going further towards the ocean is 1:600.

3.5 Water Levels

3.5.1 Tidal ranges

The water levels on the project location are given in table 5 (DHV, 2011).

LLWS	MLWS	MLWN	MHWN	MHWS	HHWS
-0.30 m	-0.25 m	-0.10 m	+0.10 m	+0.25 m	+0.45 m

Table 5: Computed tide levels (in m relative to MSL) at project location (DHV, 2011)

LLWS is set to Chart Datum (CD) so the tide levels can be translated to tide levels relative to CD, see table 6.

LLWS	MLWS	MLWN	MHWN	MHWS	HHWS
0.00 m	+0.05 m	+0.20 m	+0.40 m	+0.55 m	+0.75 m

Table 6: Computed tide levels (in m relative to CD) at project location

3.5.2 Surge

According to (Stive, 2015), a maximum occurring surge of 0.5 m can be expected at the project location. This surge will only occur during the extreme environmental conditions.

3.5.3 Sea-level rise

The sea-level is rising, but in some areas it is rising more than in others. According to (National Science Foundation, 2010) the sea-level rise is particularly high along the coastline of Sumatra, among others. The sea-level along the coasts of the northern Indian Ocean, have risen by an average of 10 mm/decade, according to (University of Colorado Boulder, 2010) and (United Nations Environment Programme (UNEP) Global Environmental Alert Service (GEAS), 2010).

However, this 10 mm/decade is an average. In some parts of the Indian Ocean the sea-level is rising more than in others. According to (Israel, 2010), the sea-level along the coast of Sumatra could rise up to 18 cm/century (18 mm/decade).

The University of Colorado (University of Colorado Boulder, 2010) and the United Nations Environment Programme (UNEP) Global Environmental Alert Service (GEAS) (United Nations Environment Programme (UNEP) Global Environmental Alert Service (GEAS), 2010) seem more reliable references than Livescience.com (Israel, 2010).

The normative sea-level rise used in the design processes is 10 mm/decade. This results in a total sea-level rise of 0.05m after a time of 50 years. Therefore, the applied sea-level rise for both the reference design as the alternative designs is set to 0.05 m.

3.5.4 Maximum water levels

The maximum water levels during the operational limits will be equal to the tidal ranges. The lowest operational water level is set to 9.5 meters as seen from the subsoil (water level is equal to CD) and the highest water level to 10.3 meters as seen from the subsoil (water level is equal to CD+HHWS+SLR).

The maximum water levels during the extreme environmental wave conditions are equal to the tidal ranges, including an addition for surge. The lowest operational water level is set to 9.5 meters as seen from the subsoil (water level is equal to CD) and the highest water level to 10.8 meters as seen from the subsoil (water level is equal to CD+HHWS+Surge+SLR).

3.6 Waves

3.6.1 Introduction

The available wave data consists of computed near shore wave conditions at a water depth of approximately 8 m. The data is from the period 1996 – 2005. The model boundaries have a resulting wave data set consisting of a time series with 6 hour interval and describes the wave parameters 'significant wave height', 'mean wave direction' and 'mean wave period'.

Yearly wave-roses are shown in figure 13 for several output locations where the wave conditions are calculated across the coast near the project location. The project location is between two output locations, w07 and w08. The wave-roses clearly show that the waves approach the shore perpendicular, from the southwest (225°). Location w07 is closest to the project location, therefore the data of location w07 will be used as input during this project. One bar of single wave-rose represents a directional sector with a bandwidth of 30°.

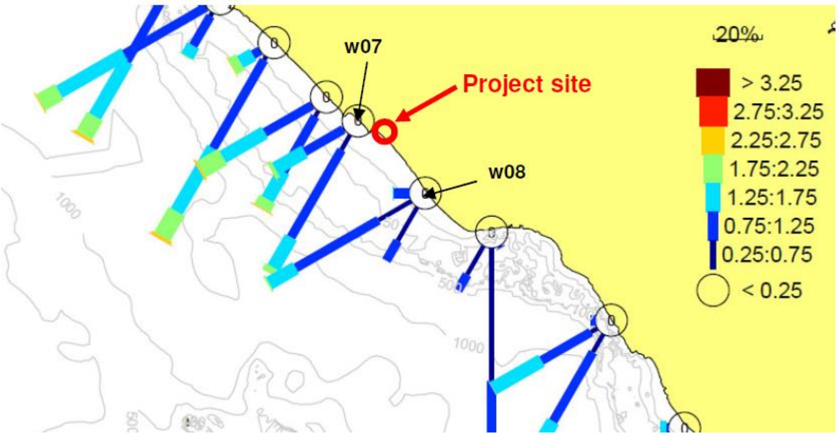

Figure 13: The yearly wave roses near the project location

3.6.2 General Waves

The local wave data obtained at location w07 is shown in table 7. Since the directional sectors have a bandwidth of 30° and all waves come from in two sectors, the total wave direction variation of the waves is 60°.

MWD (°) Hs (m)	0 – 195	195 – 225	225 – 255	255 – 360	Total (occurrence)
0.00 – 0.25	0.00	0.00	0.00	0.00	**0.00**
0.25 – 0.75	0.00	7.20	0.49	0.00	**7.69**
0.75 – 1.25	0.00	41.36	18.32	0.00	**59.68**
1.25 – 1.75	0.00	14.37	13.95	0.00	**28.32**
1.75 – 2.25	0.00	2.66	1.57	0.00	**4.23**
2.25 – 2.75	0.00	0.05	0.03	0.00	**0.08**
2.75 – 3.25	0.00	0.00	0.00	0.00	**0.00**
3.25 – 3.75	0.00	0.00	0.00	0.00	**0.00**
3.75 – 4.25	0.00	0.00	0.00	0.00	**0.00**
Total	**0.00**	**65.65**	**34.35**	**0.00**	**100.00**

Table 7: Joint probability of occurrence distribution of Direction and Wave heights at w07 for all seasons (DHV, 2011)

Computed wave periods indicate that the waves at the project location are typically swell waves. The wave period of 94% of the waves is between 6 and 16 seconds. About 76% of the waves have a wave period above 10 seconds and 24% of the waves have a wave period below 10 seconds. See table 8.

Tp (s) Hs (m)	0–2	2–4	4–6	6–8	8–10	10–12	12–14	14–16	16–18	18–35	Total
0 – 0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
0.25 – 0.75	0.00	0.02	0.08	0.19	1.69	3.28	1.87	0.42	0.10	0.04	**7.69**
0.75 – 1.25	0.00	0.02	0.84	6.02	12.59	20.31	14.99	3.88	0.95	0.08	**56.68**
1.25 – 1.75	0.00	0.00	0.04	0.44	2.24	3.50	13.21	6.63	2.05	0.20	**28.32**
1.75 – 2.25	0.00	0.00	0.01	0.01	0.27	0.24	0.74	1.79	1.07	0.10	**4.23**
2.25 – 2.75	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.00	0.05	0.00	**0.08**
2.75 – 3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.25 – 3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.75 – 4.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
Total	**0.00**	**0.04**	**0.96**	**6.67**	**16.80**	**27.33**	**30.82**	**12.72**	**4.22**	**0.42**	**100.00**

Table 8: All year joint probability of occurrence distribution of Wave periods and Wave height at location w07 (DHV, 2011)

Table 8 shows that almost all waves with a wave height of 1.25 to 1.75 meter have a period between 8 and 18 seconds, with an average period of approximately 13 a 14 seconds. This wave condition will be taken into account when describing the operational wave conditions.

Wave height Hs	PoE (%)		
	All year	SW-monsoon (May – October)	NE-monsoon (November – April)
1.00 m	60 %	80 %	20 %
1.25 m	33 %	60 %	4 %
1.50 m	13 %	30 %	0.6 %
1.75 m	5 %	8.5 %	0 %

Table 9: Approximate probability (%) of significant wave height (Hs) exceedance (DHV, 2011)

The limiting operational wave height for the tugboats at the ocean is equal to 1.5 meter (paragraph 2.3). Therefore, the occurring wave height for the service limit state (SLS) is set to 1.5 meters. It can be concluded that the yearly downtime of the jetty is approximately 13% of the time (see table 9) when a breakwater is applied and thereby the other causes of downtime are eliminated.

Table 10 shows the estimated probability of exceedance of wave height per directional sector.

Wave height Hs	Probability of exceedance for Sector 195 - 225	Probability of exceedance for Sector 225 - 255
1.00 m	32 %	24 %
1.25 m	17.1 %	15.6 %
1.50 m	8 %	6 %

Table 10: The estimated probability (%) of exceedance of wave height (Hs) per directional sector (DHV, 2011)

3.6.3 Extreme nearshore wave conditions

The extreme nearshore wave conditions for the project location are shown in table 11. The values are computed for different return periods (RP).

RP: 1/1 per year			RP: 1/25 year			RP: 1/100 year		
Hs	Tp	Dir	Hs	Tp	Dir	Hs	Tp	Dir
2.9	16.9	228.0	3.2	16.9	228.3	**3.3**	**18.6**	**228.2**

Table 11: Design wave conditions for two locations near the project location (DHV, 2011).

The significant design wave height for 1/100 year conditions is 3.3 m with a peak period of 18.6 seconds. These wave conditions will be taken into account when describing the extreme wave conditions.

3.6.4 Tsunami

The tsunami of December 26th 2004 is one of the largest tsunami in recorded human history. In deep water, the first two waves travelled with a speed of 500 to 1000 km/h, had a height of approximately 50 cm and were spaced approximately 200 km apart. Besides, tsunami waves have a very deep reach, 4000 m or more and a typical wave period of about 12 minutes (McKee, 2005).

When the tsunami reached the coast the wave height was increased to up to 30m. This caused run-ups reaching 50 m a.s.l. and 6 km inward, according to (Paris, Lavigne, Wassimer, & Sartohadi, 2007).

According to the Australian Government, Bureau of Meteorology (Australian Government, Bureau of Meteorology, 2015), the wave height of the December 26th 2004 tsunami was 10-15 m when reaching the Sumatran shore.

The characteristics of the occurring tsunamis are summarised in table 12:

Characteristic	Symbol	Value	Unit
Wave velocity	c	500 – 1000	km/h
Wave height on deep water	H_0	0.5	m
Wave length	L_0	200,000	m
Wave reach (depth)		4000	m
Wave period	T_p	12 720	min sec
Wave height on the Sumatra shore	H_s	15	m

Table 12: Tsunami wave characteristics

Because the time available for this graduation process is limited, the designs will not be designed for tsunami loading. It is advised to check the stability of the different designs under tsunami loading in the future.

3.6.5 Calculating T_m and $T_{m-1,0}$

When calculating an offshore breakwater construction, different wave periods will be taken into account for the different calculations. The values of T_m and $T_{m-1,0}$ can be determined as follows:

T_m is the mean wave period and can be estimated from T_p with the formula (DNV, 2010):

$$T_m = \frac{T_p}{1.4049} \times 1.0867$$

$T_{m-1,0}$ is the mean energy wave period and can be estimated with the formula (CIRIA; CUR; CETMEF, 2007):

$$T_p = 1.1 \times T_{m-1,0}$$

3.7 Currents

According to (DHV, 2011) the flow directions during high tide are from South-East to North-West. During low tide, flow directions are from North-West to South-East. The maximum spring tide velocities are about 0.05 m/s to 0.1 m/s.
Shore-parallel residual ocean currents may develop in both Northerly as Southerly directions. The velocity of these longshore currents has a maximum of about 0.1 m/s. Due to the wind, longshore currents may develop in the breaker zone of the coastline. Because the wind speeds are rather small, it is expected that the wind-driven currents are limited. See figure 14.

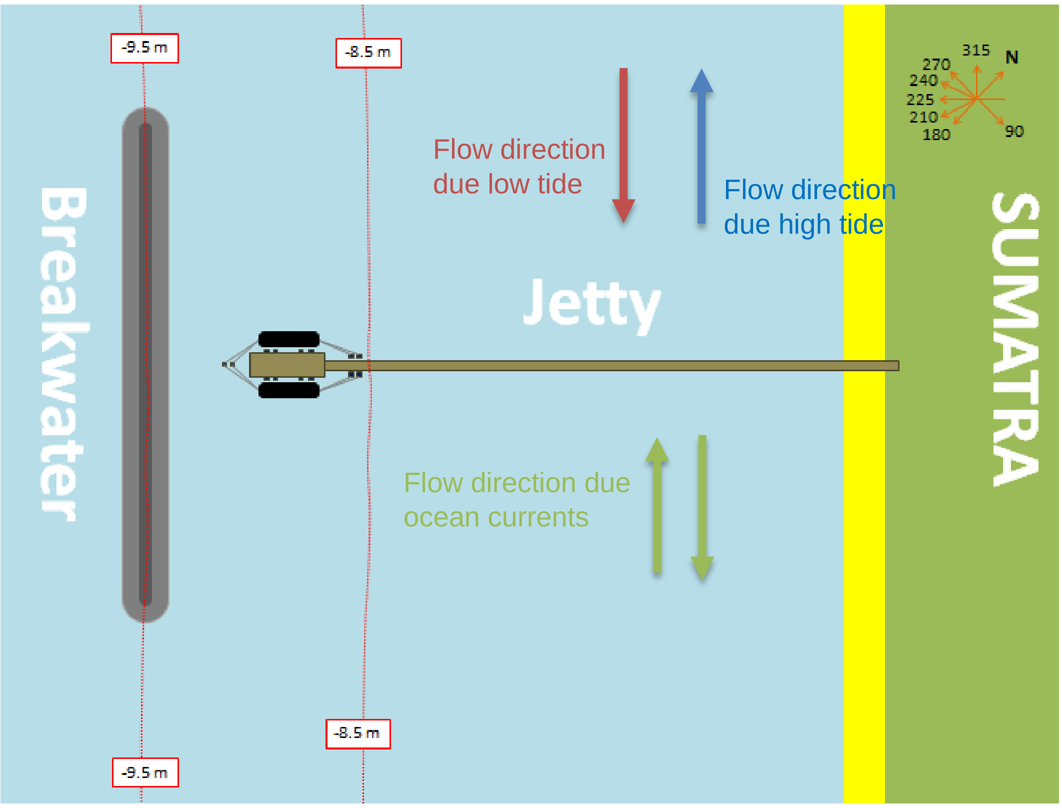

Figure 14: Direction of the currents

3.8 Geo-technics

3.8.1 Introduction

A drilling is executed near the project location (in a range of 1 km). The soil conditions of two locations, just a few meters apart, can differ much. However, there is no further information available about the soil conditions at the project location, so this drilling is used for determining the soil conditions at the project location.

3.8.2 Seabed

According to figure 15 the seabed consists of loose, medium to fine grained sand.

3.8.3 Subsoil

Figure 15 shows a schematization of the soil conditions.

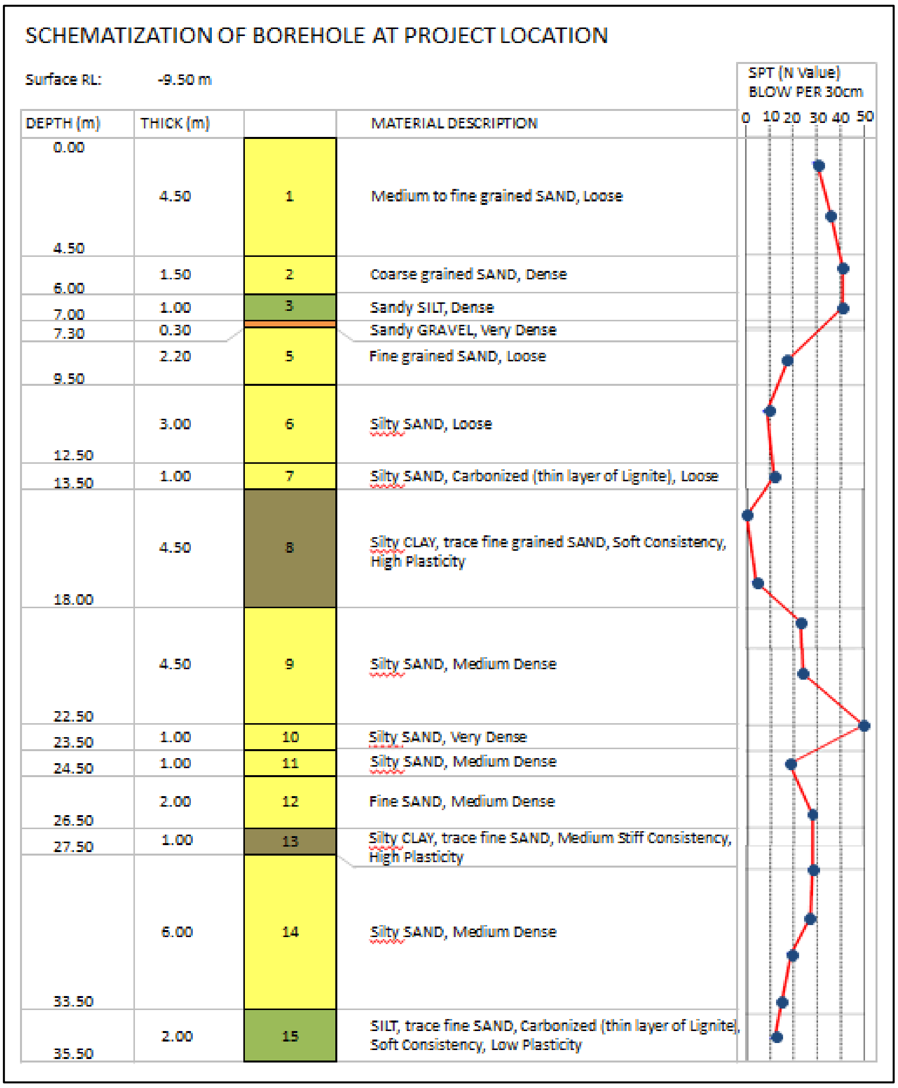

Figure 15: Schematization of a borehole at the project location

The number of blows of the sledge hammer needed for a penetration of 300 mm of the drilling sample tube is counted and provides the N-value. Therefore, the N-value provides an indication of the density of the ground.

3.8.4 Seismological Conditions

There is a lot of seismologic activity around the project location. This is because the project location is between the Sumatra fault and the subduction zone, also known as the Andaman Fault, see figure 16. In the subduction zone the Australian tectonic plate is pushed underneath the Eurasian tectonic plate, which causes a lot of tension in the subsoil. When the tension becomes too high an earthquake happens. The highest earthquake measured till today is the Sumatra Andaman Islands quake from December 26th 2004, which had a magnitude of 9.2.

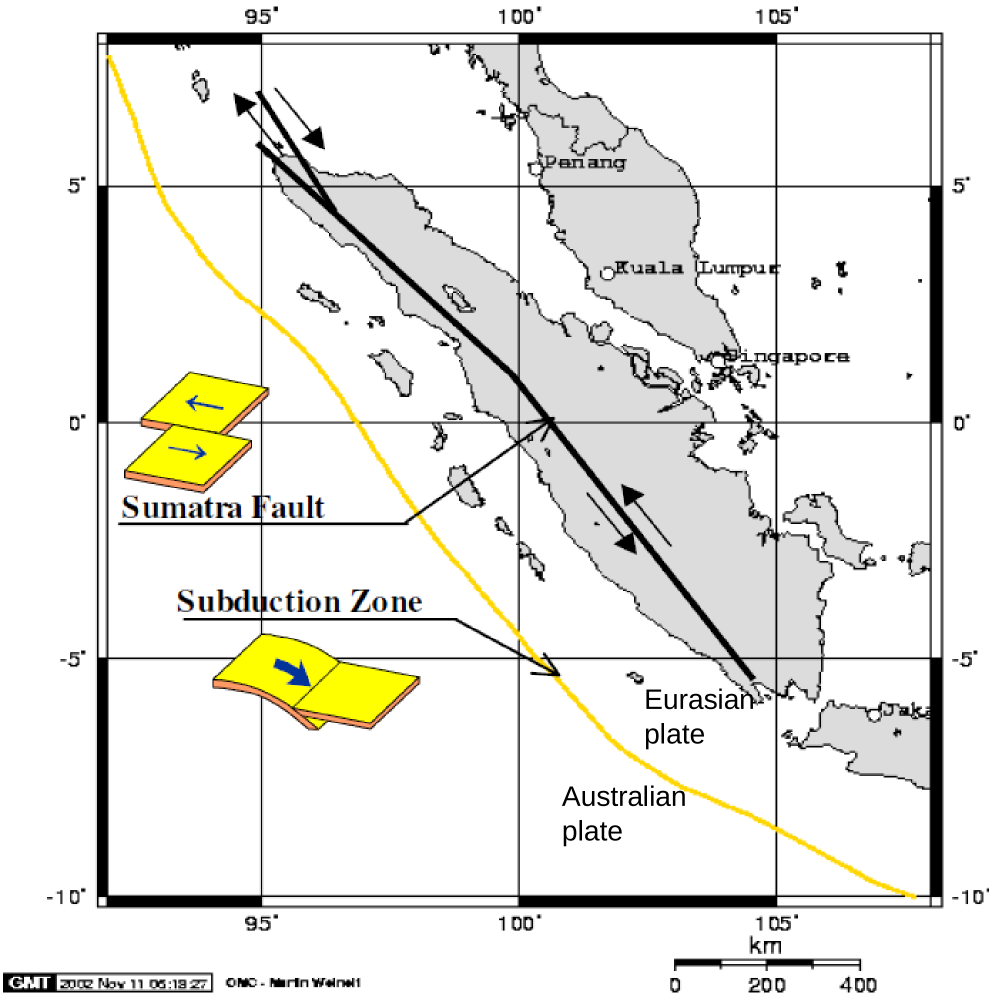

Figure 16: Tectonic setting of Sumatra, the subduction zone is the Andaman Fault (13th World Conference on Earthquake Engineering, 2004)

The project location is categorized as zone 5-6, with horizontal peak ground accelerations of 0.25-0.30 g. This earthquake has a return period of 500 years. A 9 Mw earthquake along the Andaman Fault has a return period of 250-500 year.

When calculating the earthquake resistance of a breakwater, besides the horizontal earthquake load, a vertical earthquake load has to be applied, according to (Asmerom, 2015). This vertical load is 50% of the horizontal earthquake load and has to be applied in both directions, this means that the value (of 0.15g) is applied both negative as positive.

3.9 Salinity sea water

The salinity of the sea water at the project location is approximately 32 to 32.5 psu (Practical Salinity Unit) according to (Weiqing & McCreary Jr., January 15, 2001). Practical Salanity Unit is equal to Parts Per Tonne.

According to (Qu, Du, Strachan, Meyers, & Slingo, 2005), the average sea surface temperature at the project location is 30°C.

With an online water density calculator (CSGNetwork, 2011), designed and programmed with the help of Dr. Barry Klinger of the George Mason University, the density of the seawater can be determined. This online water density calculator uses an equation described in (Millero, Chen, Bradshaw, & Schleicher, 1980). The density of the seawater is approximately 1020 kg/m^3.

3.10 Gravitational acceleration

The gravitational acceleration varies slightly on the earth. Near the equator it is slightly lower than near the poles. The gravitational acceleration on the project location is 9.78 m/s^2, according to (Elizabeth Chesick).

3.11 Morphology

3.11.1 Longshore sediment transport

Because the incoming swell waves at the project location are almost unidirectional and come in almost perpendicular to the coastline, there is a lot of uncertainty about the net longshore sediment transport. The direction of the incoming waves to the coast's normal is just a few degrees and is, depending on the prevailing season, either northward or southward. Therefore, the potential sediment transport in a particular direction is large, but it is very difficult to determine the net longshore sediment transport. The net longshore sediment transport on the project location is estimated to vary between 100,000 to 500,000 m^3/year. The equilibrium cross-shore profile will be reshaped due to land subsidence and sea level rise. The retreat of the shoreline is estimated to be some 20 to 70 m in the coming decades (DHV, 2011).

In table 13, some values of the coastal orientation and local morphology are shown.

Coastal Orientation (Degrees to North)	Equilibrium Coastline orientation (Degrees to North)	Difference between coastline orientation and equilibrium orientation (Degrees)	Average net yearly sediment transport (* 1,000 m^3/year)	
			D50 = 150 µm	D50 = 600 µm
226	222	4	280	100

Table 13: Morphological Data at project location (DHV, 2011)

3.11.2 Shoreline impact of the external impacts

Due to external influences close to the project location, the shoreline is eroding where the jetty reaches the coast.

Due to a breakwater, the wave energy that induces the longshore sediment transport is reduced. This means that both northwards and southwards sediment transport is reduced, so a large part of the yearly sediment transport will be deposited behind the breakwater. There is a lot of uncertainty in the sediment transport information available, so the exact amount of yearly deposition rates is very difficult to determine. However, there are a couple of rules of thumb that can predict the pattern of deposition at the shoreline. The shoreline behind the breakwater is moving seawards and shoreline erosion is expected at both sides of the breakwater. The shoreline may either form a bulge-like pattern or may advance until it reaches the breakwater itself. These two phenomena are called respectively a salient and a tombolo (see figure 17)

The forming of a salient has a positive effect on the erosion of the shoreline due to the breakwater rather close by to the project location.

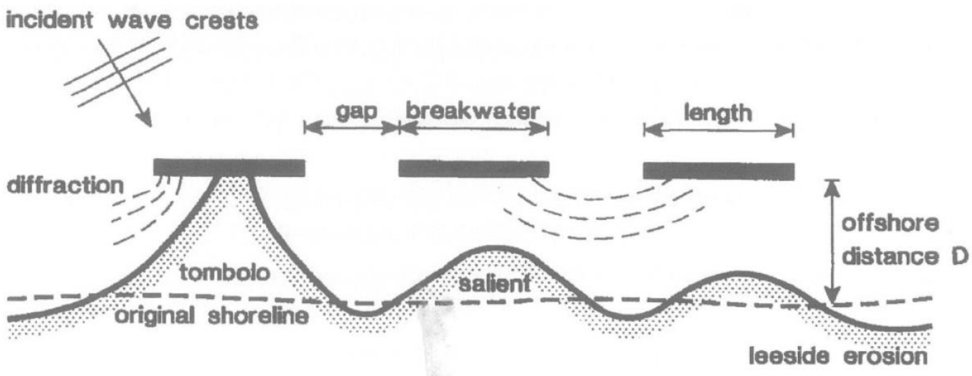

Figure 17: Salient or tombolo forming behind an offshore breakwater (DHV, 2011)

Because the terminal is behind the breakwater, tombolo forming should be avoided, so the fairway does not have to be dredged due to the erosion of sediment. According to (DHV, 2011), the following rules of thumb apply for this:

L = breakwater length
D = distance from the shoreline

L / D > 3:
A permanent tombolo is expected to develop along the shoreline, possibly already in a couple of years due to the large potential sediment transports at the project location. This means that a situation of insufficient navigation depth behind the breakwater is reached within years time.

L / D = 2 to 3:
A well-developed salient and/ or periodic tombolo is expected to form, depending on the prevailing (storm) conditions. This is also not a favourable situation.

L / D = 1 to 2:
A well-developed salient is likely to form behind the breakwater. The formation of a salient will shift the entire shoreline as well as the underwater profile in front of the salient seaward, meaning that the areas where the navigation channel and turning channel are planned will become shallower as well.

L / D = 0.5 to 1:
A weak to well-developed salient is expected to form. Although to a lesser degree, the same holds as above.

L / D = 0.2 – 0.5:
An incipient to weak salient is expected. This might be tolerated depending on the harbour configuration.

L / D < 0.2:
No significant effects on the shoreline are expected.

The distance of the breakwater from the shoreline is minimum 1150 m. The length of the breakwater is estimated to be approximately 850 m. L/D = 1150/850 = 1.35. This means that a weak to well-developed salient is expected to form. According to (DHV, 2011), a well-developed salient is expected to form. This means that the mooring and navigational area will become shallower. How shallow the mooring and navigational area will become is unknown.

3.12 Ecological

3.12.1 Sea life

There is no specific boundary determined for protecting the sea life beneath the breakwater location. It is assumed that there is no issue for the sea life, since the breakwater has to protect a jetty, which has already been constructed. This jetty already influences the sea life at the project location.

3.12.2 Noise

There is no specific boundary determined for the criteria noise during the installation of the breakwater. When much piling is required due to a foundation, this piling has to be performed in compliance with the local flora and fauna laws and regulations.

4 BREAKWATER PERFORMANCE

4.1 Introduction

In order to reduce the downtime of the terminal, some possible breakwater solutions will be investigated. Applying a breakwater in front of the terminal is not the only solution to reduce the downtime of the terminal. Several other solutions might solve this problem as well. These solutions will not be taken into account, because the client explicitly requested a breakwater in order to reduce the downtime.

In this chapter, the breakwater requirements have been determined further. These requirements are based on the feasibility study of port x (DHV, 2011), nautical guidelines (PIANC, 2014), the rock manual (CIRIA; CUR; CETMEF, 2007)(only the traditional breakwater design) and the wishes of the contractor (Stive, 2015).

4.2 Default dimensions

4.2.1 Location

The breakwater will be situated in front of the terminal, parallel to the coast, as seen in figure 18.

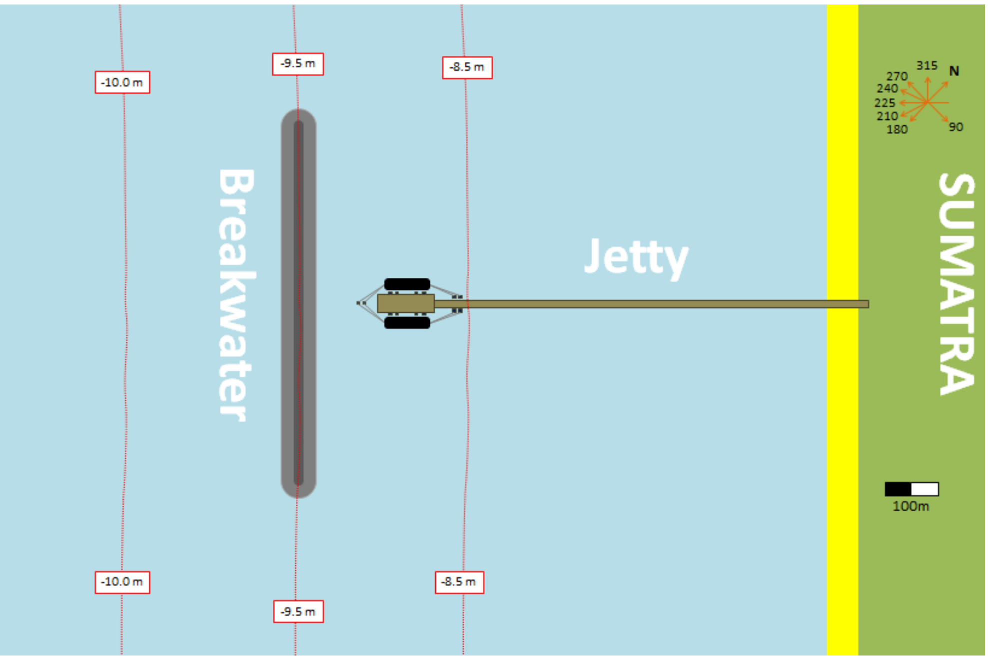

Figure 18: Schematic drawing of the project setting and breakwater situation

Applying a breakwater solution parallel to the jetty will result into two very long breakwaters, one at each side of the jetty. Thereby, most of the incoming waves are approaching from the head of the jetty, with an angle of incidence varying with 30 degrees to both sides. So incoming waves from the head will not be reduced using parallel breakwaters, without applying a breakwater parallel to the coastline. Since currents do not cause any downtime of the terminal, they don't have to be reduced. For this reason, applying a breakwater parallel to the jetty will not be investigated further.

4.2.2 Length

The length of the breakwater will be determined using diffraction models. If the maximum allowed wave height behind the breakwater and the wave reduction values for both the overtopping and transmission are known, the required diffraction coefficient can be determined. When varying with the length of the breakwater, different diffraction coefficients can be found. There are no boundaries determined which restricts in the maximum length of the breakwater. However, longer breakwaters may be more expensive due to more use of materials and a longer construction time. The most important boundary for this project is the costs, so it is preferred to reduce the length of the breakwater as much as possible. More information of the breakwater length is to be found in chapter 4.3, operational requirements.

4.2.3 Width

There is no boundary condition determined which boundaries the maximum allowed breakwater width, including eventual underwater slopes. According to the contractor, the available breakwater area towards the ocean is unlimited, starting from a water depth of 9.5 meters regarding CD with a slope of 1:600, see chapter 3.4, bathymetry.

4.2.4 Orientation

The breakwater is situated parallel to the coastline. The main reason for applying the breakwater parallel to the coastline are the quite constant approaching waves, perpendicular to the coastline, varying with 30 degrees to both the left and the right direction. The breakwater has to protect the terminal from incoming waves from both directions, so the breakwater will be designed parallel to the coastline.

4.2.5 Breakwater Heads

The breakwater heads are exposed to higher forces than the adjacent cross sections of the breakwater. To obtain the same stability for the breakwater heads as the cross section, two options are available (only applicable for the reference design). These options may be combined. The two options are:

- Apply larger units with a higher mass/density.
- Reduce the slope of the side.

First, the option of applying larger armour stones will be taken into account. When there are no larger armour stones available, the breakwater slope will be designed less steep. It is assumed that applying a slope less steep is more expensive than applying a bigger armour stone, due to more use of materials. This is only applicable for the rubble mound traditional breakwater. For a design using concrete armour units, applying a less steep slope is desired above applying larger concrete armour units, due to the interlocking effect which only occurs under a certain breakwater slope.

The breakwater heads of the alternative constructions have to withstand the extreme environmental conditions as well. Hereby, the erosion of the subsoil next to the heads of the construction has to be checked. Instability of the construction due to erosion of the subsoil is not allowed. The behavior of the waves near the breakwater heads has to be investigated looking at the 3D-effect near the breakwater heads.

Due to lack of time during the graduation process, breakwater heads have not been investigated and optimized. During the process, only the cross section of the trunk for both the reference design as the alternative concepts has been taken into account. During further research, the dimensions of the breakwater heads have to be determined.

4.2.6 Slope of the breakwater

The slope of the traditional breakwater will be designed as steep as possible, so the required material will be reduced as much as possible. If the slope turns out to be unstable, a less steep slope will be applied.

The slope of the alternative breakwater construction is dependent of its shape. The slopes of an alternative breakwater concept may be vertical. So the slopes of the alternative breakwater constructions have to be determined separately per alternative.

4.3 Operational requirements

4.3.1 Introduction

The combined wave height at the lee side of the breakwater is dependent of three occurring phenomena, the diffraction of the waves, the overtopping of waves and the wave transmission. Diffraction can be described as waves flowing around the breakwater. Overtopping can be described as waves flowing over the crest of the breakwater. Wave transmission can be described as waves flowing through the breakwater.

The significant incoming wave height for operational limits is set to 1.5 meters. If waves above the 1.5 meters will occur (DHV, 2011), tugboat assistance is not possible. The bollard forces at a moored barge will become too high when a wave height above 0.5 meters occurs (RHDHV, 2014). The maximum operational wave height at the turning circle is set to 1.0 meter (PIANC, 2014). In figure 19, the maximum allowed wave heights, for the operational limits, are schematized.

The dimensions of the turning circle (240 meters) and the minimum required distance between the jetty and the breakwaters toe (40 meters) have been determined based on the advice of a nautical expert of RHDHV.

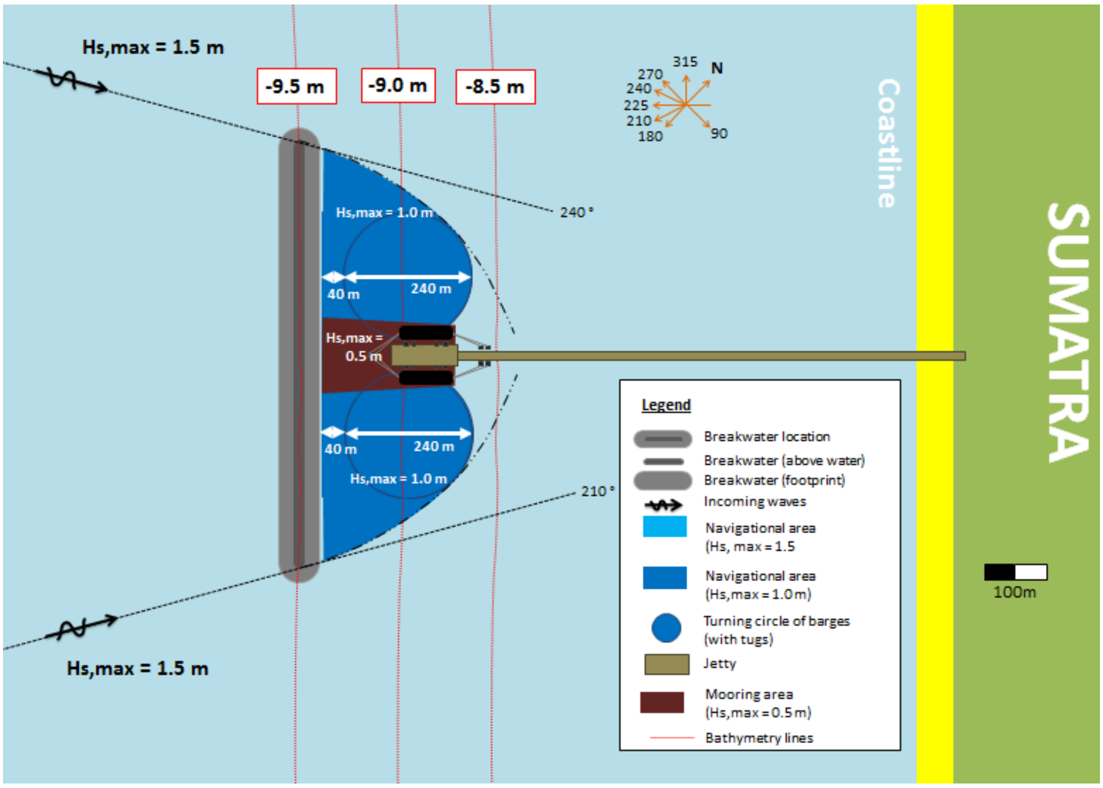

Figure 19: Wave Height al Leeside Breakwater

The combined wave height at the lee side of the breakwater can be determined as follows, the energy at the lee side of the breakwater is equal to the energy of the three independent phenomena:

$$E_c = E_D + E_T + E_o$$

Where:
E_c = Combined Wave energy
E_D = Wave energy due to diffraction
E_T = Wave energy due to transmission
E_o = Wave energy due to overtopping

The wave energy is equal to:

$$E = \frac{1}{8} \times \rho \times g \times {H_s}^2$$

The combined wave height at the lee side of the breakwater can be determined as:

$$H_s{}^c = \sqrt{(H_s{}^D)^2 + (H_s{}^T)^2 + (H_s{}^O)^2}$$

Where:

$H_s{}^c$ = The combined wave height at the lee side of the breakwater.
$H_s{}^D$= Wave height at lee side of the breakwater due to diffraction.
$H_s{}^T$= Wave height at lee side of the breakwater due to transmission.
$H_s{}^O$= Wave height at lee side of the breakwater due to overtopping.

The wave height at the lee side of the breakwater due to diffraction, transmission and overtopping are determined by multiplying the incoming wave height by the reduction factor of the three phenomena:

$$H_s{}^{D,T,O} = H_s{}^{in} \times C_{D,T,O}$$

The H_s has to be determined for each of the phenomena independently. In here C is the reduction coefficient, which has to be determined for all of the phenomena.

4.3.2 Normative daily wave direction

According to the general wave data, mentioned in chapter 3.7.2., all the waves are approaching between 195 and 255 degrees, as seen from the north. According to the feasibility study of port x (DHV, 2011), a smaller bandwidth may be applied because the extreme wave condition almost approaches perpendicular to the coast. For this reason, the normative daily wave direction has been set to 210 and 225 degrees, as seen from the north, a total bandwidth of 30 degrees.

4.3.3 Overtopping

For the design of the reference breakwater, the influence of overtopping waves at the lee side of the breakwater is determined to be zero (C_o = 0), during the operational wave conditions. The specific discharge over the breakwater will be calculated using the CUR rock manual (CIRIA; CUR; CETMEF, 2007) (chapter 5.1.1.3).
A distinction between the operational wave conditions (operational limits) and the extreme wave conditions will be made (lee side armour). The C_o coefficient is set to 0 in order to reduce the total breakwater length (a small overtopping rate will result into a smaller required diffraction coefficient and thus a longer breakwater).

The value of overtopping for the alternative breakwater designs are not specified yet. It depends on the crest height of the alternative. There are no requirements to the maximum allowed wave overtopping, as long as the maximum allowed wave height at the lee side of the breakwater is not exceeded.

4.3.4 Wave transmission

The value of wave transmission depends on the porosity of the breakwater. The more water can flow through the breakwater, the higher the factor of wave transmission. The value of wave transmission is, besides the porosity of the breakwater, dependent of the crest with of the breakwater.

For the innovative breakwater, it may be possible that the wave transmission is nihil, because of an impermeable alternative breakwater concept. For this reason, the transmission coefficient has to be determined independently per alternative. There are no requirements to the maximum allowed wave transmission, as long as the maximum allowed wave height at the lee side of the breakwater is not exceeded.

4.3.5 Diffraction

The diffraction determines the length of the breakwater. After determining the transmission through and the overtopping over the breakwater, the maximum allowable diffraction around the breakwater and the required diffraction coefficient can be determined. This required diffraction coefficient results into a minimum required length of the breakwater.

The diffraction for both the reference and the innovative breakwater will be calculated using the method of Sommerfeld. This method will give a reprehensive value of the diffraction coefficients.

For more accurate values of the diffraction coefficient, a numerical wave model has to be made. The available time during the graduation project is limited, for this reason the numerical wave model will not be performed.

4.3.6 Transmission-length diagram

With the theory of Sommerfeld a transmission-length diagram is made. This diagram is shown in appendix I. This diagram can only be used with the boundary conditions:

- The overtopping over the breakwater is nihil. Each alternative which is not low crested will be calculated with an overtopping coefficient equal to 0. Since the overtopping coefficient is hard to determine using non-numerical calculation methods.
- The centre line of the breakwater lies 280 meters away from the farthest point of the mooring area. Based a turning circle with a diameter of 240 meters and a mooring area between the jetty and the breakwater its toe of 40 meters (see chapter 4.3.1.)
- The breakwater is mirrored over the centre line of the Jetty (the length of the breakwater is equal on both sides of the centre line of the Jetty).Based on the uniform separated wave distribution.

In chapter 5 of the reference design, the length of the reference breakwater is determined with the theory of Sommerfeld. The transmission-length diagram is made using the same calculation steps as described in the reference design.

4.4 Damage

4.4.1 Wave boundary conditions

Each breakwater has to withstand the extreme wave conditions, which have been determined as:

A significant wave height of 3.3 meters and a peak period of 18.6 seconds approaching from 228.2° (as seen from the north). The designs storm has a return period of 1/100 years (Chapter 3.7.3.).

In order to simplify the calculations, the angle of incidence is set to 225°. The waves will approach the breakwater perpendicular. Moreover, it is assumed that the wave force reduction caused by the small angle of incidence is nihil.

4.4.2 Geotechnical boundary conditions

The geotechnical and seismological stability of the traditional breakwater designs will be checked, using simulation software from Deltares. The traditional breakwater designs are tested on the settlements of the subsoil, the stability of the slopes and earthquake resistance.

The settlements of the subsoil have to be compensated by increasing the crest height of the traditional breakwater(s).

To calculate the stability of the slopes, no loads other than the own weight of the breakwater has to be applied. To calculate the earthquake resistance of the breakwater, a horizontal earthquake load of 0.3g and a vertical earthquake load of 0.15g have to be applied in combination with the own weight of the breakwater. The vertical earthquake load has to be applied in both directions (positive and negative).

When calculating the stability of the slopes and the earthquake resistance, a safety has to be implemented. In order to implement this safety, the total unit weight of the soil materials have to be reduced by a factor of 1.2, (Asmerom, 2015). With this safety implemented the models have to score a safety factor of 1.0 in the models to pass the requirements.

The alternative breakwater design will be checked for the geotechnical and seismological stabilities, if there is any time left during the graduation process.

4.4.3 Damage reference design

The traditional breakwater is designed for the most common failure mechanisms, based on the loads occurring at the normative storm. In figure 20 the failure mechanisms of the traditional (rubble mound, rock armour) breakwater can be seen. During the reference design phase, different kinds of traditional breakwater designs will be investigated. Their failure mechanisms are broadly the same as the rubble mound rock armour breakwater, but some differences can occur, depending on the type of breakwater. The breakwater has to withstand the storm at the most disadvantageous water level, based on the failure mechanism.

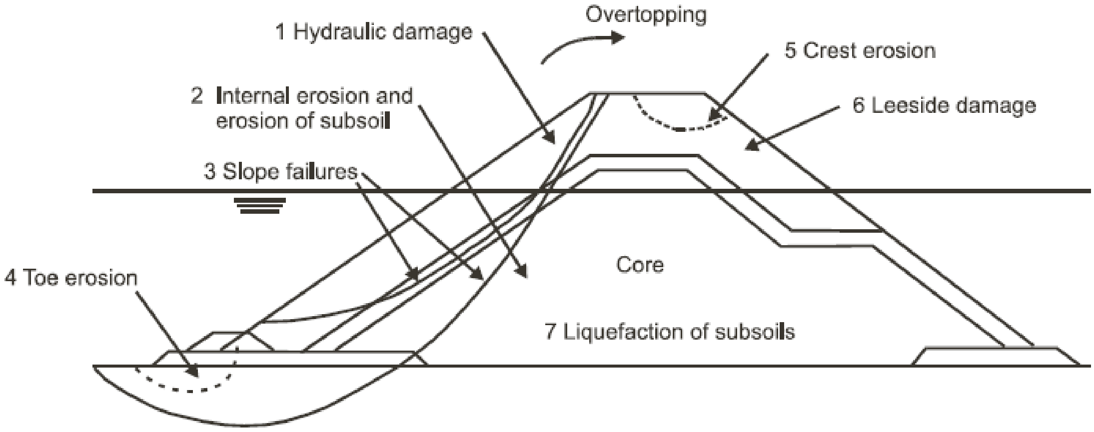

Figure 20: Failure Mechanisms Breakwater (CIRIA; CUR; CETMEF, 2007)

Damage parameter rubble mount
The allowed damage level for the traditional rubble mound double layer breakwater, determined by the client, is repairable damage. Conform the Rock Manual (CIRIA; CUR; CETMEF, 2007); the design parameter is dependent of the slope of the crest and the allowed damage level. Using the van der Meer method for deep water conditions, the damage parameters shown in table 14 are determined. This parameter is applied for the traditional, rubble mound breakwater.

Slope (Cotα)	Damage Level		
	Start of Damage	Intermediate Damage	Failure
1.5	2	3-5	8
2	2	4-6	8
3	2	6-9	12
4	3	8-12	17
6	3	8-12	17

Table 14: Damage Parameter, S_d (CIRIA; CUR; CETMEF, 2007)

Since the slope of the breakwater is not determined yet, the damage parameter cannot be demined. The maximum allowed damage level parameter will be set between the level start of damage and intermediate damage. Applying this damage level will result into repairs after the normative storm, but these reparations are accepted.

Damage parameter concrete armour units

When designing a breakwater with an armour layer of concrete armour units, the van de Meer formula is not applicable. Therefore, the Hudson formula, including the stability coefficient K_D, will be taken into account for the design calculation of the concrete armour units. In the reference design, only the concrete armour unit xBloc is taken into account. This unit can achieve very steep slopes. There is no damage allowed when using the xBloc.

The K_D factor for the xBloc is 16 in the trunk section and 13 in the head section (CIRIA; CUR; CETMEF, 2007).

4.4.4 Damage alternative concepts

Failure of the breakwater during the extreme wave and earthquake conditions is not allowed. If the breakwater construction incidentally fails, the jetty construction should not be damaged. For this reason, the breakwater construction is not allowed to slide towards the jetty construction. Thereby, loose separated parts of the breakwater construction may not be able to damage the jetty either. The boundary condition according the damage at the jetty is set by the client. The client made this boundary condition, since investment costs of the jetty where quite high.

4.5 Design lifetime

4.5.1 Traditional design lifetime

The design lifetime of the rubble mount breakwater is 50 years and has to survive a normative storm which occurs 1/100 years, which is a common lifetime for traditional breakwaters. Repairable damage is acceptable due to a storm of this kind.

4.5.2 Alternative design lifetime

The minimal required lifetime of a breakwater alternative is set to 10 years. This required lifetime is shorter than the lifetime of the reference breakwater. The main purpose of the project is to invent a low-cost breakwater alternative, even if the lifetime of the breakwater will be reduced by this costs saving. The breakwater with a reduced lifetime can be applied on project locations which require a temporary breakwater protection. During the lifetime of the innovative breakwater, the client is able to save enough money for an eventual rebuild.   To save extra money during the lifetime of the breakwater, no maintenance is allowed

4.6 Breakwater costs

The costs for the traditional breakwater design may be unlimited, since the main purpose of the reference design is to obtain a clear picture of the points of particular interest when designing a breakwater. The costs of the traditional breakwater will be utilized to schematise the savings of the alternative design compared with a traditional breakwater design.

The maximum budget for the alternative breakwater design is set to € 25,000,000.-. This is the maximum amount of money which the client can spare.

5 CONSTRUCTION MATERIALS

5.1 Introduction

This chapter describes some relevant information about the materials used for both the traditional and the innovative breakwater.

5.2 Rock

The minimum rock density for primary armour rock is taken as 2650 kg/m^3. Conform the Rock Manual (CIRIA; CUR; CETMEF, 2007); the standard rock grades are determined at the following values, see table 15.

	Class Designation	ELL	NLL	NUL	EUL	M_{em}	
Heavy	Passing requirements kg	< 5% kg	< 10% kg	> 70% kg	> 97% kg	Lower limit kg	Upper limit kg
	10,000-15,000	6,500	10,000	15,000	22,500	12,000	13,000
	6,000-10,000	4,000	6,000	10,000	15,000	7,500	8,500
	3,000-6,000	2,000	3,000	6,000	9,000	4,200	4,800
	1,000-3,000	700	1,000	3,000	4,500	1,700	2,100
	300-1,000	200	300	1,000	1,500	540	690

	Class Designation	ELL	NLL	NUL	EUL	M_{em}	
Light	Passing requirements kg	< 2% kg	< 10% kg	> 70% kg	> 97% kg	Lower limit kg	Upper limit kg
	60-300	30	60	300	450	130	190
	10-60	2	10	60	120	20	35
	40-200	15	40	200	300	80	120
	5-40	1,5	5	40	80	10	20
	15-300	3	15	300	450	45	135

	Class Designation	ELL	NLL	NUL	EUL	M_{em}
Coarse	Passing requirements	< 5% mm	< 15% mm	> 90% mm	> 98% mm	< 50 % mm
	45/125	22.4	45	125	180	63
	63/180	31.5	63	180	250	90
	90/250	45	90	250	360	125
	45/180	22.4	45	180	250	63
	90/180	45	90	180	250	NA

Table 15: Heavy, Light and Coarse standard grading requirements (CIRIA; CUR; CETMEF, 2007)

Where:
ELL = Extreme Lower Limit
NLL = Nominal Lower Limit
NUL = Nominal Upper Limit
EUL = Extreme Upper Limit

5.3 Sand

The relative density of water-saturated sand is set to 19 kN/m^3 (Blok, 2006). If more material properties of the material sand are required, in order to optimize a breakwater design, the material will be added in that specific design calculation.

5.4 Cement and aggregates

There are a lot of variables when designing a construction consisting of cement and aggregates. The determination of the required material strength will be applied in an eventual further research. The cement and aggregates do have to acquire a certain concrete cover in order to protect the rebar from corrosion.

5.5 Steel

The quality of the steel will be determined during an optimisation process, if the alternative analysis shows a steel solution. In order to determine the lifetime of a steel construction, some information of the corrosion rate is required. According to the British standards, the corrosion rates for steel constructions in temperate climates are set as follows (table 16):

Exposure zone	Corrosion rate mm/side/year	
	Mean	Upper limit
Atmospheric zone	0.04	0.10
Splash zone	0.08	0.17
Tidal zone	0.04	0.10
Intertidal low water zone	0.08	0.17
Continuous seawater immersion zone	0.04	0.13

Table 16: Corrosion rate of steel in different exposure zones (British Standards, 2000)

The mean rate is for each exposed to the environment of the zone.
The upper limit figures are the 95% probability values (5% of the corrosion exceeds this corrosion rate).

5.6 Wood

When applying wood into a certain breakwater construction, the wood requires a certain sustainability criteria, in order to guarantee a certain lifetime of the construction. If wood is constructed into a wet and dry environment, the wood will start purifying without a good sustainability criteria. The wood which will be applied into any further details will be determined in that specific document.

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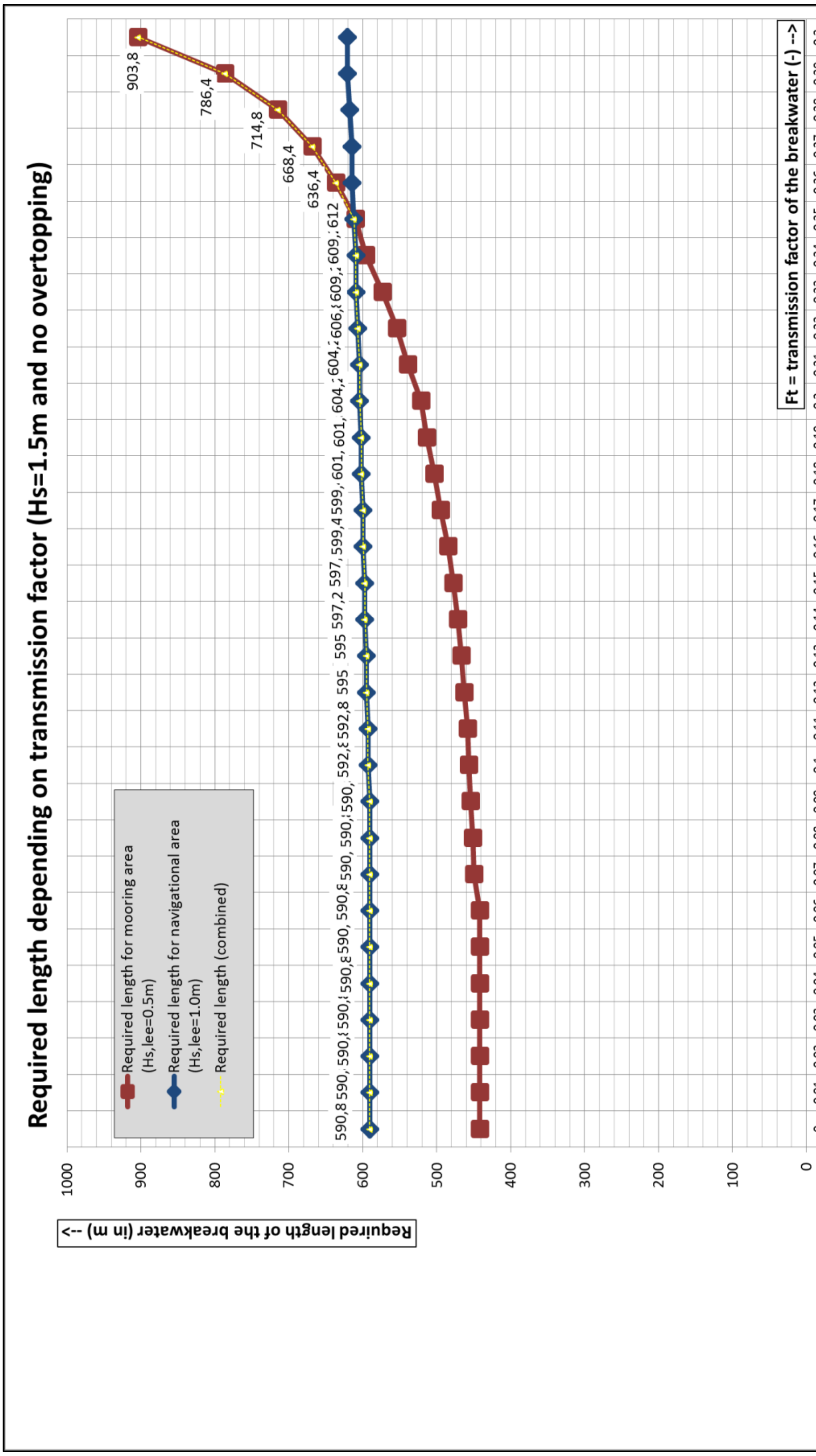
7 APPENDIX I: TRANSMISSION LENGTH DIAGRAM



Ft	0	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09	0,1	0,11	0,12	0,13	0,14	0,15
Required length for mooring area (Hs,lee=0.5m)	441,6	441,6	441,6	441,6	441,6	441,6	441,6	449,4	451	454	457	458,4	462,8	467	471,2	477,8
Required length for navigational area (Hs,lee=1.0m)	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	592,8	592,8	595	595	597,2	597,2
Required length (combined)	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	590,8	592,8	592,8	595	595	597,2	597,2

Ft	0,16	0,17	0,18	0,19	0,2	0,21	0,22	0,23	0,24	0,25	0,26	0,27	0,28	0,29	0,3
Required length for mooring area (Hs,lee=0.5m)	484,4	494,4	503	513,4	521,6	539	553,6	573,4	596	609,6	636,4	668,4	714,8	786,4	903,8
Required length for navigational area (Hs,lee=1.0m)	599,4	599,4	601,8	601,8	604,2	604,2	606,8	609,2	609,2	612	614,8	614,8	617,8	621	621
Required length (combined)	599,4	599,4	601,8	601,8	604,2	604,2	606,8	609,2	609,2	612	636,4	668,4	714,8	786,4	903,8

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Appendix 3: Reference Design

Innovative Breakwater

16 June 2015

Final Report

ROTTERDAM UNIVERSITY

UNIVERSITY OF APPLIED SCIENCES

Royal HaskoningDHV

Enhancing Society Together

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Document title	Appendix 3: Reference Design
Document short title	Reference Design
Status	Final Report
Date	16 June 2015
Project name	Innovative Breakwater

Drafted by	Jesper van Grieken, Danny Janssen	
Checked by	Ronald Stive	
Date/initials check	………………….	………………….
Approved by	Harry Dommershuijzen	
Date/initials approval	………………….	………………….

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SUMMARY

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater.

For this reason, financially feasible alternative breakwater concepts have to be developed. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference, a traditional breakwater solution will be developed as well, i.e. the Reference Design.

The purpose of this document is to create a traditional rubble mound breakwater design for the project location, i.e. the Reference Design. Starting point for the design are the boundary conditions described in the Basis of Design. The designs described in this document are preliminary designs. However, the designs will not be optimized further, because it is not the main focus of the graduation project. The main focus of this Reference Design is to get used to the design methods, learn the different aspects involved with offshore breakwaters and roughly determine the costs of a traditional breakwater for the specific situation described in the Basis of Design. These costs will be used as reference costs in the Multi-Criteria-Analysis.

CONTENTS

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1 INTRODUCTION

The purpose of a breakwater is to reduce the wave height of incoming waves so a safe environment where ships can safely moor is provided. In this document traditional breakwaters will be designed for the boundary conditions described in the Basis of Design.

Two types of traditional rubble mound breakwater designs will be made: a rubble mound breakwater with a rock armour layer and a rubble mound breakwater with concrete armour layer. The following aspects of the breakwaters will be designed, described and checked:

- The ocean side armour stones and filter layers
- The lee side armour stones and filter layers
- The crest height and width
- The toe constructions on both the ocean side as the lee side of the breakwater
- The required length
    - Transmission
    - Overtopping
    - Diffraction
- The construction costs of the designs
- The geotechnical and seismological stability
    - Settlements of the subsoil
    - Stability of the slopes
    - Seismological resistance
- The construction method of the breakwater

2 ROCK ARMOUR

2.1 Introduction

A breakwater consisting of a rock armour layer is the most common breakwater solution. The rock armour breakwater consists of a rock armour layer, a smaller graded filter layer and a core of quarry run. The dimensions of these different layers will be determined in this chapter. Thereby, the remaining dimensions of the breakwater, like the toe dimensions and the crest height and width will also be determined in this chapter.

2.2 Armour Stones

2.2.1 Introduction

The stone size of the primary armour layer of the breakwater will be determined using the formulae of van der Meer. When calculating the primary armour layer, the Hudson formula is also applicable. The difference between the van der Meer formulae and the Hudson formulae is the complexity of both formulas. The formula of van der Meer includes the effects of storm duration, wave period, the structure's permeability and a clearly defined damage level. For this reason, the formula of van der Meer will be taken into account for the calculation of the rock armour layer of the rubble mound breakwater instead of Hudson's formula.

For the determination of the armour stone units, the normative storm conditions will be taken into account. The boundary conditions, as a result of these normative storm conditions, are shown in table 1.

Boundary	Description	Value
H_s	Significant wave height	3.3 m
T_p	Peak wave period	18.6 s
T_m	Mean wave period	14.4.s
α	Structure slope angle	26.5° (Slope 1:2)
g	Gravitational acceleration	9.81 m/s^2
ρ_{water}	Specific density seawater	1020 kg/m^3
ρ_{stone}	Specific density stones	2650 kg/m^3
S_{0m}	Fictitious wave steepness mean wave period	0.01
ξ_m	Surf similarity parameter (mean wave)	4.95
T_{storm}	Duration normative storm	6 hours

Table 1: Boundary conditions for determining the rock armour layer

The wave steepness (s_{0m}) and the surf similarity parameter (ξ_m) can be determined with (CIRIA; CUR; CETMEF, 2007):

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

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

To reduce the use of material the structure slope is determined as steep as possible (1:2).

2.2.2 Critical wave conditions

The van der Meer method for determining the armour stone stability is separated into plunging and surging wave conditions. Plunging waves will occur when $\xi_m < \xi_{cr}$ and surging waves when $\xi_m \geq \xi_{cr}$. The critical surf similarity parameter (ξ_{cr}) is determined with (CIRIA; CUR; CETMEF, 2007):

$$\xi_{cr} = (\frac{c_{pl}}{c_s} \times P^{0.31} \times \sqrt{\tan\alpha})^{\frac{1}{P+0.5}}$$

Where:

C_{pl}	Empirical coefficient (No requirement for failure)	6.2
C_s	Empirical coefficient (No requirement for failure)	1.0
P	Permeability structure (Armour layer and filter layer)	0.4
α	Angle of crest (1:2)	26.5°

$$\xi_{cr} = 3.77$$

$\xi_m \geq \xi_{cr}$ (4.95 ≥ 3.77),surging waves will occur.

Therefore, the formula of van der Meer is equal to (CIRIA; CUR; CETMEF, 2007):

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

It is assumed that the most unfavourable load case will occur when considering the turning point between plunging and surging waves (ξ_{cr}), instead of the peak wave conditions. For this reason, a second load case is determined with: ξ_{cr}=ξ_m. (CIRIA; CUR; CETMEF, 2007)

$$\xi_{cr} = \frac{\tan\alpha}{\sqrt{\frac{2\pi \times H_s}{g \times T_{m,cr}{}^2}}}$$

In order to calculate the critical wave period, the formula can be rewritten to:

$$T_{m,cr} = \sqrt{\frac{2\pi \times H_s \times \xi_{cr}{}^2}{g \times \tan^2\alpha}}$$

Where:

$T_{m,cr}$	Critical mean wave period	[s]
ξ_{cr}	Critical surf similarity parameter	3.77
H_s	Significant wave height	3.3 m
α	Angle of crest (1:2)	26.5°
g	Gravitational acceleration	9.78 m/s^2

$$T_{m,cr} = 10.97\ s$$

The critical peak period is equal to:

$$T_{p,cr} = 14.19s$$

More than 50% of the waves during the average yearly wave conditions have a peak wave period between the 10 and 14 seconds, regardless the wave height. So, it is assumed that a wave with a peak period of 14.19 seconds and a significant wave height of 3.3 meter are able to occur at the project location.

The armour stone dimensions will be calculated using both the peak wave conditions and the critical wave conditions. In order to visualise the difference of these boundary conditions, both conditions will be calculated independently.

2.2.3 Armour layer

The armour stone dimensions will be calculated using both the critical wave period and the mean wave period, using the formula of van der Meer for surging waves (CIRIA; CUR; CETMEF, 2007):

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

Peak wave period

Where:

D_{n50}	Median nominal diameter	Variable (m)
H_s	Significant wave height	3.3 m
Δ	Relative buoyant density $\left(\frac{\rho_{stone}}{\rho_{wat}}\right) - 1$	1.6
C_s	Empirical coefficient surging waves (No requirement failure)	1.0
P	Permeability structure (Armour layer and filter layer)	0.4
S_d	Damage parameter (Start of damage)	2
N	Number of waves at toe $\left(\frac{Storm\ duration\ (6\ hr)}{T_m\ (14.4\ s)}\right)$	1501
ξ_m	Surf similarity parameter (mean wave)	4.95
α	Angle of crest (1:2)	26.5°

According to the above equation (for the normative storm conditions),

$$D_{n50} = 1.23\ m$$

Critical wave period

Where:

D_{n50}	Median nominal diameter	Variable (m)
H_s	Significant wave height	3.3 m
Δ	Relative buoyant density $\left(\frac{\rho_{stone}}{\rho_{wat}}\right) - 1$	1.598
C_s	Empirical coefficient surging waves (No requirement failure)	1.0
P	Permeability structure (Armour layer and filter layer)	0.4
S_d	Damage parameter (Start of damage)	2
N	Number of waves at to $\left(\frac{Storm\ duration\ (6\ hr)}{T_{m,cr}\ (10.95\ s)}\right)$	1971
$\xi_{m,cr}$	Critical surf similarity parameter (mean wave)	3.77
α	Angle of crest (1:2)	26.5°

$$D_{n50} = 1.42\ m$$

Calculating the D_{n50} for both the peak wave period and the critical wave period, it can be concluded that the most unfavourable wave condition is the critical wave condition. This situation results into the required rock armour stone size. The minimum required median diameter of the rock armour stone is, $D_{n50} = 1.42\ m$.

The required M_{50} (mass of particle for which 50% of the granular material is lighter), can be calculated using the following equation (CIRIA; CUR; CETMEF, 2007):

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

Where:

ρ_{stone}	Specific density of stone	2650 kg/m^3
D_{n50}	Median nominal diameter	1.42 m

$$M_{50} = 7587\ kg$$

For this average weight, a standard gradation of 6,000-10,000 kg is required. The nominal diameter of this gradation is $D_{n50} = 1.45\ m$. In practice, these armour stones are not easy to mine from a quarry, it will result in a huge amount of quarry run when producing this gradation, because many of the stones will not achieve the minimal required gradation size. For this reason, a lighter armour stone gradation is desired. The only variable in the van der Meer formula is the crest slope. For this reason a crest slope of 1:3 will be applied.

Slope 1:3

Using the same formula as described above, using the following parameters:

Boundary	Description	Value
H_s	Significant wave height	3.3 m
T_p	Peak wave period	18.6 s
T_m	Mean wave period	14.4.s
α	Structure slope angle	18.4° (Slope 1:3)
g	Gravitational acceleration	9.78 m/s^2
ρ_{water}	Specific density seawater	1020 kg/m^3
ρ_{stone}	Specific density stones	2650 kg/m^3
T_{storm}	Duration normative storm	6 hours
ξ_{cr}	Critical surf similarity parameter	3.0
$T_{m,cr}$	Critical mean wave period	13.1 s
N	Number of waves at toe $\left(\frac{Storm\ duration\ (6\ hr)}{T_m\ (13.1\ s)}\right)$	1644

Table 2: Boundary conditions slope 1:3

Using the following formulae with the above parameters (CIRIA; CUR; CETMEF, 2007):

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

$$D_{n50} = 1.24\ m$$

The required M_{50} (mass of particle for which 50% of the granular material is lighter), can be calculated using the following equation (CIRIA; CUR; CETMEF, 2007):

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

Where:

ρ_{stone}	Specific density of stone	2650 kg/m^3
D_{n50}	Median nominal diameter	1.24 m

$$M_{50} = 5053\ kg$$

For this average weight, a standard gradation of 3000-6000 kg is required. The nominal diameter of this gradation is $D_{n50} = 1.19\ m$.

2.2.4 Filter layer

The relation between the diameter of the filter layer and the armour layer is approximately 2.2 to 2.5, dependant of the standard grading ratio. The D_{n50} of the armour layer is 1.19 m. This means the required D_{n50} of the filter layer is (CIRIA; CUR; CETMEF, 2007):

$$\frac{D_{50a}}{D_{50u}} = 2.2\ to\ 2.5$$

Where:

D_{50u}	Nominal gradation diameter filter layer	Variable (m)
D_{50a}	Nominal gradation diameter armour layer	1.19 m

$$\frac{D_{50a}}{D_{50u}} = 2.2 \rightarrow D_{50u} = 0.54\ m$$

$$\frac{D_{50a}}{D_{50u}} = 2.5 \rightarrow D_{50u} = 0.48\ m$$

The M_{50} for these nominal diameters is:

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

$$M_{50,u} = 417 - 293\ kg$$

This result in a filter layer with the gradation 300-1000 kg, the D_{n50} of this standard gradation is 0.63 m.

2.2.5 Core

The relation between the diameter of the filter layer and the core is approximately 2.2 to 2.5, dependant of the standard grading ratio. The D_{n50} of the filter layer is 0.63 m, so the required D_{n50} of the core is:

$$\frac{D_{50u}}{D_{50c}} = 2.2\ to\ 2.5$$

Where:

D_{50u}	Nominal gradation diameter filter layer	0.63
D_{50c}	Nominal gradation diameter core	Variable (m)

$$\frac{D_{50u}}{D_{50c}} = 2.2 \rightarrow D_{50c} = 0.29\ m$$

$$\frac{D_{50u}}{D_{50c}} = 2.5 \rightarrow D_{50c} = 0.25\ m$$

The M_{50}, for these nominal diameters is equal to:

$$M_{50} = \rho_{stone} \times {D_{n50}}^3$$

$$M_{50,c} = 65 - 41\ kg$$

This results in a core with a quarry run gradation of 1-500 kg, the $M_{50}$of this quarry run is 50 kg. According to (Asmerom, 2015) quarry run is the most common core material. The quarry run is a waste product of a quarry and for this reason, less expensive than a more specific rock gradation.

2.2.6 Layer Thickness

The layer thickness can be determined using the equation (CIRIA; CUR; CETMEF, 2007):

$$t_d = n \times k_t \times D_{n50}$$

Where:

t_d	Theoretical layer thickness	variable (m)
n	Number of layers	2
k_t	Layer thickness coefficient	0.91
D_{n50}	Nominal stone diameter	1.19/0.63 m (Armour and Filter)

The k_t factor is defined using a general guidance from the Rock Manual (CIRIA; CUR; CETMEF, 2007). The double layer will be placed using a standard degree of compaction. The survey of the layers will be done using the spherical foot staff. The applied rock will be quite blocky with a BLc coefficient of 0.65. (The BLc coefficient is a coefficient for the blockiness of rock, it is calculated by dividing the volume of a block by the volume of the enclosing XYZ orthogonal, (CIRIA; CUR; CETMEF, 2007))

Using the above equation, the layer thickness of the armour layer ($t_{d,a}$) and the filter layer ($t_{d,u}$) are:

$$t_{d,a} = 2 \times 0.91 \times 1.19 = 2.2\ m$$

$$t_{d,u} = 2 \times 0.91 \times 0.63 = 1.1\ m$$

2.3 Crest width

The crest width is equal to 3 x D_{n50} (armour layer). According the overtopping manual (EA; ENW; KFKI, 2007), a crest width of 3 x D_{n50} is an average value of the crest width. So the crest width is:

$$W_c = 3 \times D_{n50,a}$$

Where:

W_c	Crest width	Variable (m)
$D_{n50,a}$	Nominal diameter of the armour layer	1.19 m

$$W_c = 3.6\ m$$

2.4 Crest height (SLS)

2.4.1 Introduction

The crest height of the breakwater is calculated, using the operational boundary conditions. The critical boundary condition for this calculation is the volume of overtopping over the breakwater. The overtopping may not influence the wave height at the lee side of the breakwater. For the calculation of the crest height, the following boundary conditions are determined:

Boundary	Description	Value
H_s/H_{m0} (Spectral wave)	Significant wave height	1.5 m
T_p	Peak wave period	17 s
T_m	Mean wave period	13.s
$T_{m-1.0}$	Mean energy wave period	15.5 s
α	Structure slope angle	26.5° (Slope 1:2)
g	Gravitational acceleration	9.81 m/s^2
$\xi_{m-1.0}$	Surf parameter spectral waves	7.88
$s_{m-1.0}$	Fictitious wave steepness	0.004

Table 3: Dimensions for determining crest height

Tm = Mean energy wave (DNV, 2010).

$$T_m = \frac{T_p}{1.4049} \times 1.0867$$

Tm-1.0 = Energy wave period (CIRIA; CUR; CETMEF, 2007).

$$T_p = 1.1 \times T_{m-1.0}$$

The surf similarity parameter is equal to (CIRIA; CUR; CETMEF, 2007):

$$\xi_{m-1.0} = \frac{\tan\alpha}{\sqrt{s_{m-1.0}}}$$

And the wave steepness for the mean energy period (CIRIA; CUR; CETMEF, 2007):

$$s_{m-1.0} = \frac{2\pi \times H_{m0}}{g \times T_{m-1.0}{}^2}$$

2.4.2 Specific overtopping discharge

The crest height can be determined using Owen's method or using the TAW method. The maximum specific discharge q, at the end of the breakwater is set to 1x10$^{-4}\,\frac{m^3}{s}/m$. Using this specific discharge, the influence of the overtopping on the wave height at the lee side of the breakwater will be nihil.

The specific discharge at the end of the breakwater is dependant of the crest width. When waves are overtopping, the breakwater will dissipate some of the wave energy into the crest; therefore a reduction factor may be applied, a schematic drawing of the dissipated water can be seen in Figure 1.

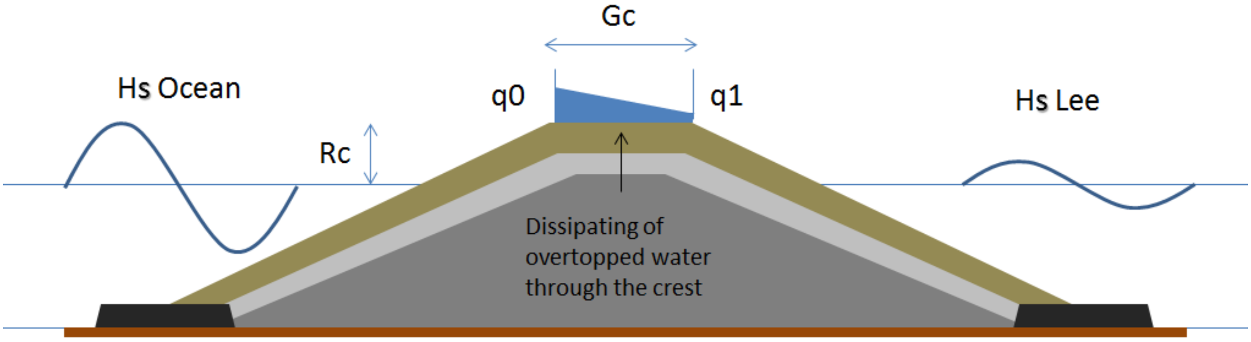

Figure 1: Dissipating water through crest

The difference between the discharges q_0 and q_1 can be determined by using a reduction factor. The maximum allowed operational overtopping discharge at location q_1 is set to 1x10$^{-4}\,\frac{m^3}{s} \times m$. The reduction factor on the overtopping discharge can be applied (EA; ENW; KFKI, 2007):

$$C_r = 3.06 \times \exp(-1.5 \frac{G_c}{H_{m0}})$$

Where:

G_c	Crest width (3 D_{n50})	3.6 m
H_{m0}	Significant wave height	1.5 m

$$C_r = 0.08$$

The overtopping discharge at the ocean side of the breakwater is equal to:

$$q_0 = \frac{q_1}{C_r}$$

Where:

q_0	Specific discharge ocean side breakwater	Variable ($\frac{m^3}{s}/m$)
q_1	Specific discharge ocean lee breakwater	1x10$^{-4}\,\frac{m^3}{s}/m$
C_r	Reduction factor specific discharge	0.08

$$q_0 = 1.25 \times 10^{-3} \ \frac{m^3}{s}/m$$

2.4.3 Owens method

Owens method is applicable when the main deep water wave steepness is, 0.035 < S_{0m} < 0.055. The mean deep water wave steepness is 0.0056, which means that Owens method is not applicable for this specific situation. Moreover, Owens method is drafted in 1980 while the TAW-method is drafted in 2002. The results of Owens method are less accurate than the TAW-method. (CIRIA; CUR; CETMEF, 2007)

$$s_{0m} = \frac{2\pi \times H_{m0}}{g \times {T_m}^2}$$

2.4.4 TAW-method

The TAW method can be divided into two different ranges of wave conditions, breaking waves and non-breaking waves. When $\gamma_b \times \xi_{m-1.0} < 2$, waves are breaking and when $\gamma_b \times \xi_{m-1.0} > 2$, waves are not breaking. γ_b Is the reduction factor for berm influence, since there is no berm applied in the design γ_b = 1.0. So, $1 \times 7.88 > 2$, which means the non-breaking wave equation is applied (CIRIA; CUR; CETMEF, 2007):

$$\frac{q_0}{\sqrt{g{H_{m0}}^3}} = C\ \exp(-D\frac{R_c}{H_{m0}}\frac{1}{\gamma_f \gamma_b})$$

Where:

q_0	Specific discharge (point 0)	1.25x 10^{-3}
g	Gravitation constant	9.78 m/s^2
H_{m0}	Significant wave height	1.5 m
C	Safety Coefficient TAW-method	0.20
D	Safety Coefficient TAW-method	2.30
R_c	Crest height	Variable (m)
γ_f	Roughness reduction factor	
	(2 layers of rock, permeable core)	0.8
γ_b	Berm reduction factor (No berm)	1.00

In order to calculate the crest height, the formula can be rewritten to:

$$R_c = \frac{\log(\frac{q}{c \times \sqrt{g{H_{m0}}^3}}) \times H_{m0} \times \gamma_f \gamma_b}{-D}$$

$$R_c = 1.55\ m$$

2.5 Crest height and Lee side armour (ULS)

2.5.1 Introduction

When the lee side armour is calculated for the ultimate limit state (ULS) with the crest height calculated for the service limit state (SLS), an armour layer with an M50 (Mass of particle of which 50% of the granular material is lighter) of 25 ton is required. Since the ocean side armour layer consists of armour stone units with a gradation between 3000-6000 kg, the crest height is re-calculated.

The design is calculated for the ultimate limit state (ULS). The boundary conditions shown in table 4 are used.

Boundary	Description	Value
H_s/H_{m0} (Spectral wave)	Significant wave height	3.3 m
T_p	Peak wave period	18.6 s
$T_{m-1.0}$	Mean energy wave period	16.9 s
α_{ocean}	Structure slope angle (ocean side)	18.4° (Slope 1:1.3)
G	Gravitational acceleration	9.78 m/s^2
$\xi_{s-1,0}$	Surf similarity parameter	3.87 [-]
T_{storm}	Duration of normative storm	6 hr (21600 s)

Table 4: Boundary conditions lee armour size

2.5.2 Rock armour layer on the lee side slope

To reduce the use of material, the slope of the rear side is set as steep as a rock armour layer permits (1:1.5). The required stone size, D_{n50} (m), at the rear side of the breakwater for a given amount of acceptable damage, S_d, can be estimated with the equation (Van Gent & Pozueta, 2004):

$$D_{n50} = 0.008\left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)(\cot\varphi)^{-2.5/6}\left(1+10*exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

However, both de D_{n50} and R_{c-rear} (crest height on the rear side of the crest) are unknown. Therefore a D_{n50} of 1.19 m is assumed to make an estimation of the crest height (rock grading of 3000 - 6000 kg, as used at the ocean slope). The estimated crest height is roughly 5 m.

The required D_{n50} for the armour layer can be determined:

$$D_{n50} = 0.008 \left(\frac{S_d}{\sqrt{N}}\right)^{-1/6} \left(\frac{u_{1\%} T_{m-1,0}}{\Delta^{0.5}}\right) (\cot\varphi)^{-2.5/6} \left(1 + 10 * exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

Where:

S_d	=	damage level parameter	=	3	[-]
N	=	number of waves	=		[-]
H_s	=	significant wave height of incident waves at the toe of the structure	=	3.3	[m]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]
φ	=	angle of the rear side slope (α_{rear})	=	33.69	[°]
R_{c-rear}	=	crest freeboard relative to the water level at rear side of the crest	=	5	[m]
$u_{1\%}$	=	maximum velocity (depth-average) at the rear side of the crest during a wave overtopping event, exceeded by 1% of the incident waves, given by the equation:	=		[m/s]

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

Where:

B	=	crest width	=	3.6	[m]
R_c	=	crest level relative to still water at the seaward side of the crest	=	5	[m]
γ_f	=	roughness factor of the seaward slope	=	0.55	[-]
γ_{f-c}	=	roughness factor of the crest	=	0.55	[-]
$R_{u1\%}$	=	fictitious run-up level exceeded by 1 per cent of the incident waves	=		[m]

The velocity, $u_{1\%}$, is related to the rear-side of the crest for situations with $R_{u1\%} \geq R_c$, in which the (fictious) run-up level, $R_{u1\%}$, is obtained using one of the following equations:

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

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

Where:

c_0, c_1, c_2	=	coefficients: $c_0 = 1.45$, $c_1 = 5.1$, $c_2 = 0.25\, c_1{}^2/c_0$			[-]
p	=	$0.5\ c_1/c_0$	=	1.76	[-]
γ	=	reduction factor; $\gamma = \gamma_f \gamma_\beta$, taking into account the effects of angular wave attack, γ_β, which can be approximated by: $\gamma_\beta = 1 - 0.0022\beta$, where $\beta \leq 80°$, and roughness γ_f	=	0.55	[-]
$\xi_{s-1,0}$	=	surf similarity parameter	=	3.87	[-]

Since $\xi_{s-1,0} > p$, $R_{u1\%}$ can be determined: $R_{u1\%} = 7.15\, m$

With $R_{u1\%}$, $u_{1\%}$ is calculated: $u_{1\%} = 7.03\, m/s$

To determine the required D_{n50}, only the amount of waves, N, is not yet known. The amount of waves can be calculated with:

$$N = \frac{T}{T_{m-1,0}}$$

Where:

T	=	the duration of the storm	=	21600	[s]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]

The amount of waves is calculated as: $N = 1278$

The required stone size, D_{n50} (m), at the rear side of the breakwater can be determined: $D_{n50} = 1.16\, m$

With the D_{n50}, the rock grading can be defined as 3000 – 6000 kg ($M_{50} = 4203\, kg$), with a crest height of 5 meters.

With R_c, $R_{c,tot}$ can be determined:

$$R_{c,tot} = R_c + \Delta HHWS + Sea\ Level\ Rise + Surge = 5 + 0.75 + 0.065 + 0.5 \\ = 6.315\, m\ above\ CD\ (LLWS)$$

2.6 Toe design

2.6.1 Introduction

The toe dimensions will be calculated using the following boundary conditions:

Boundary	Description	Value
Hs	Significant wave height	3.3 m
ρ_{water}	Specific density seawater	1020 kg/m^3
ρ_{stone}	Specific density stones	2650 kg/m^3
Δ	Relative buoyant density $\left(\frac{\rho_{stone}}{\rho_{wat}}\right) - 1$	1.6
h	Water depth (LLWS)	9.5 m

Table 5: Boundary conditions toe design

2.6.2 Toe armour size

The minimum required toe armour size can be determined using the equation shown below (CIRIA; CUR; CETMEF, 2007), the formulae may be applied in the range of 3 < h_t/D_{n50} < 25:

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

Where:

N_{od}	Damage number (Start of Damage)	0.5
h_t	Depth of toe below water level	Variable (m)

A safe boundary for applying the equation for calculating the toe armour size is (CIRIA; CUR; CETMEF, 2007): $\frac{h_t}{H_s} < 2$. The relative toe depth (h_t) is not determined yet. The significant wave height is 3.3 m. A safe value for h_t can be determined:

$$h_t < 2 * H_s \rightarrow h_t < 2 * 3.3 \rightarrow\ h_t < 6.6\ m$$

With the value for h_t known, D_{n50} of the toe can be calculated:

$$\frac{3.3}{1.6 \times D_{n50}} = \left(2 + 6.2\left(\frac{6.6}{9.5}\right)^{2.7}\right) 0.5^{0.15}$$

$$\frac{H_s}{\Delta D_{n50}} = 3.89$$

$$D_{n50} = 0.53\ m$$

And,

$$M_{50} = \rho_{stone} \times D_{n50}{}^3$$

$$M_{50} = 395\ kg$$

The minimum required toe grading is 300-1000 kg, with a D_{n50} of 0.6 m. This gradation is equal to the armour layer gradation.

$\frac{h_t}{D_{n50}}$ = 12.7 → 3 < h_t/D_{n50} < 25 → 3 < 12.7 < 25, the formula may be applied.

$\frac{h_t}{H_s} < 2$ → $\frac{6.6}{3.3} < 2$ → $2 \leq 2$, a safe boundary for the toe dimensions is applied.

2.6.3 Toe dimensions (Ocean Side)

The relative toe height is set to 6.6m. The total height of the toe is 9.5– 6.6 = 2.9 m.

Toe armour thickness

The thickness of the toe can be defined as (CIRIA; CUR; CETMEF, 2007):

$$t_{d,toe} = 2 \times k_t \times D_{n50}$$

Where:

k_t	Layer thickness coefficient (See par 2.2.6 Layer Thickness)	0.91
D_{n50}	Nominal diameter toe gradation	0.63 m

$$t_{d,toe} = 1.2\ m$$

The remainder height of the toe (2.9 – 1.2 = 1.7 m) has to be completed using the core material of the breakwater.

Toe width

The minimal toe width, conform the rock manual, is equal to 3 times the nominal toe diameter. So the toe width is designed at 1.6 meters.
The shoulder width is, conform the Rock manual, not less than 2 meters. Since, the size of the toe is only 1.6 meters; the shoulder will be constructed equal to its minimal width. The slopes of the toe are 1:1. A schematic drawing of the toe structure can be seen in Figure 2.

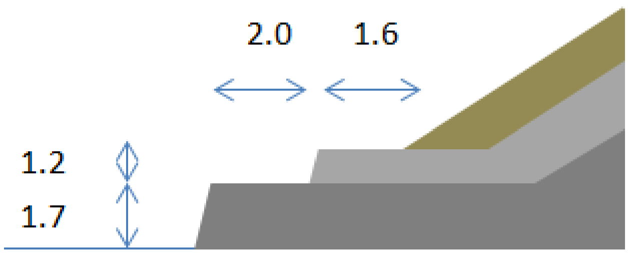

Figure 2: Schematic Toe

2.6.4 Toe dimensions (Lee side)

The dimensions of the lee side toe construction are determined using a rule of thumb, which is mentioned in the Rock Manual (CIRIA; CUR; CETMEF, 2007). The rule of thumb determines the lee side toe dimensions as follows:

h_l	Water depth at the lee side toe	3 m
S_l	Shoulder width lee side toe (0.5 x Thickness under layer)	0.55 m

A schematic drawing of the lee side toe can be seen in .

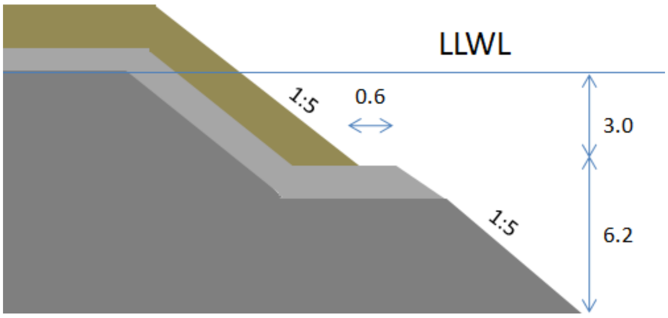

Figure 3: Lee side toe

2.7 Lifetime

If the rubble mound which is applied in the breakwater passes all of the tests, mentioned in the rock manual (CIRIA; CUR; CETMEF, 2007), a minimal durability of 50 years can be guaranteed. Thereby, the breakwater has to be constructed within the placing tolerances mentioned in the rock manual.

2.8 Cross-sectional drawings

Cross-sectional drawings of the trunk of the breakwaters can be found in appendix 1:

- Appendix 1 page 1/1: Cross-sectional drawing of a breakwater with a rock armour layer.

3 CONCRETE ARMOUR UNITS

3.1 Introduction

There are different types of concrete armour units. These concrete armour units can vary in the placement pattern, the number of layers, the complexity of the shape of the units and the way the units provide their own stability.

Due to the practicability during construction and the cost a single layer concrete armour unit with a random placement pattern is desired. This results in five possible concrete armour units:

- Cube
- Stabit
- Accropode
- Core-loc
- Xbloc

Only one of these five concrete armour units is chosen as concrete armour unit for the concrete armour breakwater. The graduation students have chosen the Xbloc armour units for the concrete armour breakwater. Moreover, according to (CIRIA; CUR; CETMEF, 2007) the Xbloc units have the highest stability number, $\frac{H_s}{\Delta D_n}$, in comparison with the other concrete armour units (the Core-loc concrete armour units have the same stability number as the Xbloc units).

3.2 Concrete Armour Units

3.2.1 Introduction

Because Xbloc armour units form an interlocking single layer armour layer, no damage is allowed. According to (CIRIA; CUR; CETMEF, 2007) storm duration and wave period have no influence on the hydraulic stability of the Xbloc armour units.

The concrete armour units are determined using the boundary conditions shown in table 6:

Boundary	Description	Value
h	Water depth	10.8 m
H_s	Significant wave height	3.3 m
ρ_w	Density of seawater	1020 kg/m^3
ρ_c	Density of concrete	2350 – 2500 kg/m^3
α	Slope of the breakwater	1:1.33 (3:4)

Table 6: Boundary conditions for concrete armour unit layer

To reduce the use of material the structure slope is determined as steep as possible (3:4).

3.2.2 Volume concrete armour units

According to (Delta Marine Consultants, 2014), the required Xbloc size can be determined with the following formula:

$$V = \left[\frac{H_s}{2.77 * \Delta}\right]^3$$

Where:

V	Xbloc volume	in m^3
H_s	Significant wave height	3.3 m
Δ	Relative concrete density	$\rho_c/\rho_w - 1$
ρ_c	Density of concrete	2450 kg/m^3
ρ_w	Density of seawater	1020 kg/m^3

The required Xbloc volume can be calculated:

$$V = \left[\frac{H_s}{2.77 * \left(\frac{\rho_c}{\rho_w} - 1\right)}\right]^3 = \left[\frac{3.3}{2.77 * \left(\frac{2450}{1020} - 1\right)}\right]^3 = 0.61\ m^3$$

3.2.3 Correction factor on unit weight

According to (Delta Marine Consultants, 2014), local phenomena can influence the required unit size. For this situation the water depth influences the required unit size.

The correction factors in table 7 apply on the unit weight for breakwaters in situations with large water depth (Delta Marine Consultants, 2014):

Correction Factor	
1.5	For a water depth > 2.5 H_s
2	For a water depth > 3.5 H_s

Table 7: Correction factor due to water depth

For this specific situation a correction factor of 1.5 is required:
$(3.5 * 3.3 > 10.8 > 2.5 * 3.3)$

$$V = 1.5 * 0.61 = 0.92\ m^3$$

3.2.4 Properties of Xbloc layer

Table 8 shows the characteristics of the Xbloc armour layer, according to (Delta Marine Consultants, 2014):

Symbol	Value		Description
V	1.00	m^3	Unit volume
$\rho_{concrete}$	2450	kg/m^3	Concrete density
H_s	3.88	m	Maximum design wave height
D	1.44	m	Unit height
W	2.45	t	Unit weight
h	1.4	m	Thickness of armor layer
N	57.81	$1/100m^2$	Packing density
	0.58	m^3/m^2	Concrete volume
D_x	1.90	m	Placement distance horizontal
D_y	0.91	m	Placement distance up-slope
	58.7	%	Porosity of armour layer
	0.06-0.3	t	Rock grading for under layer
f	0.8	m	Thickness of under layer

Table 8: Characteristics of Xbloc armour layer with an Xbloc unit volume of 1.0 m^3

3.2.5 Under layer and core material

Table 8, in 0,0 shows that the under layer consists of stones with a weight of 60 – 300 kg and has a thickness of 0.8 m.

The core material of the breakwater is quarry run from a local quarry.

3.3 Crest width and armour size

3.3.1 Crest armour size

The armour on the crest of the breakwater is in general the same as the armour on the seaside slope of the breakwater. This means that the crest has an armour layer of Xbloc units with a volume of 1 m^3 and the armour layer has a thickness of 1,4m. The under layer is also the same as on the seaside slope of the breakwater, a layer of rocks with a grading of 60 – 300 kg and a layer thickness of 0.8 m.

3.3.2 Crest width

According to (CIRIA; CUR; CETMEF, 2007) the crest should be wide enough to permit at least three to five concrete armour units to be placed on the crest. This means that the crest should have a width of:

$$3 * 1.44 \approx 5\ m\ to\ 5 * 1.44 \approx 7.5\ m$$

According to (Delta Marine Consultants, 2014), the design company of the Xbloc, the crest should be at least 2.28 times the D of the unit. This means that the crest should be at least $2.28 * 1.44 = 3.28\ m$ wide.

The crest should be as narrow as possible to reduce the materials needed. However, there has to be enough space on top of the crest for the equipment used during construction. So the crest width is determined to be 5 meters.

3.4 Crest height (SLS)

3.4.1 Introduction

The crest height of the breakwater for the service limit state (SLS) is calculated, using the operational boundary conditions, see table 9.

Boundary	Description	Value
H_s	Significant wave height	1.5 m
H_{m0}	Significant wave height calculated from the spectrum ($\approx H_s$)	1.5 m
T_p	Peak wave period	17 s
T_m	Mean wave period	13.s
$T_{m-1,0}$	Mean energy wave period	15.5 s
A	Structure slope angle	36.9° (Slope 1:1.33)
G	Gravitational acceleration	9.78 m/s^2
$\xi_{m-1,0}$	Surf parameter spectral waves	11.84
$s_{m-1,0}$	Fictitious wave steepness	0.0040

Table 9: Operational boundary conditions

T_m is the mean wave period and can be estimated from T_p with the formula (DNV, 2010):

$$T_m = \frac{T_p}{1.4049} \times 1.0867$$

$T_{m-1,0}$ is the mean energy wave period and can be estimated with the formula (CIRIA; CUR; CETMEF, 2007):

$$T_p = 1.1 \times T_{m-1,0}$$

The surf similarity parameter ($\xi_{m-1,0}$) is equal to (CIRIA; CUR; CETMEF, 2007):

$$\xi_{m-1,0} = \frac{\tan \alpha}{\sqrt{s_{m-1,0}}}$$

And the fictitious wave steepness for the mean energy period ($s_{m-1,0}$) can be calculated with the formula (CIRIA; CUR; CETMEF, 2007):

$$s_{m-1,0} = \frac{2\pi \times H_{m0}}{g \times T_{m-1,0}{}^{2}}$$

3.4.2 Specific overtopping discharge

The crest height can be determined using Owen's method or using the TAW method. The maximum allowed specific discharge q1, at the end of the breakwater is set to $1\text{x}10^{-4}\,\frac{m^3}{s}/m$. Using this specific discharge, the influence of the overtopping on the significant wave height at the lee side of the breakwater will be negligible.

The specific discharge at the end of the breakwater is dependant of the crest width. When waves are overtopping the breakwater, some of the energy will be dissipated into the crest. Therefore a reduction factor has to be taken into account. A schematic drawing of the dissipated water can be seen in Figure 4 .

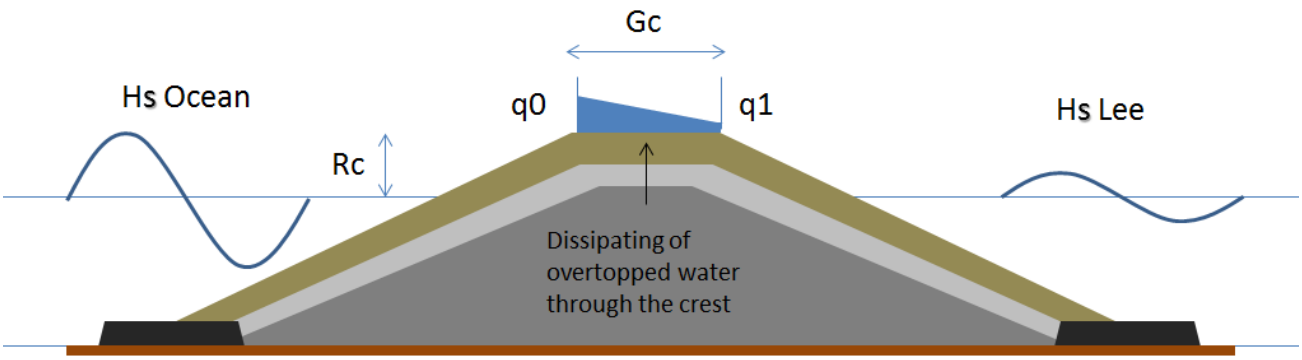

Figure 4: Dissipating water through crest

The difference between the discharges q_0 and q_1 can be determined using a reduction factor. The maximum allowed operational overtopping discharge at location q_1 is $1x10^{-4}$ $\frac{m^3}{s}/m$. The reduction factor shown below can be applied on the overtopping discharge (EA; ENW; KFKI, 2007).

$$C_r = 3.06 \times \exp\left(-1.5\frac{G_c}{H_{m0}}\right) = 3.06 \times \exp\left(-1.5\frac{5}{1.5}\right) = 0.0206$$

Where:

C_r	Reduction factor specific discharge	
G_c	Crest width	5 m
H_{m0}	Significant wave height	1.5 m

The overtopping discharge at the ocean side of the breakwater is equal to:

$$q_0 = \frac{q_1}{C_r}$$

Where:

q_0	Specific discharge ocean side of breakwater	in $\frac{m^3}{s}/m$
q_1	Specific discharge lee side of breakwater	$1*10^{-4}$ $\frac{m^3}{s}/m$
C_r	Reduction factor specific discharge	0.0206

$$q_0 = \frac{q_1}{C_r} = \frac{1*10^{-4}}{0.0206} = 4.85*10^{-3}\,\frac{m^3}{s}/m$$

3.4.3 Owens method

Owens method is applicable when the main deep water wave steepness is, $0.035 < S_{0m} < 0.055$ (CIRIA; CUR; CETMEF, 2007). The mean deep water wave steepness is equal to 0.0040 (see paragraph 3.4.1), which means that Owens method is not applicable for this specific situation.

3.4.4 TAW-method

The TAW method can be divided into two different ranges of wave conditions, breaking waves and non-breaking waves. If $\gamma_b \times \xi_{m-1.0} < 2$, waves are breaking and if $\gamma_b \times \xi_{m-1.0} > 2$, waves are not breaking. γ_b is the reduction factor for berm influence, since there is no berm applied in the design γ_b = 1.0. $\xi_{m-1.0}$ is the surf similarity parameter, the value of $\xi_{m-1.0}$ is shown in paragraph 3.4.1. ($\xi_{m-1.0}$=11.84).

$1 \times 11.84 > 2$, which means, the non-breaking wave equation has to be applied (CIRIA; CUR; CETMEF, 2007):

$$\frac{q_0}{\sqrt{gH_{m0}{}^3}} = C\exp\left(-D\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_b}\right)$$

Where:

q_0	Specific discharge (point 0)	$4.85 \cdot 10^{-3}\ \frac{m^3}{s}/m$
g	Gravitation constant	9.78 m/s^2
H_{m0}	Significant wave height	1.5 m
C	Safety Coefficient TAW-method	0.20 (-)
D	Safety Coefficient TAW-method	2.30 (-)
R_c	Crest height above CD (LLWS)	in m
γ_f	Roughness reduction factor (Xbloc)	
	($\gamma_b * \xi_{m-1,0} > 10$ so γ_f=1.00)	1.00 (-)
γ_b	Berm reduction factor (No berm)	1.00 (-)

In order to calculate the crest height, the formula can be rewritten to:

$$R_c = \frac{\log\left(\frac{q_0}{C \times \sqrt{gH_{m0}{}^3}}\right) \times H_{m0} \times \gamma_f\gamma_b}{-D}$$

3.4.5 Total crest height

The total crest height, as seen from CD (LLWS) is equal to:

$$R_{c,tot} = R_c + \Delta HHWS + Sea\ Level\ Rise + Surge$$

Where:

R_c	Crest height above CD (LLWS)	in m
$\Delta HHWS$	Difference between CD and high water level	0.75 m
$Sea\ Level\ Rise$	Expected sea level rise during the design lifetime of the breakwater	0.065 m
$Surge$	The maximum occurring surge	0.5 m

R_c can be determined:

$$R_c = \frac{\log\left(\frac{q_0}{C \times \sqrt{gH_{m0}{}^3}}\right) * H_{m0} * \gamma_f\gamma_b}{-D} = \frac{\log\left(\frac{4.85 * 10^{-3}}{0.20 \times \sqrt{9.78 * 1.5^3}}\right) * 1.5 * 1.00 * 1.00}{-2.30}$$
$$= 1.49\ m$$

With R$_{c,}$ R$_{c,tot}$ can be determined:

$$R_{c,tot} = R_c + \Delta HHWS + Sea\ Level\ Rise + Surge = 1.49 + 0.75 + 0.065 + 0.5$$
$$= 2.805\ m\ above\ CD\ (LLWS)$$

3.5 Crest height & lee side armour (ULS)

3.5.1 Introduction

When the lee side armour is calculated for the ultimate limit state (ULS) with the crest height calculated for the SLS, an armour layer with an M_{50} (Mass of particle which 50% of the granular material is lighter) of 25 ton is required. Since the ocean side armour layer consists of Xbloc units with a weight of 2.45 ton, the crest height is re-calculated.

Two different designs are calculated:
- Xbloc unit armour layer on the ocean side slope and crest of the breakwater with a rock armour layer on the lee side slope.
- Xbloc unit armour layer on the ocean side slope, the crest and the lee side slope of the breakwater.

The two different designs are calculated for the ultimate limit state (ULS), with the boundary conditions shown in table 10.

Boundary	Description	Value
H_s	Significant wave height	3.3 m
T_p	Peak wave period	18.6 s
$T_{m-1.0}$	Mean energy wave period	16.9 s
α_{ocean}	Structure slope angle (ocean side)	36.9° (Slope 1:1.33)
G	Gravitational acceleration	9.78 m/s^2
$\xi_{s-1,0}$	Surf similarity parameter	8.71 [-]

Table 10: Boundary conditions for the ultimate limit state (ULS)

3.5.2 Rock armour layer on the lee side slope

To reduce the use of material, the rear side slope is as steep as a rock armour layer permits (1:1.5). The required stone size, D_{n50} (m), at the rear side of the breakwater for a given amount of acceptable damage, S_d, can be estimated with the equation (Van Gent & Pozueta, 2004):

$$D_{n50} = 0.008\left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)(\cot\varphi)^{-2.5/6}\left(1+10*exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

However, both de D_{n50} and R_{c-rear} (crest height on the rear side of the crest) are unknown. Therefore a D_{n50} of 0.64 m is assumed (rock grading of 300 - 1000 kg) to make an estimation of the crest height. The estimated crest height roughly is 6 m (CD +7.32 m).

The required D_{n50} for the armour layer can be determined for a crest height of 6 m:

$$D_{n50} = 0.008\left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)(\cot\varphi)^{-2.5/6}\left(1+10*exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

Where:

S_d	=	damage level parameter	=	3	[-]
N	=	number of waves	=		[-]
H_s	=	significant wave height of incident waves at the toe of the structure	=	3.3	[m]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]
φ	=	angle of the rear side slope (α_{rear})	=	33.69	[°]
R_{c-rear}	=	crest freeboard relative to HHWS+SLR+Surge at the rear side of the crest (CD+1.32)	=	6	[m]
$u_{1\%}$	=	maximum velocity (depth-average) at the rear side of the crest during a wave overtopping event, exceeded by 1% of the incident waves, given by the equation:	=		[m/s]

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

Where:

B	=	crest width	=	5	[m]
R_c	=	crest level relative to HHWS+SLR+Surge at the seaward side of the crest (CD+1.32)	=	6	[m]
γ_f	=	roughness factor of the seaward slope	=	0.45	[-]
γ_{f-c}	=	roughness factor of the crest	=	0.45	[-]
$R_{u1\%}$	=	fictitious run-up level exceeded by 1 per cent of the incident waves	=		[m]

The velocity, $u_{1\%}$, is related to the rear-side of the crest for situations with $R_{u1\%} \geq R_c$, in which the (fictious) run-up level, $R_{u1\%}$, is obtained using one of the following equations:

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

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

Where:

c_0, c_1, c_2	=	coefficients: $c_0 = 1.45$, $c_1 = 5.1$, $c_2 = 0.25\,c_1{}^2/c_0$			[-]
p	=	$0.5\ c_1/c_0$			[-]
γ	=	reduction factor; $\gamma = \gamma_f\gamma_\beta$, taking into account the effects of angular wave attack, γ_β, which can be approximated by: $\gamma_\beta = 1 - 0.0022\beta$, where $\beta \leq 80°$, and roughness γ_f	=	0.45	[-]
$\xi_{s-1,0}$	=	surf similarity parameter	=	8.71	[-]

Since $\xi_{s-1,0} > p$, $R_{u1\%}$ can be determined: $R_{u1\%} = 6.81\ m$

With $R_{u1\%}$, $u_{1\%}$ is calculated: $u_{1\%} = 4.15\ m/s$

To determine the required D_{n50}, only the amount of waves, N, is not yet known. The amount of waves can be calculated with:

$$N = \frac{T}{T_{m-1,0}}$$

Where:

T	=	the duration of the storm	=	21600	[s]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]

The amount of waves is calculated as: $N = 1278$

The required stone size, D_{n50} (m), at the rear side of the breakwater can be determined:
$D_{n50} = 0.66$

With the D_{n50}, the rock grading can be defined as 300 – 1000 kg ($M_{50} = 782\ kg$).

3.5.3 Xbloc units armour layer on lee side slope

To reduce the use of material the rear side slope is as steep as an Xbloc units armour layer permits (3:4). The required stone size, D_{n50} (m), at the rear side of the breakwater for a given amount of acceptable damage, S_d, can be estimated with the equation (Van Gent & Pozueta, 2004):

$$D_{n50} = 0.008\left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)(\cot\varphi)^{-2.5/6}\left(1+10*exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

However, both de D_{n50} and $R_{c\text{-}rear}$ (crest height on the rear side of the crest) are unknown. It is desirable that the Xbloc units on the rear side of the breakwater are the same size or smaller than the Xbloc units on the front side of the breakwater. So an estimation of the crest height is made, based on the Xbloc units with a volume of 1.0 m^3 (the same as the Xbloc units on the front slope). The estimated crest height is 6 m (CD +7.32 m).

The required D_{n50} for the armour layer can be determined:

$$D_{n50} = 0.008\left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)(\cot\varphi)^{-2.5/6}\left(1+10*exp\left(\frac{-R_{c-rear}}{H_s}\right)\right)^{1/6}$$

Where:

S_d	=	damage level parameter	=	1	[-]
N	=	number of waves	=		[-]
H_s	=	significant wave height of incident waves at the toe of the structure	=	3.3	[m]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]
φ	=	angle of the rear side slope (α_{rear})	=	36.87	[°]
R_{c-rear}	=	crest freeboard relative to HHWS+SLR+Surge at the rear side of the crest (CD+1.32)	=	6	[m]
$u_{1\%}$	=	maximum velocity (depth-average) at the rear side of the crest during a wave overtopping event, exceeded by 1% of the incident waves, given by the equation:	=		[m/s]

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

Where:

B	=	crest width	=	5	[m]
R_c	=	crest level relative to still water at the seaward side of the crest	=	6	[m]
γ_f	=	roughness factor of the seaward slope	=	0.45	[-]
γ_{f-c}	=	roughness factor of the crest	=	0.45	[-]
$R_{u1\%}$	=	fictitious run-up level exceeded by 1 per cent of the incident waves	=		[m]

The velocity, $u_{1\%}$, is related to the rear-side of the crest for situations with $R_{u1\%} \geq R_c$, in which the (fictitious) run-up level, $R_{u1\%}$, is obtained using one of the following equations:

$$R_{u1\%}/(\gamma H_s) = c_0 \xi_{s-1,0} \quad \text{for} \quad \xi_{s-1,0} \leq p$$

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

Where:

c_0, c_1, c_2	=	coefficients: $c_0 = 1.45$, $c_1 = 5.1$, $c_2 = 0.25\, c_1{}^2/c_0$			[-]
p	=	$0.5\ c_1/c_0$			[-]
γ	=	reduction factor; $\gamma = \gamma_f \gamma_\beta$, taking into account the effects of angular wave attack, γ_β, which can be approximated by: $\gamma_\beta = 1 - 0.0022\beta$, where $\beta \leq 80°$, and roughness γ_f	=	0.45	[-]
$\xi_{s-1,0}$	=	surf similarity parameter	=	8.71	[-]

Since $\xi_{s-1,0} > p$, $R_{u1\%}$ can be determined: $R_{u1\%} = 6.81\, m$

With $R_{u1\%}$, $u_{1\%}$ is calculated: $u_{1\%} = 4.15\, m/s$

To determine the required D_{n50}, only the amount of waves, N, is not yet known. The amount of waves can be calculated with:

$$N = \frac{T}{T_{m-1,0}}$$

Where:

T	=	the duration of the storm	=	21600	[s]
$T_{m-1,0}$	=	the energy wave period	=	16.9	[s]

The amount of waves is calculated as: $N = 1278$

The required stone size, D_{n50} (m), at the rear side of the breakwater can be determined: $D_{n50} = 0.89$

With the D_{n50}, the M_{50} can be determined and so the volume of the Xbloc unit required: $V_{lee\ side,required} = 1\, m^3$

Note: The method used is designed to calculate rock armour layers at the rear side of a breakwater, not concrete armour unit layers. However, it is assumed this method can be used to estimate an Xbloc unit armour layer on the rear side of the breakwater if a S_d factor of 1 is used, because no damage is allowed for Xbloc unit armour layers.

3.6 Larger Xbloc units

3.6.1 Introduction

According to (Stive, 2015), larger, oversized, Xbloc units are often applied in breakwater projects, because of the practicability during construction and the better stability of the rear slope. Therefore two larger Xbloc units will be considered:

- Xbloc units with a unit volume of 1.5 m^3
- Xbloc units with a unit volume of 3.0 m^3

Since the Xbloc units with a volume of 1.0 m^3 are stable on the front slope, the larger Xbloc units will certainly be stable on the front slope. Therefore only the rear slope of the breakwater is recalculated for the larger Xbloc units.

The crest height and the lee side armour layer for the larger Xbloc units are calculated in the same way as the Xbloc units with a unit volume of 1.0 m^3. Therefore only the results that differ from the results in paragraph 3.5.3 will be described here.

3.6.2 Volume unit of 1.5 m^3

The properties of an Xbloc armour unit layer with Xbloc units with a unit volume of 1.5 m^3 are shown in table 11.

Symbol	Value		Description
V	1.50	m^3	Unit volume
$\rho_{concrete}$	2400	kg/m^3	Concrete density
H_s	4.22	m	Maximum design wave height
D	1.65	m	Unit height
W	3.6	t	Unit weight
h	1.6	m	Thickness of armor layer
N	44.12	$1/100m^2$	Packing density
	0.66	m^3/m^2	Concrete volume
D_x	2.18	m	Placement distance horizontal
D_y	1.04	m	Placement distance up-slope
	58.7	%	Porosity of armour layer
	0.3-1.0	t	Rock grading for under layer
f	1.3	m	Thickness of under layer

Table 11: Characteristics of Xbloc armour layer with a Xbloc unit volume of 1.5 m^3

The required crest height relative to HHWS+SLR+Surge (CD+1.32), when an armour layer consisting of Xbloc units with a unit volume of 1.5 m^3 is applied, is 5.5 m (CD +6.82).

3.6.3 Volume unit of 3 m^3

The properties of an Xbloc armour unit layer with Xbloc units with a unit volume of 3.0 m^3 are shown in table 12.

Symbol	Value		Description
V	3.00	m^3	Unit volume
$\rho_{concrete}$	2400	kg/m^3	Concrete density
H_s	5.32	m	Maximum design wave height
D	2.08	m	Unit height
W	7.2	t	Unit weight
h	2.0	m	Thickness of armor layer
N	27.79	$1/100m^2$	Packing density
	0.83	m^3/m^2	Concrete volume
D_x	2.75	m	Placement distance horizontal
D_y	1.31	m	Placement distance up-slope
	58.7	%	Porosity of armour layer
	0.3-1.0	t	Rock grading for under layer
f	1.3	m	Thickness of under layer

Table 12: Characteristics of Xbloc armour layer with a Xbloc unit volume of 3.0 m3

The required crest height relative to HHWS+SLR+Surge (CD+1.32), when an armour layer consisting of Xbloc units with a unit volume of 3.0 m^3 is applied, is 5.0 m (CD +6.32).

3.7 Toe design

3.7.1 Introduction

According to (Delta Marine Consultants, 2014), a typical toe layout on a sandy seabed, for the front slope of a breakwater with Xbloc armour units, looks like shown in figure 5. The need of a geotextile is depending on the soil conditions of the seabed and the geotechnics of the construction.

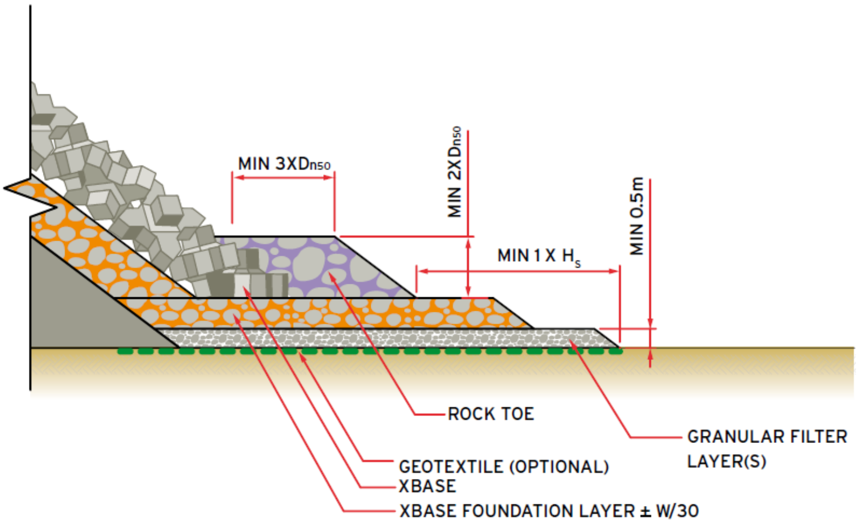

Figure 5: Typical toe layout on a sandy seabed for the front slope of a breakwater with Xbloc armour units (Delta Marine Consultants, 2014)

The size of the rock toe in front of the Xbase unit can be calculated by the formula (Delta Marine Consultants, 2014):

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

Where:

D_{n50}	=	Median nominal diameter of rock	=		[-]
H_s	=	Design significant wave height	=	3.3	[m]
h_t	=	Water depth above toe	=		[m]
h	=	Water depth in front of toe	=	9.5	[m]
N_{od}	=	Damage value Number of displaced units	=	0.5	[-]
Δ	=	Relative concrete density	=		[-]

(Delta Marine Consultants, 2014) advises a value of 0.5 for N_{od}.
The relative concrete density, Δ, is 1.40 for the Xbloc units with a unit volume of 1.0 m^3 ($\rho_{concrete}$ = 2450 kg/m^3) and is 1.35 for Xbloc units with a unit volume of 1.5 or 3.0 m^3 ($\rho_{concrete}$ = 2400 kg/m^3).
The water depth above the toe can be expressed in D_{n50}, so the only unknown parameter is D_{n50}.

The armour layer at the lee side of the breakwater is required to reach at least 0,5 to 1 times H_s deep relative to the lowest water level, LLWS (CD +0.00), for $0.8 > R_{c,tot}/H_s > 1.2$, according to (Delta Marine Consultants, 2014).

3.7.2 Ocean side toe for Xbloc front slope

The D_{n50} calculated for the rock toe results in a rock grading of 300 – 1000 kg. The under layer for the toe requires a rock grading of minimum 60 - 300 kg. There is chosen to use the same rock grading as used in the under layer of the front slope Xbloc armour layer, because of the practicality during construction.

This results in the toe constructions shown on the cross-sectional drawings in appendix 2.

3.7.3 Lee side toe

The armour layer at the lee side of the breakwater is required to reach at least 1.65 to 3.30 meters below the lowest water level, which means CD -1.65 m to CD -3.30 m.

The design of the lee side toe can be seen in the cross-sectional drawings in appendix 2.

3.8 Lifetime

If the rubble mound which is applied in the breakwater passes all of the tests, mentioned in the rock manual (CIRIA; CUR; CETMEF, 2007), a minimal durability of 50 years can be guaranteed. Thereby, the breakwater has to be constructed within the placing tolerances mentioned in the rock manual.

3.9 Cross-sectional drawings

Cross-sectional drawings of the trunk of the breakwaters can be found in appendix 2:

- Appendix 2 page 1/4: Cross-sectional drawing of a breakwater with an Xbloc unit armour layer, with a Xbloc unit volume of 1.0 m^3, on the front slope and a rock armour layer on the rear slope of the breakwater.

- Appendix 2 page 2/4: Cross-sectional drawing of a breakwater with an Xbloc unit armour layer, with a Xbloc unit volume of 1.0 m^3, on the front slope and on the rear slope of the breakwater.

- Appendix 2 page 3/4: Cross-sectional drawing of a breakwater with an Xbloc unit armour layer, with a Xbloc unit volume of 1.5 m^3, on the front slope and on the rear slope of the breakwater.

- Appendix 2 page 4/4: Cross-sectional drawing of a breakwater with an Xbloc unit armour layer, with a Xbloc unit volume of 3.0 m^3, on the front slope and on the rear slope of the breakwater.

4 LENGTH OF THE BREAKWATER

4.1 Introduction

The required length of the breakwater is determined by the maximum allowed wave height at the lee side of the breakwater for the operational conditions. There are two boundary locations for the maximum allowed wave height at the lee side of the breakwater. The first boundary location is on the edge of the turning circle of the barge combination, with a maximum allowed wave height of 1.0 meter. The second boundary location is on the edge of the mooring area of the barges, with a maximum allowed wave height of 0.5 meter. Figure 6 shows the boundary locations.

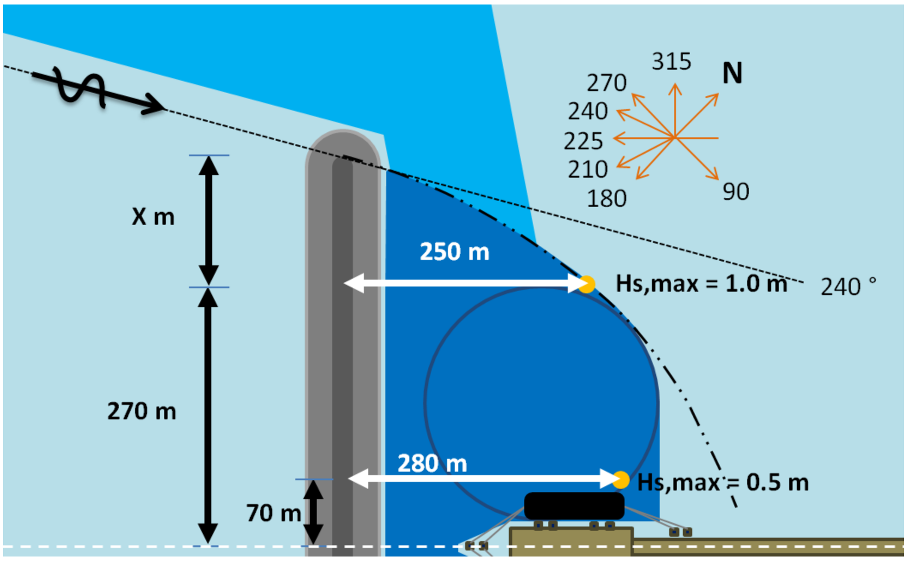

Figure 6: Boundary locations of maximum allowed wave height

The combined wave height at the lee side of the breakwater can be with the rule: the energy at the lee side of the breakwater is equal to the energy of the three independent phenomena combined:

$$E_c = E_D + E_T + E_o$$

Where:
E_c = Combined Wave energy
E_d = Wave energy due to diffraction
E_t = Wave energy due to transmission
E_o = Wave energy due to overtopping

The wave energy is equal to:

$$E = \frac{1}{8} \times \rho \times g \times H_s^{\,2}$$

The combined wave height at the lee side of the breakwater can be with:

$$H_s{}^c = \sqrt{(H_s{}^D)^2 + (H_s{}^T)^2 + (H_s{}^O)^2}$$

Where:

$H_s{}^c$ = The combined wave height at the lee side of the breakwater.
$H_s{}^D$= Wave height at lee side of the breakwater due to diffraction.
$H_s{}^T$= Wave height at lee side of the breakwater due to transmission.
$H_s{}^O$= Wave height at lee side of the breakwater due to overtopping.

4.2 Wave transmission

The wave transmission is determined with the coefficient of transmission, C_t (-):

$$C_t = H_t/H_i$$

Where:
C_t = Coefficient of transmission.
H_i = The incident significant wave height.
H_t = The transmitted significant wave height.

According to (CIRIA; CUR; CETMEF, 2007), the coefficient of transmission, C_t, is 0.10 for $1.2 < R_c/H_s$. However, a note is made that larger wave periods always give higher wave transmission coefficients. So the coefficient of transmission for the traditional breakwaters is assumed to be 0.20.

The transmitted significant wave height can be determined:

$$H_t(= H_s{}^T) = H_i * C_t = 1.5 * 0.20 = 0.30\ m$$

4.3 Overtopping

The overtopping discharge can be determined following the same method as in paragraph 2.4.2 and 3.4.2. However, now the specific discharge is unknown but the crest height of the different designs is known. All the crest heights of the designs are higher than the calculated minimum crest height in paragraph 2.4.2 and 3.4.2, due to the stability of the lee side slope, see paragraph 2.5 and 3.5.

It can be concluded that overtopping does not contribute to the significant wave height at the lee side of the breakwater ($H_s{}^O$= 0 m).

4.4 Diffraction

4.4.1 Introduction

To determine the required length of the breakwater the maximum allowed contribution of the diffraction is required. The required length of the breakwater to ensure that this maximum allowed contribution is not exceeded can be calculated with the linear wave-theory (Battjes, 1999) and the Cornu spiral (method of Sommerfeld). The required breakwater length for the boundary conditions on both boundary locations is determined.

4.4.2 Boundary location 1: edge of the turning circle

The maximum allowed significant wave height due to diffraction at the lee side of the breakwater ($H_s{}^D$) can be determined with:

$$H_s{}^D = \sqrt{(H_s{}^c)^2 - \left(H_s{}^T\right)^2 - \left(H_s{}^O\right)^2}$$

Since $H_s{}^T$ and $H_s{}^O$ are known (respectively 0.30 m and 0.0 m), as is the maximum allowed $H_s{}^C$ (1.0 m), the maximum allowed $H_s{}^D$ can be calculated. $H_s{}^D$ = 0.95 m.

The required diffraction factor k_d on the location can be determined with the formula:

$$k_d = H_s{}^D / H_S$$

The minimum required diffraction factor is: 0.64

With this minimum required diffraction factor, the location on the Cornu spiral can be determined with the formula:

$$k_d = d_w / d_c$$

Where:

d_w= The distance from point w to the centre on the Cornu spiral
d_c = The distance from centre to centre on the Cornu spiral

The value w can be determined on the Cornu spiral (with d_w= 15cm and d_c = 9.54cm), see figure 7 on the next page.

w = 0.015

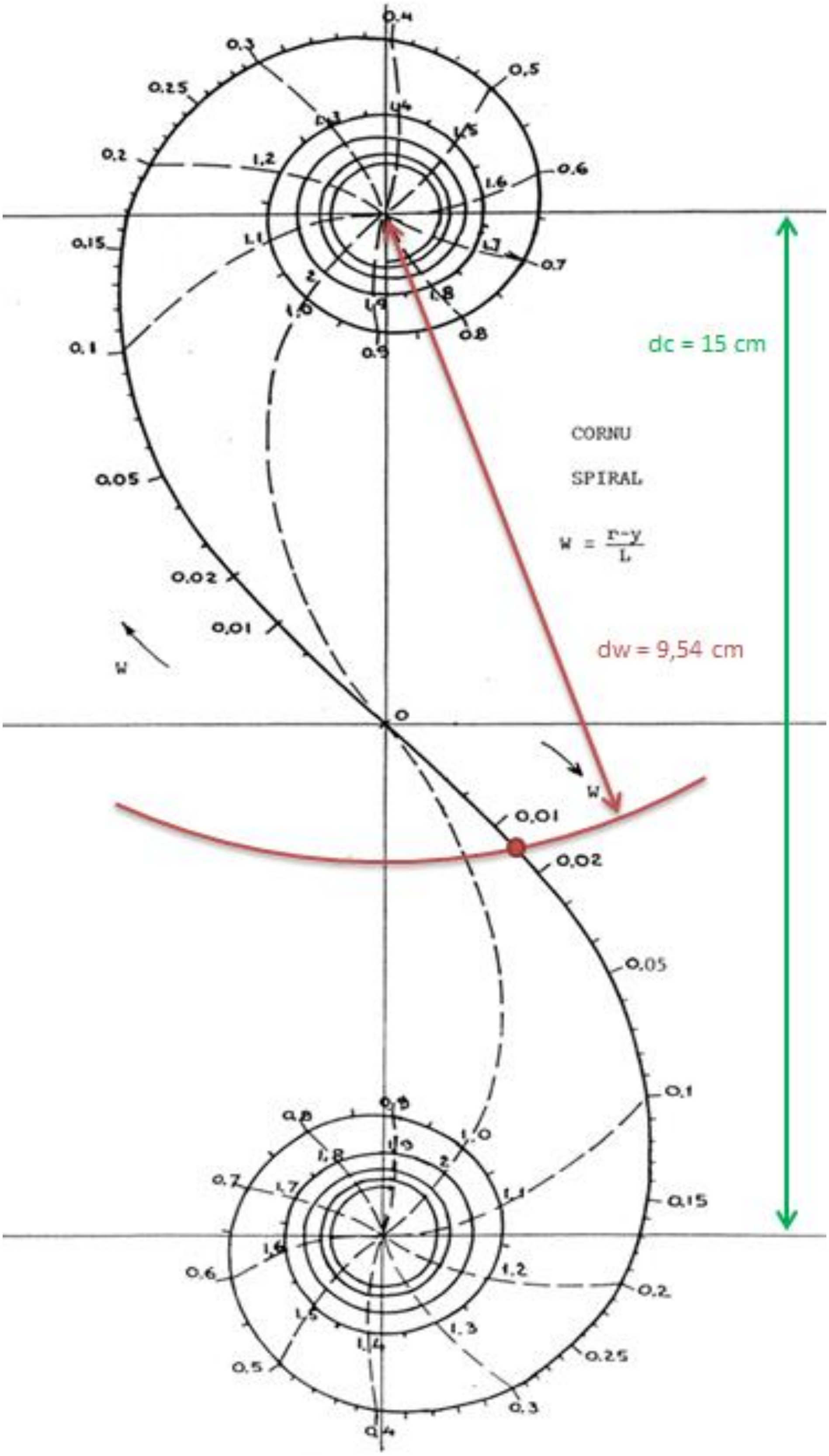

Figure 7: Cornu spiral (Korte golven & Getijden, 2009) for boundary location 1

The value w is equal to:

$$\frac{r - y}{L}$$

Where:

r = The distance between the boundary location and the head of the breakwater.
y = The distance in the direction of the incoming waves from the head of the breakwater to the perpendicular of the boundary location.
L = Wave length of the incoming waves.

See figure 8.

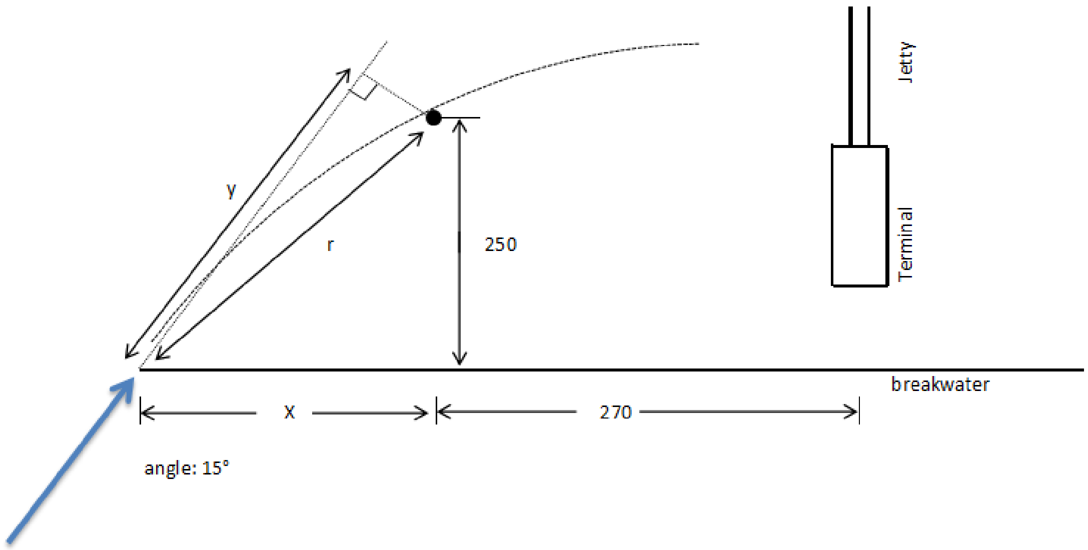

Figure 8: Schematisation of boundary location 1

In this situation r and y are:

$$r = \sqrt{x^2 + 250^2}$$
$$y = \cos\left(90 - 15 - \tan^{-1}\left(\frac{250}{x}\right)\right) * r$$

The wave length (L) of the incoming waves can be determined using the linear wave theory described by (Battjes, 1999). The wave length is between 160 and 171 meters, depending on the water level.

The value of x can be determined:

x = approximately 35 meters

This means that the minimum required total length of the breakwater for the boundary conditions on boundary location 1 is 610 meter.

4.4.3 Boundary location 2: edge of the mooring area

The maximum allowed significant wave height due to diffraction at the lee side of the breakwater (${H_s}^D$) can be determined with:

$${H_s}^D = \sqrt{({H_s}^c)^2 - \left({H_s}^T\right)^2 - \left({H_s}^O\right)^2}$$

Since ${H_s}^T$ and ${H_s}^O$ are known (respectively 0.30 m and 0.0 m), as is the maximum allowed ${H_s}^C$ (0.5 m), the maximum allowed ${H_s}^D$ can be calculated. ${H_s}^D$ = 0.4 m.

The required diffraction factor k_d on the location can be determined with the formula:

$$k_d = {H_s}^D / H_S$$

The minimum required diffraction factor is: 0.27

With this minimum required diffraction factor, the location on the Cornu spiral can be determined with the formula:

$$k_d = d_w / d_c$$

Where:

d_w= The distance from point w to the centre on the Cornu spiral
d_c = The distance from centre to centre on the Cornu spiral

The value w can be determined on the Cornu spiral (with d_w=15cm and d_c =8.93cm), see figure 9 on the next page.

w = 0.12

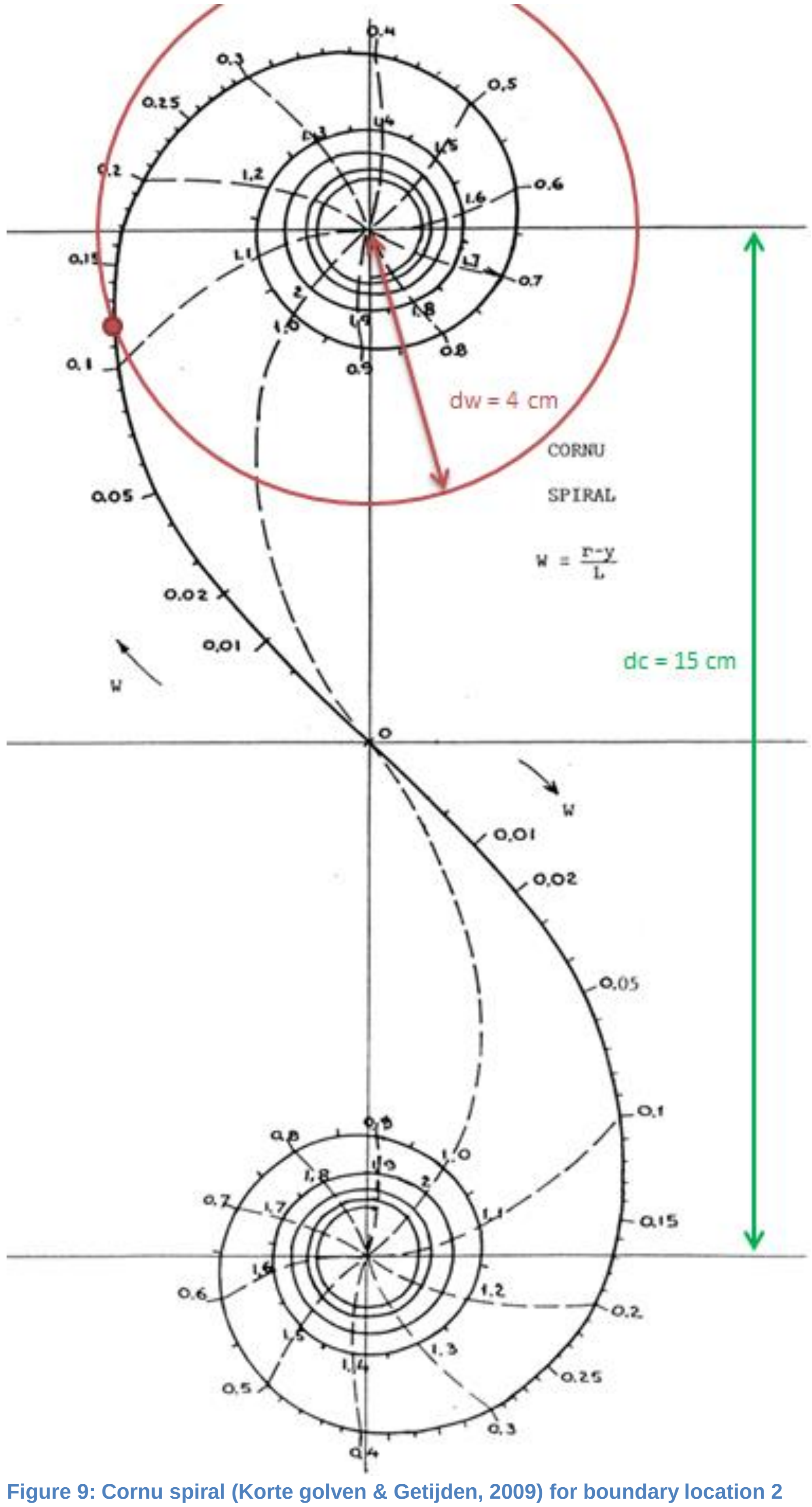

Figure 9: Cornu spiral (Korte golven & Getijden, 2009) for boundary location 2

The value w is equal to:

$$\frac{r-y}{L}$$

Where:

r = The distance between the boundary location and the head of the breakwater.
y = The distance in the direction of the incoming waves from the head of the breakwater to the perpendicular of the boundary location.
L = Wave length of the incoming waves.

See figure 10.

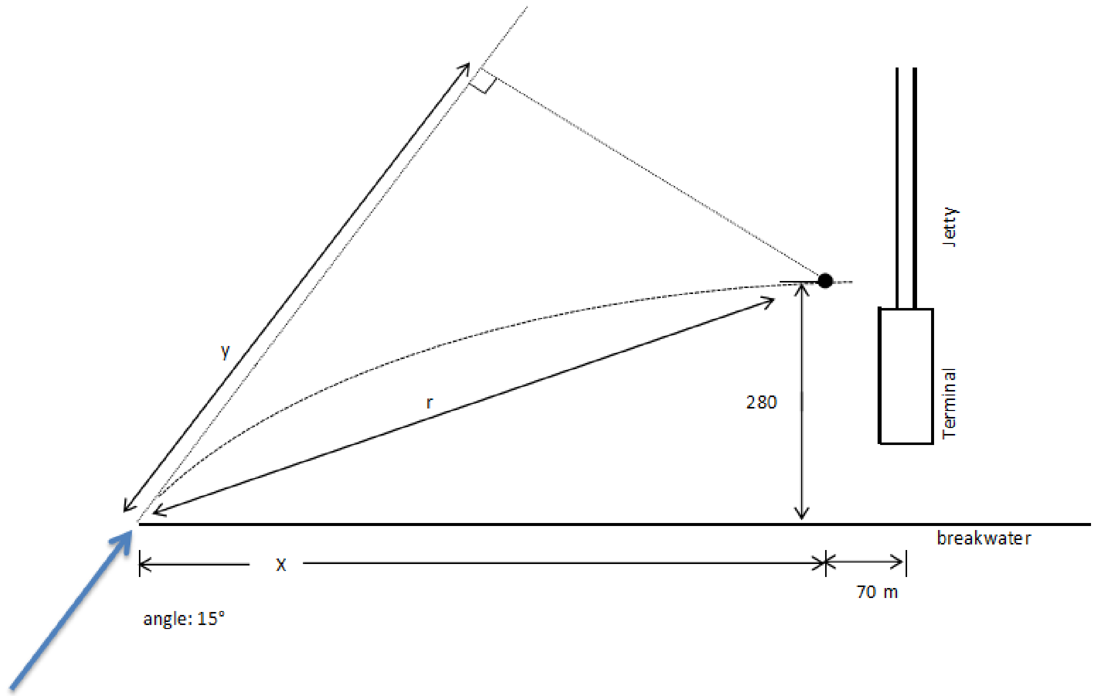

Figure 10: Schematisation of boundary location 2

In this situation r and y are:

$$r = \sqrt{x^2 + 280^2}$$

$$y = \cos\left(90 - 15 - \tan^{-1}\left(\frac{280}{x}\right)\right) * r$$

The wave length (L) of the incoming waves can be determined using the linear wave theory described by (Battjes, 1999). The wave length is between 160 and 171 meters, depending on the water level.

The value of x can be determined:

x = approximately 200 meters

This means that the minimum required total length of the breakwater for the boundary conditions on boundary location 2 is 540 meter.

4.5 Required length

The minimum required length of the traditional breakwater is the maximum value of the required lengths for the boundary conditions on the two mentioned locations. This means that the minimum required length of the traditional breakwater is 610 meters.

5 CONSTRUCTION METHOD

5.1 Introduction

The construction of an offshore, detached, breakwater is quite an operation. According to (CIRIA; CUR; CETMEF, 2007), there are two methods for constructing a breakwater:

- Land-based breakwater construction method
- Waterborne breakwater construction method

Both methods can be applied for the rubble mound breakwater on the project location. However, when the land-based breakwater construction method is applied, a temporary auxiliary construction is required. Both methods will be explained further in this chapter

5.2 Land-based breakwater construction method

To apply the land-based construction method, an auxiliary construction of approximately 150 meters has to be installed on the head of the Jetty, see figure 11. With the auxiliary construction in place, the construction of the breakwater can begin.

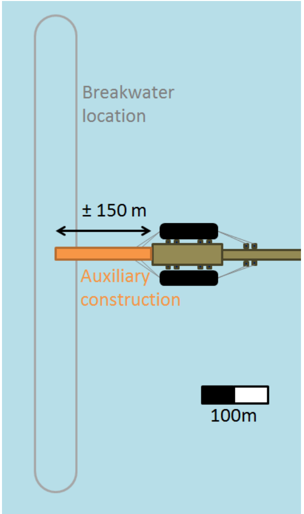

Figure 11: Location of the auxiliary construction

A typical plan view of a land-based breakwater construction operation, split in six phases, is shown in figure 12. The six phases are, according to (CIRIA; CUR; CETMEF, 2007):

1. Placing of quarry run core by dump trucks.
2. Placing remainder of core by crawler crane and/or excavator.
3. Placing scour protection with a crawler crane.
4. Placing of under layer by crawler crane or excavator.
5. Placing of toe on seaward slope by crane or excavator.
6. Placing of armour layer on seaward slope by crane or excavator.

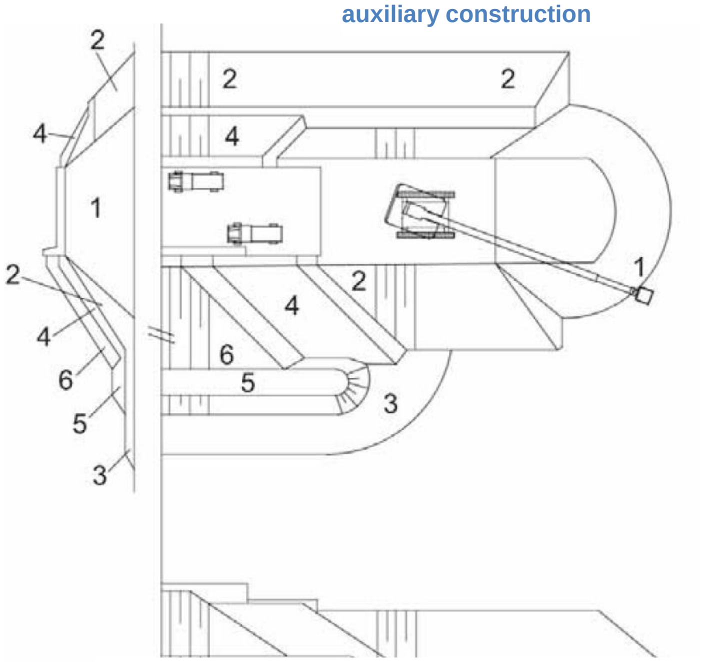

Figure 12: Typical plan view, side view and cross-section of land-based breakwater construction (CIRIA; CUR; CETMEF, 2007)

5.3 Waterborne breakwater construction method

To apply the waterborne breakwater construction method, no auxiliary construction is needed. A typical plan view of a waterborne breakwater construction operation, split in five phases, is shown in figure 13. The five phases are, according to (CIRIA; CUR; CETMEF, 2007):

1. Placing scour protection, from side stone-dumping vessels for example.

2. Placing of quarry run core from split-hopper barge (up to 3m below water level), then tipping with wheel loader from flat-top barge or by floating crane.

3. Trimming of slopes and placing of under layer by floating hydraulic excavator and/or floating crane.

4. Placing of toe on seaward slope by side stone-dumping vessel or floating crane.

5. Placing of armour layer slope by floating crane.

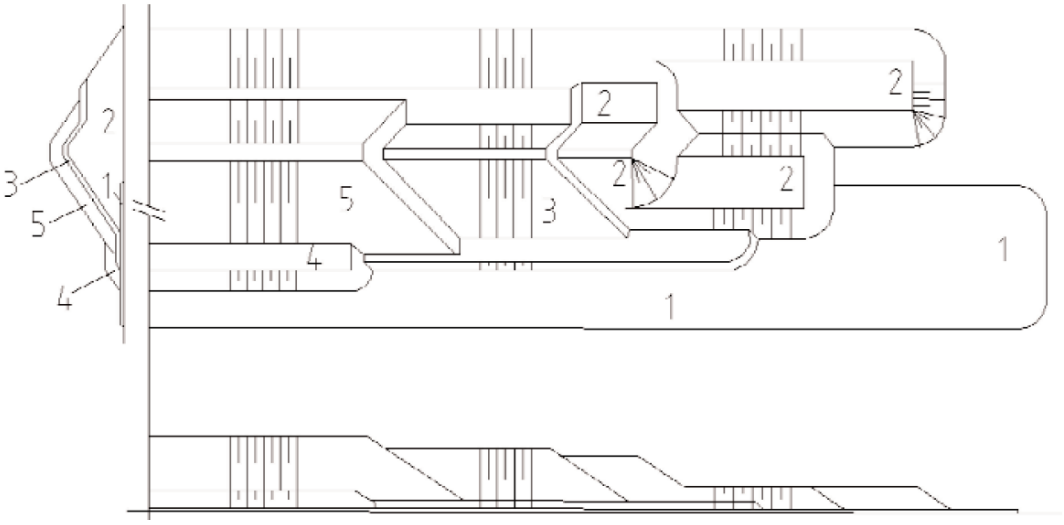

Figure 13: Typical plan view, side view and cross-section of waterborne breakwater construction (CIRIA; CUR; CETMEF, 2007); equipment not shown

5.4 Selected construction method

Both the land-based and the waterborne breakwater construction method can be applied. The equipment costs and mobilization and demobilization costs are assumed to be approximately the same for both construction methods. However, it is expected that the waterborne breakwater construction method has a greater risk of downtime during construction due to the swell waves. Therefore, the selected construction method is the method of the land-based breakwater described in chapter 5.2.

6 COSTS

6.1 Introduction

The costs of the reference designs are calculated to make a comparison on costs between a traditional breakwater and the alternative concepts possible. The unit prices of the materials are rough approximations and these can vary much due to many different factors (for example: the work load of local quarries). The determined costs do not include the costs the breakwater heads, due to a lack of time the breakwater heads have not been designed. An overview of the total costs calculation can be found in appendix III.

6.2 Unit price

The unit prices used for calculating the total material costs are shown in table 13:

Material	Unit price
Xbloc (1 m^3)	€ 1,000 / pcs
Xbloc (1.5 m^3)	€ 1,500 / pcs
Xbloc (3 m^3)	€ 2,000 / pcs
Rock 3000 – 6000 kg	€ 80 / m^3
Rock 300 – 1000 kg	€ 60 / m^3
Rock 60 – 300 kg	€ 50 / m^3
Quarry run	€ 40 / m^3

Table 13: Cost ratios (DHV, 2011) (Stive, 2015).

6.3 Amount of materials used

The total amount of the materials used in a, 1 meter thick, cross section of the variants is calculated. The results are shown in table 14.

	Rubble mound breakwater	Xbloc breakwater Option 1	Option 2	Option 3	Option 4	
Xbloc (1 m^3)	-	16	26	-	-	pcs/m
Xbloc (1.5 m^3)	-	-	-	19	-	pcs/m
Xbloc (3 m^3)	-	-	-	-	11	pcs/m
3000-6000 kg	124	-	-	-	-	m^3/m
300-1000 kg	79	26	3	71	68	m^3/m
60-300 kg	-	44	43	-	-	m^3/m
Quarry run	431	388	369	321	288	m^3/m

Table 14: Amount of materials used in a 1m thick cross section of the variants (rounded figures)

The different options of Xbloc breakwaters are:

- Option 1: Xbloc (1 m^3) unit armour layer on the front slope and rock armour layer on the back slope.
- Option 2: Xbloc (1 m^3) unit armour layer on the front and the back slope.
- Option 3: Xbloc (1.5 m^3) unit armour layer on the front and the back slope.
- Option 4: Xbloc (3 m^3) unit armour layer on the front and the back slope.

6.4 Total material costs

The total material usage of for the breakwater is calculated by multiplying the required amount of materials per meter breakwater, shown in table 14, with the total required length of the breakwater (610 meter, see chapter 4.5). The total material costs of the breakwater designs are calculated by multiplying the total material usage with the corresponding unit prices. Table 15 shows the total material costs

	Rubble mound breakwater	Xbloc breakwater				
		Option 1	Option 2	Option 3	Option 4	
Total material usage						
Xbloc (1 m^3)	-	10,056	15,844	-	-	pcs
Xbloc (1.5 m^3)	-	-	-	11,339	-	pcs
Xbloc (3 m^3)	-	-	-	-	6,748	pcs
3000-6000 kg	75,351	-	-	-	-	m^3
300-1000 kg	48,182	15,971	1,797	43,129	41,595	m^3
60-300 kg	-	26,988	26,263	-	-	m^3
Quarry run	262,820	236,882	225,347	195,923	175,622	m^3
Unit prices						
Xbloc (1 m^3)	1,000	1,000	1,000	1,000	1,000	€/$_{pcs}$
Xbloc (1.5 m^3)	1,500	1,500	1,500	1,500	1,500	€/$_{pcs}$
Xbloc (3 m^3)	2,000	2,000	2,000	2,000	2,000	€/$_{pcs}$
3000-6000 kg	80	80	80	80	80	€/m^3
300-1000 kg	60	60	60	60	60	€/m^3
60-300 kg	50	50	50	50	50	€/m^3
Quarry run	40	40	40	40	40	€/m^3
Material costs						
Xbloc (1 m^3)	-	10,056,123	15,844,181	-	-	€
Xbloc (1.5 m^3)	-	-	-	17,008,827	-	€
Xbloc (3 m^3)	-	-	-	-	13,496,891	€
3000-6000 kg	6,028,069	-	-	-	-	€
300-1000 kg	2,890,930	958,272	107,802	2,587,711	2,495,723	€
60-300 kg	-	1,349,423	1,313,127	-	-	€
Quarry run	10,512,804	9,475,282	9,013,864	7,836,909	7,024,877	€
Total material costs						
	19,431,803	21,839,101	26,278,973	27,433,447	23,017,491	€

Table 15: Total material costs

6.5 Total costs

The total costs for the breakwater construction is calculated by multiplying the total material costs with factor for the construction costs, a factor for unknown / unexpected costs due to the complexity of the construction and with a factor for the indirect costs of the contractor.

$$€_{total} = €_{material} * f_{construction} * f_{complexity} * f_{contractor}$$

The factor $f_{construction}$ is a compensation for the costs for the used equipment, the costs for the installation of construction parts, the costs for transportation of construction parts, the costs for preparation work onshore and other constructing related costs.

The factor $f_{complexity}$ is a compensation factor for the complexity of the construction and construction parts from which the costs are unknown.

The factor $f_{contractor}$ is a compensation factor for the survey costs, engineering costs, the construction site and site office costs, some unexpected expenses, profits and other contractor related costs.

After consultation with experts from RHDHV, the factors are determined to be:

$$f_{construction} = 1.3$$
$$f_{complexity} = 1.1$$
$$f_{contractor} = 1.35$$

The construction factor is set to 1.3 because of the large amount of equipment required and the transportation required for the materials.

The complexity factor is set to 1.1 because of the unknown costs for the auxiliary construction.

The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), because probably a quarry has to be opened to provide the required amount of rock. Moreover, the whole construction process has to be supervised, a site office has to be build and the contractor wants to make some profit on the project.

The total costs can be seen in table 16.

	Rubble mound breakwater	Xbloc breakwater				
		Option 1	Option 2	Option 3	Option 4	
Total material costs						
	19,431,803	21,839,101	26,278,973	27,433,447	23,017,491	€
Multiplication factors						
$f_{construction}$	1.3	1.3	1.3	1.3	1.3	-
$f_{complexity}$	1.1	1.1	1.1	1.1	1.1	-
$f_{contractor}$	1.35	1.35	1.35	1.35	1.35	-
Total costs						
	37,513,096	42,160,384	50,731,558	52,960,270	44,435,267	€

Table 16: Total construction costs.

6.6 Accuracy costs determination

To estimate the accuracy of the costs determination, all the multiplication factors are increased and decreased by 0.1 Table 17 shows an overview of the bandwidth of the costs determination.

	Rubble mound breakwater	Xbloc breakwater				
		Option 1	Option 2	Option 3	Option 4	
Total costs						
	37,513,096	42,160,384	50,731,558	52,960,270	44,435,267	€
Total costs with increased factors (+0.1)						
Costs	47,335,872	53,200,050	64,015,579	66,827,878	56,070,609	€
Deviation	9,822,776	11,039,666	13,284,021	13,867,608	11,635,341	€
Deviation	26.18	26.18	26.18	26.18	26.18	%
Total costs with decreased factors (-0.1)						
Costs	29,147,705	32,758,652	39,418,460	41,150,171	34,526,237	€
Deviation	-8,365,391	-9,401,733	-11,313,098	-11,810,099	-9,909,030	€
Deviation	-22.30	-22.30	-22.30	-22.30	-22.30	%

Table 17: Bandwidth of the costs determination

From table 17 can be concluded that the total costs may vary from -25% to +30%.

7 GEOTECHNICS AND SEISMOLOGICAL RESISTANCE

7.1 Introduction

The geotechnical and seismological stability of the breakwater designs is checked, using different types of simulation software from Deltares. Two breakwater designs are tested: the rock armour breakwater and the concrete armour unit breakwater with Xbloc units ($3m^3$) on both the front and the rear slope. These two designs are chosen because the rock armour breakwater is the cheapest design and the concrete armour unit breakwater with $3m^3$ Xbloc units is the cheapest of the designs with concrete armour units on both the front slope and the rear slope. The breakwater designs are tested on the settlements of the subsoil, the stability of the slopes and earthquake resistance.

7.2 Settlements subsoil

7.2.1 Introduction

The settlements of the subsoil are determined with the simulation program D-Settlement. The input and results for the simulations can be found in appendix IV.

7.2.2 Rock armour breakwater

The maximum occurring settlement for the rock armour breakwater after 10000 days is 1.112 meters. According to the simulation in D-Settlement, the subsoil will not settle any further as a result of the application of the breakwater. To compensate the occurring settlement, the crest level of the breakwater needs to be increased with 1.2 meters. Liquefaction due to applying the breakwater is not expected.

7.2.3 Concrete armour unit breakwater

The maximum occurring settlement for the concrete armour unit breakwater after 10000 days is 1.075 meters. According to the simulation in D-Settlement, the subsoil will no settle any further as a result of the application of the breakwater. To compensate the occurring settlement, the crest level of the breakwater needs to be increased with 1.2 meters. Liquefaction due to applying the breakwater is not expected.

7.3 Stability slopes

7.3.1 Introduction

The stability of the slopes is checked using the simulation program D-Geo Stability. For these simulations, earthquakes are not taken into account and geotextiles are only applied when necessary. Damage is not allowed. As recommended by (Asmerom, 2015), the total unit weight of the soil materials is divided by a factor 1.2 to implement a level of safety. Although damage is not allowed, a minimum safety factor of only 1.0 is required. This is due to the safety in the soil material parameters. The input and results for the simulations can be found in appendix V.

7.3.2 Rock armour breakwater

Both the front slope as the rear slope of the rock armour breakwater are stable without the need of applying a geotextile, according to the simulation in D-Geo Stability. The front slope has a probability of failure of 6.36E-12, this results in a safety factor of 2.21. The rear slope has a probability of failure of 3.80E-03, this results in a safety factor of 1.53.

7.3.3 Concrete armour unit breakwater

Both the front slope as the rear slope of the concrete armour unit breakwater are stable without the need of applying a geotextile, according to the simulation in D-Geo Stability. The front slope has a probability of failure of 1.44E-03, this results in a safety factor of 1.61. The rear slope has a probability of failure of 2.55E-03, this results in a safety factor of 1.57.

Note: The interlocking strength of the Xbloc units is not taken into account.

7.4 Seismological resistance

7.4.1 Introduction

The seismological resistance of the breakwater designs is checked using the simulation program D-Geo Stability. For these simulations a horizontal earthquake factor of 0.3g and a vertical earthquake factor of 0.15g are applied to the models. The vertical load is applied in both direction, this means that the value is applied both negative as positive. When necessary, geotextiles are applied. Damage is not allowed. As described in/recommended by (Asmerom, 2015), the total unit weight of the soil materials is divided by a factor 1.2 to implement a level of safety. Although damage is not allowed, a minimum safety factor of only 1.0 is required. This is due to the safety in the soil material parameters. The input and results for the simulations can be found in appendix VI.

7.4.2 Rock armour breakwater

According to the simulation in D-Geo Stability, both the front slope as the rear slope of the rock armour breakwater are not stable during an earthquake, even when a geotextile is applied. The front slope has a probability of failure of 9.90E-01, this results in a safety factor of 0.77. The rear slope has a probability of failure of 9.78E-01, this results in a safety factor of 0.74.

7.4.3 Concrete armour unit breakwater

According to the simulation in D-Geo Stability, both the front slope as the rear slope of the concrete armour unit breakwater are not stable during an earthquake, even when a geotextile is applied. The front slope has a probability of failure of 9.76E-01, this results in a safety factor of 0.75. The rear slope has a probability of failure of 9.78E-01, this results in a safety factor of 0.74.

Note: The interlocking strength of the Xbloc units is not taken into account.

8 CONCLUSION

Both the rock armour breakwater and the concrete armour unit breakwater are unstable during the event of an earthquake and both designs did not have taken into account the settlement of the subsoil. Therefore both designs and related costs have to be redesigned to get a design that meets all the requirements. However, due to a lack of time, it is decided to omit these redesigns.

The rock armour breakwater is more beneficial than the concrete armour unit breakwater due to less construction costs (total costs of approximately € 37,500,000.-). Therefore, the rock armour breakwater is used as reference design during the continuation of this graduation project. The rubble mound breakwater designs will not be redesigned to meet the requirements on the settlement of the subsoil and the seismological resistance. Moreover, the concept designs in the next phases of the graduation process will not be designed for settlement of the subsoil and the seismological resistance.

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10 APPENDIX I: ROCK ARMOUR RUBBLE MOUND BREAKWATER

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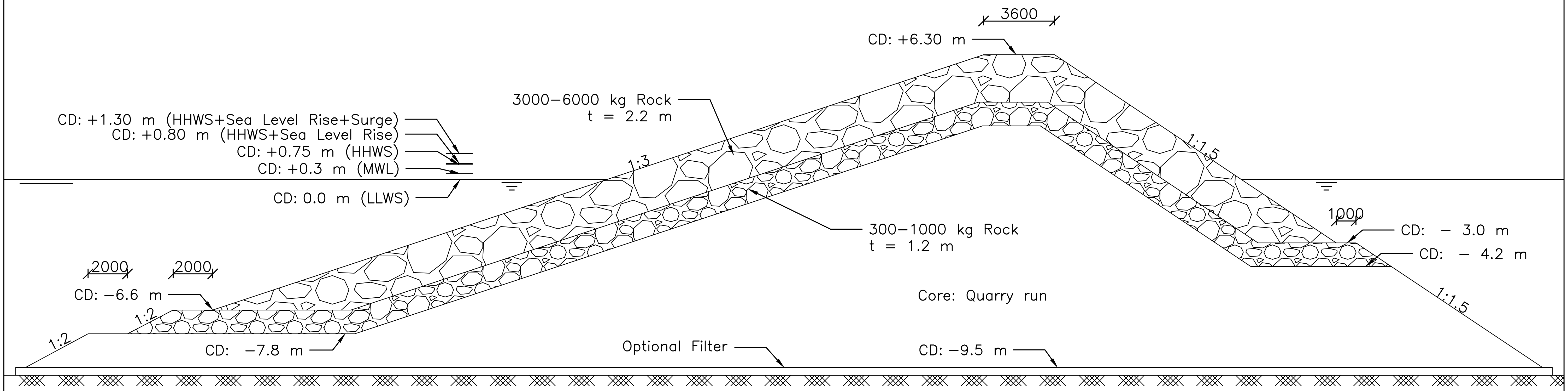

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Designed by D. Janssen	Checked by J. v. Grieken	Approved by - date	Filename RMB.dwg	Date 05-03-2015	Scale 1:200
Information Traditional Rubble Mound Breakwater		Rubble Mound Breakwater			
				Edition V1	Sheet 1/1

10 APPENDIX II: CONCRETE ARMOUR RUBBLE MOUND BREAKWATER

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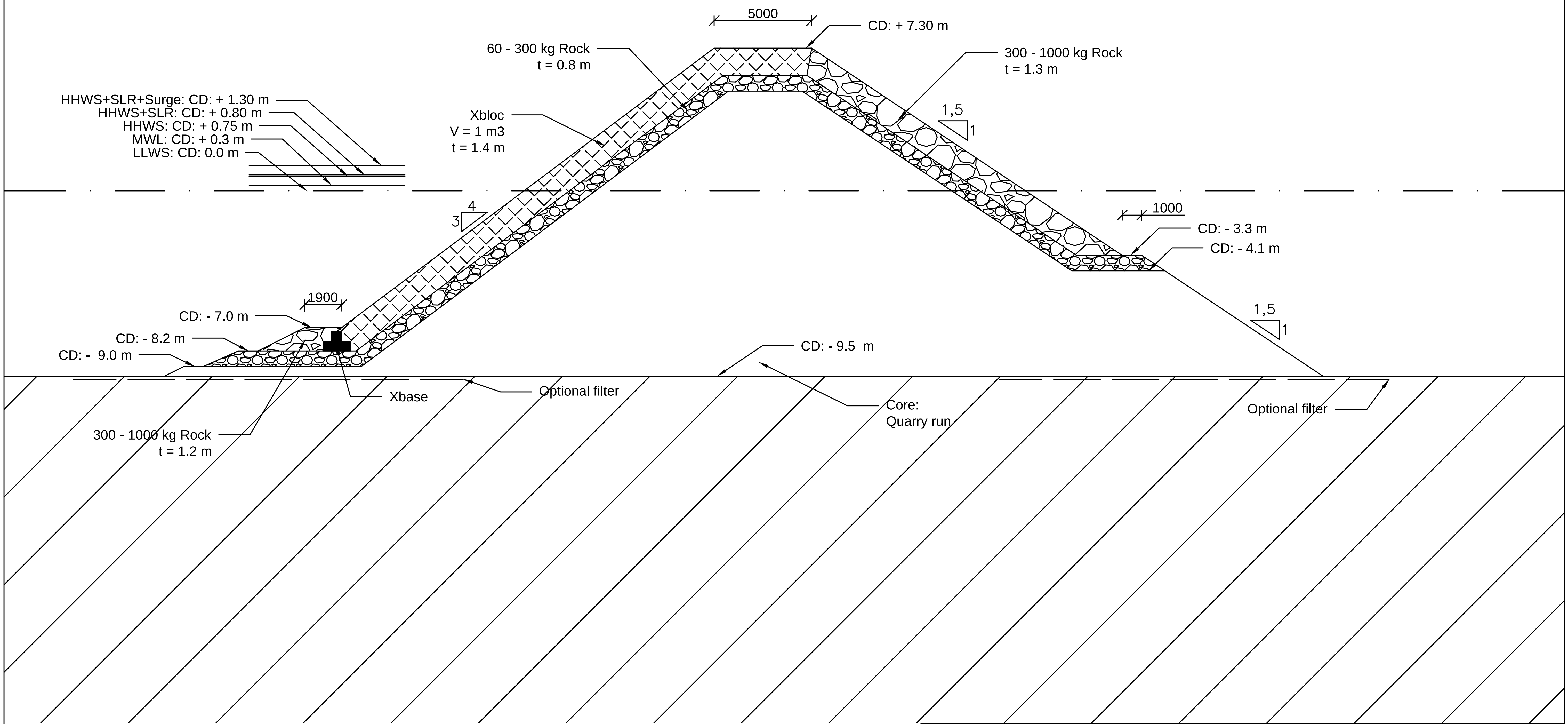
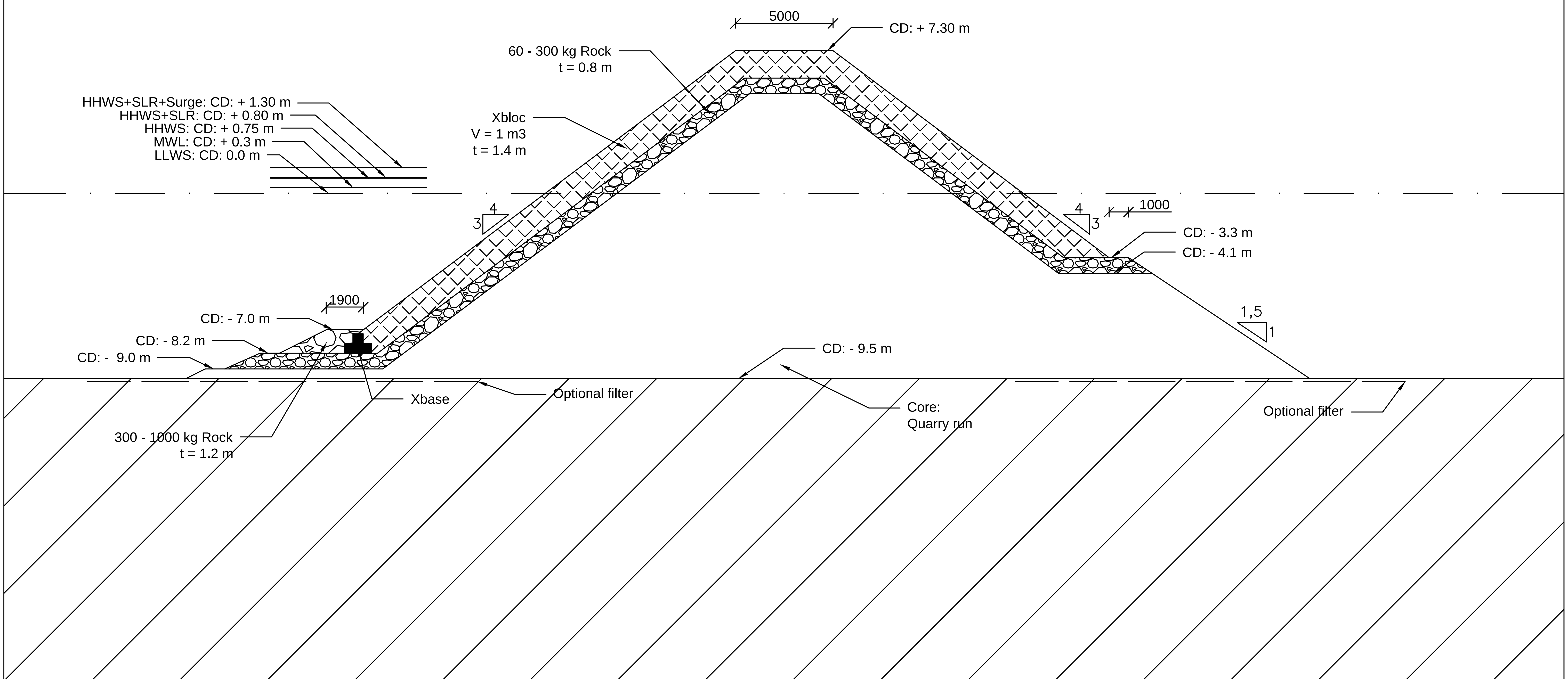
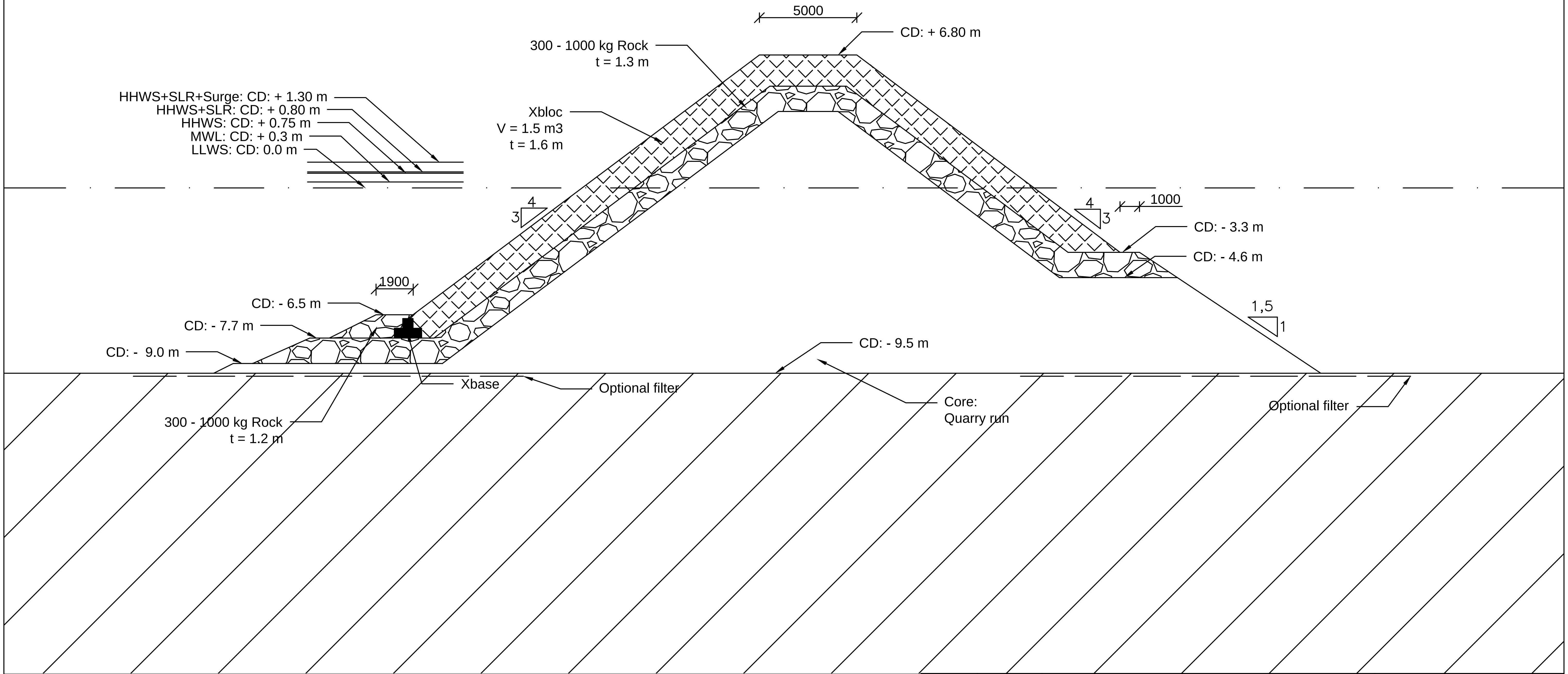


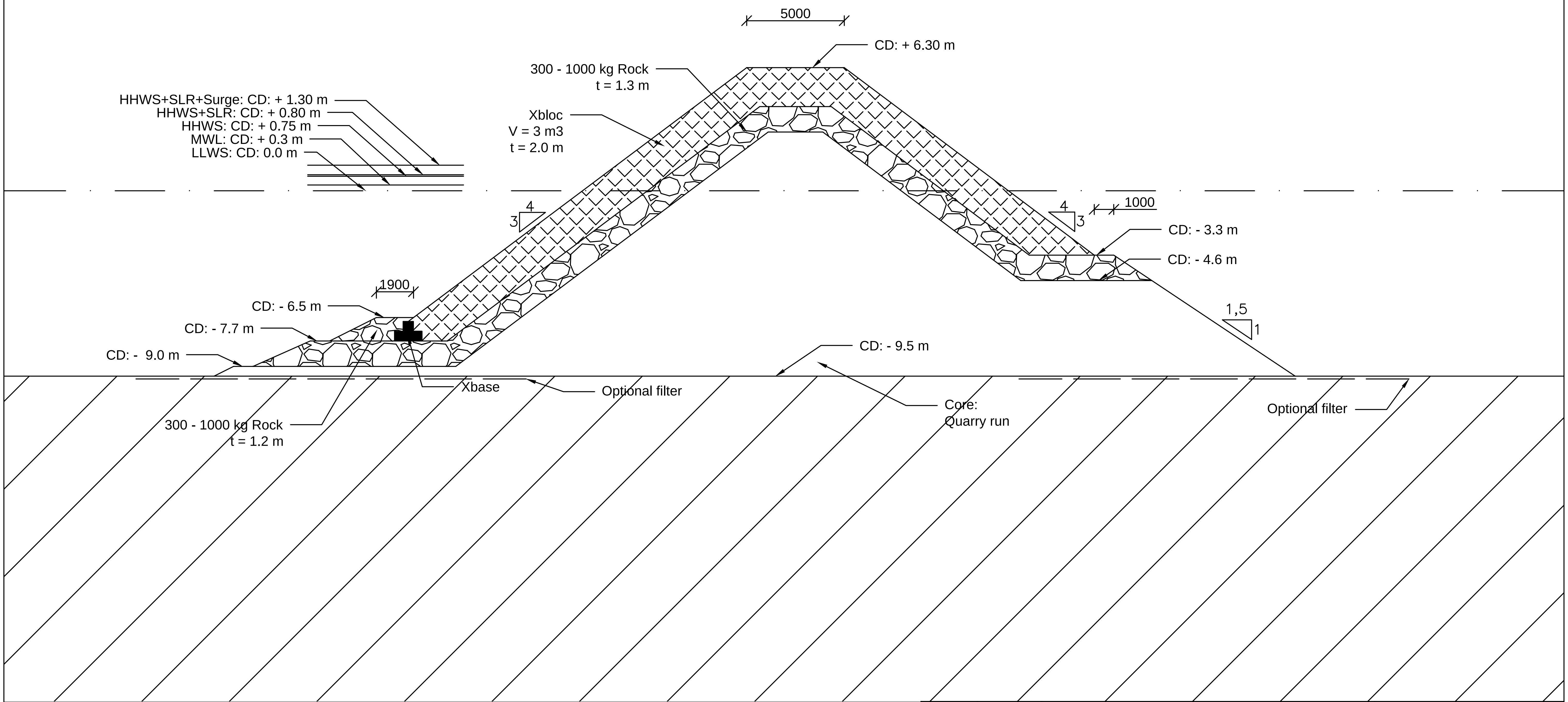
Itemref	Quantity	Title/Name, designation, material, dimension etc		Article No./Reference	
Designed by J. van Grieken	Checked by D. Janssen	Approved by - date	Filename BWXBLOC.dwg	Date 09/03/2015	Scale 1:200
Information Traditional Breakwater With Xbloc (1m3) Unit Armour Layer On Front Slope And Rock Armour Layer On Back Slope		Breakwater Design		Edition V1	Sheet 1/4



Itemref	Quantity	Title/Name, designation, material, dimension etc		Article No./Reference	
Designed by J. van Grieken	Checked by D. Janssen	Approved by - date	Filename BWXBLOC.dwg	Date 09/03/2015	Scale 1:200
Information Traditional Breakwater With Xbloc (1m3) Unit Armour Layer On Front And Back Slope		Breakwater Design		Edition V1	Sheet 2/4



Itemref	Quantity	Title/Name, designation, material, dimension etc		Article No./Reference	
Designed by J. van Grieken	Checked by D. Janssen	Approved by - date	Filename BWXBLOC.dwg	Date 09/03/2015	Scale 1:200
Information Traditional Breakwater With Xbloc Unit (1,5m3) Armour Layer On Front And Back Slope		Breakwater Design		Edition V1	Sheet 3/4



Itemref	Quantity	Title/Name, designation, material, dimension etc	Article No./Reference
Designed by J. van Grieken	Checked by D. Janssen	Approved by - date	Filename BWXBLOC.dwg	Date 09/03/2015	Scale 1:200

Information	
Traditional Breakwater With Xbloc Unit (3m3) Armour Layer On Front And Back Slope	Breakwater Design
	Edition V1 | Sheet 4/4

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11 APPENDIX III: RUBBLE MOUND BREAKWATER COSTS

The volumes of the required material are determined by calculating the area of the cross section of the materials and multiplying these with the required breakwater length.

		Rubble mount	Xbloc front Rock rear	xbloc 1m3	xbloc 1,5 m3	xbloc 3 m3
Costs traditional breakwater design						
Material usage						
xbloc (1 m3) (2450kg/m3)						
Volume per m (area in .dwg)	mm3/m	0	39923244,00	62902083,00	0,00	0,00
Volume per m	m3/m	0,00	39,92	62,90	0,00	0,00
layer thickness	m	1,40	1,40	1,40	1,40	1,40
concrete volume (xbloc.com)	m3/m2	0,58	0,58	0,58	0,58	0,58
concrete in BW	m3/m	0,00	16,54	26,06	0,00	0,00
packing density	1/100m2	57,81	57,81	57,81	57,81	57,81
layer area	m2/m	0,00	28,52	44,93	0,00	0,00
number of Xbloc per m	pcs/m	0,00	16,49	25,97	0,00	0,00
xbloc (1,5 m3) (2400 kg/m3)						
Volume per m (area in .dwg)	mm3/m	0	0	0	67412083	0
Volume per m	m3/m	0,00	0,00	0,00	67,41	0,00
layer thickness	m	1,60	1,60	1,60	1,60	1,60
concrete volume (xbloc.com)	m3/m2	0,66	0,66	0,66	0,66	0,66
concrete in BW	m3/m	0,00	0,00	0,00	27,81	0,00
packing density	1/100m2	44,12	44,12	44,12	44,12	44,12
layer area	m2/m	0,00	0,00	0,00	42,13	0,00
number of Xbloc per m	pcs/m	0,00	0,00	0,00	18,59	0,00
xbloc (3 m3) (2400 kg/m3)						
Volume per m (area in .dwg)	mm3/m	0	0	0	0	79618750
Volume per m	m3/m	0,00	0,00	0,00	0,00	79,62
layer thickness	m	2,00	2,00	2,00	2,00	2,00
concrete volume (xbloc.com)	m3/m2	0,83	0,83	0,83	0,83	0,83
concrete in BW	m3/m	0,00	0,00	0,00	0,00	33,04
packing density	1/100m2	27,79	27,79	27,79	27,79	27,79
layer area	m2/m	0,00	0,00	0,00	0,00	39,81
number of Xbloc per m	pcs/m	0,00	0,00	0,00	0,00	11,06
3000 - 6000 kg						
Volume per m (area in .dwg)	mm3/m	123525999	0	0	0	0
Volume per m	m3/m	123,53	0,00	0,00	0,00	0,00
300 - 1000 kg						
Volume per m (area in .dwg)	mm3/m	78987160	26182292	2945416	70702496	68189163
Volume per m	m3/m	78,99	26,18	2,95	70,70	68,19
60 - 300 kg						
Volume per m (area in .dwg)	mm3/m	0	44243393	43053330	0	0
Volume per m	m3/m	0,00	44,24	43,05	0,00	0,00
quarry run						
Volume per m (area in .dwg)	mm3/m	430852632	388331241	369420636	321184803	287904803
Volume per m	m3/m	430,85	388,33	369,42	321,18	287,90

Costs traditional breakwater design

		Rubble mount	Xbloc front Rock rear	xbloc 1m3	xbloc 1,5 m3	xbloc 3 m3
Total material usage per m						
number of Xbloc (1 m3)	pcs/m	0,00	16,49	25,97	0,00	0,00
number of Xbloc (1,5 m3)	pcs/m	0,00	0,00	0,00	18,59	0,00
number of Xbloc (3 m3)	pcs/m	0,00	0,00	0,00	0,00	11,06
3000 - 6000 kg	m3/m	123,53	0,00	0,00	0,00	0,00
300 - 1000 kg	m3/m	78,99	26,18	2,95	70,70	68,19
60 - 300 kg	m3/m	0,00	44,24	43,05	0,00	0,00
quarry run	m3/m	430,85	388,33	369,42	321,18	287,90
Total material usage						
number of Xbloc (1 m3)	pcs	0	10056	15844	0	0
number of Xbloc (1,5 m3)	pcs	0	0	0	11339	0
number of Xbloc (3 m3)	pcs	0	0	0	0	6748
3000 - 6000 kg	m3	75351	0	0	0	0
300 - 1000 kg	m3	48182	15971	1797	43129	41595
60 - 300 kg	m3	0	26988	26263	0	0
quarry run	m3	262820	236882	225347	195923	175622
Unit prices						
unit price Xbloc (1 m3)	€/pcs	€ 1.000,00	€ 1.000,00	€ 1.000,00	€ 1.000,00	€ 1.000,00
unit price Xbloc (1,5 m3)	€/pcs	€ 1.500,00	€ 1.500,00	€ 1.500,00	€ 1.500,00	€ 1.500,00
unit price Xbloc (3 m3)	€/pcs	€ 2.000,00	€ 2.000,00	€ 2.000,00	€ 2.000,00	€ 2.000,00
unit price 3000 - 6000 kg	€/m3	€ 80,00	€ 80,00	€ 80,00	€ 80,00	€ 80,00
unit price 300 - 1000 kg	€/m3	€ 60,00	€ 60,00	€ 60,00	€ 60,00	€ 60,00
unit price 60 - 300 kg	€/m3	€ 50,00	€ 50,00	€ 50,00	€ 50,00	€ 50,00
unit price quarry run	€/m3	€ 40,00	€ 40,00	€ 40,00	€ 40,00	€ 40,00
Material costs per m						
costs Xbloc (1 m3)	€/m	€ -	€ 16.485,45	€ 25.974,07	€ -	€ -
costs Xbloc (1,5 m3)	€/m	€ -	€ -	€ -	€ 27.883,32	€ -
costs Xbloc (3 m3)	€/m	€ -	€ -	€ -	€ -	€ 22.126,05
costs 3000 - 6000 kg	€/m	€ 9.882,08	€ -	€ -	€ -	€ -
costs 300 - 1000 kg	€/m	€ 4.739,23	€ 1.570,94	€ 176,72	€ 4.242,15	€ 4.091,35
costs 60 - 300 kg	€/m	€ -	€ 2.212,17	€ 2.152,67	€ -	€ -
costs quarry run	€/m	€ 17.234,11	€ 15.533,25	€ 14.776,83	€ 12.847,39	€ 11.516,19
total material costs						
costs Xbloc (1 m3)	€	-	10.056.123,35	15.844.181,04	-	-
costs Xbloc (1,5 m3)	€	-	-	-	17.008.826,93	-
costs Xbloc (3 m3)	€	-	-	-	-	13.496.890,88
costs 3000 - 6000 kg	€	6.028.068,75	-	-	-	-
costs 300 - 1000 kg	€	2.890.930,06	958.271,89	107.802,23	2.587.711,35	2.495.723,37
costs 60 - 300 kg	€	-	1.349.423,49	1.313.126,57	-	-
costs quarry run	€	10.512.804,22	9.475.282,28	9.013.863,52	7.836.909,19	7.024.877,19
material costs per meter	€/m	€ 31.855,41	€ 35.801,80	€ 43.080,28	€ 44.972,86	€ 37.733,59
length of breakwater	m	610	610	610	610	610
total material costs	€	€ 19.431.803,03	€ 21.839.101,00	€ 26.278.973,35	€ 27.433.447,47	€ 23.017.491,44
Multiplification factors						
construction factor	-	1,3	1,3	1,3	1,3	1,3
complexity factor	-	1,1	1,1	1,1	1,1	1,1
indirect costs constructor factor	-	1,35	1,35	1,35	1,35	1,35
Total costs						
Total costs	€	€ 37.513.095,75	€ 42.160.384,48	€ 50.731.558,04	€ 52.960.270,35	€ 44.435.267,23
Determining bandwidth						
Factors +0.1	€	€ 47.335.872,18	€ 53.200.050,04	€ 64.015.579,07	€ 66.827.878,05	€ 56.070.609,15
Deviation	€	€ 9.822.776,43	€ 11.039.665,56	€ 13.284.021,03	€ 13.867.607,70	€ 11.635.341,92
Deviation	%	26,18%	26,18%	26,18%	26,18%	26,18%
Factors -0.1	€	€ 29.147.704,54	€ 32.758.651,50	€ 39.418.460,02	€ 41.150.171,21	€ 34.526.237,16
Deviation	€	€ -8.365.391,20	€ -9.401.732,98	€ -11.313.098,03	€ -11.810.099,14	€ -9.909.030,07
Deviation	%	-22,30%	-22,30%	-22,30%	-22,30%	-22,30%

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12 APPENDIX IV: SETTLEMENTS WITH D-SETTLEMENT

12.1 Introduction

The settlements of the subsoil are determined with the simulation program D-Settlement. The 2D models are made in a NEN-Bjerrum calculation model, with a Terzaghi consolidation model. Two models are made: a model for the breakwater with rock armour on both the front and the rear side of the breakwater and a model for the breakwater with Xbloc concrete armour units of 3m^3 on both the front and the rear side of the breakwater.

This appendix describes first the input that is used in both models. After that, the two different models including the results are described.

12.2 General Input

12.2.1 Introduction

The general input for the models in D-Settlement is provided in the Basis of Design and in the report of the borehole provided by the client. In this chapter the geometry of the subsoil, the specifications of the soil layers and the calculation options in the models are described.

12.2.2 Geometry

The subsoil is schematized as shown in figure 14. The range of the model is from $x = -50$ to $x = 150$.

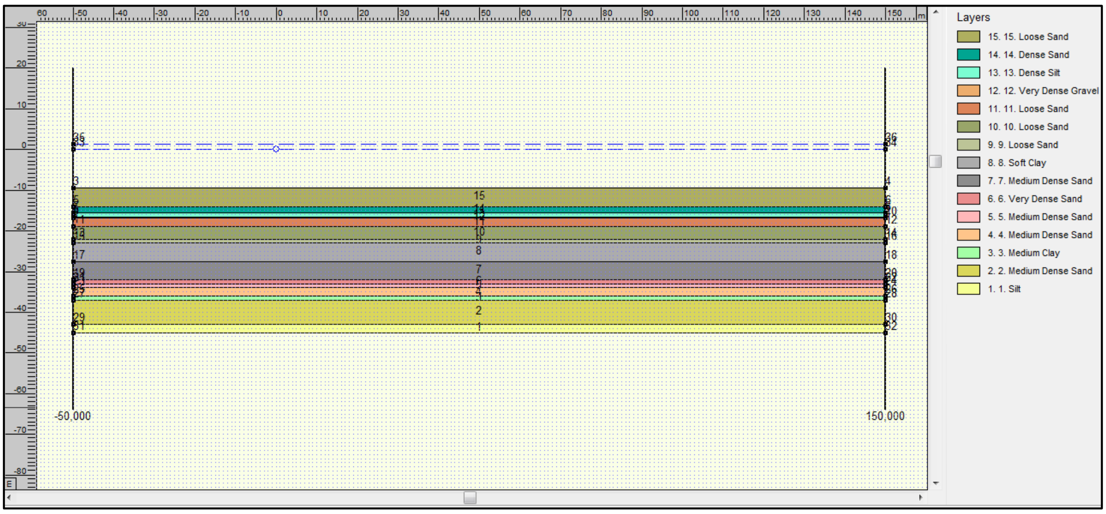


Figure 14: Schematization of the subsoil

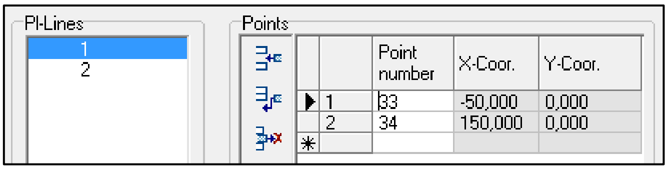
The coordinates of the points used in the schematization are shown in table 18.

	X Co-ordinate [m]	Y Co-ordinate [m]
1	-50,000	-16,500
2	150,000	-16,500
3	-50,000	-9,500
4	150,000	-9,500
5	-50,000	-14,000
6	150,000	-14,000
7	-50,000	-15,500
8	150,000	-15,500
9	-50,000	-16,800
10	150,000	-16,800
11	-50,000	-19,000
12	150,000	-19,000
13	-50,000	-22,000
14	150,000	-22,000
15	-50,000	-23,000
16	150,000	-23,000
17	-50,000	-27,500
18	150,000	-27,500
19	-50,000	-32,000
20	150,000	-32,000
21	-50,000	-33,000
22	150,000	-33,000
23	-50,000	-34,000
24	150,000	-34,000
25	-50,000	-36,000
26	150,000	-36,000
27	-50,000	-37,000
28	150,000	-37,000
29	-50,000	-43,000
30	150,000	-43,000
31	-50,000	-45,000
32	150,000	-45,000
33	-50,000	0,000
34	150,000	0,000
35	-50,000	1,320
36	150,000	1,320

Table 18: The coordinates of the points in the schematisation

12.2.3 Water

The water levels have the coordinates shown in figure 15 and 16.

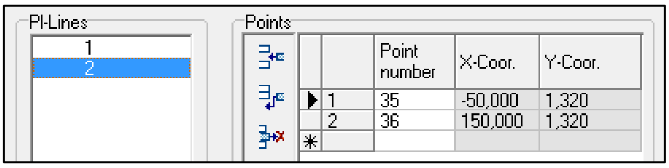


PI-Lines: 1, 2

Points

	Point number	X-Coor.	Y-Coor.
1	33	-50,000	0,000
2	34	150,000	0,000

Figure 15: The coordinates of the LLWS



PI-Lines: 1, 2

Points

	Point number	X-Coor.	Y-Coor.
1	35	-50,000	1,320
2	36	150,000	1,320

Figure 16: The coordinates of the HHWS+SLR+Surge

The unit weight of the water is set to $\left(\frac{1020*9.78}{1000}\right) = 9.98\ kN/m^3$.

12.2.4 Soil layers

Table 19 shows the parameters used in the models.

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
15.	Loose Sand	4.50	18.0	20.2	0	30
14.	Dense Sand	1.50	21.0	23.0	0	45
13.	Dense Silt	1.00	18.0	18.0	7	32
12.	Very Dense Gravel	0.30	22.0	24.0	0	35
11.	Loose Sand	2.20	18.0	20.0	0	27
10.	Loose Sand	3.00	19.0	21.0	22	28
9.	Loose Sand	1.00	19.0	21.0	22	28
8.	Soft Clay	4.50	15.0	15.0	20	25
7.	Medium Dense Sand	4.50	20.5	22.5	22	30
6.	Very Dense Sand	1.00	22.0	24.0	22	34
5.	Medium Dense Sand	1.00	20.5	22.5	22	30
4.	Medium Dense Sand	2.00	20.5	22.5	0	38
3.	Medium Clay	1.00	16.0	16.0	20	25
2.	Medium Dense Sand	6.00	20.5	22.5	22	30
1.	Silt	2.00	17.0	17.0	7	30

Table 19: Used parameters in the models

Where:

γ_{dry} = Dry unit weight
γ_{sat} = wet unit weight
c = cohesion
φ = Soil friction angle

12.2.5 Calculation options

The used calculation options are as shown in figure 17.

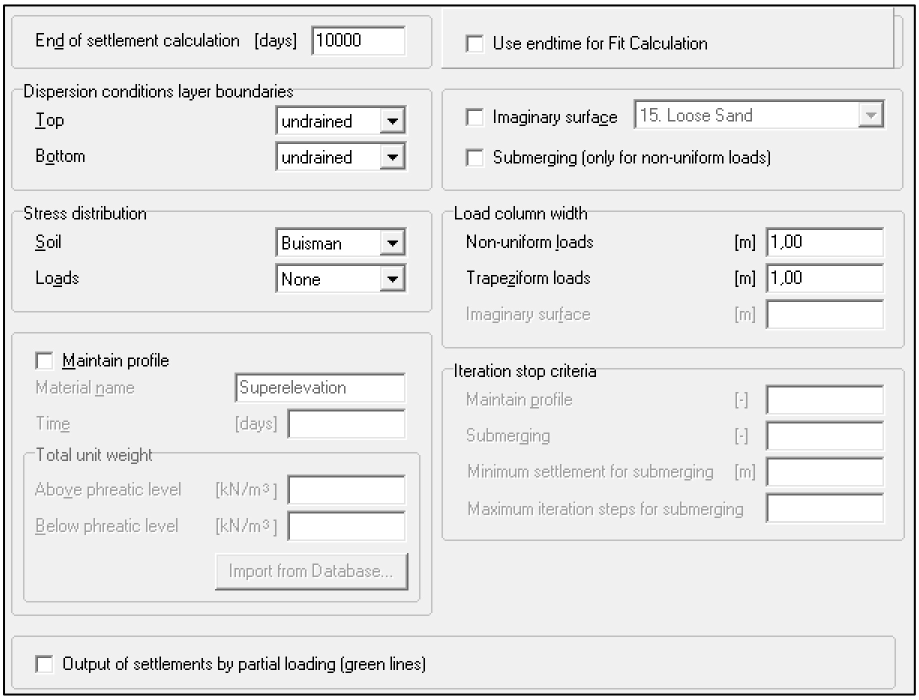


Figure 17: The used calculation options

12.3 Rock armour breakwater

12.3.1 Input

An overview of the specific input for the model of the rock armour breakwater is shown in figure 18.

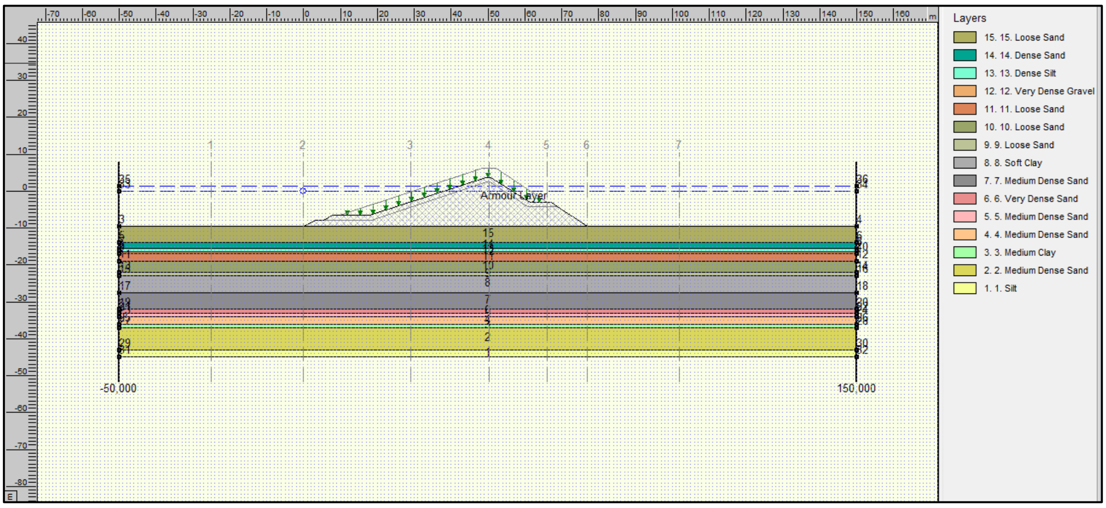


Figure 18: An overview of the specific input for the rock armour breakwater

The x-coordinates of the verticals are shown in table 20. The properties of the loads that represent the different layers of the breakwater are shown in figure 19, 20 and 21.

	X co-ordinate [m]
1	-25,000
2	0,000
3	29,180
4	50,360
5	66,140
6	76,890
7	101,890

Table 20: X-coordinates of the verticals

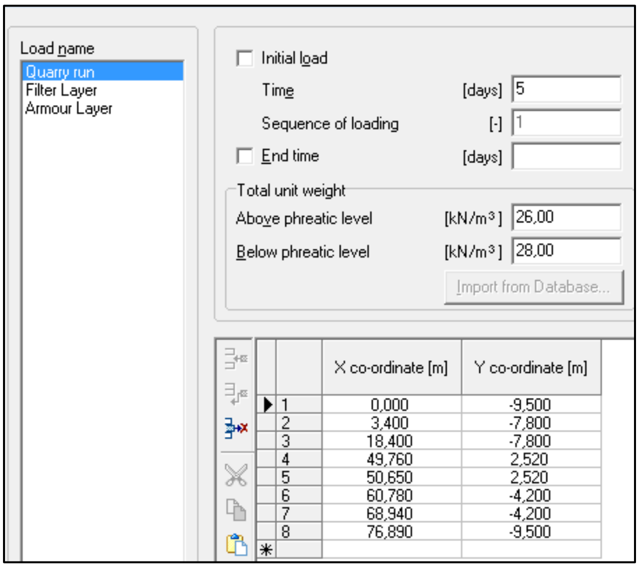


Figure 19: Load of Quarry Run

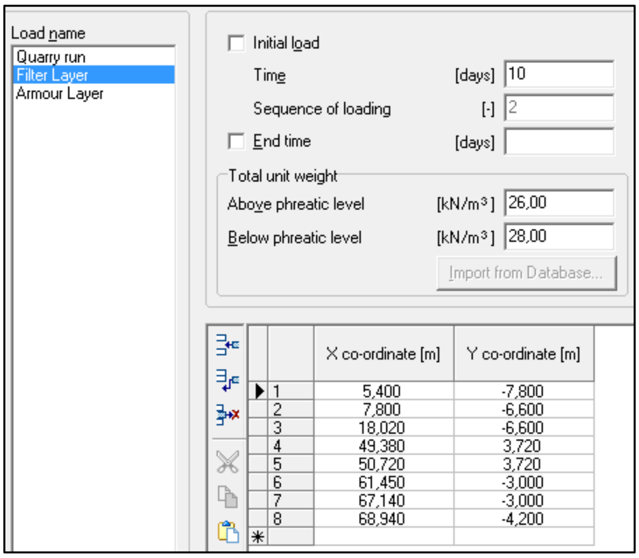


Figure 20: Load of Filter Layer

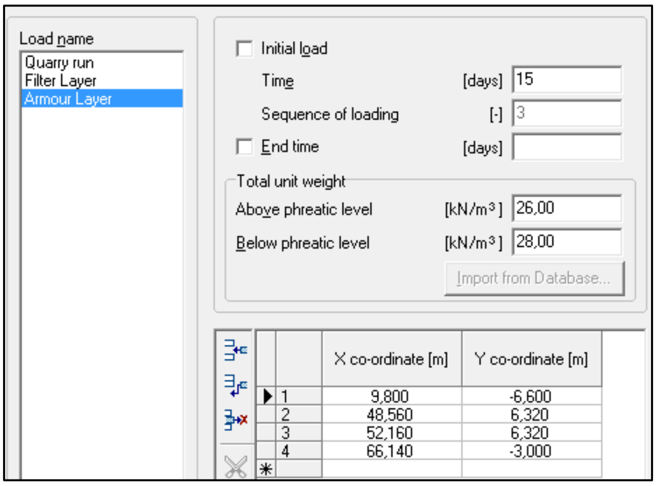


Figure 21: Load of Armour Layer

12.3.2 Results

The results of the settlement simulation are shown in table 21.

Vertical number	Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	10000	0,424	100,000	0,000
2	10000	0,562	100,000	0,000
3	10000	0,990	100,000	0,000
4	10000	1,119	100,000	0,000
5	10000	0,913	100,000	0,000
6	10000	0,653	100,000	0,000
7	10000	0,439	100,000	0,000

Table 21: The results of the settlement simulation

The maximum settlement after 10000 days is 1.119 meters. The subsoil will not settle any further as a result of the application of the breakwater.

12.4 Concrete armour unit breakwater

12.4.1 Input

An overview of the specific input for the model of the concrete armour unit breakwater is shown in figure 22.

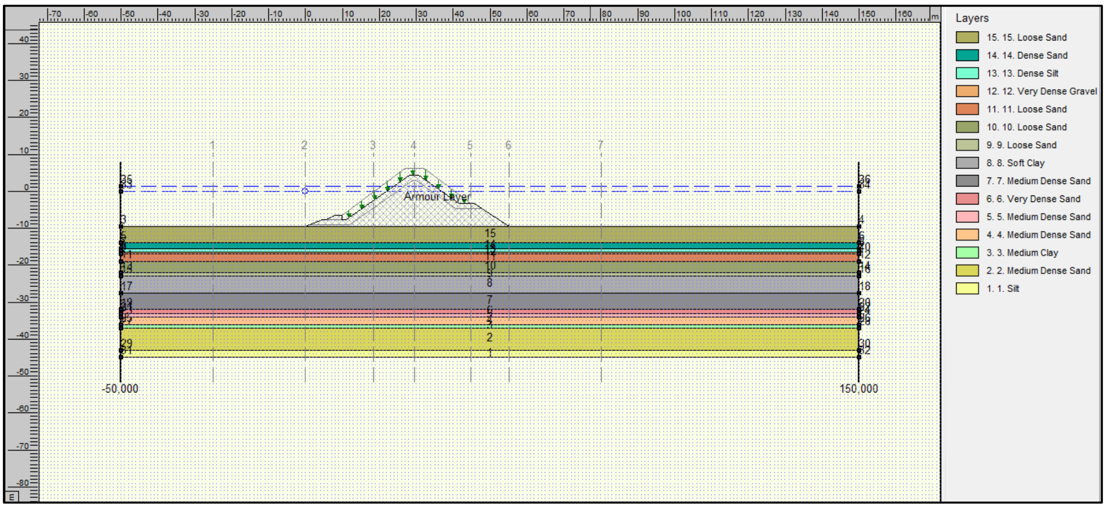


Figure 22: An overview of the specific input for the concrete armour unit breakwater

The x-coordinates of the verticals are shown in table 22. The properties of the loads that represent the different layers of the breakwater are shown in figure 23, 24, 25 and 26.

	X co-ordinate [m]
1	-25,000
2	0,000
3	18,450
4	29,500
5	44,830
6	55,130
7	80,130

Table 22: X-coordinates of the verticals

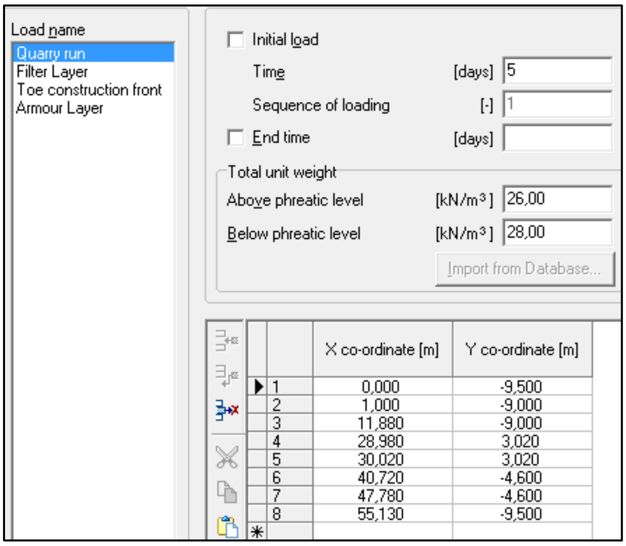


Figure 23: Load of Quarry Run

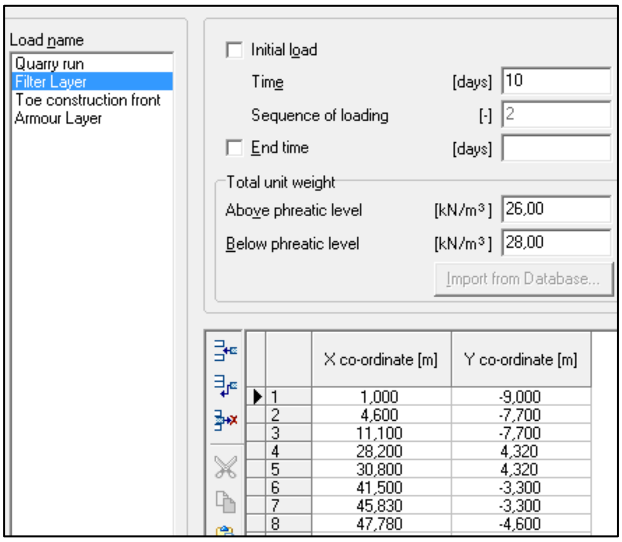


Figure 24: Load of Filter Layer

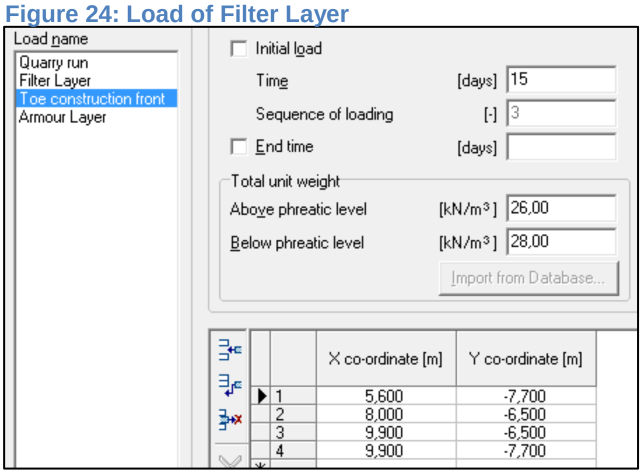


Figure 25: Load of Toe construction on the sea side of the breakwater

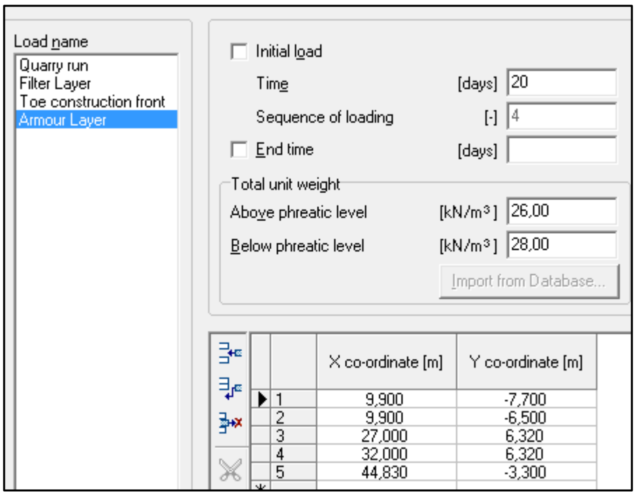


Figure 26: Load of Concrete Armour Layer

12.4.2 Results

The results of the settlement simulation are shown in table 23.

Vertical number	Time [days]	Settlement [m]	Part of final settlement [%]	Residual settlements [m]
1	10000	0,433	100,000	0,000
2	10000	0,596	100,000	0,000
3	10000	0,970	100,000	0,000
4	10000	1,082	100,000	0,000
5	10000	0,899	100,000	0,000
6	10000	0,650	100,000	0,000
7	10000	0,438	100,000	0,000

Table 23: The results of the settlement simulation

The maximum settlement after 10000 days is 1.082 meters. The subsoil will not settle any further as a result of the application of the breakwater.

13 APPENDIX V: STABILITY SLOPES WITH D-GEO STABILITY

13.1 Introduction

The stability of the slopes of the breakwater designs are checked using the simulation program D-Geo Stability. For these simulations, earthquakes are not taken into account and geotextiles are only applied when necessary. Damage is not allowed. As recommended by (Asmerom, 2015), the total unit weight of the soil materials is divided by a factor 1.2 to implement a level of safety. Although damage is not allowed, a minimum safety factor of only 1.0 is required. This is due to the safety in the soil material parameters.

Two models are made: a model for the breakwater with rock armour on both the front and the rear side of the breakwater and a model for the breakwater with Xbloc concrete armour units of $3m^3$ on both the front and the rear side of the breakwater.

This appendix describes first the input that is used in both models. After that, the two different models including the results are described.

Note: The interlocking strength of the Xbloc units is not taken into account.

13.2 General Input

13.2.1 Introduction

The general input for the models in D-Geo Stability is provided in the Basis of Design and in the report of the borehole provided by the client. In this chapter the geometry of the subsoil, the specifications of the soil layers and the calculation options in the models are described.

13.2.2 Geometry

The subsoil is schematized as shown in figure 27. The range of the model is from $x = -50$ to $x = 150$.

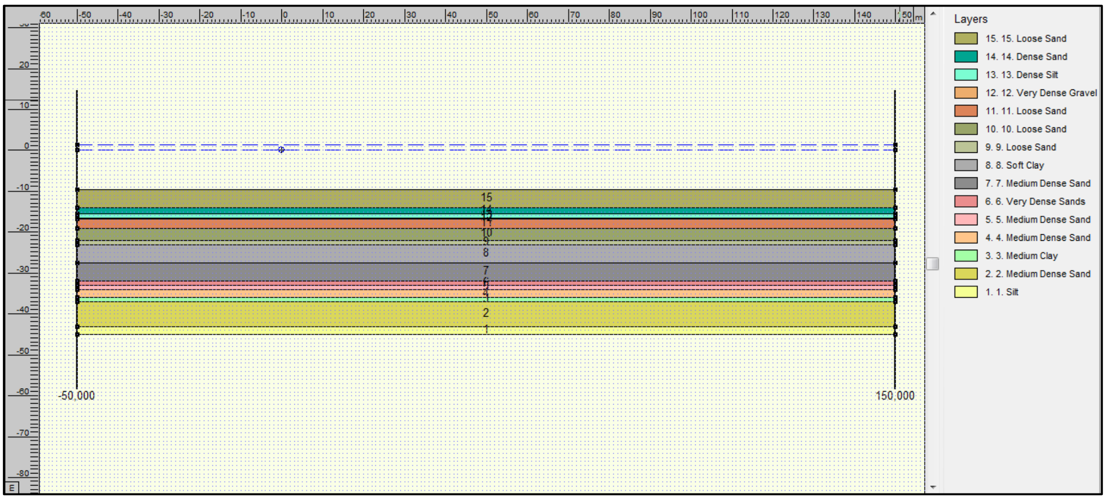


Figure 27: Schematization of the subsoil

The coordinates of the points used in the schematization are shown in table 24.

	X Co-ordinate [m]	Y Co-ordinate [m]
1	-50,000	0,000
2	150,000	0,000
3	-50,000	1,320
4	150,000	1,320
5	-50,000	-9,500
6	150,000	-9,500
7	-50,000	-14,000
8	150,000	-14,000
9	-50,000	-15,500
10	150,000	-15,500
11	-50,000	-16,500
12	150,000	-16,500
13	-50,000	-16,800
14	150,000	-16,800
15	-50,000	-19,000
16	150,000	-19,000
17	-50,000	-22,000
18	150,000	-22,000
19	-50,000	-23,000
20	150,000	-23,000
21	-50,000	-27,500
22	150,000	-27,500
23	-50,000	-32,000
24	150,000	-32,000
25	-50,000	-33,000
26	150,000	-33,000
27	-50,000	-34,000
28	150,000	-34,000
29	-50,000	-36,000
30	150,000	-36,000
31	-50,000	-37,000
32	150,000	-37,000
33	-50,000	-43,000
34	150,000	-43,000
35	-50,000	-45,000
36	150,000	-45,000

Table 24: The coordinates of the points in the schematisation

13.2.3 Water

The water levels have the coordinates shown in figure 28 and 29.

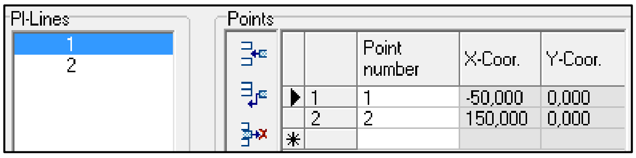


Figure 28: The coordinates of the LLWS

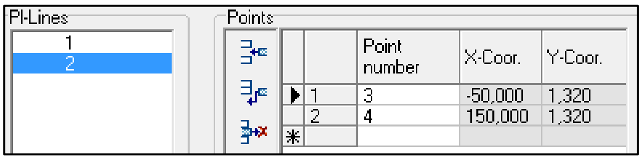


Figure 29: The coordinates of the HHWS+SLR+Surge

The unit weight of the water is set to $\left(\frac{1020*9.78}{1000}\right) = 9.98\ kN/m^3$.

13.2.4 Soil layers

Table 25 shows the parameters used in the models.

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m3]	[kN/m3]	[kN/m2]	[°]
15.	Loose Sand	4.50	15.00	16.67	0	30
14.	Dense Sand	1.50	17.50	19.17	0	45
13.	Dense Silt	1.00	15.00	15.00	7	32
12.	Very Dense Gravel	0.30	18.33	20.00	0	35
11.	Loose Sand	2.20	15.00	16.67	0	27
10.	Loose Sand	3.00	15.83	17.50	22	28
9.	Loose Sand	1.00	15.83	17.50	22	28
8.	Soft Clay	4.50	12.50	12.50	20	25
7.	Medium Dense Sand	4.50	17.08	18.75	22	30
6.	Very Dense Sand	1.00	18.33	20.00	22	34
5.	Medium Dense Sand	1.00	17.08	18.75	22	30
4.	Medium Dense Sand	2.00	17.08	18.75	0	38
3.	Medium Clay	1.00	13.33	13.33	20	25
2.	Medium Dense Sand	6.00	17.08	18.75	22	30
1.	Silt	2.00	14.12	14.12	7	30

Table 25: Used parameters in the models

Where:

γ_{dry} = Dry unit weight (reduced by a factor of 1.2 to implement safety)
γ_{sat} = wet unit weight (reduced by a factor of 1.2 to implement safety)
c = cohesion
φ = Soil friction angle

13.2.5 Earthquake

There is no earthquake factor applied.

13.2.6 Calculation options

The used calculation options are as shown in figure 30 and 31.

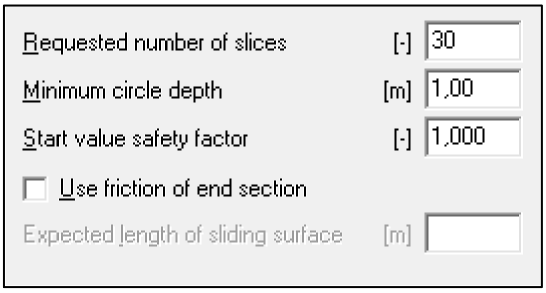


Figure 30: The used calculation options

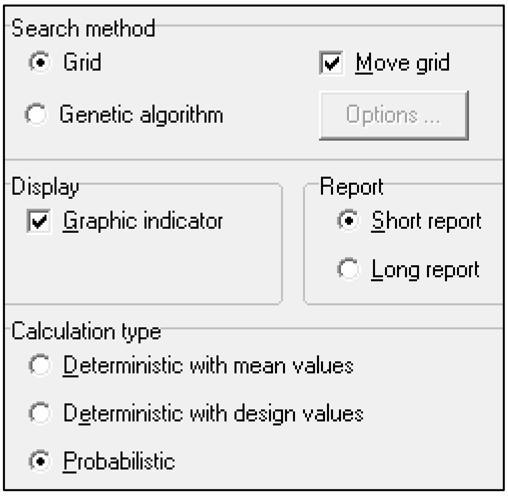


Figure 31: The used calculation options

13.3 Rock armour breakwater

13.3.1 Input

13.3.1.1 General

An overview of the specific input for the model of the rock armour breakwater is shown in figure 32.

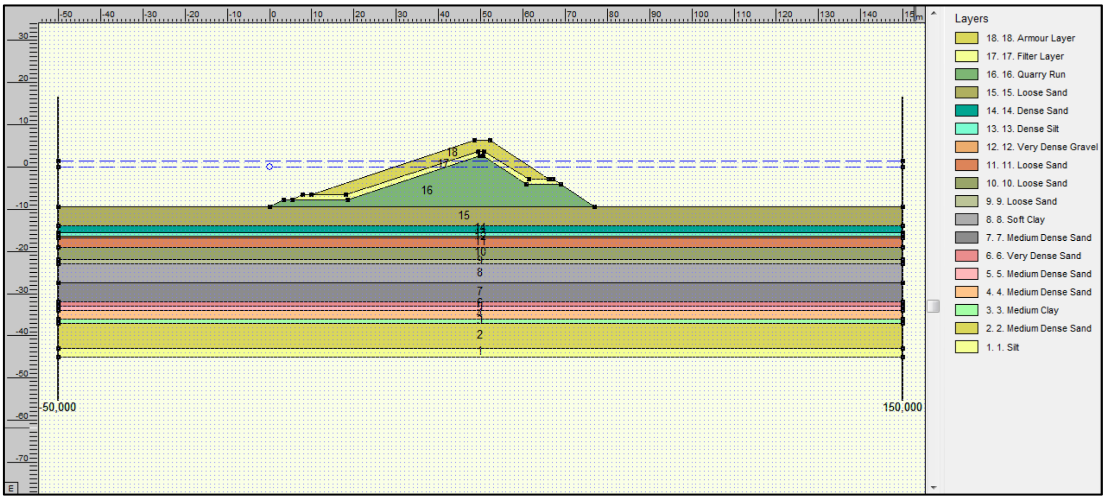


Figure 32: An overview of the specific input for the rock armour breakwater

The coordinates of the points used to create the breakwater are shown in table 26.

The soil parameters used for the breakwater are shown in table 27.

A geotextile is not applied.

X Co-ordinate [m]	Y Co-ordinate [m]
0,000	-9,500
3,400	-7,800
5,400	-7,800
7,800	-6,600
9,800	-6,600
18,020	-6,600
48,560	6,320
52,160	6,320
61,450	-3,000
66,140	-3,000
67,140	-3,000
68,940	-4,200
76,890	-9,500

Table 26: The coordinates of the points used to create the breakwater model

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
18.	Armour Layer	2.60	21.67	23.33	0	50
17.	Filter Layer	1.20	21.67	23.33	0	50
16.	Quarry Run	variable	21.67	23.33	0	45

Table 27: The soil parameters used for the breakwater model

Where:

γ_{dry} = Dry unit weight (reduced by a factor of 1.2 to implement safety)
γ_{sat} = wet unit weight (reduced by a factor of 1.2 to implement safety)
c = cohesion
φ = Soil friction angle

13.3.1.2 Front Slope

The slip circle definition used for the front slope is shown in figure 33.

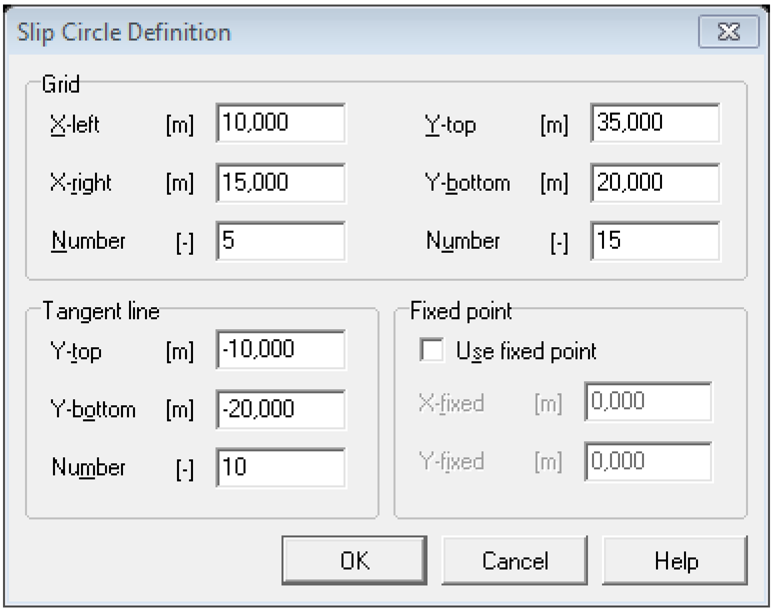


Figure 33: The slip circle definition used for the front slope

13.3.1.3 Rear Slope

The slip circle definition used for the rear slope is shown in figure 34.

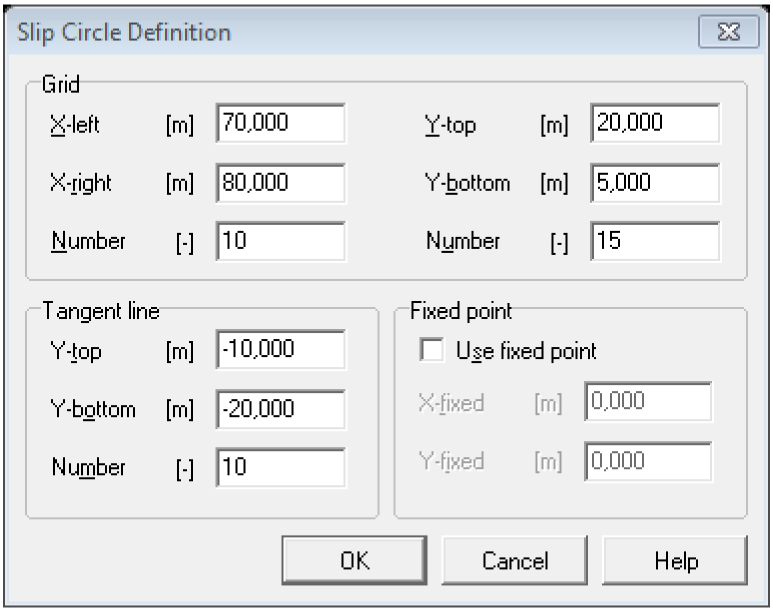


Figure 34: The slip circle definition used for the rear slope

13.3.2 Results

13.3.2.1 Front Slope

The simulation results for the front slope are shown in figure 35 and 36. Figure 35 shows the results for LLWS, figure 36 for HHWS+SLR+Surge (SLR = Sea Level Rise).

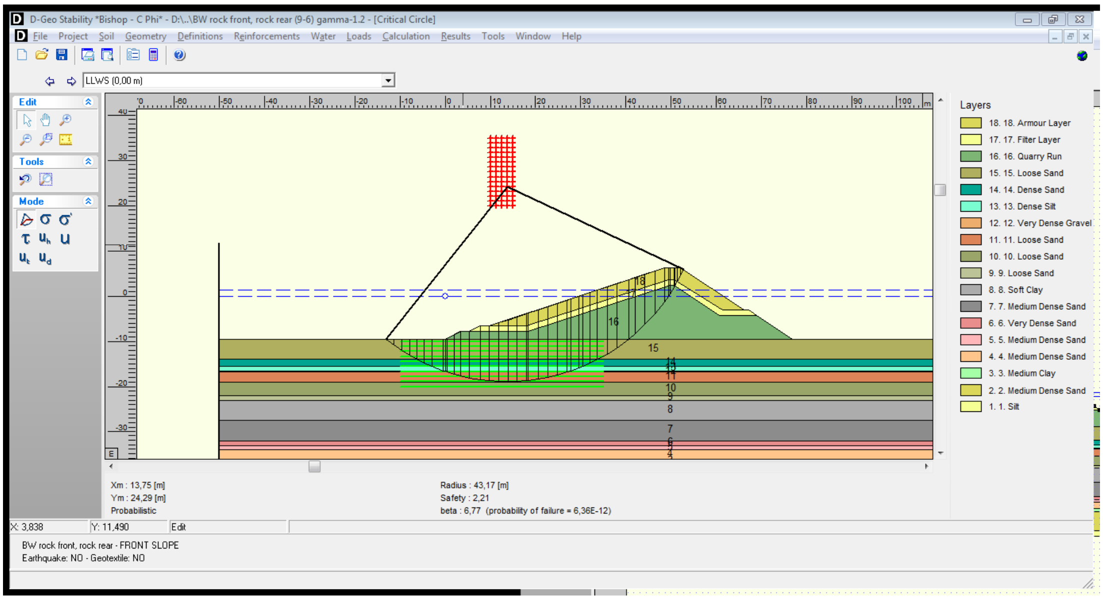


Figure 35: The simulation results for the front slope (LLWS)

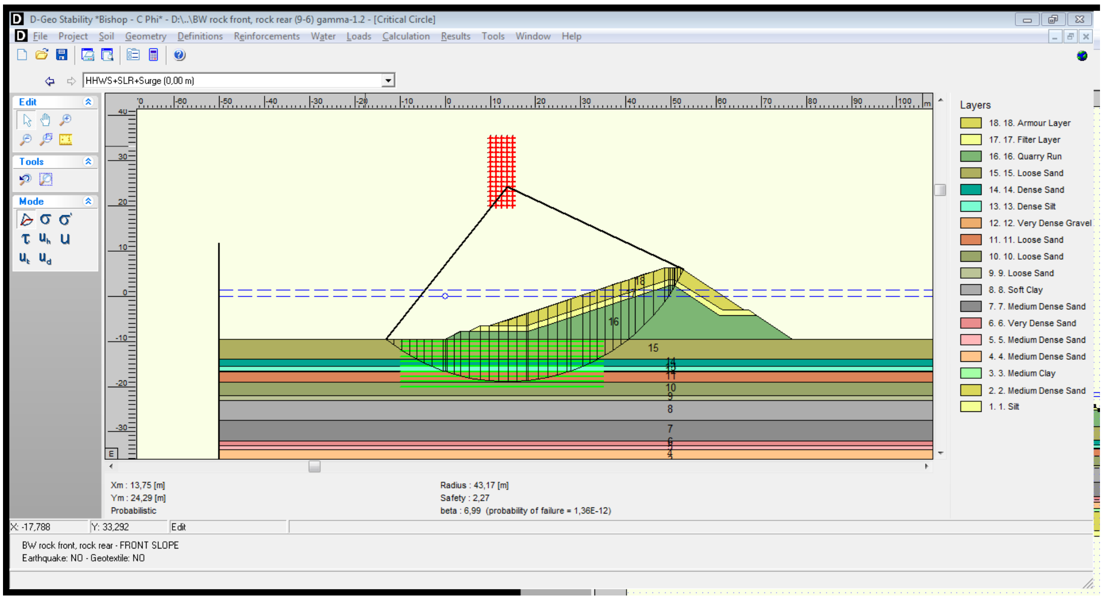


Figure 36: The simulation results for the front slope (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 28.

	Xm	Ym	Radius	Safety	beta	Probability of failure
	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	13.75	24.29	43.17	2.21	6.77	6.36E-12
HHWS +SLR +Surge	13.75	24.29	43.17	2.27	6.99	1.36E-12

Table 28: The normative slip circle properties for the front slope

The slip circle for LLWS has the biggest probability of failure, therefore LLWS is normative. The safety is bigger than 1.0 so the design complies with the requirements.

13.3.2.2 Rear Slope

The simulation results for the rear slope are shown in figure 37 and 38. Figure 37 shows the results for LLWS, figure 38 for HHWS+SLR+Surge (SLR = Sea Level Rise).

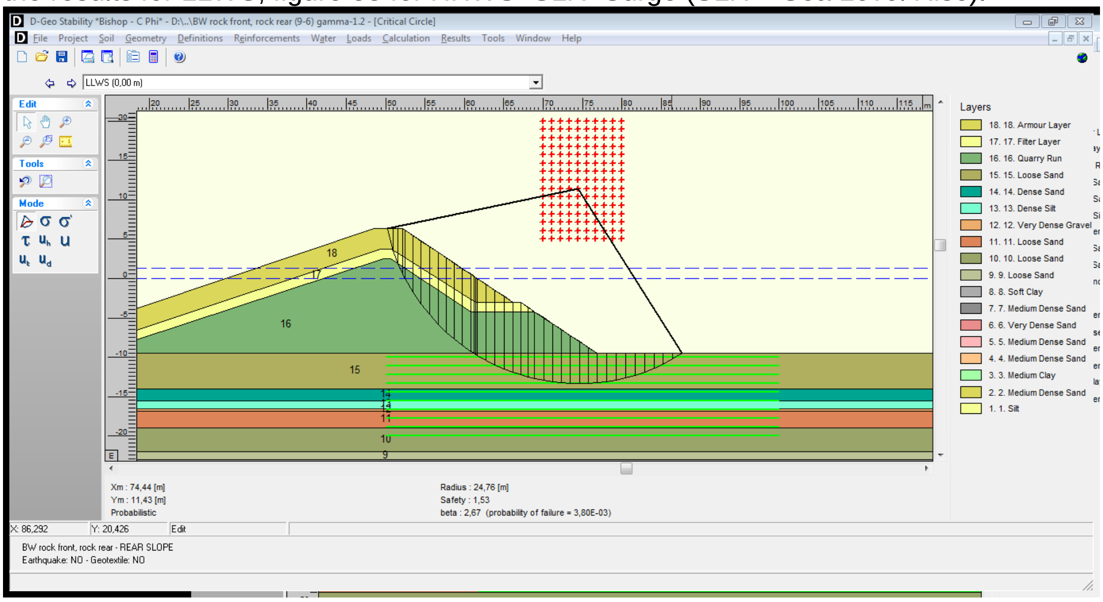


Figure 37: The simulation results for the rear slope (LLWS)

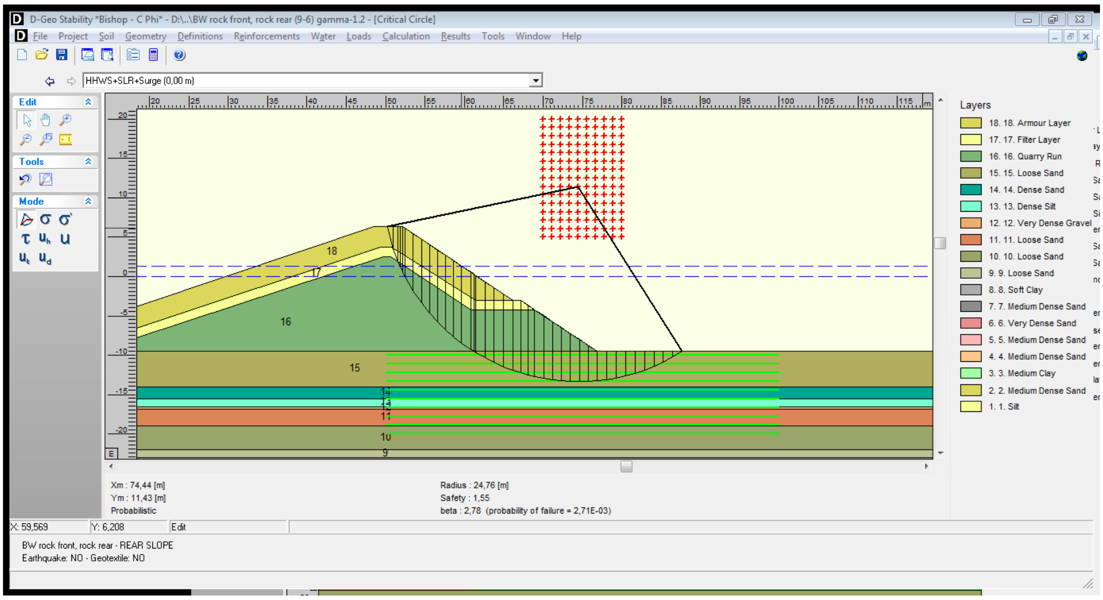


Figure 38: The simulation results for the rear slope (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 29.

	Xm	Ym	Radius	Safety	beta	Probability of failure
	[m]	[m]	[m]	[-]	[-]	[-]
LWS	74.44	11.42	24.76	1.53	2.67	3.80E-03
HHWS +SLR +Surge	74.44	11.42	24.76	1.55	2.78	2.71E-03

Table 29: The normative slip circle properties for the rear slope

The slip circle for LLWS has the biggest probability of failure, therefore LLWS is normative. The safety is bigger than 1.0 so the design complies with the requirements.

13.4 Concrete armour unit breakwater

13.4.1 Input

13.4.1.1 General

An overview of the specific input for the model of the concrete armour unit (Xbloc 3m³) breakwater is shown in figure 39.

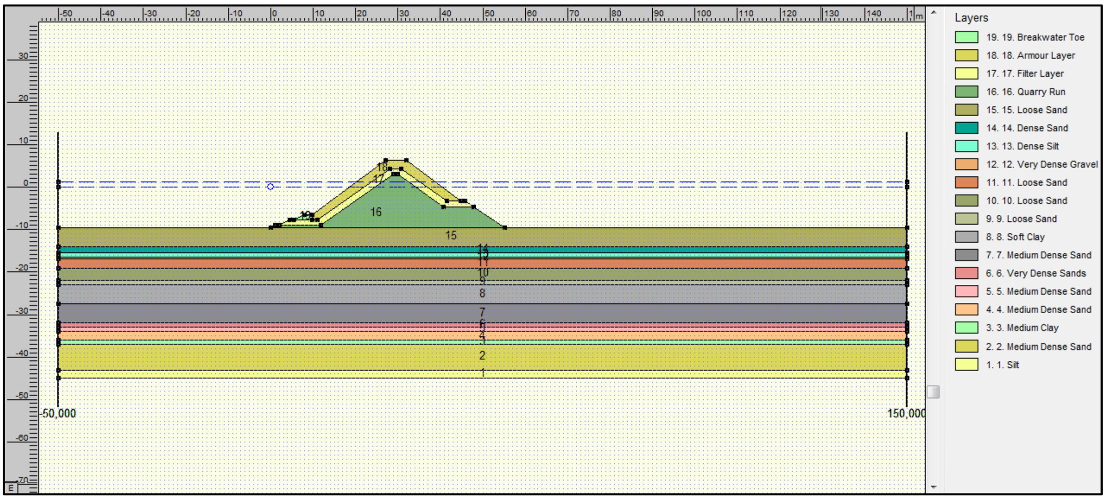


Figure 39: An overview of the specific input for the concrete armour unit breakwater

The coordinates of the points used to create the breakwater are shown in table 30.

The soil parameters used for the breakwater are shown in table 31.

A geotextile is not applied.

X Co-ordinate [m]	Y Co-ordinate [m]
0,000	-9,500
1,000	-9,000
2,000	-9,000
4,600	-7,700
5,600	-7,700
8,000	-6,500
9,900	-6,500
9,900	-7,700
11,100	-7,700
11,880	-9,000
27,000	6,320
28,200	4,320
28,980	3,020
32,000	6,320
30,800	4,320
30,020	3,020
40,720	-4,600
41,500	-3,300
44,830	-3,300
45,830	-3,300
47,780	-4,600
55,130	-9,500

Table 30: The coordinates of the points used to create the breakwater model

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m³]	[kN/m³]	[kN/m²]	[°]
19.	Breakwater Toe	1.20	21.67	23.33	0	45
18.	Armour Layer	2.00	11.50	14.92	0	50
17.	Filter Layer	1.30	21.67	23.33	0	50
16.	Quarry Run	variable	21.67	23.33	0	45

Table 31: The soil parameters used for the breakwater model

Where:

γ_{dry} = Dry unit weight (reduced by a factor of 1.2 to implement safety)
γ_{sat} = wet unit weight (reduced by a factor of 1.2 to implement safety)
c = cohesion
φ = Soil friction angle

13.4.1.2 Front Slope

The slip circle definition used for the front slope is shown in figure 40.

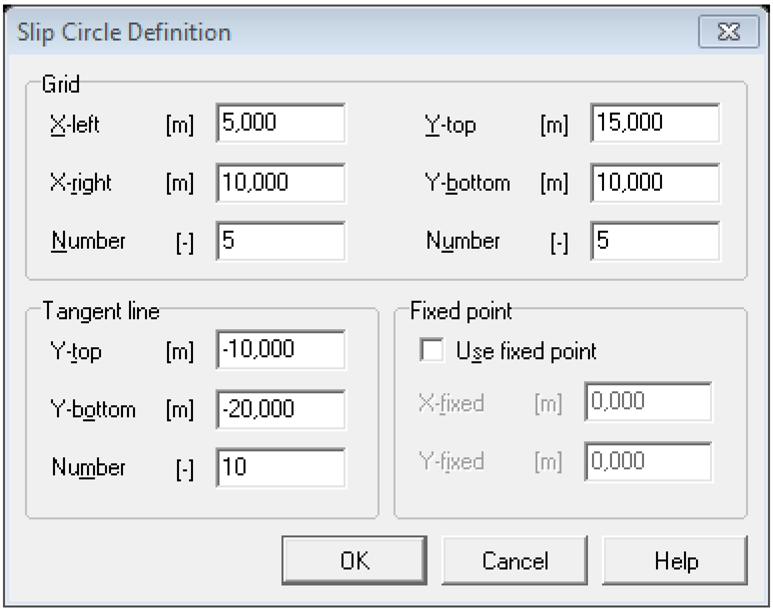


Figure 40: The slip circle definition used for the front slope

13.4.1.3 Rear Slope

The slip circle definition used for the rear slope is shown in figure 41.

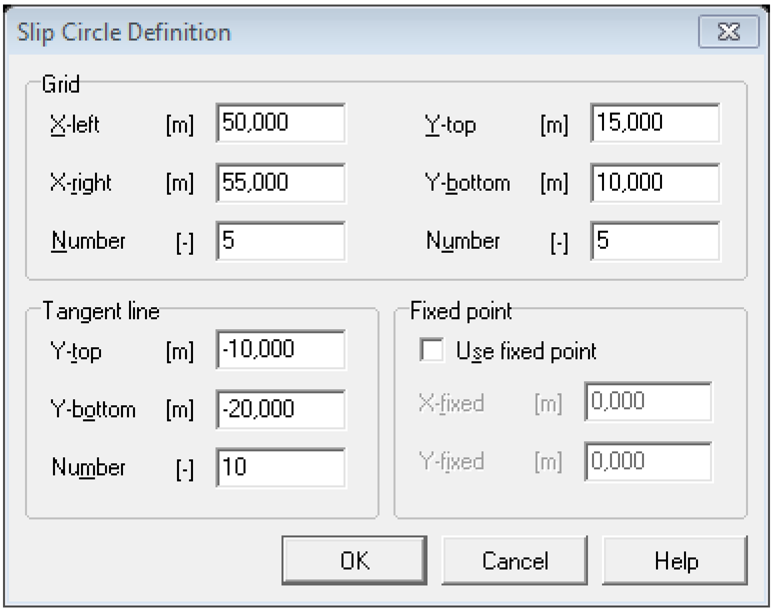


Figure 41: The slip circle definition used for the rear slope

13.4.2 Results

13.4.2.1 Front Slope

The simulation results for the front slope are shown in figure 42 and 43. Figure 42 shows the results for LLWS, figure 43 for HHWS+SLR+Surge (SLR = Sea Level Rise).

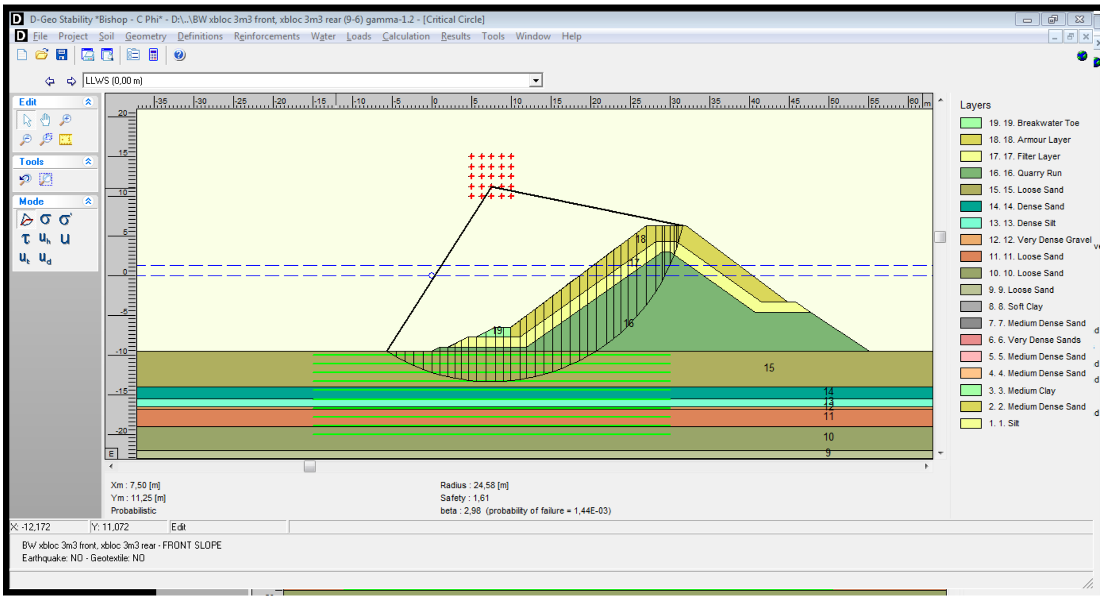


Figure 42: The simulation results for the front slope (LLWS)

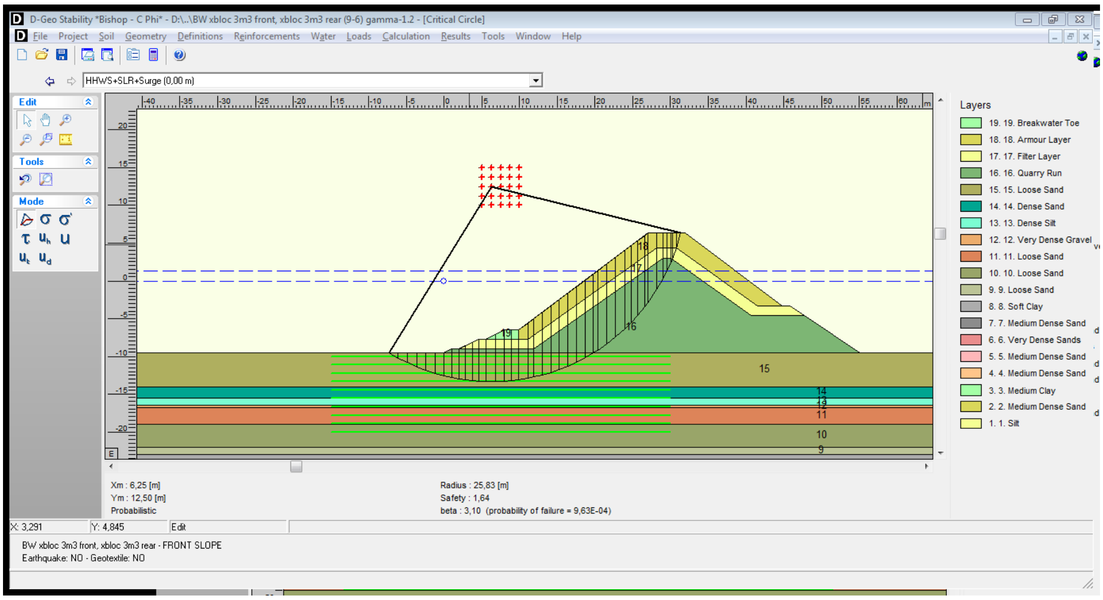


Figure 43: The simulation results for the front slope (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 32.

	Xm	Ym	Radius	Safety	beta	Probability of failure
	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	7.50	11.25	24.58	1.61	2.98	1.44E-03
HHWS +SLR +Surge	6.25	12.50	25.83	1.64	3.10	9.63E-04

Table 32: The normative slip circle properties for the front slope

The slip circle for LLWS has the biggest probability of failure, therefore LLWS is normative. The safety is bigger than 1.0 so the design complies with the requirements.

13.4.2.2 Rear Slope

The simulation results for the rear slope are shown in figure 44 and 45. Figure 44 shows the results for LLWS, figure 45 for HHWS+SLR+Surge (SLR = Sea Level Rise).

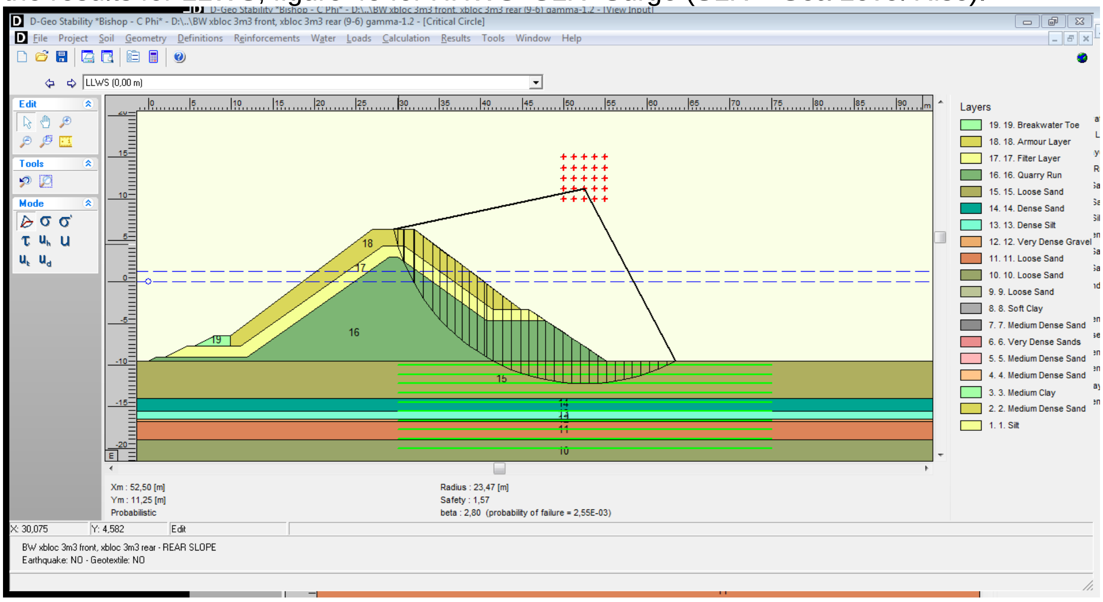


Figure 44: The simulation results for the rear slope (LLWS)

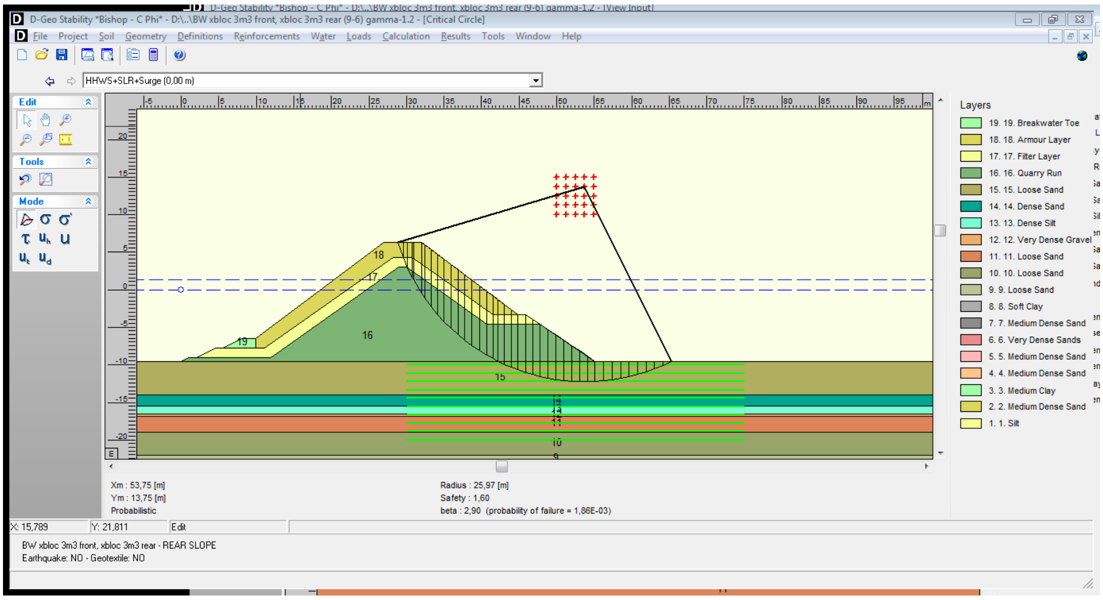


Figure 45: The simulation results for the rear slope (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 33.

	Xm	Ym	Radius	Safety	beta	Probability of failure
	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	52.50	11.25	23.47	1.57	2.80	2.55E-03
HHWS +SLR +Surge	53.75	13.75	25.97	1.60	2.90	1.86E-03

Table 33: The normative slip circle properties for the rear slope

The slip circle for LLWS has the biggest probability of failure, therefore LLWS is normative. The safety is bigger than 1.0 so the design complies with the requirements.

14 APPENDIX VI: SEIESMOLOGICAL RESISTANCE WITH D-GEO STABILITY

14.1 Introduction

The seismological resistance of the breakwater designs is checked using the simulation program D-Geo Stability. For these simulations a horizontal earthquake factor of 0.3g and a vertical earthquake factor of 0.15g are applied to the models.

The vertical earthquake load is applied in both direction, this means that the value is applied both negative as positive. When necessary, geotextiles are applied. Damage is not allowed. As recommended by (Asmerom, 2015), the total unit weight of the soil materials is divided by a factor 1.2 to implement a level of safety. Although damage is not allowed, a minimum safety factor of only 1.0 is required. This is due to the safety in the soil material parameters.

Two models are made: a model for the breakwater with rock armour on both the front and the rear side of the breakwater and a model for the breakwater with Xbloc concrete armour units of $3m^3$ on both the front and the rear side of the breakwater.

This appendix describes first the input that is used in both models. After that, the two different models including the results are described.

Note: The interlocking strength of the Xbloc units is not taken into account.

14.2 General Input

14.2.1 Introduction

The general input for the models in D-Geo Stability is provided in the Basis of Design and in the report of the borehole provided by the client. In this chapter the geometry of the subsoil, the specifications of the soil layers and the calculation options in the models are described.

14.2.2 Geometry

The subsoil is schematized as shown in figure 46. The range of the model is from $x = -50$ to $x = 150$.

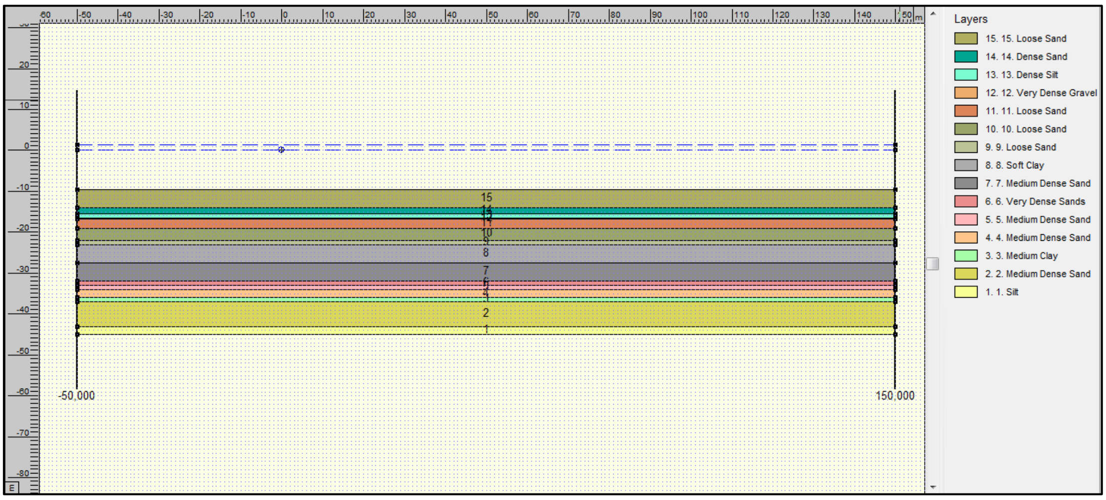


Figure 46: Schematization of the subsoil

The coordinates of the points used in the schematization are shown in table 34.

	X Co-ordinate [m]	Y Co-ordinate [m]
1	-50,000	0,000
2	150,000	0,000
3	-50,000	1,320
4	150,000	1,320
5	-50,000	-9,500
6	150,000	-9,500
7	-50,000	-14,000
8	150,000	-14,000
9	-50,000	-15,500
10	150,000	-15,500
11	-50,000	-16,500
12	150,000	-16,500
13	-50,000	-16,800
14	150,000	-16,800
15	-50,000	-19,000
16	150,000	-19,000
17	-50,000	-22,000
18	150,000	-22,000
19	-50,000	-23,000
20	150,000	-23,000
21	-50,000	-27,500
22	150,000	-27,500
23	-50,000	-32,000
24	150,000	-32,000
25	-50,000	-33,000
26	150,000	-33,000
27	-50,000	-34,000
28	150,000	-34,000
29	-50,000	-36,000
30	150,000	-36,000
31	-50,000	-37,000
32	150,000	-37,000
33	-50,000	-43,000
34	150,000	-43,000
35	-50,000	-45,000
36	150,000	-45,000

Table 34: The coordinates of the points in the schematisation

14.2.3 Water

The water levels have the coordinates shown in figure 47 and 48.

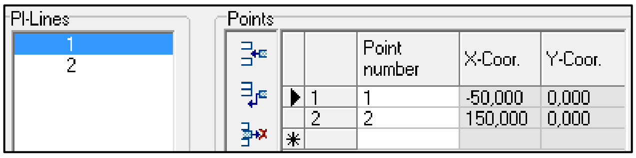


Figure 47: The coordinates of the LLWS

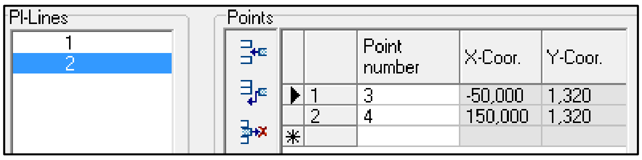


Figure 48: The coordinates of the HHWS+SLR+Surge

The unit weight of the water is set to $\left(\frac{1020*9.78}{1000}\right) = 9.98\ kN/m^3$.

14.2.4 Soil layers

Table 35 shows the parameters used in the models.

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
15.	Loose Sand	4.50	15.00	16.67	0	30
14.	Dense Sand	1.50	17.50	19.17	0	45
13.	Dense Silt	1.00	15.00	15.00	7	32
12.	Very Dense Gravel	0.30	18.33	20.00	0	35
11.	Loose Sand	2.20	15.00	16.67	0	27
10.	Loose Sand	3.00	15.83	17.50	22	28
9.	Loose Sand	1.00	15.83	17.50	22	28
8.	Soft Clay	4.50	12.50	12.50	20	25
7.	Medium Dense Sand	4.50	17.08	18.75	22	30
6.	Very Dense Sand	1.00	18.33	20.00	22	34
5.	Medium Dense Sand	1.00	17.08	18.75	22	30
4.	Medium Dense Sand	2.00	17.08	18.75	0	38
3.	Medium Clay	1.00	13.33	13.33	20	25
2.	Medium Dense Sand	6.00	17.08	18.75	22	30
1.	Silt	2.00	14.12	14.12	7	30

Table 35: Used parameters in the models

Where:

γ_{dry} = Dry unit weight (reduced by a factor of 1.2 to implement safety)
γ_{sat} = wet unit weight (reduced by a factor of 1.2 to implement safety)
c = cohesion
φ = Soil friction angle

14.2.5 Earthquake

The earthquake properties applied are shown in figure 49. The degree of consolidation is set to 100% for all the layers.

Figure 49: The earthquake properties applied

14.2.6 Calculation options

The used calculation options are as shown in figure 50 and 51.

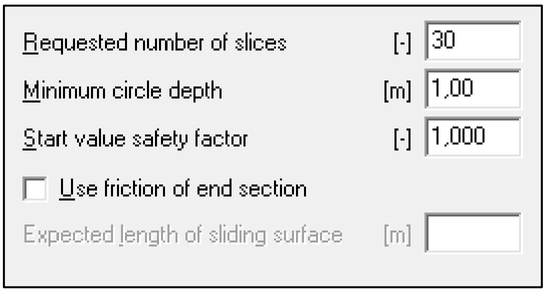


Figure 50: The used calculation options

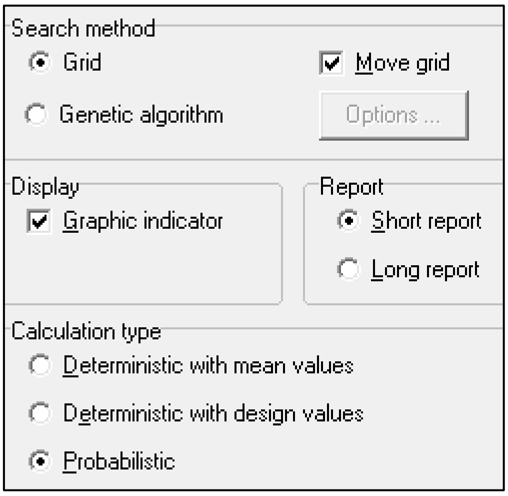


Figure 51: The used calculation options

14.3 Rock armour breakwater

14.3.1 Input

14.3.1.1 General

An overview of the specific input for the model of the rock armour breakwater is shown in figure 52.

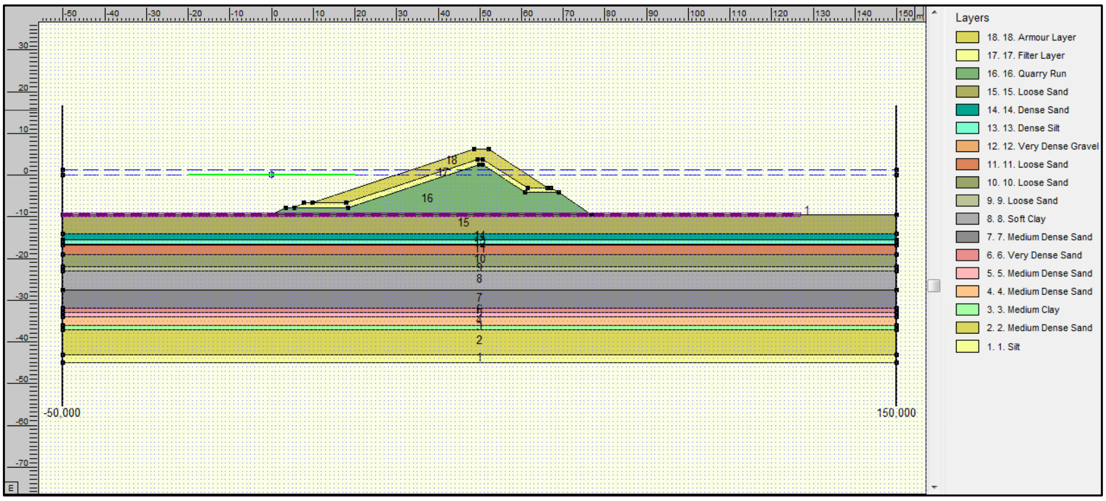


Figure 52: An overview of the specific input for the rock armour breakwater

The coordinates of the points used to create the breakwater are shown in table 36.

The soil parameters used for the breakwater are shown in table 37.

A geotextile is applied between the subsoil and the breakwater, from 50 meters in front of the breakwater (X=-50.000, Y=-9.500) to 50 m behind the breakwater (X=126.890, Y=-9.500). The geotextile has an effective tensile strength of 200.00 kN/m and a reduction area of 5.00 meter.

X Co-ordinate [m]	Y Co-ordinate [m]
0,000	-9,500
3,400	-7,800
5,400	-7,800
7,800	-6,600
9,800	-6,600
18,020	-6,600
48,560	6,320
52,160	6,320
61,450	-3,000
66,140	-3,000
67,140	-3,000
68,940	-4,200
76,890	-9,500

Table 36: The coordinates of the points used to create the breakwater model

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
18.	Armour Layer	2.60	21.67	23.33	0	50
17.	Filter Layer	1.20	21.67	23.33	0	50
16.	Quarry Run	variable	21.67	23.33	0	45

Table 37: The soil parameters used for the breakwater model

Where:

γ_{dry} = Dry unit weight (reduced by a factor of 1.2 to implement safety)

γ_{sat} = wet unit weight (reduced by a factor of 1.2 to implement safety)

c = cohesion

φ = Soil friction angle

14.3.1.2 Front Slope

The slip circle definition used for the front slope is shown in figure 53.

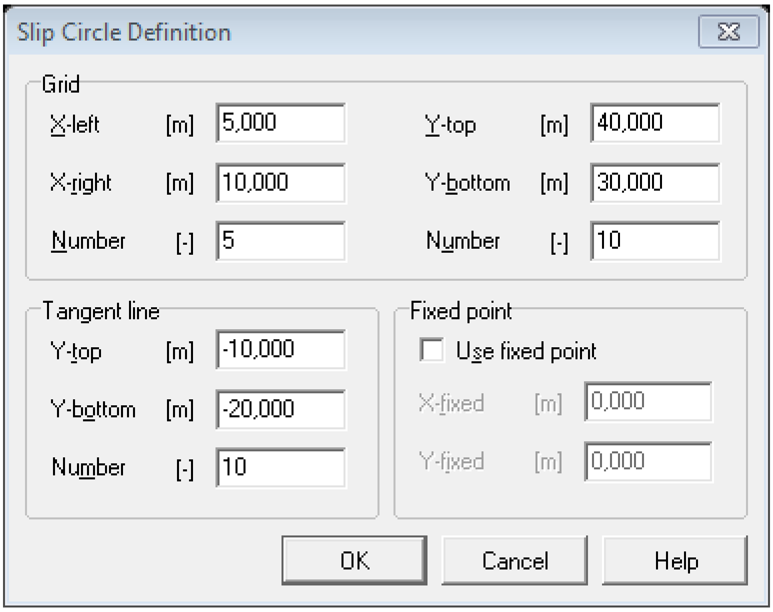


Figure 53: The slip circle definition used for the front slope

14.3.1.3 Rear Slope

The slip circle definition used for the rear slope is shown in figure 54.

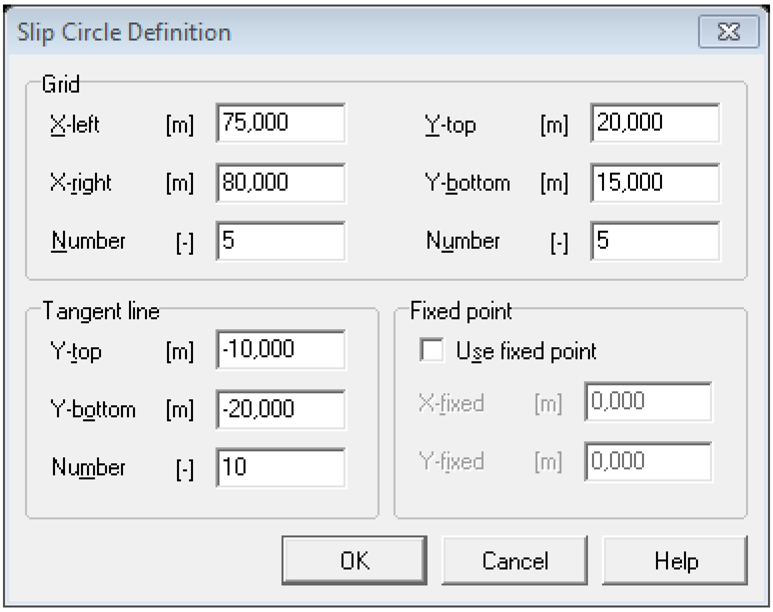


Figure 54: The slip circle definition used for the rear slope

14.3.2 Results

14.3.2.1 Front Slope

The simulation results for the front slope are shown in figure 55, 56, 57 and 58. Figure 55 shows the results for LLWS with a vertical earthquake load of +0.15g, figure 56 for HHWS+SLR+Surge (SLR = Sea Level Rise). Figure 57 shows the results for LLWS with a vertical earthquake factor of -0.15g, figure 58 for HHWS+SLR+Surge.

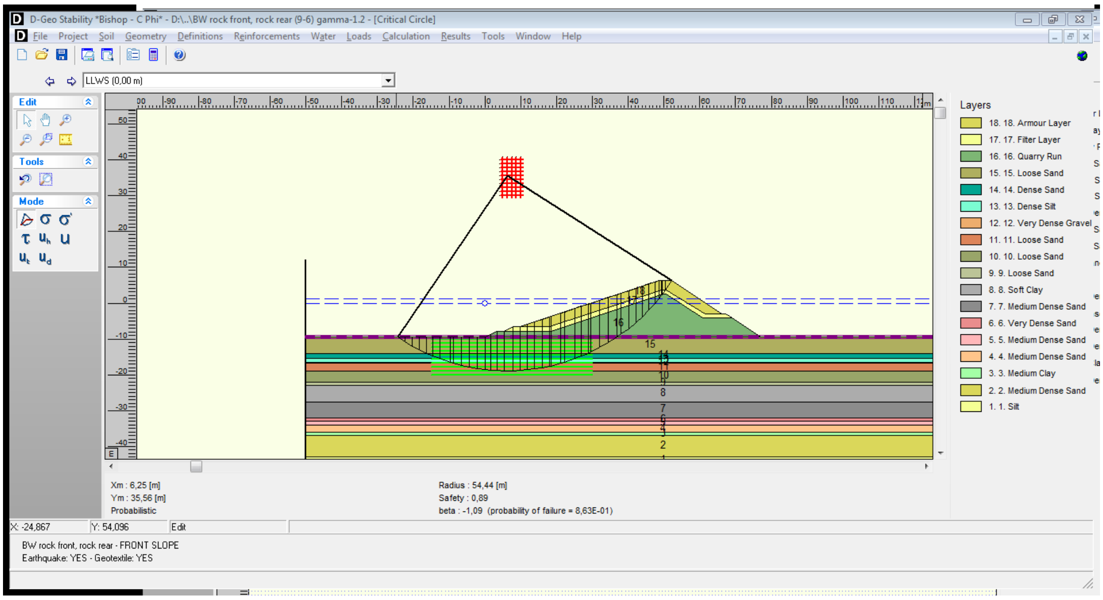


Figure 551: The simulation results for the front slope with a vertical earthquake factor of +0.15 (LLWS)

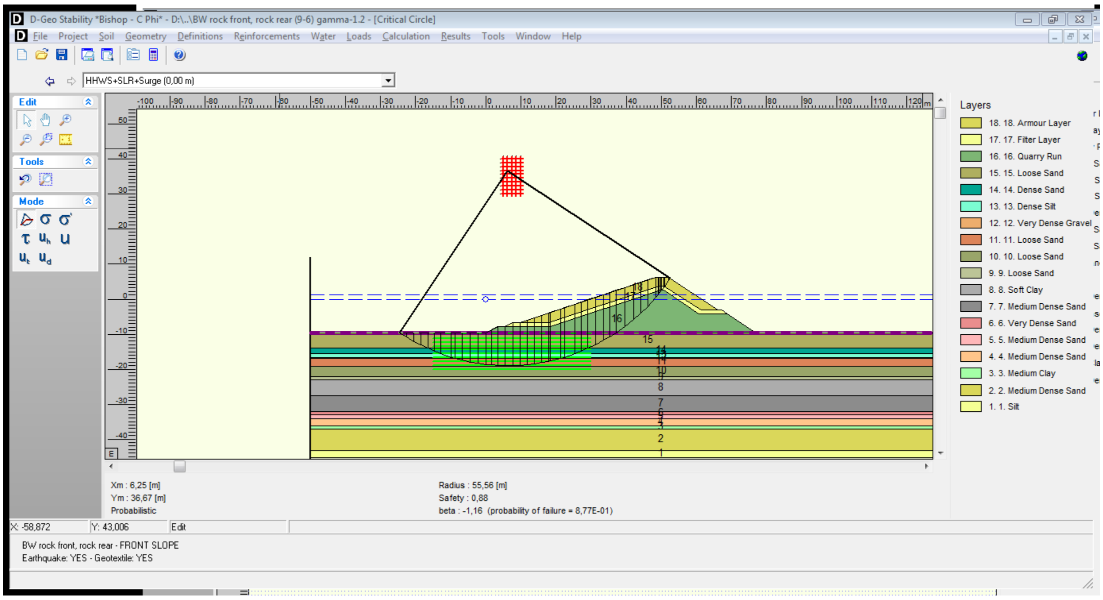


Figure 56: The simulation results for the front slope with a vertical earthquake factor of +0.15 (HHWS+SLR+Surge)

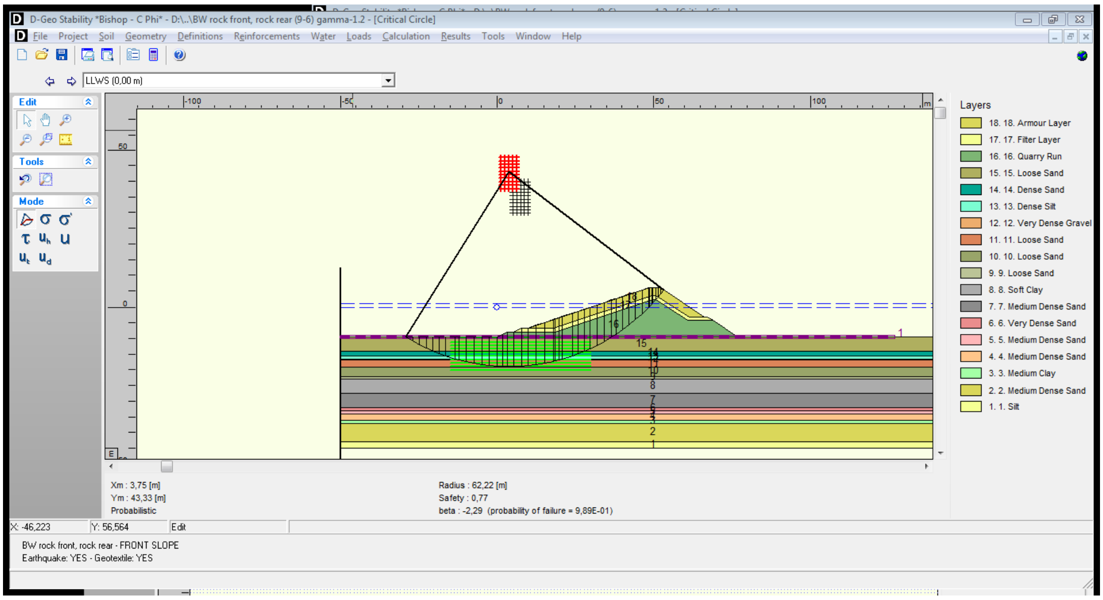


Figure 57: The simulation results for the front slope with a vertical earthquake factor of -0.15 (LLWS)

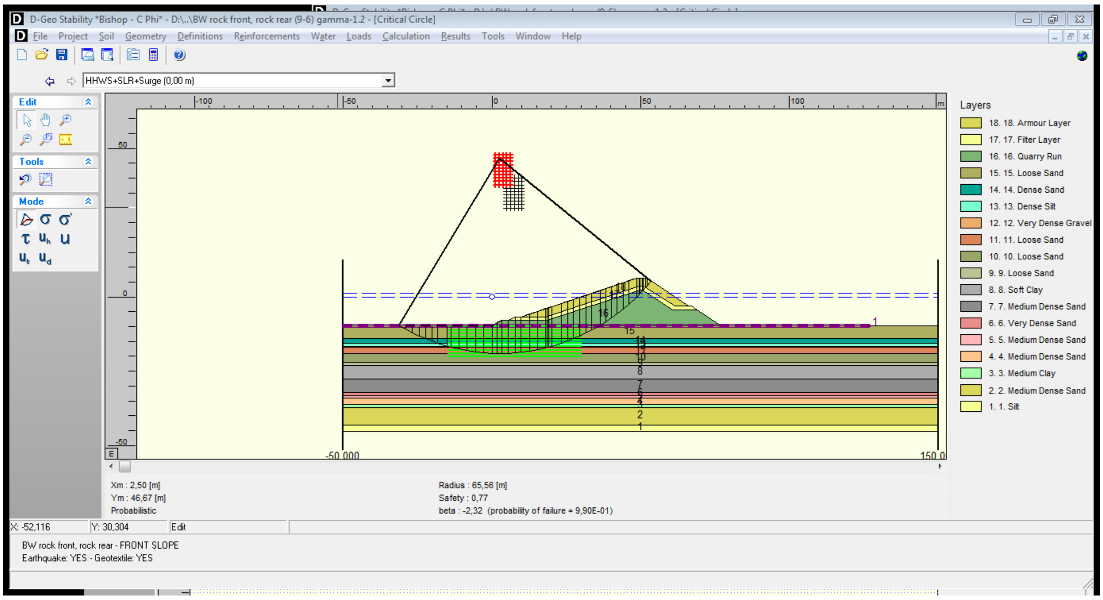


Figure 58: The simulation results for the front slope with a vertical earthquake factor of -0.15 (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 38.

	Vertical earthquake factor	Xm	Ym	Radius	Safety	beta	Probability of failure
	[g]	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	+0.15	6.25	35.56	54.44	0.89	-1.09	8.63E-01
HHWS +SLR +Surge	+0.15	6.25	36.67	55.56	0.88	-1.16	8.77E-01
LLWS	-0.15	3.75	43.33	62.22	0.77	-2.29	9.89E-01
HHWS +SLR +Surge	-0.15	2.50	46.67	65.56	0.77	-2.32	9.90E-01

Table 38: The normative slip circle properties for the front slope

The slip circle for HHWS+SLR+Surge with a vertical earthquake factor of -0.15 has the biggest probability of failure, therefore HHWS+SLR+Surge is normative. The safety is not bigger than 1.0 so the design does not comply with the requirements.

14.3.2.2 Rear Slope

The simulation results for the rear slope are shown in figure 59, 60, 61 and 62. Figure 59 shows the results for LLWS with a vertical earthquake load of +0.15g, figure 60 for HHWS+SLR+Surge (SLR = Sea Level Rise). Figure 61 shows the results for LLWS with a vertical earthquake factor of -0.15g, figure 62 for HHWS+SLR+Surge.

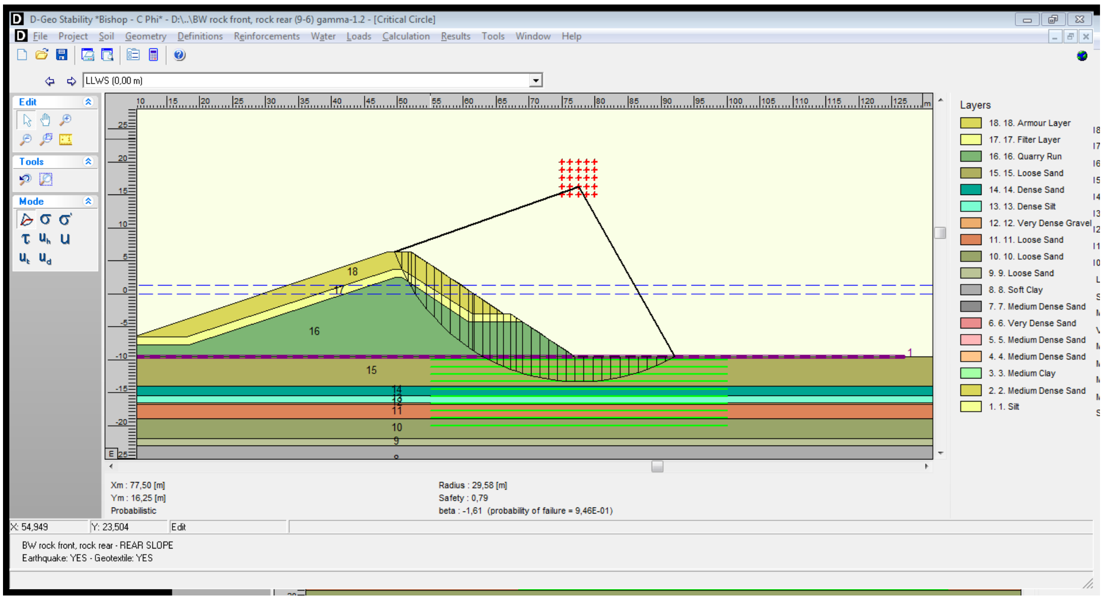


Figure 59: The simulation results for the front slope with a vertical earthquake factor of +0.15 (LLWS)

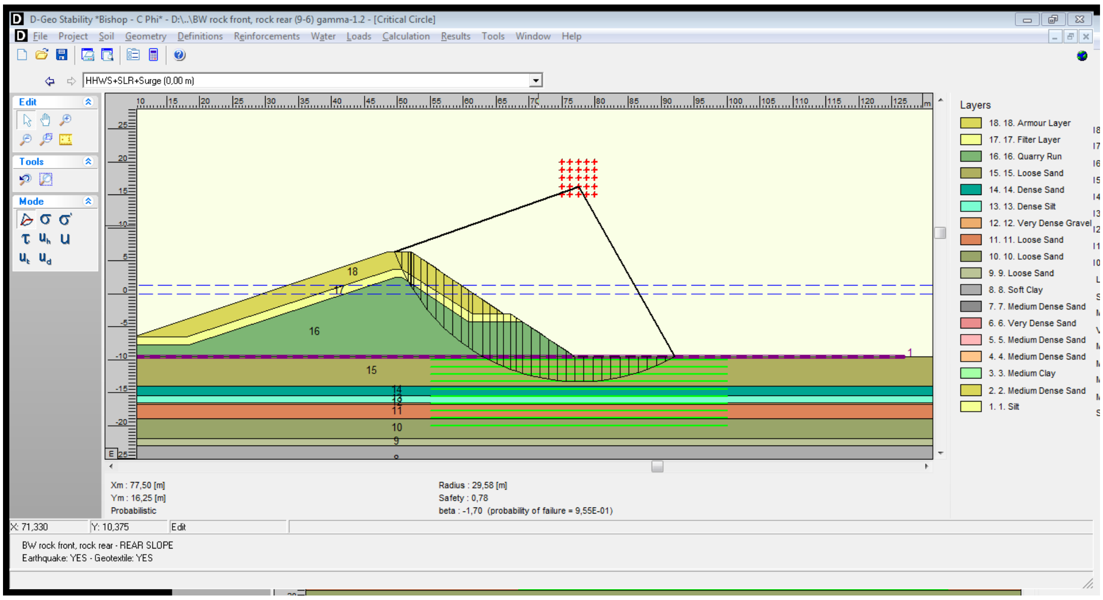


Figure 60: The simulation results for the front slope with a vertical earthquake factor of +0.15 (HHWS+SLR+Surge)

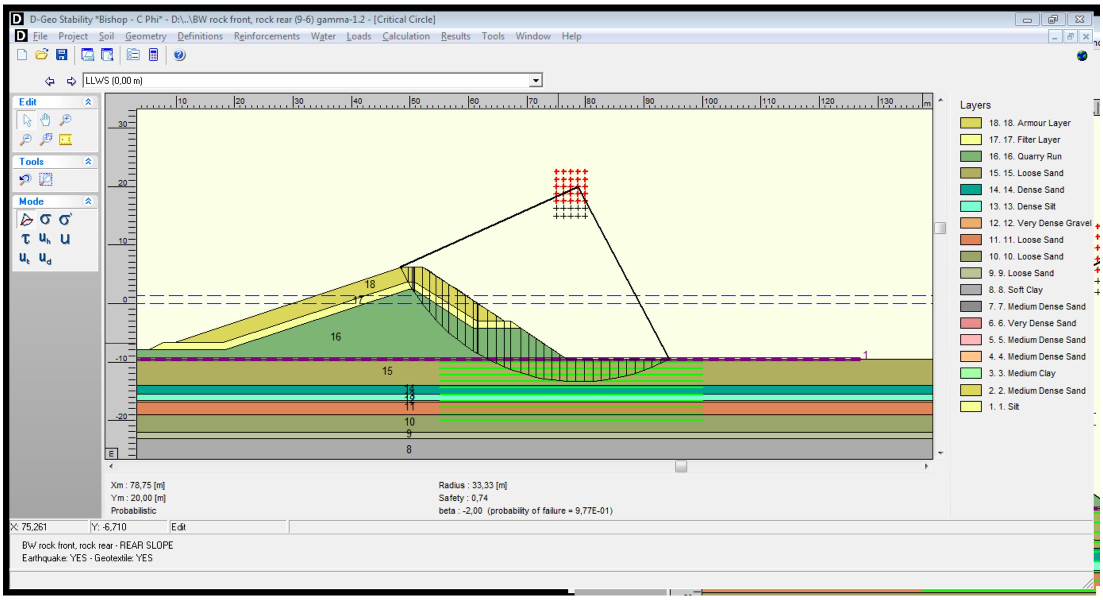


Figure 61: The simulation results for the front slope with a vertical earthquake factor of -0.15 (LLWS)

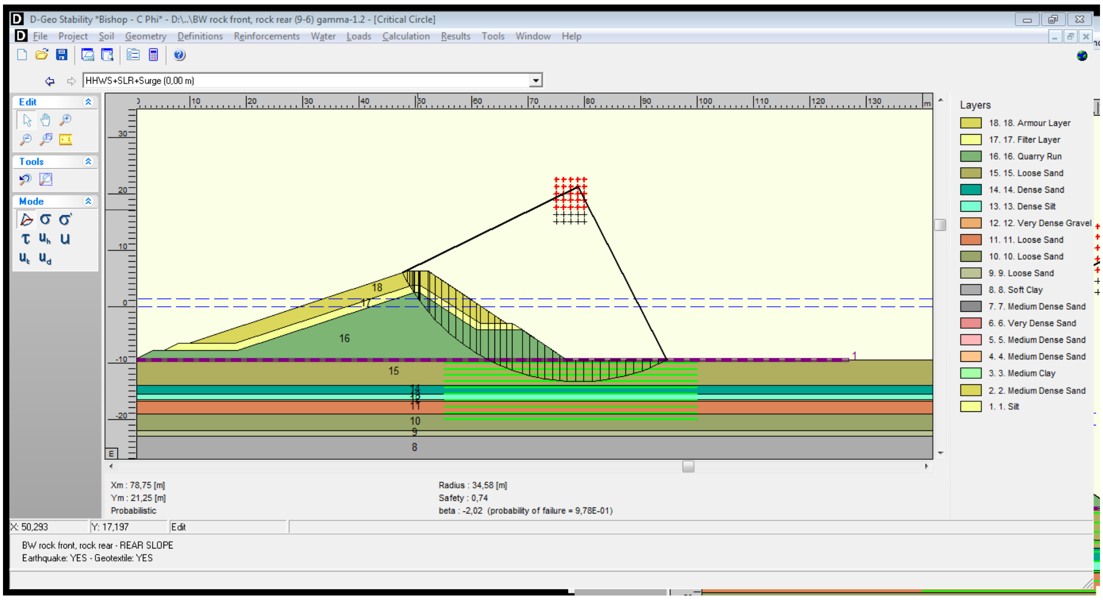


Figure 62: The simulation results for the front slope with a vertical earthquake factor of -0.15 (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 39.

	Vertical earthquake factor	Xm	Ym	Radius	Safety	beta	Probability of failure
	[g]	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	+0.15	77.50	16.25	29.58	0.79	-1.61	9.46E-01
HHWS +SLR +Surge	+0.15	77.50	16.25	29.58	0.78	-1.70	9.55E-01
LLWS	-0.15	78.75	20.00	33.33	0.74	-2.00	9.77E-01
HHWS +SLR +Surge	-0.15	78.75	21.25	34.58	0.74	-2.02	9.78E-01

Table 39: The normative slip circle properties for the rear slope

The slip circle for HHWS+SLR+Surge with a vertical earthquake factor of -0.15 has the biggest probability of failure, therefore HHWS+SLR+Surge is normative. The safety is not bigger than 1.0 so the design does not comply with the requirements.

14.4 Concrete armour unit breakwater

14.4.1 Input

14.4.1.1 General

An overview of the specific input for the model of the concrete armour unit (Xbloc 3m^3) breakwater is shown in figure 63.

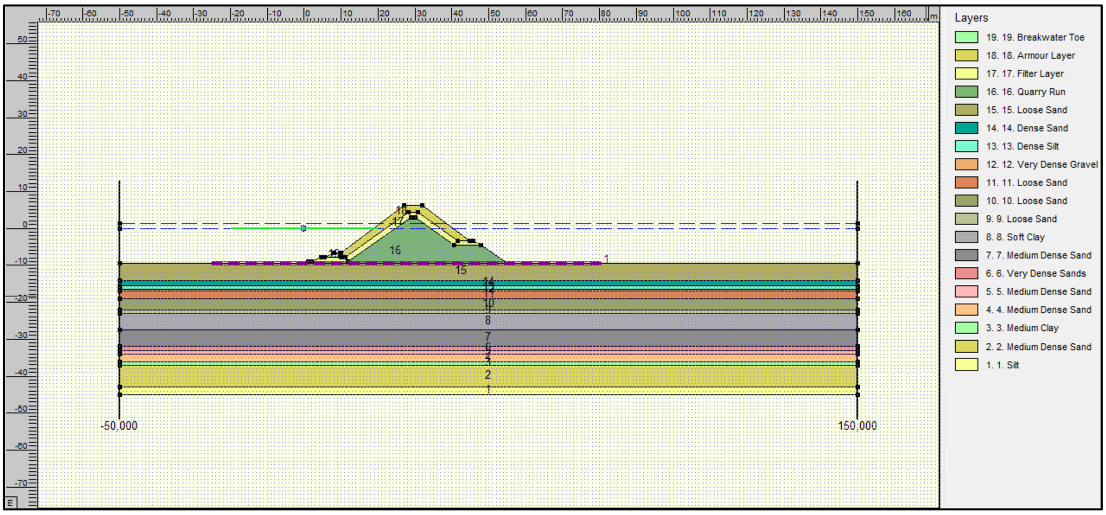


Figure 63: An overview of the specific input for the concrete armour unit breakwater

The coordinates of the points used to create the breakwater are shown in table 40.

The soil parameters used for the breakwater are shown in table 41.

A geotextile is applied between the subsoil and the breakwater, from 50 meters in front of the breakwater (X=-50.000, Y=-9.500) to 50 m behind the breakwater (X=80.130, Y=-9.500). The geotextile has an effective tensile strength of 200.00 kN/m and a reduction area of 5.00 meter.

X Co-ordinate [m]	Y Co-ordinate [m]
0,000	-9,500
1,000	-9,000
2,000	-9,000
4,600	-7,700
5,600	-7,700
8,000	-6,500
9,900	-6,500
9,900	-7,700
11,100	-7,700
11,880	-9,000
27,000	6,320
28,200	4,320
28,980	3,020
32,000	6,320
30,800	4,320
30,020	3,020
40,720	-4,600
41,500	-3,300
44,830	-3,300
45,830	-3,300
47,780	-4,600
55,130	-9,500

Table 40: The coordinates of the points used to create the breakwater model

No.	Material	Thickness	γ_{dry}	γ_{sat}	c	φ
		[m]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[°]
19.	Breakwater Toe	1.20	21.67	23.33	0	45
18.	Armour Layer	2.00	11.50	14.92	0	50
17.	Filter Layer	1.30	21.67	23.33	0	50
16.	Quarry Run	variable	21.67	23.33	0	45

Table 41: The soil parameters used for the breakwater model

Where:

γ_{dry}	=	Dry unit weight (reduced by a factor of 1.2 to implement safety)
γ_{sat}	=	wet unit weight (reduced by a factor of 1.2 to implement safety)
c	=	cohesion
φ	=	Soil friction angle

14.4.1.2 Front Slope

The slip circle definition used for the front slope is shown in figure 64.

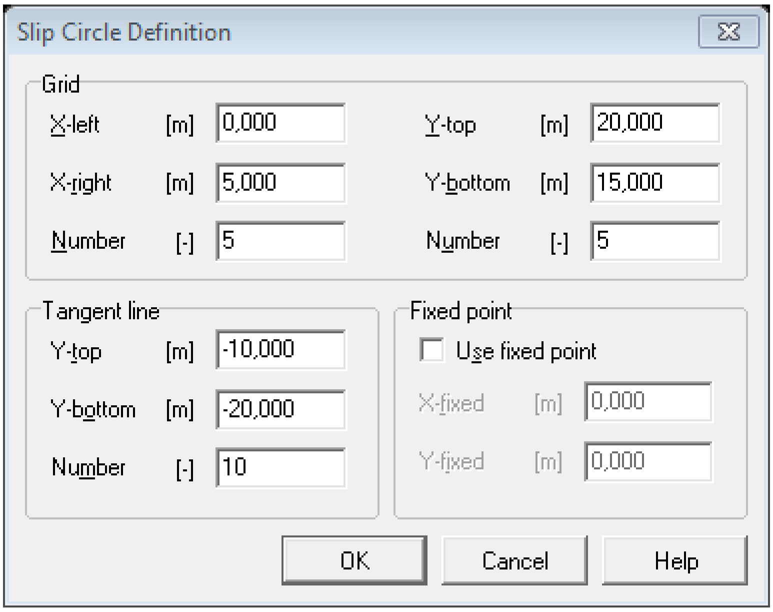


Figure 64: The slip circle definition used for the front slope

14.4.1.3 Rear Slope

The slip circle definition used for the rear slope is shown in figure 65.

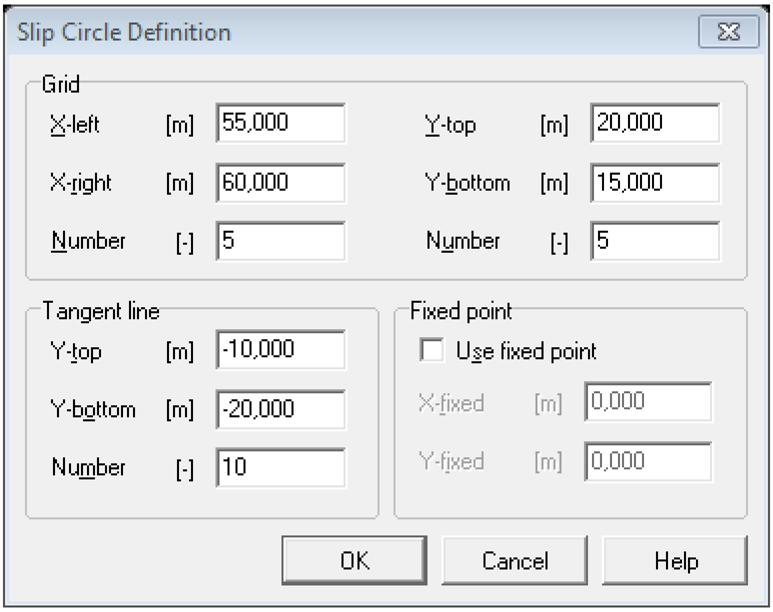


Figure 65: The slip circle definition used for the rear slope

14.4.2 Results

14.4.2.1 Front Slope

The simulation results for the front slope are shown in figure 66, 67, 68 and 69. Figure 66 shows the results for LLWS with a vertical earthquake load of +0.15g, figure 67 for HHWS+SLR+Surge (SLR = Sea Level Rise). Figure 68 shows the results for LLWS with a vertical earthquake factor of -0.15g, figure 69 for HHWS+SLR+Surge.

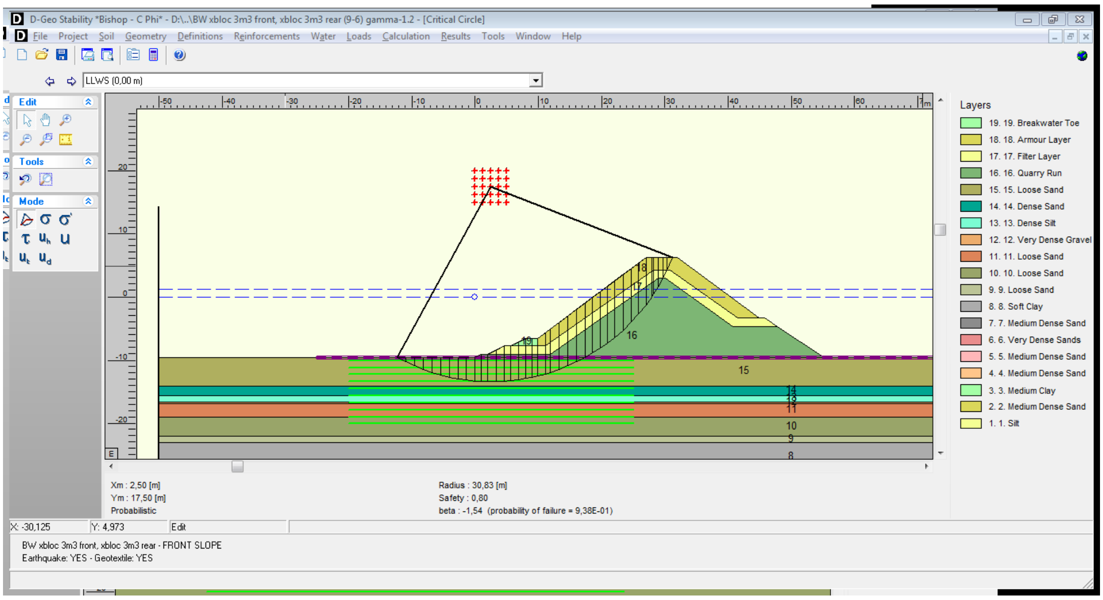


Figure 66: The simulation results for the front slope with a vertical earthquake factor of +0.15 (LLWS)

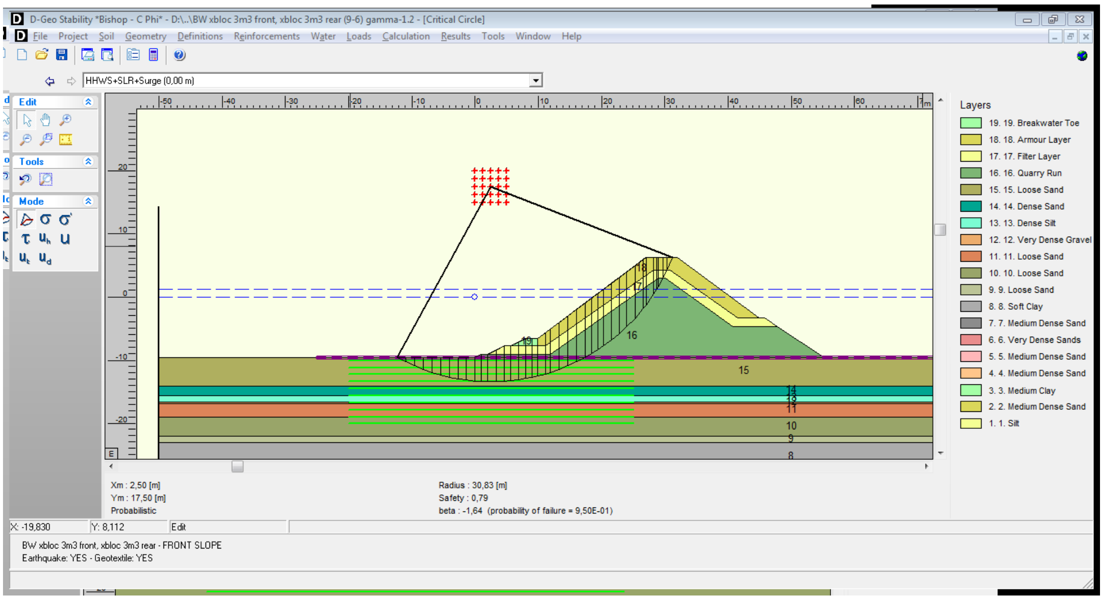


Figure 67: The simulation results for the front slope with a vertical earthquake factor of +0.15 (HHWS+SLR+Surge)

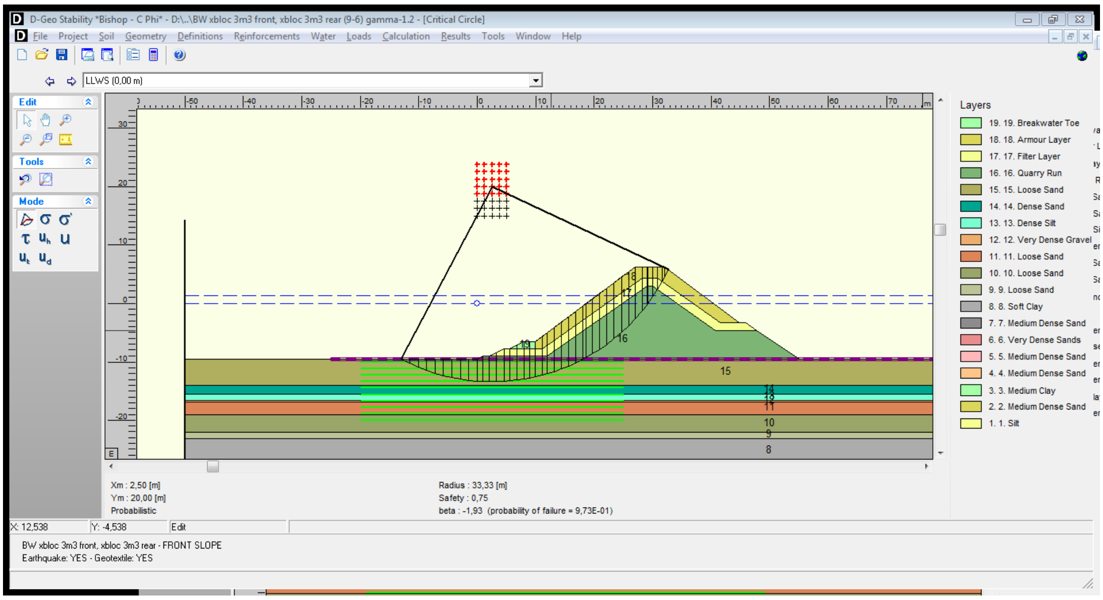


Figure 68: The simulation results for the front slope with a vertical earthquake factor of -0.15 (LLWS)

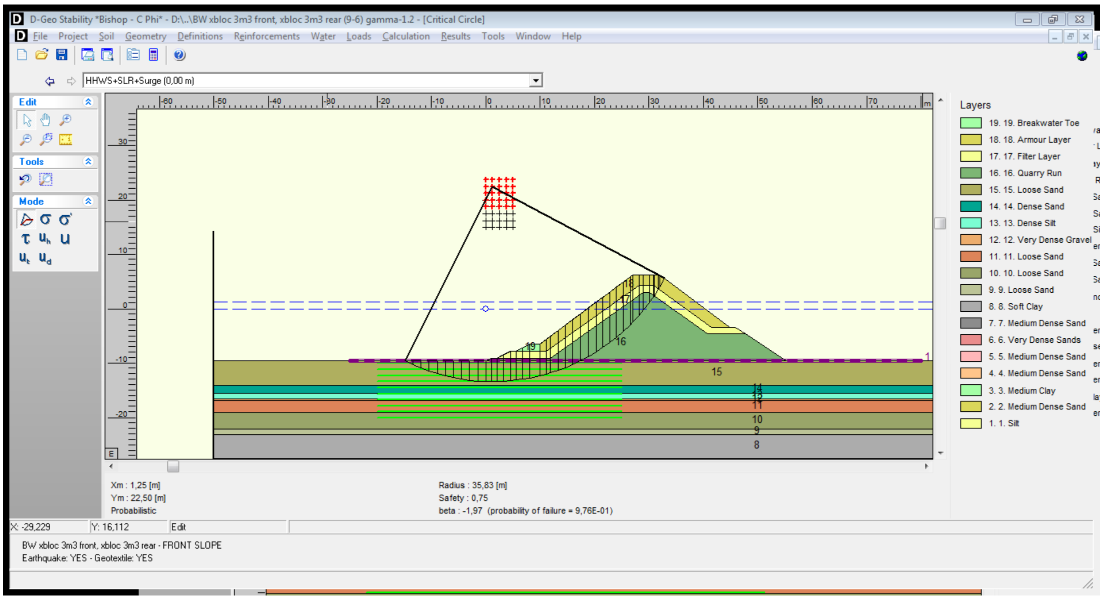


Figure 69: The simulation results for the front slope with a vertical earthquake factor of -0.15 (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 41.

	Vertical earthquake factor	Xm	Ym	Radius	Safety	beta	Probability of failure
	[g]	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	+0.15	2.50	17.50	30.83	0.80	-1.54	9.38E-01
HHWS +SLR +Surge	+0.15	2.50	17.50	30.83	0.79	-1.64	9.50E-01
LLWS	-0.15	2.50	20.00	33.33	0.75	-1.93	9.73E-01
HHWS +SLR +Surge	-0.15	1.25	22.50	35.83	0.75	-1.97	9.76E-01

Table 41: The normative slip circle properties for the front slope

The slip circle for HHWS+SLR+Surge with a vertical earthquake factor of -0.15 has the biggest probability of failure, therefore HHWS+SLR+Surge is normative. The safety is not bigger than 1.0 so the design does not comply with the requirements.

14.4.3 Rear Slope

The simulation results for the rear slope are shown in figure 70, 71, 72 and 73. Figure 70 shows the results for LLWS with a vertical earthquake load of +0.15g, figure 71 for HHWS+SLR+Surge (SLR = Sea Level Rise). Figure 72 shows the results for LLWS with a vertical earthquake factor of -0.15g, figure 73 for HHWS+SLR+Surge.

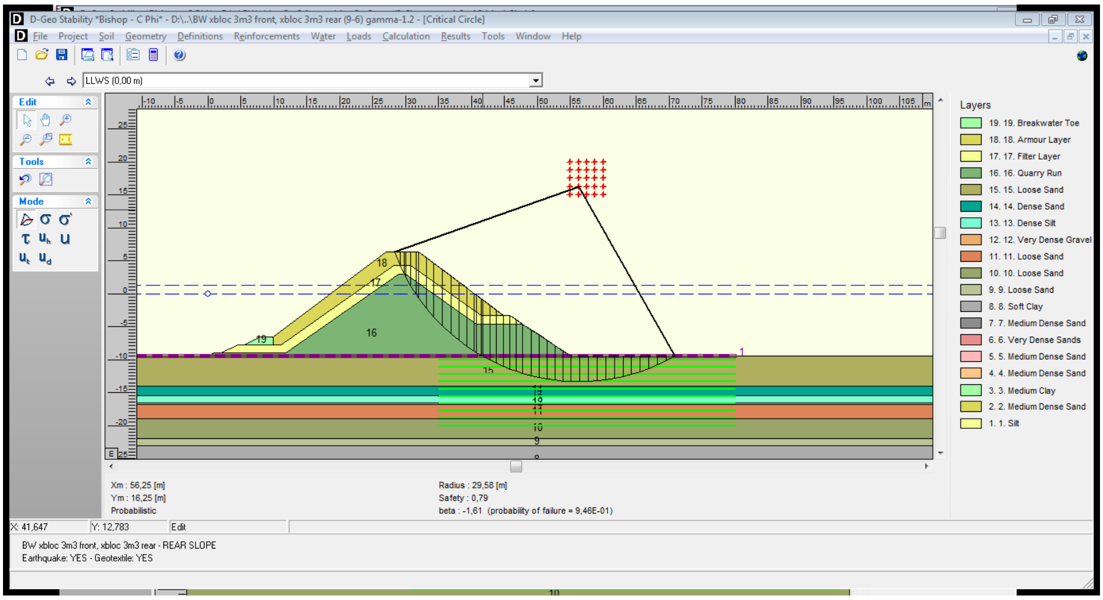


Figure 70: The simulation results for the front slope with a vertical earthquake factor of +0.15 (LLWS)

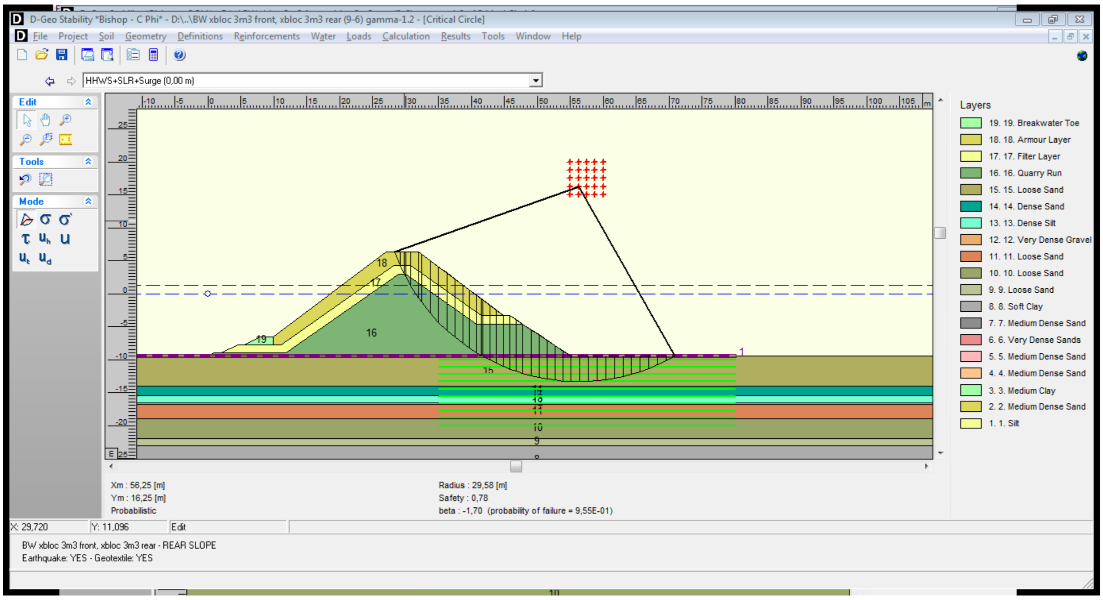


Figure 71: The simulation results for the front slope with a vertical earthquake factor of +0.15 (HHWS+SLR+Surge)

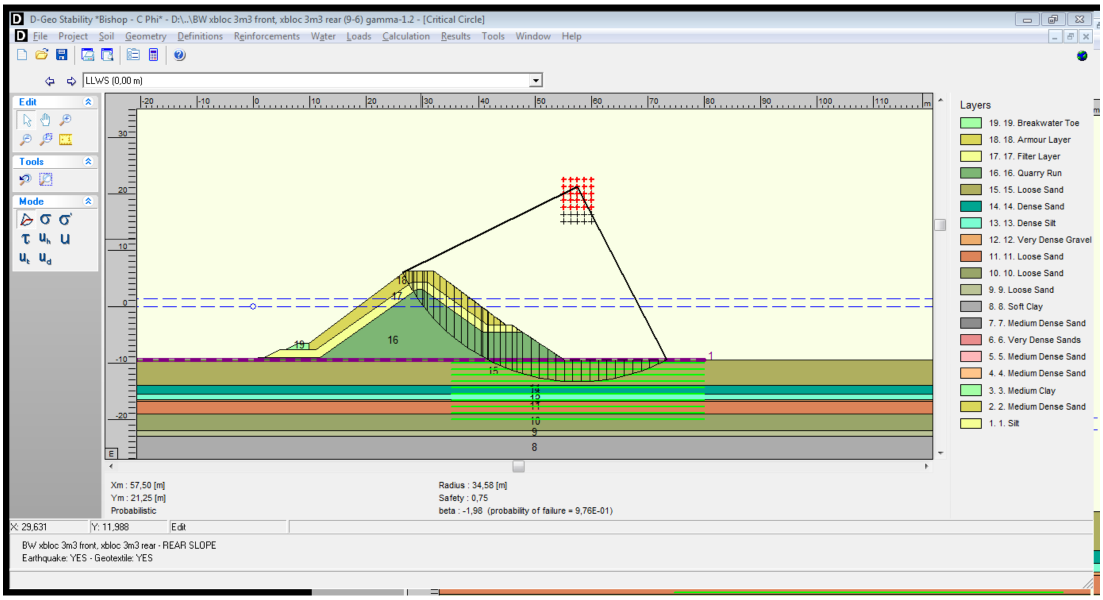


Figure 72: The simulation results for the front slope with a vertical earthquake factor of -0.15 (LLWS)

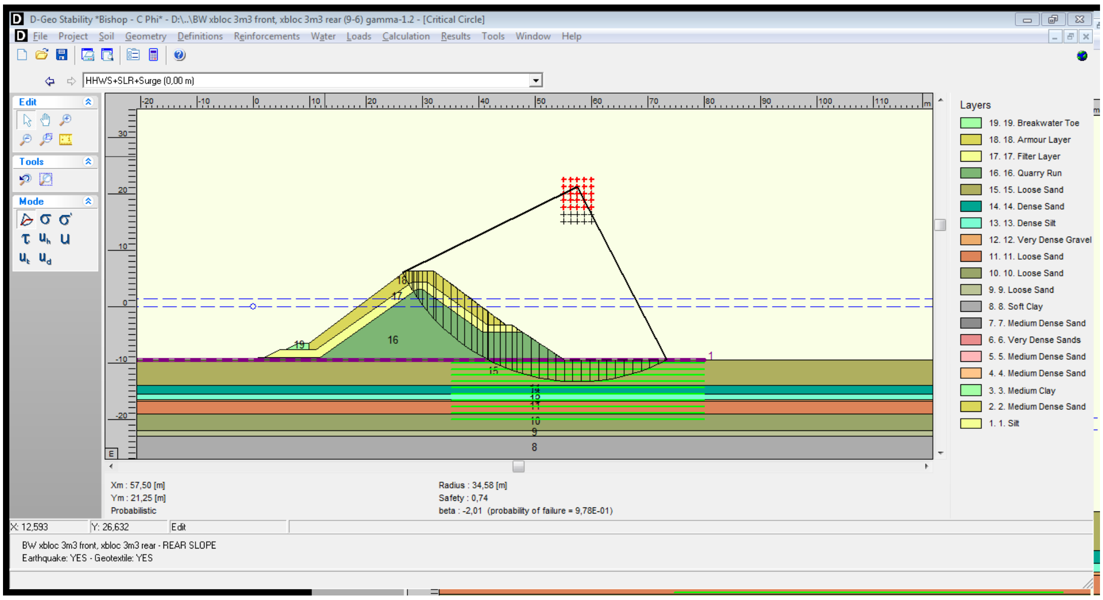


Figure 73: The simulation results for the front slope with a vertical earthquake factor of -0.15 (HHWS+SLR+Surge)

The normative slip circle properties are shown in table 42.

	Vertical earthquake factor	Xm	Ym	Radius	Safety	beta	Probability of failure
	[g]	[m]	[m]	[m]	[-]	[-]	[-]
LLWS	+0.15	56.25	16.25	29.58	0.79	-1.61	9.46E-01
HHWS +SLR +Surge	+0.15	56.25	16.25	29.58	0.78	-1.70	9.55E-01
LLWS	-0.15	57.50	21.25	34.58	0.75	-1.98	9.76E-01
HHWS +SLR +Surge	-0.15	57.50	21.25	34.58	0.74	-2.01	9.78E-01

Table 42: The normative slip circle properties for the rear slope

The slip circle for HHWS+SLR+Surge with a vertical earthquake factor of -0.15 has the biggest probability of failure, therefore HHWS+SLR+Surge is normative. The safety is not bigger than 1.0 so the design does not comply with the requirements.

Appendix 4: Brainstorm Session

Innovative Breakwater

16 June 2015

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CONTENTS

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1 INTRODUCTION

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because of underestimating the severe environmental conditions, both during construction and operation, it was wrongly decided to omit a solution for the swell waves. Moreover, the project budget was too limited to build a traditional breakwater. The current estimated yearly downtime is approximately between the 80 and 90%, which makes the business case of the terminal not lucrative.

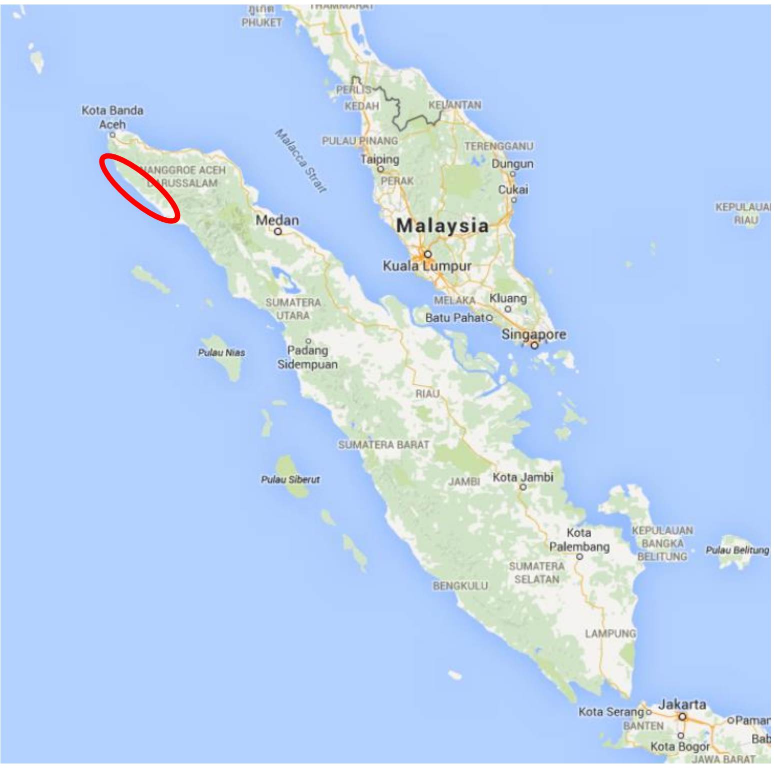


Figure 1: Location project

In order to make the terminal profitable, a breakwater has to be constructed, however, the available budget is too limited to build a traditional breakwater construction. For this reason, alternative and innovative breakwater solutions are being examined in order to significantly reduce the costs of the breakwater construction.

The purpose of this brainstorm session is to generate and schematize a huge amount of innovative breakwater solutions, which are applicable at the project location. These breakwater solutions will be optimized and designed further during the graduation period. The following chapters describe the environmental conditions at the project location and the minimal required boundary conditions.

2 ENVIROMENTAL CONDITIONS

2.1 Bathymetry

A schematic drawing of the bathymetry at the project location can be seen in Figure 2. The water depth at the breakwater location is approximately **9.5 meters**, as seen from chart datum.

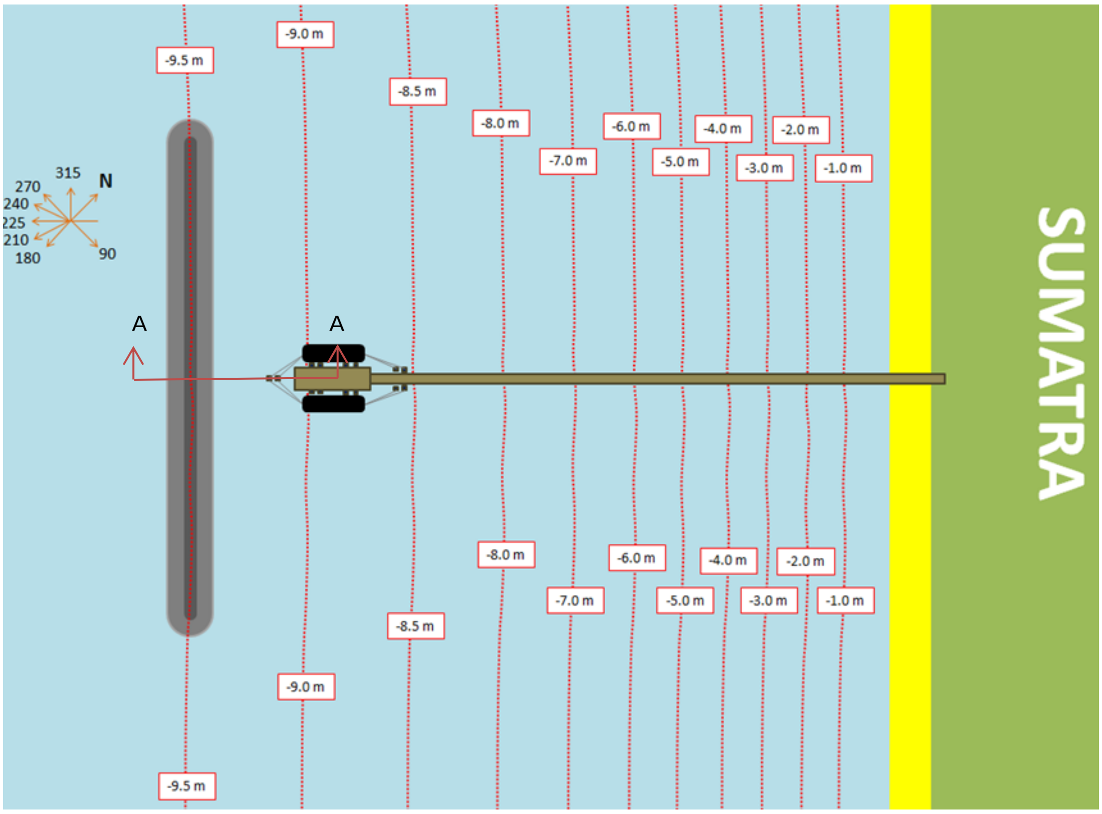


Figure 2: Bathymetry project location

Figure 3 shows the head of the jetty and the location of the breakwater (cross section A-A).

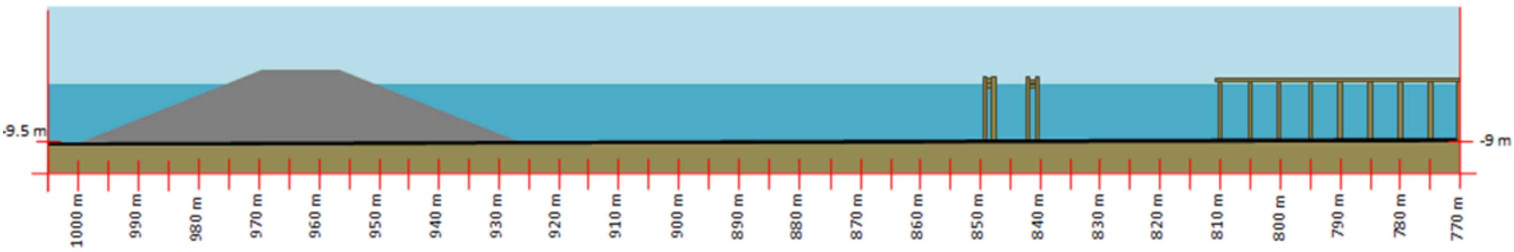


Figure 3: Head of the terminal and (traditional) breakwater location (cross section A-A)

2.2 Subsoil

The subsoil at the project location exists of medium dense sand. The granular size of the sand is silty to fine. A schematization of the subsoil at the project location can be seen in Figure 4.

SCHEMATIZATION OF SUBSOIL AT PROJECT LOCATION

Surface RL: -9.50 m

DEPTH (m)	THICK (m)		MATERIAL DESCRIPTION
0.00 4.50	4.50	1	Medium to fine grained SAND, Loose
6.00	1.50	2	Coarse grained SAND, Dense
7.00	1.00	3	Sandy SILT, Dense
7.30	0.30		Sandy GRAVEL, Very Dense
9.50	2.20	5	Fine grained SAND, Loose
12.50	3.00	6	Silty SAND, Loose
13.50	1.00	7	Silty SAND, Carbonized (thin layer of Lignite), Loose
18.00	4.50	8	Silty CLAY, trace fine grained SAND, Soft Consistency, High Plasticity
22.50	4.50	9	Silty SAND, Medium Dense
23.50	1.00	10	Silty SAND, Very Dense
24.50	1.00	11	Silty SAND, Medium Dense
26.50	2.00	12	Fine SAND, Medium Dense
27.50	1.00	13	Silty CLAY, trace fine SAND, Medium Stiff Consistency, High Plasticity
33.50	6.00	14	Silty SAND, Medium Dense
35.50	2.00	15	SILT, trace fine SAND, Carbonized (thin layer of Lignite), Soft Consistency, Low Plasticity

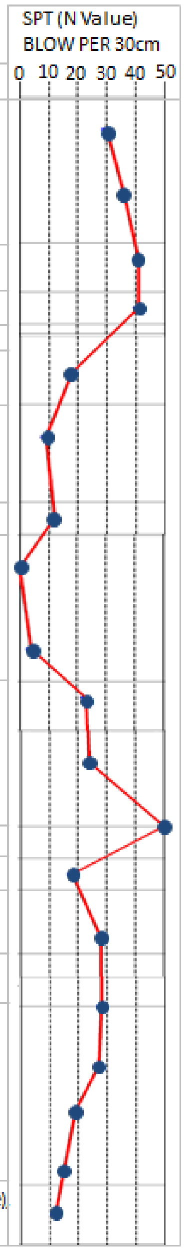


Figure 4: Schematisation of subsoil at project location

2.3 Tides

The tidal range at the project location is equal to **0.75 meters**.

Due to surge, the water level will rise an additional 0.5 meter, during the normative storm condition. Under the operational conditions, no surge will apply.

2.4 Currents

The maximum spring tide current velocity is equal to **0.1 m/s**.

2.5 Waves

2.5.1 Operational conditions

Tugboats will stop operating when waves above 1.5 meters will occur. For this reason, the operational limit of the terminal is set to **1.5 m**; the most unfavourable wave period corresponding with this wave height is equal to **17 s**. The waves are approaching within a bandwidth of **30°**; the centreline of the bandwidth is perpendicular to the coast.

Tp (s) / Hs (m)	0–2	2–4	4–6	6–8	8–10	10–12	12–14	14–16	16–18	18–35	Total
0 – 0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
0.25 – 0.75	0.00	0.02	0.08	0.19	1.69	3.28	1.87	0.42	0.10	0.04	**7.69**
0.75 – 1.25	0.00	0.02	0.84	6.02	12.59	20.31	14.99	3.88	0.95	0.08	**56.68**
1.25 – 1.75	0.00	0.00	0.04	0.44	2.24	3.50	13.21	6.63	2.05	0.20	**28.32**
1.75 – 2.25	0.00	0.00	0.01	0.01	0.27	0.24	0.74	1.79	1.07	0.10	**4.23**
2.25 – 2.75	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.00	0.05	0.00	**0.08**
2.75 – 3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.25 – 3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.75 – 4.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
Total	**0.00**	**0.04**	**0.96**	**6.67**	**16.80**	**27.33**	**30.82**	**12.72**	**4.22**	**0.42**	**100.00**

Table 1: All year wave distribution

2.5.2 Normative storm condition

The normative storm condition has a 1/100 years return period. The significant wave height during this storm is **3.3 meters** with a peak period of **18.6 seconds**. The angle of incidence of the normative storm is assumed to be perpendicular to the coast.

2.6 Seismological Conditions

The normative earthquake has a horizontal peak ground acceleration of **0.30 g**.

3 REQUIREMENTS ALTERNATIVES

3.1 Design lifetime

The **minimal** required design lifetime of the alternative breakwater is **10 years**. During the breakwater lifetime, no maintenance is allowed. Damage due to extreme wave or seismological conditions is permitted, but failure is not. Damage at the breakwater may not lead to any extra downtime during operational conditions.

3.2 Operational conditions

The operational conditions are summarized in Figure 5. At the terminal, a maximum significant wave height of **0.5 meters** is allowed. The maximum allowed wave height at the turning circles is **1.0 meters**.

The maximum wave height contribution at the lee side of the breakwater due to overtopping and transmission has to be between **0 and 0.5 m**, so the operational limits can be guaranteed. There is no boundary determined which reduces the maximal breakwater length (diffraction), as long as the combination of the three components: overtopping, transmission and diffraction don't exceed the operational conditions.

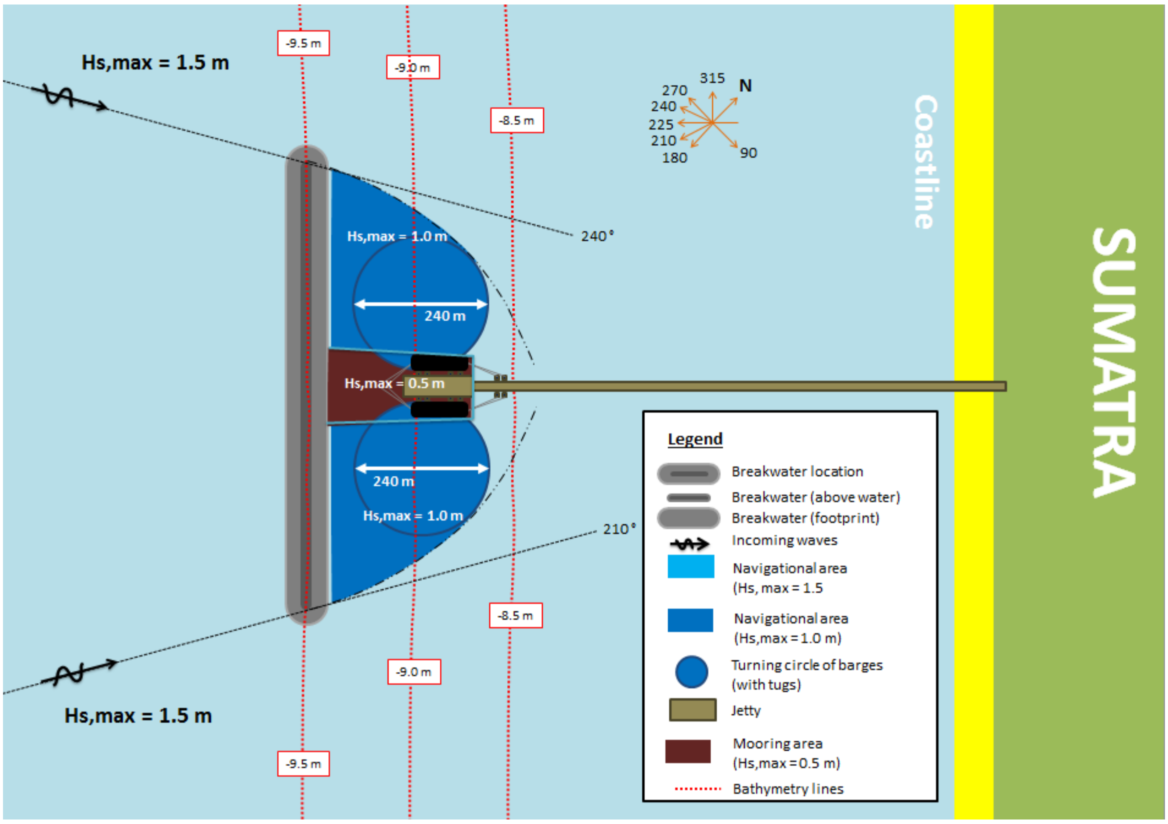


Figure 5: Operational Conditions

3.3 Failure

Failure of the breakwater during the extreme wave and earthquake conditions is not allowed. If the breakwater construction incidentally fails, the jetty construction should not be damaged. For this reason, the breakwater construction is not allowed to slide towards the jetty construction. Thereby, loose separated parts of the breakwater construction should not be able to damage the jetty either.

3.4 Costs

The innovative breakwater construction maximal costs **half the price** of a traditional breakwater.

4 APPENDIX I: POWERPOINT PRESENTATION

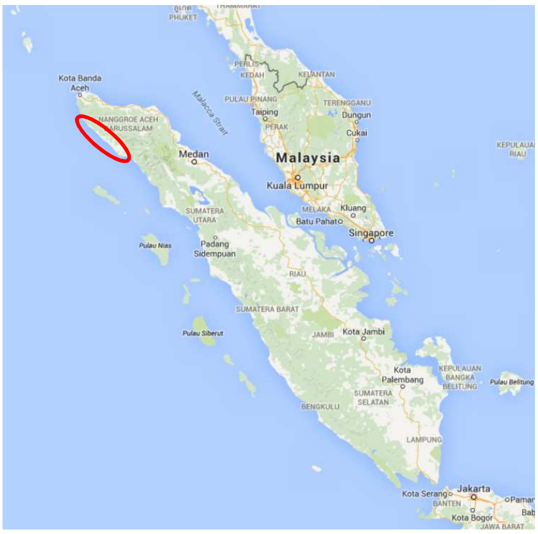
Alternative Breakwater

Introduction Brainstorm session

Danny Janssen, Jesper van Grieken
11 March 2015

Introduction

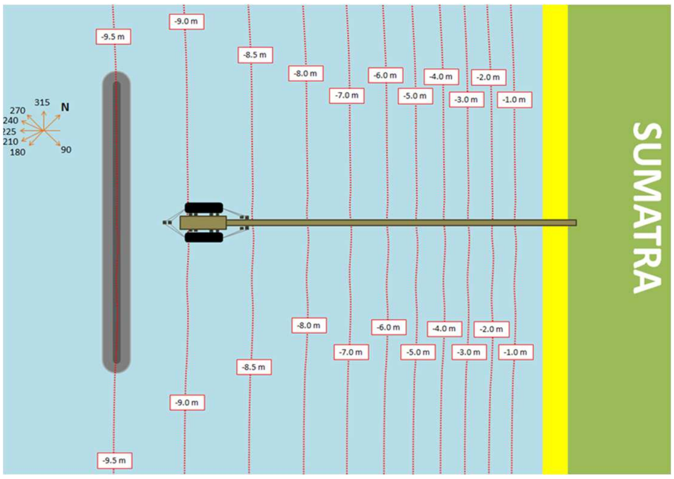
- Barge loading terminal
- Current downtime 80-90%

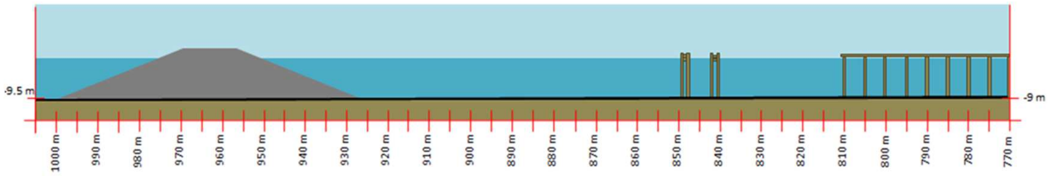


Environmental Conditions

Bathymetry

- Water depth 9.5 meters





Environmental Conditions

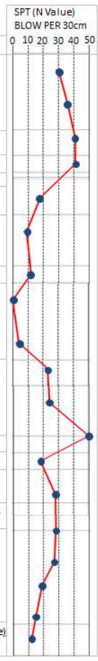
Subsoil

- Medium dense sand
- Granular size of sand is silty to fine

SCHEMATIZATION OF SUBSOIL AT PROJECT LOCATION

Surface RL: -9.50 m

DEPTH (m)	THICK (m)		MATERIAL DESCRIPTION
0.00 4.50	4.50	1	Medium to fine grained SAND, Loose
6.00	1.50	2	Coarse grained SAND, Dense
7.00	1.00	3	Sandy SILT, Dense
7.30	0.30		Sandy GRAVEL, Very Dense
9.50	2.20	5	Fine grained SAND, Loose
12.50	3.00	6	Silty SAND, Loose
13.50	1.00	7	Silty SAND, Carbonized (thin layer of Lignite), Loose
18.00	4.50	8	Silty CLAY, trace fine grained SAND, Soft Consistency, High Plasticity
22.50	4.50	9	Silty SAND, Medium Dense
23.50	1.00	10	Silty SAND, Very Dense
24.50	1.00	11	Silty SAND, Medium Dense
26.50	2.00	12	Fine SAND, Medium Dense
27.50	1.00	13	Silty CLAY, trace fine SAND, Medium Stiff Consistency, High Plasticity
33.50	6.00	14	Silty SAND, Medium Dense
35.50	2.00	15	SILT, trace fine SAND, Carbonized (thin layer of Lignite), Soft Consistency, Low Plasticity



Environmental Conditions

Tides and Currents

- Tidal range is equal to 0.75 m
- 0.5 m surge normative storm condition
- 0.0 m surge operational conditions
- Maximum spring tide current velocity 0.1 m/s

Environmental Conditions

Waves

- **Operational**
- H_s = 1.5 m, T_p = 17 s

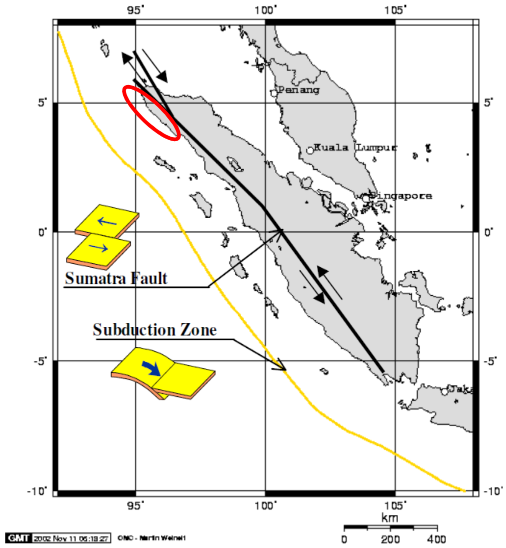
Tp (s) / Hs (m)	0–2	2–4	4–6	6–8	8–10	10–12	12–14	14–16	16–18	18–35	Total
0 – 0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25 – 0.75	0.00	0.02	0.08	0.19	1.69	3.28	1.87	0.42	0.10	0.04	**7.69**
0.75 – 1.25	0.00	0.02	0.84	6.02	12.59	20.31	14.99	3.88	0.95	0.08	**56.68**
1.25 – 1.75	0.00	0.00	0.04	0.44	2.24	3.50	13.21	6.63	2.05	0.20	**28.32**
1.75 – 2.25	0.00	0.00	0.01	0.01	0.27	0.24	0.74	1.79	1.07	0.10	**4.23**
2.25 – 2.75	0.00	0.00	0.00	0.00	0.02	0.00	0.01	0.00	0.05	0.00	**0.08**
2.75 – 3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.25 – 3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
3.75 – 4.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**
Total	0.00	**0.04**	**0.96**	**6.67**	**16.80**	**27.33**	**30.82**	**12.72**	**4.22**	**0.42**	**100.00**

- **Normative Storm Conditions**
- Rp = 1/100 years, H_s = 3.3 m, T_p = 18.6 s

Environmental Conditions

Seismological

- The normative earthquake has a 0.30 g peak acceleration



Requirements Alternatives

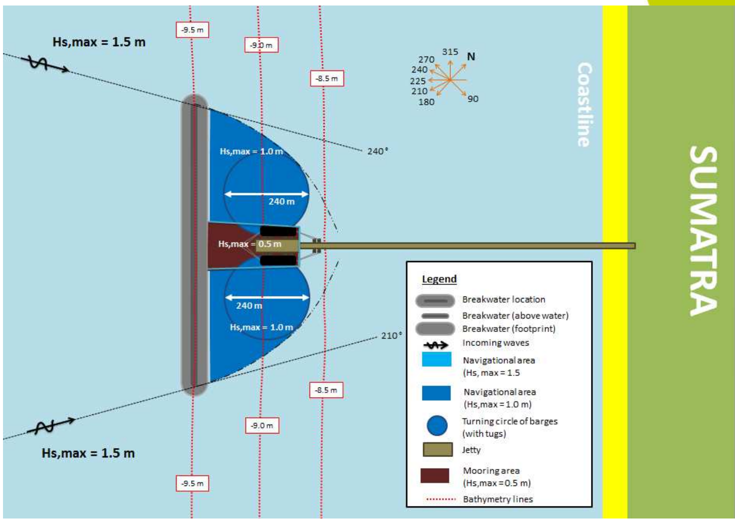
Design Lifetime

- Minimal lifetime of 10 years
- No maintenance allowed
- Damage during breakwater lifetime is allowed, failure is not
- Damage may not lead to any extra downtime during operational conditions

Requirements Alternatives

Operational Conditions

- $H_{s,}$ terminal = 0.5 m
- $H_{s,}$ turning circle = 1.0 m
- No limit to breakwater length
- Overtopping and transmission wave height contribution at lee side between 0 - 0.5 m



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Requirements Alternatives

Failure and Costs

- Failure is not allowed
- Damage at jetty due to (partly) failure of breakwater is not allowed
- Breakwater construction is not able to slide towards the jetty
- The innovative breakwater maximal costs half the price of a traditional breakwater

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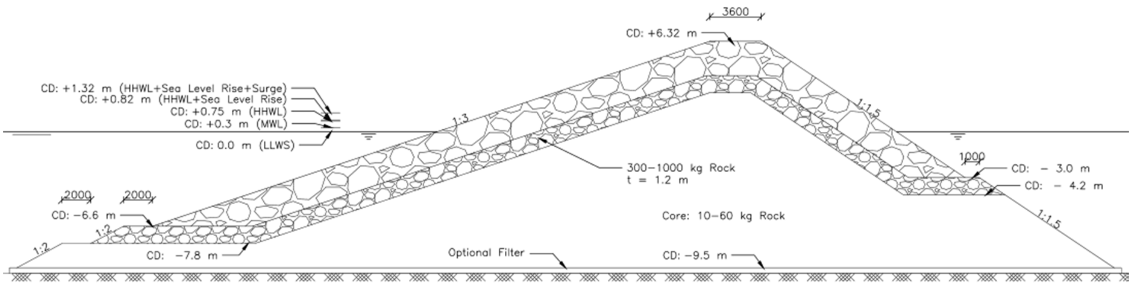
Reference Design



- Rubble Mount breakwater
- €62,000.- (+/- 25%) m^1

Brainstorm Session

Categories

1. Sand
2. Piles and Tubes
3. Slender Constructions
4. Floating
5. Waste Products
6. Containers
7. Old ‘’ships’’
8. Hybrid solutions (combinations of solutions)
9. Building with nature
10. Crazy Ideas

Brainstorm Session

Sand

- For example:
- Geo-tubes
- Dikes

Brainstorm Session

Plies and Tubes

- For example:
- Pile Pyramid
- Vertical Piles

Brainstorm Session

Slender Constructions

- For example:
- Concrete slab with steel support

Brainstorm Session

Floating

- For example:
- Steel plates with anchors

Brainstorm Session

Waste Products

- For example:
- Old Tires
- Cars

Brainstorm Session

Containers

- For example:
- Pyramid of containers

Brainstorm Session

Old Ships

- For example:
- Oil tankers

Brainstorm Session

Hybrid solutions

- Combination of Innovative solutions

Brainstorm Session

Building with nature

- For example:
- Mangrove

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Brainstorm Session

Crazy ideas

- Everything you can imagine

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Thanks for your innovation

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Appendix 5: Long list
Innovative Breakwater

16 June 2015
Final Report

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Document title	Appendix 5: Long list
Document short title	Long list
Status	Final Report
Date	16 June 2015
Project name	Innovative Breakwater

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Approved by	Harry Dommershuijzen	
Date/initials approval	………………….	………………….

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SUMMARY

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater.

For this reason, financially feasible alternative breakwater concepts have to be developed. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference, a traditional breakwater solution will be developed as well, i.e. the Reference Design.

The purpose of this document is to summarize a longlist of breakwater concepts. Some advantages and disadvantages of each breakwater concept are described independently. A first filtering at not feasible and too expensive breakwater concepts will be done. Next, feasible breakwater concepts and applicable materials will be combined with each other, so the most favourable combinations can be determined.

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CONTENTS

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1 INTRODUCTION

This document contains a long list of alternative breakwater concepts, which may serve at the set project location described in the basis of design. These ideas have been made up by the graduation students themselves and by colleagues of RHDHV during a brainstorm session. This document can be seen as a summarisation of the brainstorm session results complemented with breakwater concepts which are made-up during the graduation process.

This document gives a description of each breakwater concept which has been made up during the process. Each of these breakwater concepts will be compared on its advantageous and it's disadvantageous. If a breakwater concept is very complex or consists of expensive materials, the breakwater will not proceed to the short list, since the main purpose of the process is to define a low-cost breakwater. Higher complexity and/or higher material costs will result into higher breakwater costs. The distinction between the long list and the short list is made to reduce the required amount of breakwater concepts. If each of the breakwater concepts mentioned in the long list will be investigated in the short list there will be not enough time during the graduation process.

A second criterion is the rate of innovation of the breakwater concept. If the concept consists of a regular breakwater construction, it will not proceed to the short list. The above arguments are based on the main question of this project:

Design an alternative, low cost, non-traditional breakwater solution for of a specified location at the NW-coast of Sumatra.

Besides the criteria mentioned in the main question of the project, the client requested a maintenance free breakwater, this breakwater requirement can be seen in the basis of design. If an alternative breakwater construction requires maintenance, the construction will not proceed to the short list.

Each breakwater concept which is assumed to be low-costs, non-traditional and maintenance free will proceed to the short list. The process for determining the best alternative can be seen in
Figure 1.

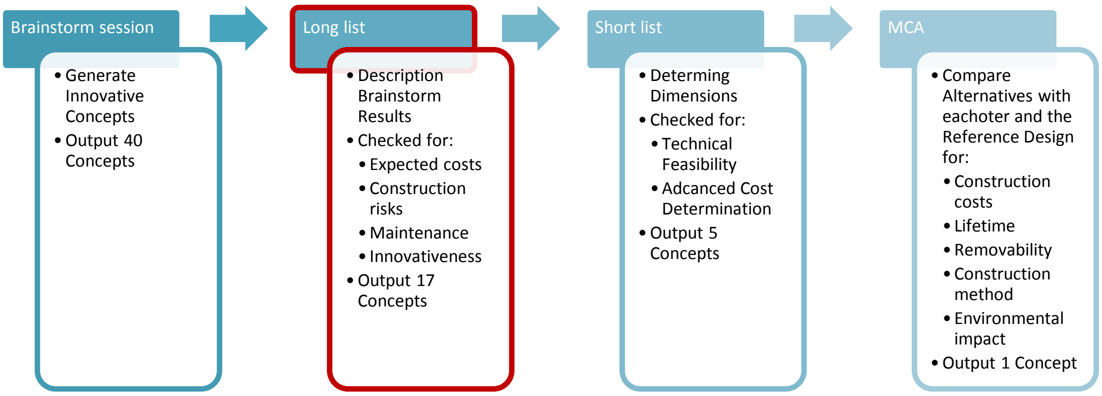


Figure 1: Process for determining "best alternative"

2 LONG LIST

2.1 Sand constructions

2.1.1 Island

When applying a sand supplementation in front of the jetty, a dynamic island will occur. Advantages and disadvantaged of this breakwater solution are:

Advantages

- Easy to construct.

Disadvantages

- Sedimentation of the fairway near the jetty.
- Island will erode due to wave forces.

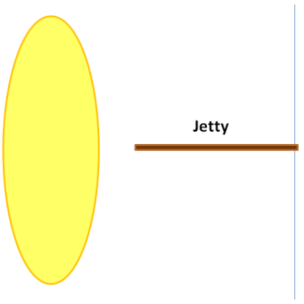


Figure 2: Island Breakwater

A supplemented island in front of the jetty is assumed to be not lucrative. This solution will result in much maintenance at the island and the jetty, because of the expected effects of erosion and sedimentation. Maintenance at the breakwater is not permitted during the lifetime of the breakwater. Due to the expected maintenance and the unpredictability of the sedimentation and erosion, the island breakwater will not be taken into account further.

2.1.2 Natural coast

A natural coast can be compared with option 2.1.1; however this solution contains an additional natural bank protection, like dunes or other plantation.

Advantages

- Easy to construct.
- Plantation will bind the sand together.

Disadvantages

- It is possible that the plantation will not grow at the supplemented island.
- Plantation requires time to develop.
- Non bounded sand will be moved towards the jetty.

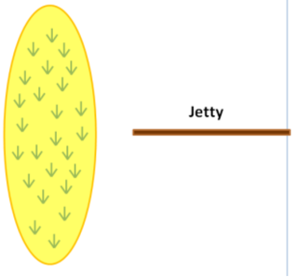


Figure 3: Island breakwater with dunes

The risks of plantation failure are considered to be too high. Thereby, it will take a while before the sand will be kept in place by the roots of the dunes. A third disadvantageous of the natural coast solution are the required slopes, since the dunes will not start growing below the water level. For this reason huge slopes or a secondary plantation is required in order to achieve stable breakwater slopes.

2.1.3 Geo-containers with sand

Another method to create an innovative breakwater is to fill bags or geo-containers with sand and stacking these sand bags up. These geo-containers will protect the sand from eroding away. The separate geo-containers need enough weight, so the bags will not flush away during the normative storm conditions.

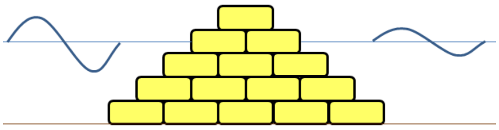


Figure 4: Geo-Container breakwater

Advantages

- The geo-containers can be stacked up very easily.
- The construction is assumed to be quite stable

Disadvantages

- Each bag has to be filled independently

Applying geo-containers, in order to reduce the wave height, seems to be a feasible solution. Geo-containers have been implemented in projects quite often; however a geo-container breakwater has never been constructed. For this reason, the geo-containers will be investigated further.

2.1.4 Create refraction with islands

Refraction is a phenomenon which will make waves bend towards the perpendicular to the coast, when approaching shallow water. When creating some islands in front of the jetty, containing of a very gentle slope, waves will refract towards the islands.

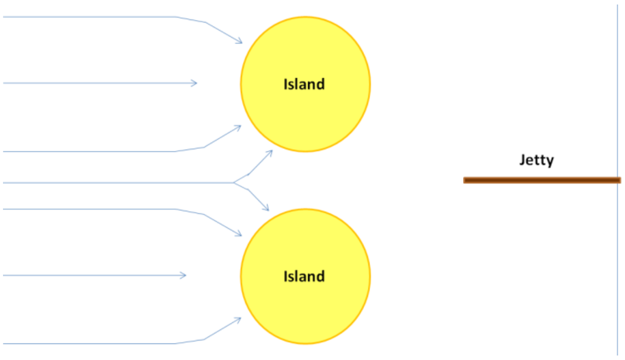


Figure 5: Refraction caused by islands

Advantages

- Waves will refract toward the islands, when approaching from different directions.

Disadvantages

- Island will erode, if not fortified.

Applying some offshore islands, in order to reduce the wave height of the jetty is definitely a plausible wave reduction solution. However, the islands may have to be protected against erosion. Applying offshore islands in order to create refraction will be considered further.

2.1.5 Create refraction with gully

Another way to create refraction is dredging a gully in front of the jetty. If waves are approaching perpendicular to the coast, the waves will start refracting towards the shallower, not dredged part of the ocean.

Advantages

- A gully is easy to construct (using a trailing suction hopper dredger)

Disadvantages

- Refraction effect will become less when waves are approaching from large angles relative to the gully. Gully effect mostly effective for waves more or less parallel with the gully.
- Sedimentation of the gully will occur during the gullys lifetime, so maintenance consisting of dredging works is required.

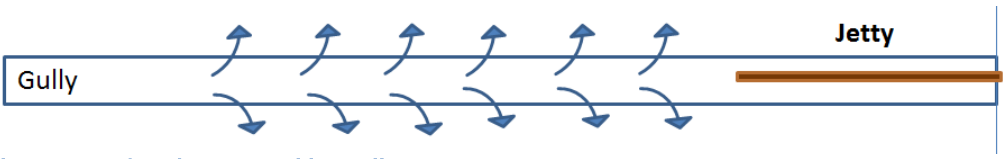


Figure 6: Refraction caused by gully

The wave reducing solution has to be maintenance free during its lifetime. When dredging a gully into the ocean, this requirement cannot be guaranteed due to expected sedimentation of the gully. For this reason, the solution of creating refraction using a gully won't be investigated further.

2.2 Piles and tubes

2.2.1 Vertical piles; single row, multiple rows

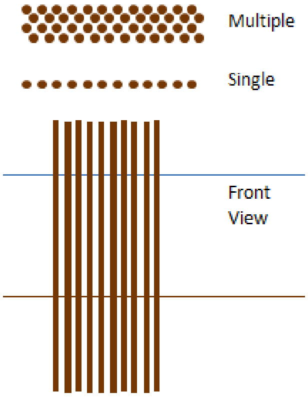


Figure 7: Single / Multiple pile row(s)

Constructing a vertical pile row is another option to reduce the wave height at the jetty. This solution can be performed using one single pile row or multiple pile rows in sequence. These piles can be constructed using different types of materials, like wood, steel or bamboo.

Advantages

- Vertical piles are easy to construct.

Disadvantages

- Lee side wave height may exceed the maximum wave height, due to a too high transmission through the breakwater.

This solution is determined to be feasible. The most critical requirement for this alternative is the wave height at the lee side of the breakwater caused by transmission. If this construction can't fulfil the determined operational conditions, the breakwater is not considered to be feasible at the project location.

2.2.2 Pyramid of piles and/or tubes

If stacking a couple of piles or tubes, in order to create a pile pyramid, a wave reducing construction will occur. The piles and tubes can be applied of different materials like wood, steel or bamboo.

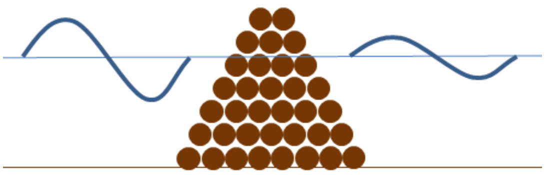


Figure 8: Pile pyramid

Advantages

- The breakwater can be pre-fabricated into different breakwater parts.

Disadvantages

- Connecting the piles to each other is very difficult.
- Risk of damage to jetty due to single pile/tube failure.

The pile/tube pyramid breakwater solution is determined to be plausible at the set project location. If applying relative cheap products as building material, like bamboo or wood, the price of the construction may be very attractive. The most complicated part of this breakwater solution is the connection method which connects the piles together. This connection method has to be determined further during the process.

2.2.3 Combined wall

A combined wall is often applied as a ground retaining structure, but it can also be applied as a breakwater. This construction contains steel hollow tubes, joint by sheet piles. In order to reduce the tubes diameter, a strut may be applied. The strut can be performed diagonal and vertical.

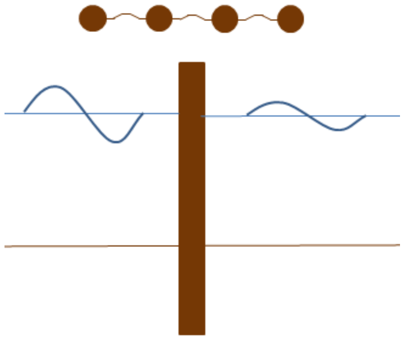


Figure 9: Combined wall

Advantages

- The transmission is expected to be nihil.

Disadvantages

- Huge forces at construction caused by the waves due to the closed construction.
- Steel constructions are often very expensive

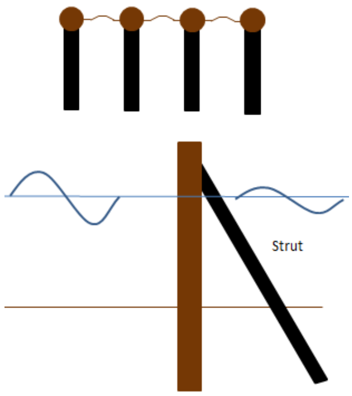


Figure 10: Combined wall with diagonal strut

The main material of the combined wall construction is steel. The price per unit steel is expected to be quite high. Thereby, the construction consists of a closed construction, which will not allow any transmission through the breakwater. This will last in high wave forces at the construction, which results into big material usage, in order to achieve a certain construction strength. Thereby, applying a combined wall has been applied many times in many ports, which makes the concept not innovative. For the above mentioned reasons, this breakwater concept will not be investigated further.

2.2.4 Mikado piles

The Mikado breakwater is realised by separately throwing tree branches or other construction materials at the bottom of the ocean. This breakwater solution will not be optimized further because the construction is too unpredictable. The construction risks due to partly failure of the breakwater are assumed to be too high. If a single tube will be separated from the Mikado breakwater, damage at the jetty may occur. A set boundary condition for the alternative breakwater is that no damage at the jetty due to (partly) failure of the breakwater is allowed. This boundary condition cannot be guaranteed when constructing a Mikado piles breakwater, without fortifications. If these fortifications are applied, the Mikado piles breakwater will become comparable to the pyramid of piles breakwater.

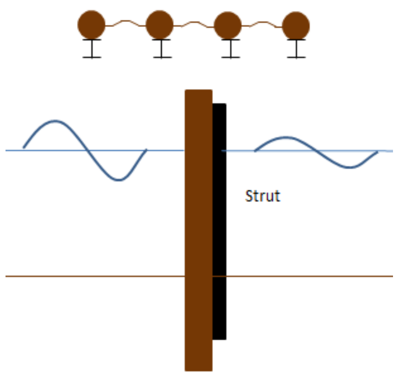


Figure 11: Combined wall with vertical strut

2.2.5 Pipe with angle

When constructing tubes under a horizontal or vertical angle, as seen from the front view. The wave forces will be reduced due to the smashing effect of the waves. In order to make the construction functional, the hollow pipes have to be constructed under a specific angle. Thereby, the pipes have to be supported by a supporting construction, like a concrete cover, in order to protect the pipes of deformations, to keep the piles in the right position.

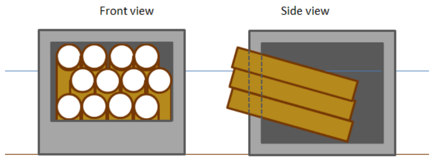


Figure 12: Pipe with vertical angle breakwater

Advantages

- The hollow pipes may be performed using second hand hollow pipes.

Disadvantages

- The pipes have to be kept in place in order to guarantee the wave reducing effect of the breakwater.
- Constructing this breakwater solution is assumed to be quite difficult. Since the piles requires a specific angle in order to achieve the required wave reduction. Thereby, variation in wave angle requires a different angle of the pipes.

This breakwater solution will not be optimized further during the process. It is assumed that the total costs of the breakwater are too high, due to use of materials and to the construction costs. Thereby, the risks of construction deformations during the construction lifetime, due to earthquakes and hydraulic are assumed to be quite high. In order to manage these construction risks, the construction has to be fortified; this will last in higher construction costs.

2.2.6 PVC Pipe with curve

The PVC pipe with curve solution, is based on the pipe with angle solution, however for this solution flexible pipes are taken into account instead of rigid pipes.

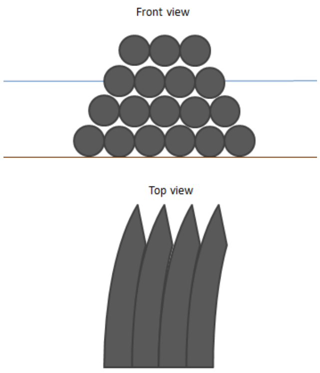


Figure 13: PVC pipe with curve

Advantages

- The total costs of the construction materials are assumed to be quite payable.

Disadvantages

- The pipes have to be bound together, in order to protect the pipes of displacement caused by the environmental conditions. Each pipe has to be connected independently.
- The relative density of PVC is expected to be quite low.

This breakwater solution will not be optimized further. The risks of single pipe failure are too high due to the low own weight of the pipes. Due to single pipe failure, damage to the jetty may apply, damage to the jetty is not allowed. Thereby, constructing these pipes into a specific curve requires a very complicated construction method. The curves have to be kept in place in order to prevent deformations due to the environmental conditions. For the above reasons, the pipe with curve will not be taken into account in the short list.

2.3 Slender Constructions

2.3.1 Jacket breakwater

When constructing a breakwater based on the shape of an offshore jacket, the breakwater is able to withstand relatively high forces. If the ocean facing side of the jacket is closed, (almost) no wave transmission will occur through the breakwater. The jacket construction can be performed using different kinds of materials.

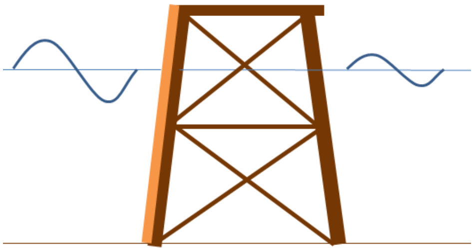


Figure 14: Jacket breakwater

Advantages

- The transmission is expected to be nihil.
- Jacket construction distributes forces equally.

Disadvantages

- The construction of the jacket breakwater takes quite long (connecting all pieces together).
- High material usage expected.

Considering both the advantages and the disadvantages of the jacket breakwater, the breakwater is assumed to be lucrative. The costs of the jacket breakwater may be high, if the entire construction is realized in steel. For this reason, jacket constructions consisting of other construction materials will be compared for example bamboo and wood in order to give a good cost comparison.

2.3.2 Caisson breakwater with sheet piles

Combining a caisson breakwater with sheet piles, will result into a construction using less concrete than the original caisson breakwater.

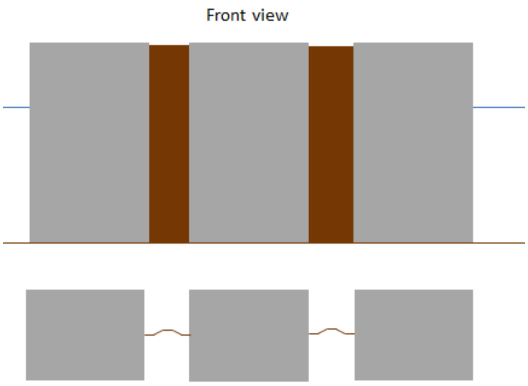


Figure 15: Caisson sheet pile breakwater

Advantages

- Relative strong construction

Disadvantages

- A special connecting piece has to be designed in order to connect the caisson with the sheet piles.
- Closed concrete structures are vulnerable during earthquakes.

According to the feasibility study of (DHV, 2011), a caisson is not feasible at the project location, because of the high risks of earthquakes. According to (Stive, 2015), a caisson breakwater is possible on the project location. However, to make it stable, the construction will have very big dimensions and therefore will be very expensive. Thereby, a complex connection method between the caisson and the sheet pile has to be developed, in order to achieve the required connection. Due to the above mentioned arguments, it is assumed that the costs of the caisson breakwater with sheet piles will be too high.

2.3.3 Slab with support

Another innovative breakwater solution is a breakwater existing of pre-fabricated slabs supported by beams. The construction can be built in steel, wood, bamboo or concrete.

Advantages

- A relative simple construction.

Disadvantages

- Construction has to be fortified at the subsoil, in order to avoid the breakwater construction sliding towards the jetty.
- Connection between slab and support may be hard to construct.

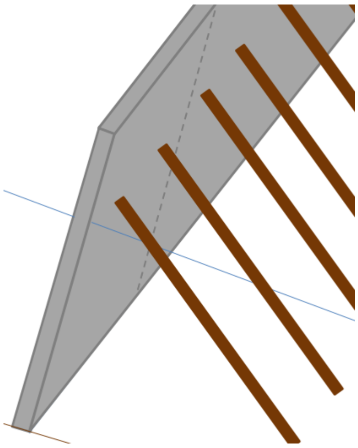


Figure 16: Slap of Concrete breakwater

The construction may be feasible for the project location if build in prefabricated concrete elements, where the supporting beams and the slab are already connected on shore (like schematized in figure 15). The concepts of steel, wood and bamboo are also considered feasible but harder to construct than the concrete version. If constructing a steel, wooden or bamboo slab with support, a construction comparable to the jacket breakwater will be applied. Therefore only the concrete slab with support breakwater will be investigated further.

2.3.4 Hollow breakwater

A hollow breakwater significantly reduces the required materials for a breakwater construction. By applying some pre-fabricated concrete units, a hollow breakwater can be constructed.

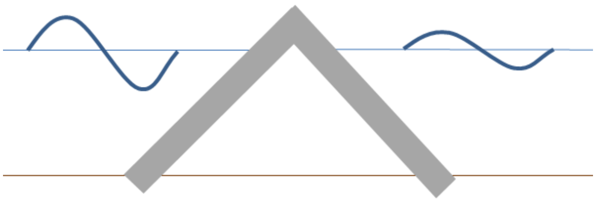


Figure 17: Hollow breakwater

Advantages

- The pre-fabricated concrete units are easy to put in place.

Disadvantages

- The concrete units are vulnerable for seismological activity.
- Huge concrete units required in order to achieve the required water depth and withstand the huge wave forces.

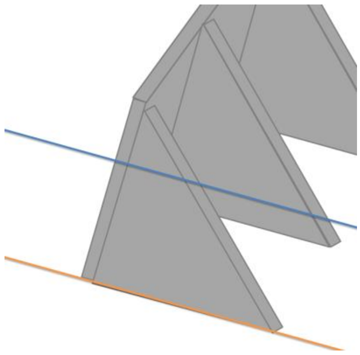


Figure 17: Merging between slab with support and hollow breakwater

The hollow breakwater as schematized in figure 16 is assumed to be not applicable at the set project location. The seismological conditions are too critical for the concrete structure at the project location, so huge reinforcements are required. However, when the concept is built like schematized in figure 17 (with application of backfill if needed), the concept is assumed to be feasible. The concept shown in figure 17 is a merging of the slab with support concept with the hollow breakwater. This concept will be the only concept of these two categories to be investigated further.

2.3.5 Bellows structure

Another breakwater construction can be performed by inflating a balloon with air of fill a balloon with water, this balloon has to be anchored to the subsoil in order to eliminate the buoyancy forces. This balloon can be performed with materials like rubber or plastic.

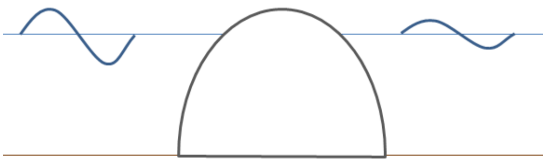


Figure 18: Bellows Structure

Advantages

- The bellows structure may be performed using recycled rubber or plastic.

Disadvantages

- Huge anchorage is required in order to withstand the vertical buoyant forces.
- Leakage of the balloon will cause construction failure.

This breakwater concept is determined to be feasible at the current project location; however the costs of the construction due to the required anchorage structure may be very expensive.

2.4 Floating constructions

Another direction for innovative breakwater concepts is applying a floating construction. This direction is discouraged by the colleges of RHDHV, due to the swell wave climate. These swell waves will make the construction heave at the same speed and frequency as the incoming waves. Due to this heaving, the waves will not be reduced by the breakwater construction. Beside, to guarantee that the transmission through and underneath the breakwater is not too high, the floating construction has to cover a significant part of the water depth.

The heaving can be compensated in two ways:

- Anchoring
- Increasing the size of the floating breakwater

Anchoring

To prevent a floating construction from heaving, it can be anchored to the seabed. The anchoring forces required in the design situation are expected to be very high. Therefore anchoring is considered as financially not feasible.

Increasing the size of the floating breakwater

According to (Groenewegen, 2015), floating breakwaters need a minimum width of at least 1 time the wavelength. The incoming waves have a wavelength of approximately 175 to 200 meters. This means the required minimum width of the breakwater is approximately 175 to 200 meters. Therefore, increasing the size of the floating breakwater is considered as financially not feasible.

Figure 19: Floating construction with swell wave

2.5 Waste products

2.5.1 Busses/Car wreckages

By stacking car and/or bus wreckages, a breakwater construction can be constructed. If applying car wreckages in a breakwater construction, no oil may be left behind in the car wreckages to prevent the water from getting polluted

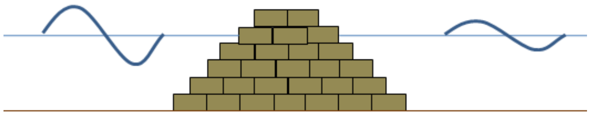


Figure 20: Busses/car wreckages breakwater

Advantages.

- Car wreckages are assumed to be quite cheap.

Disadvantages

- Risk of leaking oil into the ocean.

The advantages of applying old car wreckages are very encouraging. The risk of leaking oil into the ocean can be prevented by cleaning the car wreckages from fuel and oil, this will last in extra construction costs, but will prevent an environmental hazard. For this reason, this solution will be investigated further during the process.

2.5.2 Old airplane pieces

Instead of using busses and cars in order to create a breakwater construction, old airplane pieces could serve as a breakwater construction as well. However, it is assumed that there are more car and bus wreckages available than airplane wreckages. For this reason only the car wreckages solution will be investigated further instead of the airplane wreckages solution. If the car wreckage breakwater turns out to be feasible, an airplane wreckage breakwater might be feasible too (dependant on the material quantity and availability).

2.5.3 Old tires

Another material in the category waste products are old car tires. When stacking these tires, a breakwater construction will be emerged.

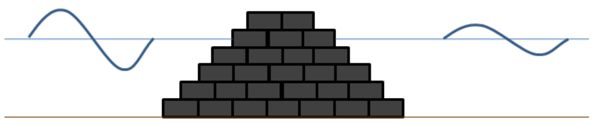


Figure 21: Old tire breakwater

Advantages

- The unit price of a single tire are assumed to be quite cheap.

Disadvantages

- Difficult to link the tires together.
- Risk of single tire failure.

It is assumed that there are a lot of tires per breakwater cross section required, in order to achieve a stable breakwater construction. However, the costs of a single tire are assumed to be quite payable. When applying a huge amount of tires of a low cost price per tire, the total breakwater its costs may be quite lucrative, for this reason, the tire breakwater will be investigated further.

2.6 Containers

2.6.1 Container pyramid

A breakwater solution using containers can be applied by stacking the containers, this result into a container pyramid. This container pyramid can be constructed using different types of container configurations. The containers can be placed horizontal in direction of the waves or in the direction of the shoreline.

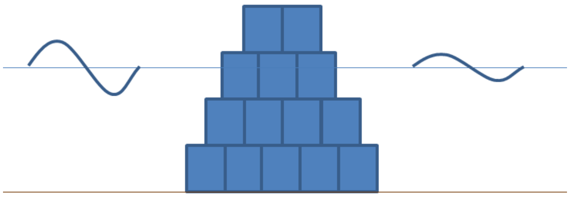


Figure 22: Container pyramid

Another variable in the container pyramid is the amount of fill; the containers can be filled completely or not at all. If the container is filled, the container is able to withstand more wave forces before flipping over, due to extra gravitational forces.

Advantages

- Second hand containers are assumed to be not expensive.
- It is assumed that the container frame can absorb quite many forces.

Disadvantages

- Difficult to link the containers together.
- Each container has to be filled independently.

Applying containers using a pyramid construction is assumed to be a feasible solution. The highest risk is corrosion during the breakwater's lifetime; the containers can be prevented from corrosion by applying a container protection. The container pyramid and the possible container configurations will be investigated further during the process.

2.6.2 Core of containers

A second container breakwater solution is created by replacing the breakwater core (quarry run) with old containers.

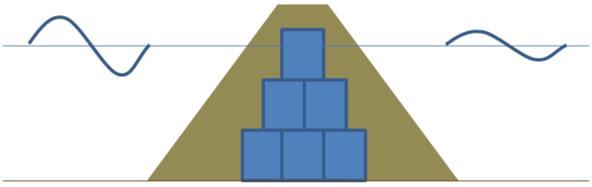


Figure 23: Core of containers

Advantages

- The required containers are assumed to be cheaper than quarry run.

Disadvantages

- Collapsing of containers might occur due to the weight of the rubble mound armour layer.

At this moment, the container core breakwater solution is determined to be financially lucrative. During further investigation the construction might turn out to be unfeasible due to container failure.

2.6.3 Container wall with struts

When applying a vertical container wall, supported with diagonal or vertical supporting beams, the vertical container may be able to withstand the wave conditions. This solution can be applied using different container configurations as well, see 2.6.1 container pyramid.

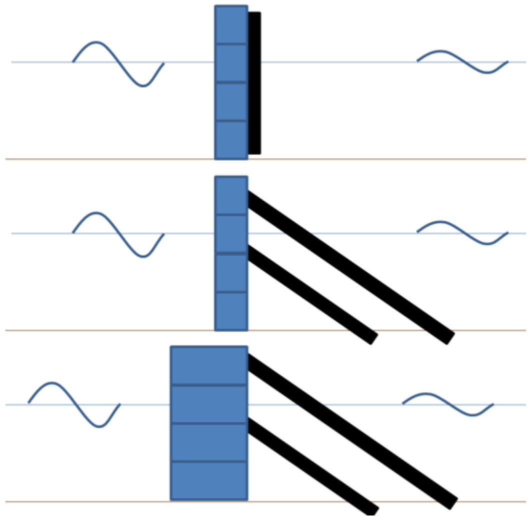


Figure 24: Container wall with struts

Advantages

- Applying containers instead of a steel combined wall is assumed to be cheaper.

Disadvantages

- Huge forces at the breakwater construction may result into an unstable breakwater construction.
- Diagonal struts have to be constructed before the containers can be placed.

This solution is assumed to be financially lucrative. The most critical design aspect for this breakwater solution is the required fortifications. Technical calculations have to determine if there are any fortifications required in order to increase the strength of the containers.

2.6.4 Container protection

The lifetime of, in particular, the container walls are estimated to be quite low. Due to corrosion of the steel, holes in the container wall are expected, which will last in transmission through the breakwater. In order to extend the lifetime of the containers, two different protection methods for extending the lifetime of the container have been made up.

Coating
When coating the containers with a special coating, the lifetime of the container can be extended. When coating both sides of the container walls, the lifetime may increase more.

Steel shield
A second way of increasing the container its lifetime can be performed by welding steel plated in front of the container walls. These steel plates will increase the thickness of the container walls and herewith the time it takes to completely rust away the container wall.

2.6.5 Container connections

It may be required to connect the used containers together due to the internal stability of the different breakwater concepts. The containers could be connected in the way they are connected on container ships, with twist-locks and corner castings. However, this requires the containers to be stacked exactly on top of each other, which is assumed to be extremely difficult in the occurring swell waves. Therefore, there are two container connection methods made up for situations where container connections may be necessary.

Steel cables
A possible solution of a container connection is to string the containers together using steel cables. When putting these steel cables under a tensile force, the containers will be connected together. This connection can be applied in both the horizontal and vertical direction of the cross section.

Dowels
A second method for connecting the containers together is using a dowel connection. This connection will be realized using a blot connection. This connection will be completed using a bolt at both sides of the connection.

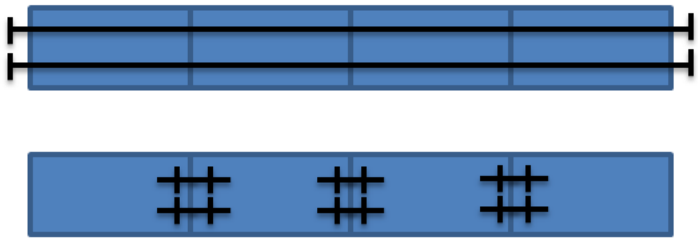


Figure 25: Container connections

2.7 Ships

If sinking an old ship to the bottom of the ocean, in front of the jetty, the waves at the lee side of the ships will be reduced. There are different kinds of old ships which may be applied as an innovative breakwater solution. A list of possible applicable ships types in determined as follows:

- Oil ships
- Barges
- LNG ships
- Cruise ships
- Car carriers
- Concrete ships (comparable with caisson)

The most beneficial ship based on the required amount of ships and their price, will be optimized in the short list. The sinking ship breakwater solution is expected to be financially lucrative at the project location.

2.8 Building with nature

2.8.1 Mangrove

A breakwater solution in the category building with nature, is realizing a mangrove forest in order to reduce the waves.

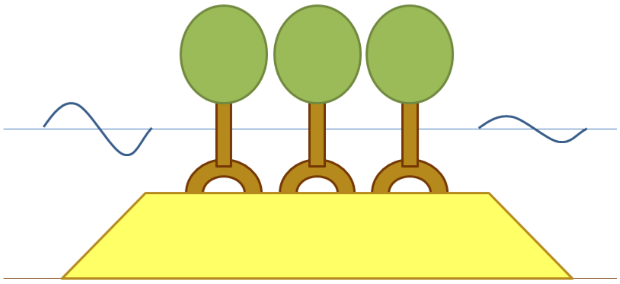


Figure 26: Mangrove breakwater

Advantages

- The mangrove trees will positively influence the water quality at the jetty location.

Disadvantages

- The mangrove trees will take a while to develop, for this reason it takes several years before the mangrove breakwater fulfils its task (reducing wave height).
- The mangrove trees may not start growing at the breakwater location (plantation failure).

The mangrove breakwater will not be considered further. It will take a while before the mangrove trees consists of enough roots, which are required in order to reduce the wave height. A second issue when constructing a mangrove breakwater is the maximum allowed water depth which the mangrove trees require in order to start growing. In order to achieve this maximum required water depth, a sand nourishment have to be reclaimed. The sand will likely erode due to waves and currents, until the roots of the mangrove trees will bind the sand together. Due to the required time it takes the mangrove trees breakwater to achieve its operational wave conditions and the expected sedimentation during this required time, the mangrove breakwater will not be investigated further in the short list. It will take too long before the mangrove trees achieve its operational conditions.

2.9 Crazy ideas

2.9.1 Lava hole

The idea of a lava hole is based on a volcano eruption. When drilling through the top layer of the earth crust until the magma is reached, a custom made volcanic eruption will occur. If the hot lava touches the cold water, the lava will petrify. The petrified lava will form a very strong breakwater.

The lava hole solution is determined to be not feasible at the project location. The lava stream cannot be controlled when floating out of the bore hole; this will result into dangerous situations for the construction workers and the entire surroundings. Thereby the costs of the boring equipment are assumed to be very expensive. For this reason the lava hole solution will not be optimized further.

2.9.2 Vertical tubes with band/slab

The vertical tube wall with a band/slab is a variation of the combined wall. The sheet piles will be replaced by a slab made of rubber or plastic. If it occurs that the horizontal forces at the tubes are too high, a strut can be applied, as seen in chapter 2.2.3, combined wall.

Advantages

- If the slabs are constructed as a waste product, the breakwater can be assumed as sustainable.

Disadvantages

- If struts are required in order to reinforce the construction, the total price of the breakwater solution will rise significantly.
- A connection method between the tube and the slab has to be designed.

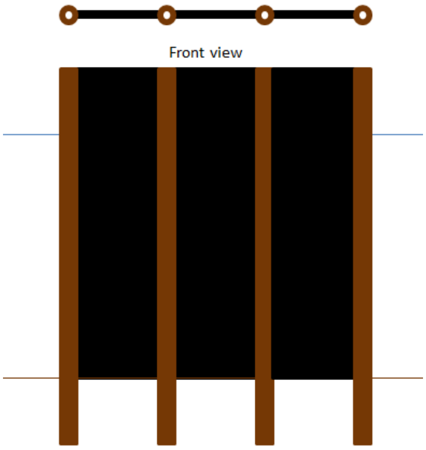


Figure 27: Vertical tubes with band/slab

For this moment, the vertical tube with band/slab solution is assumed to be a lucrative breakwater concept at the project location. During the further investigation process, the connection between the slabs and the tubes has to be determined. This connection may be the most complex part of this breakwater concept, since it has to transfer many forces form the slab to the steel piles.

2.9.3 Natural sedimentation

When building a breakwater in the extension of the jetty, natural sedimentation will occur. The sedimentation will serve as a wave reducing construction, if the sedimentation is perched.

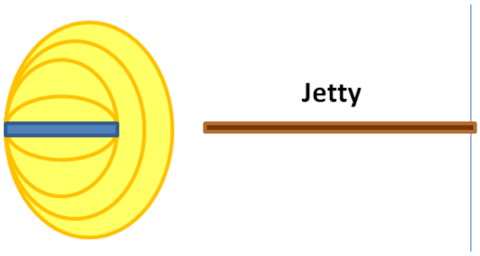


Figure 28: Natural sedimentation

Advantages

- A relative small breakwater construction is required in compression with the reference breakwater. This construction is only constructed for obstructing the sediment flow.

Disadvantages

- It will take a while for the sedimentation to perch at the breakwater location.
- A huge amount of sedimentation is required in order to achieve the required operational conditions.

The solution natural sedimentation is determined to be not plausible at the project location. It will take too long before the breakwater construction achieves its operational requirements. Thereby, the sedimentation transport is obstructed due to external impacts near the project location.

2.9.4 Offshore platform

When buying an old offshore platform and sinking this platform down in front of the jetty, a breakwater construction will occur.

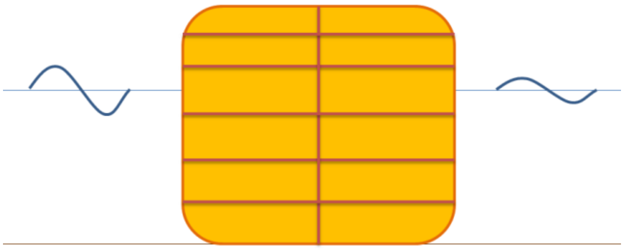


Figure 29: offshore platform

Advantages

- The platforms are huge and strong constructions which are built to resist influences of the sea.

Disadvantages

- The offshore constructions are very heavy, for this reason soil reinforcement or foundation work may be required in order to prevent soil failure.
- The offshore platform has to be cleaned completely; to be sure the water near the breakwater will not be polluted.

This solution will not be optimized further during the process. It is assumed that there are too little second hand offshore platforms available in order to create a breakwater. Thereby, the used materials for the offshore platform construction are quite expensive and may be reused for other purposes as well. Cleaning the entire offshore platform in order to avoid ocean pollution is assumed to be quite expensive either. For these reasons, the offshore platform solution will not be investigated further.

2.9.5 Iceberg without ice

Another alternative breakwater solution is the iceberg without ice solution. This solution contains lightweight floating materials like light expended clay aggregate or coconuts. These lightweight materials will be packed together using a string bag with very small gaps.

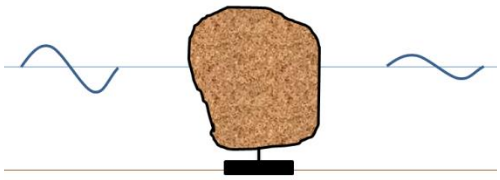


Figure 30: Iceberg without ice

Advantages

- The construction time of the construction is assumed to be short.

Disadvantages

- If a hole in the string bag occurs, the entire breakwater construction will fail.
- Floating structures are expected to be not lucrative, see chapter 2.4..

The iceberg without ice construction is assumed to be not lucrative at the project location. Due to the swell waves, the construction have to be enormous in order to avoid the construction from heaving. As mentioned in chapter 2.4, floating constructions, the minimal required breakwater width is approximately 200 meters. Thereby, the risks of construction failure due to a gap in the string bag have to be managed, in order to avoid construction failure. The risk can be avoided by applying a string bag of enough strength, which will increase the breakwater its costs.

2.10 Removable breakwaters

2.10.1 Removable breakwater

When constructing a breakwater, which is designed for the operational wave conditions and will be removed during the normative storm conditions, the breakwater can be designed lighter. This lighter design will result into less use of material due to reduced forces at the breakwater construction.

Applying this removable breakwater will not be investigated being an obligated requirement. If the breakwater turns out to be removable without any huge structural changes, the removable breakwater concept will be optimized further.

Thereby, there is a risk of removing the breakwater too late. The weather can be predicted in general. However, the weather may change unexpected, which may lead to a storm. If the construction is not able to withstand this storm, massive damage at the breakwater may occur. Due to this risk, the removable breakwater will not be taken into account further.

2.10.2 Rent a Breakwater

Rent a breakwater is a cost recovery model. This model can be applied if a contractor requires a temporary breakwater. When designing a removable breakwater with a huge lifetime, it can be used multiple times at different locations; during the lifetime of the breakwater the collected rent will be higher than the construction costs.

This cost recovery model will not be investigated during the process. Creating a breakwater which is able to withstand every possible environmental condition is expected to be difficult, due to the much variables at a project location, for example wave height, wave length, quality of subsoil, earthquakes, wind, currents, temperature and required breakwater performances (breakwater length, height and permeability).

2.11 Alternative combinations

2.11.1 Soft construction enclosed by hard construction

A combination between two solutions can be performed by combining an island alternative with a hard solution alternative, like containers or car wreckages. When protecting the soft breakwater heads with a hard solution, less erosion of the breakwater heads is expected. If the forces at the ocean site of the breakwater will result in much cross transport, a rubble mound rock armour have to be applied in order to protect the sand of washing away.

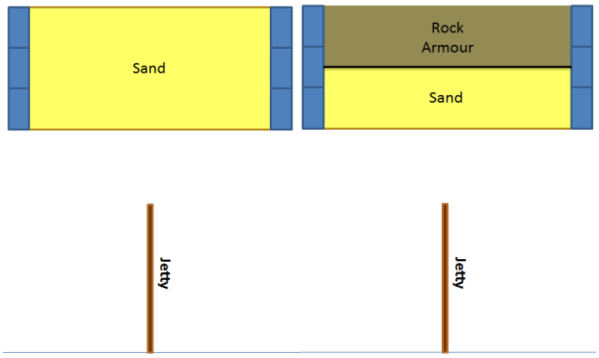


Figure 31: Combination of island with breakwater head protection

Advantages

- The entire breakwater costs when applying sand and a waste product protection are expected to be quite low.

Disadvantages

- Huge cross transport erosion rates may apply due to extreme storm conditions.

Verifying the advantages and disadvantages, the construction is assumed to be feasible. During further investigation, the morphological effects on the protected offshore islands have to be determined, in order to define the dimensions of an eventually required rubble mound rock armour protection. However, it is assumed that applying a rock armour layer will result into construction costs higher than the reference design, due to the combination between the rock armour layer and the fixed construction.

2.11.2 Waste products enclosed by piles

Another breakwater combination is schematized when enclosing a waste product, like cars or containers within some vertical piles. Applying this combination requires less waste materials in comparative to the waste product pyramid solutions.

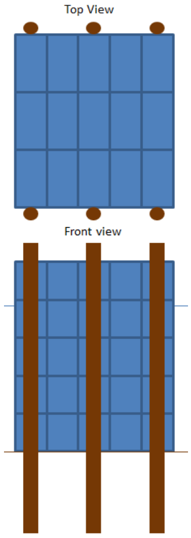


Figure 32: Waste product enclosed by piles

Advantages

- Less waste material products are required in comparative to the pyramid solutions.

Disadvantages

- Strict measurements during the construction phase of the construction are required in order to guarantee that the waste materials fit inside the piles.

A rough material versus costs estimation has to be established in order to decide with of the solution is the most lucrative. Both solutions are assumed to be lucrative, due to the application of waste within the breakwater. It is assumed that the unit prices for waste materials are quite attractive.

2.11.3 Replacing armour layer or core

When replacing the rock material of the reference breakwater design with alternative materials like cars of containers, the costs of the breakwater can be reduced. This solution is comparable to the container core solution. However, it may also be possible to replace the breakwater armour layer with an alternative material, like cars or containers. The following possible combinations of reference design breakwater combinations are possible, table 1:

Alternative	Core	Armour layer
Busses/Cars	Yes	Yes
Airplane Pieces	Yes	Yes
Tires	Yes	No
Containers	Yes	Yes

Table 1: Replacing armour layer

The busses/cars, tires and container core will be investigated further in the next phase of the progress. The airplane pieces core breakwater will not be investigated due to the estimated lack of airplane pieces.
The replaced armour layer combination will not be investigated further during the progress. It is assumed that the savings for replacing the armour layer are not that lucrative, caused by the costs deviations of both of the breakwater solutions.

2.11.4 Combinations with berm breakwaters

When constructing a breakwater with a berm in front of the slope, the wave forces at the breakwater itself will be reduced, this results into reduced main breakwater dimensions. The concept of the berm breakwater is schematized in figure 32. Table 2 summarizes which breakwater solution may be combined with a berm in front of the breakwater.

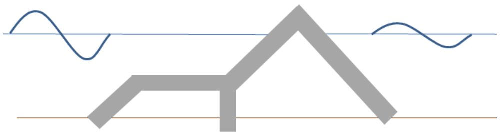


Figure 32: Combination with berm

Alternative	Berm applicable
Islands	Yes
Geo-Container	Yes
Pyramid of piles	Yes
Jacket breakwater	Yes
Slab with support / Hollow breakwater	Yes
Busses/car wreckages	Yes
Old airplane pieces	Yes
Old tires	Yes
Container pyramid	Yes
Container Wall	Yes
Vertical tubs with band/slab	Yes

Table 2: Combination berm breakwaters

If the most advantageous alternative breakwater solution may be combined with a berm, the savings of a berm construction will be investigated during the alternative optimisation progress. If the alternative shows out to be more favourable, a berm construction will be applied.

2.11.5 Tires and piles

If the vertical pile row is combined with the tire solution, a vertical pile row with tires will occur. By sliding the tires over the vertical piles, the diameter of the piles will increase, so the total required vertical piles will be reduced.

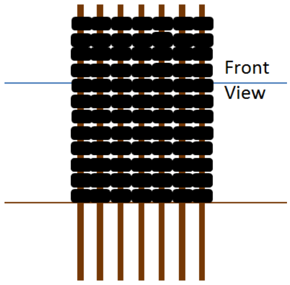


Figure 33: Tires and piles concept

Advantages

- It is assumed that the costs of applying tires instead of piles are less expensive than an entire vertical pile breakwater.

Disadvantages

- The gravitational force of a single tire in water is quite low, for this reason the weight of the tires have to be increased, in order to make sure that each pile is filled with tires from top to bottom.

The vertical tires and piles breakwater might be more lucrative than the pile row breakwater. If the transmission through the tire pile breakwater is less than transmission through the vertical pile row breakwater, the tire pile breakwater may be very lucrative. For the above reasons, the pile tire breakwater solution will be taken into account during the further process.

2.11.6 Container armour layer with tire core

The idea is that the core of the breakwater is made of old tires and the armour layer is made of reused containers A schematization of this concept is shown in figure 34.

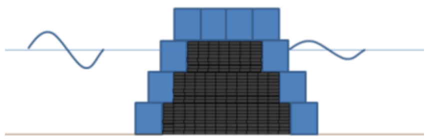


Figure 34: Container armour layer with tire core concept

Advantages

- It is expected that the materials applied are cheaper than the materials for a rubble mound breakwater.

Disadvantages

- The construction may not be stable without some measures.

The container armour layer with tire core is expected to be not lucrative at the project location. However, the required materials are less expensive than the materials of the reference design breakwater. The construction requires a lot of fortifications in order to achieve a stable breakwater construction. The containers and the tires have to be connected to each other in order to achieve a certain stability in order to prevent the loose breakwater materials from moving. Due to this required fortifications, the breakwater is assumed to be not lucrative at the project location.

3 MATERIALS

3.1 Introduction

Some of the solutions, which were discussed in chapter 2, could be constructed using different types of material. Besides the common construction materials concrete, steel, wood, sand and rock, some other innovative materials may be used as a construction material as well. In this chapter, the applicability of some innovative materials can be found.

3.2 Not common construction materials

3.2.1 Sand, mixed in place

If mixing sand, cement and water together, sand mixed in place will occur. This mixture can withstand more shear stress before it starts moving by the environmental forces. Due to the fact that untreated sand is less expensive than the mixed in place sand, normal sand is preferred above the sand mixed in place. The sand mixed in place is comparable to the common building material grout. If this suspension will be installed directly into the ocean, it will disperse in the ocean. This will make the mixture only applicable for on-shore reinforcements. Thereby, it is assumed that applying construction waste is less expensive as a fill material than the sand, mixed in place. For the reasons mentioned above, sand, mixed in place/grout, will not be considered as building material in the next phase.

3.2.2 Palm trees

There are a lot of palm oil farms at Sumatra. The palm oil is obtained from the fruit of the oil palm. If these palms are not productive every part of the tree can reused by the local inhabitants. However, most of the palm tree plantations are burned down at the end of the season, because this is the cheapest way to get rid of the old trees. Felling the trees is more expensive for the land owners. Therefore, palm trees will not be applied as a building material.

3.2.3 Construction waste

Construction waste could be used as an alternative core material instead of quarry run. Thereby construction waste can also be applied instead of sand, if applying sand appears to be unstable.

At Sumatra, there are still a lot of buildings which were partly destroyed during the 2004 tsunami. These buildings can be demolished completely in order to achieve some extra construction waste. An additional advantage of demolishing these old buildings is the increased liveability at Sumatra.

However, it is expected that the total cost reduction of construction waste as replacement for quarry run or sand is very limited. Therefore, construction waste will not be applied as building material in the next phase.

3.2.4 Waste products of local industry

The main waste product from the palm oil industry is called palm kernel expeller. This product can be used as animal feed, as a base paper and as fertilizer. This leftover is dumped into the water by local farmers very often, which results into serious water pollution. Since the breakwater will be constructed into the ocean, this alternative material will not be used in order to protect the project location from serious pollution.

3.2.5 Polyethylene

Polyethylene is the most common form of plastic. This material can be used as a construction material instead of wood or steel. Before applying this material, some properties of the polyethylene have to be determined. This material is determined to be a plausible construction material.

3.2.6 Bitumen

Bitumen can be used as a binding agent for smaller materials like sand or construction waste. Applying bitumen will result into a higher maximum permissible shear stress value. Bitumen can be applied as a breakwater construction binding material, if the non-bounded material appears to be too weak.

3.2.7 Bamboo

Bamboo is a fast-growing, relative strong construction material, which is available in Indonesia. In order to achieve more construction strength, the bamboo can be connected together. A disadvantage of bamboo is the irregular form of the bamboo stalks. Altogether, bamboo is assumed to be a plausible and cheap substitute material for steel, wood and concrete.

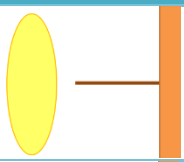
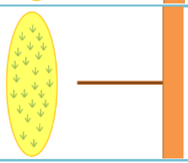
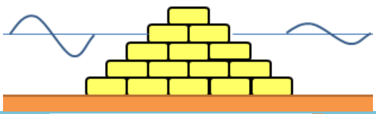
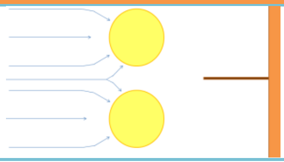
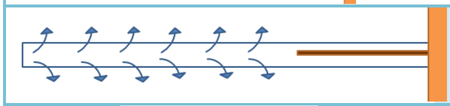
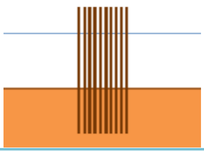
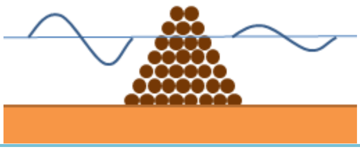
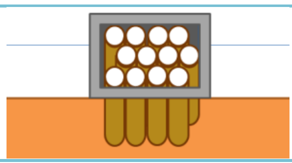
3.2.8 Gel

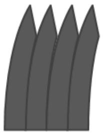
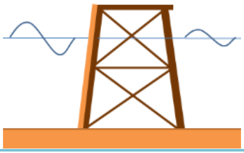
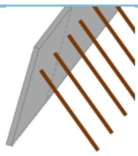
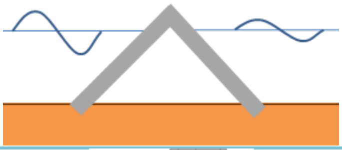
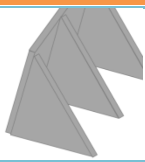
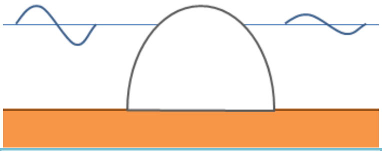
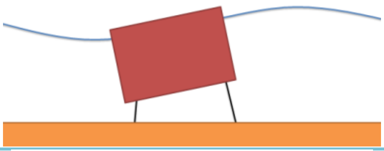
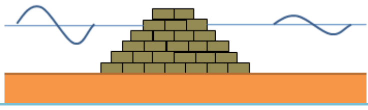
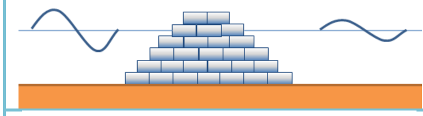
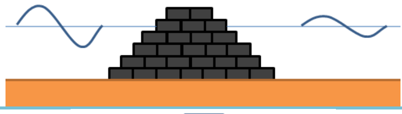
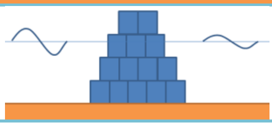
Another innovative material for an alternative breakwater construction is a gel suspension, investigated into the graduation report ‘’gelsluis’’ for the university of Rotterdam. According to the graduation supervisor, this gel cannot be applied as an innovative breakwater construction material, because of decomposition due to environmental impacts. For this reason, the gel breakwater material will not be investigated further.

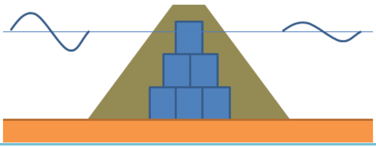
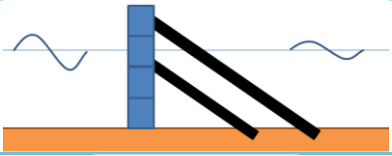
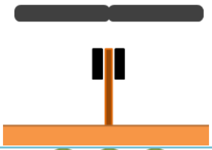
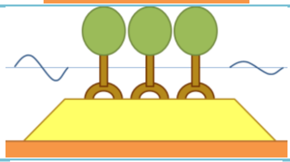
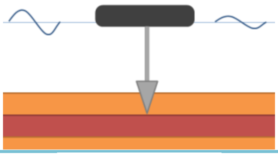
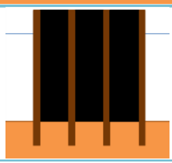
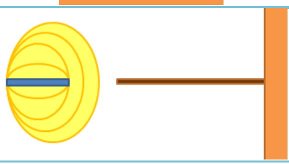
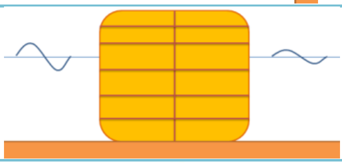
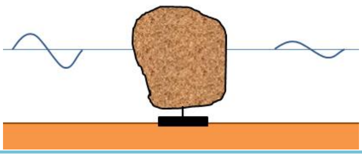
4 SUMMARY

4.1 Breakwater concepts

In table 3, a summary of the breakwater concepts which will proceed to the short list (SL) can be seen.

Pictogram		Alternative	To SL?	Reason if no
	1	Island	NO	Maintenance required
	2	Natural coast	NO	Plantation development time
	3	Geo-containers with sand	YES	
	4	Create refraction with islands	YES	
	5	Create refraction with	NO	Maintenance required
	6	Vertical piles; single row, multiple rows	YES	
	7	Pyramid of piles and/or tubes	YES	
	8	Combined wall	NO	Too expensive
	9	Mikado piles	NO	Behaviour piles unpredictable
	10	Pipe with angle	NO	Construction too complicated

Pictogram	Alternative		To SL?	Reason if no
	11	PVC Pipe with curve	NO	Single pipe failure
	12	Jacket breakwater	YES	
	13	Caisson breakwater with sheet piles	NO	Too expensive
	14a	Slab with support	NO	Combined with 14 b to 14 c
	14b	Hollow breakwater	NO	Combined with 14 a to 14 c
	14c	Slab with support	YES	
	15	Bellows structure	YES	
	16	Floating constructions	NO	Too expensive
	17	Busses/Car wreckages	YES	
	18	Old airplane pieces	NO	Lack of available materials
	19	Old tires	YES	
	20	Container pyramid	YES	

Pictogram		Alternative	To SL?	Reason if no
	21	Core of containers	NO	Taken into account in 34
	22	Container wall with struts	YES	
	23	Ships	YES	
	24	Mangrove	NO	Plantation development time
	25	Lava hole	NO	Uncontrollable
	26	Vertical tubes with band/slab	YES	
	27	Natural sedimentation	NO	Develop time sedimentation
	28	Offshore platform	NO	Lack of available materials
	29	Iceberg without ice	NO	Floating solution
No picture available	30	Removable breakwater	NO	Risks of removing breakwater to late
No picture available	31	Rent-A-Breakwater	NO	Variety in boundary conditions too high

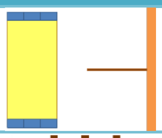
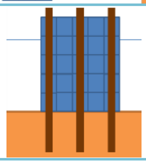
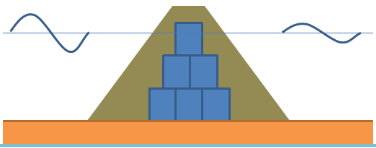
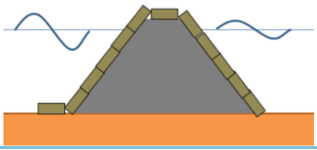
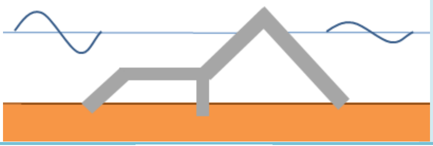
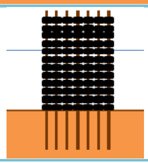
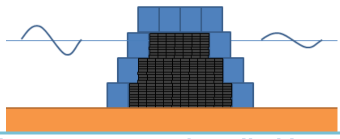
Pictogram	Alternative		To SL?	Reason if no
	32	Soft construction enclosed by hard construction	YES	
	33	Waste products enclosed by piles	YES	
	34	Replacing core with tires, containers or cars	YES	
	35	Replacing armour layer	NO	Savings replacing armour layer not lucrative
	36	Combinations with berm breakwaters	NO	May be an optimisation in further design phase
	37	Tires and piles	YES	
	38	Container armour layer with tire core	NO	Too expensive due to fortifications

Table 3: Summary of applied breakwater concepts

4.2 Construction materials

In table 4, a summary of applicable construction materials can be seen. These material properties will be investigated further during the next phase, if the opportunity occurs of combining these materials with the breakwater alternatives.

Nr.	Par. §	Description	Plausible	Reason if no	Usage
1		Steel	Yes		Construction
2		Wood	Yes		Construction
3		Concrete	Yes		Construction
4		Rock	Yes		Construction
5		Sand	Yes		Fill material
6	3.2.1	Sand, mixed in place	No	Dispersion and high costs	
7	3.2.2	Palm trees	No	Fully used in own industry	
8	3.2.3	Construction waste	No	Very limited cost reductions expected	
9	3.2.4	Waste products of local industry	No	Water pollution	
10	3.2.5	Polyethylene	Yes		Construction
11	3.2.6	Bitumen	Yes		Binding material
12	3.2.7	Bamboo	Yes		Construction
13	3.2.8	Gel	No	Decomposition	

Table 4: Summary breakwater materials

5 CONCLUSION

Table 5 shows the concepts that proceed to the next phase of the project with the materials investigated for the concept.

Nr.	Par. §	Description	Materials	Remarks
3	2.1.3	Geo-containers with sand	Sand, Geotextile	
4	2.1.4	Create refraction with islands	Rock	
6	2.2.1	Vertical piles; single row, multiple rows	Steel, Wood	
7	2.2.2	Pyramid of piles and/or tubes	Steel, Bamboo, Polyethylene, Concrete, Wood	
12	2.3.1	Jacket breakwater	Steel, Bamboo, Concrete, Wood	
14	2.3.3	Slab with support	Concrete	**Combined to one solution (slab with support)**
14	2.3.4	Hollow breakwater	Concrete	
15	2.3.5	Bellows structure	Concrete, Rubber	
17	2.5.1	Busses/Car wreckages	Wreckages	
19	2.5.3	Old tires	Tires	
20	2.6.1	Container pyramid	Containers	
21	2.6.2	Core of containers	Containers	**Is included in solution 34**
22	2.6.3	Container wall with struts	Containers, Steel	
23	2.7	Ships	Old Ships	
26	2.9.2	Vertical tubes with band/slab	Steel, Rubber	
32	2.11.1	Soft construction enclosed by hard construction	Sand, (and Steel, Wood or Concrete)	Named: Sandwich breakwater
33	2.11.2	Waste products enclosed by piles	Steel, Wood, Wreckages, Containers	Named: Enclosed waste
34	2.11.3	Replacing . Tires, Containers and cars	Tires, Containers, Wreckages	
36	2.11.5	Tires and piles	Tires, Steel	
37	2.11.6	Container armour layer with tire core	Containers, Tires	

Table 5: Concepts that proceed to the next phase and materials per concept

6 BIBLIOGRAPHY

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Groenewegen, P. (2015, May 13). (J. van Grieken, Interviewer)
Stive, R. (2015, May 12). Progress meeting. (D. Janssen, & J. van Grieken, Interviewers)

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Appendix 6: Short list
Innovative Breakwater

16 June 2015
Final Report

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Document title	Appendix 6: Short list
Document short title	Short list
Status	Final Report
Date	16 June 2015
Project name	Innovative Breakwater

Drafted by	Jesper van Grieken, Danny Janssen
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Approved by	Harry Dommershuijzen
Date/initials approval	…………………. ………………….

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SUMMARY

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater.

For this reason, financially feasible alternative breakwater concepts have to be developed. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference, a traditional breakwater solution will be developed as well, i.e. the Reference Design.

The purpose of this document is to filter the feasible breakwater solutions, coming from the longlist, to a smaller list of breakwater alternatives which will be put into a MCA afterwards.

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CONTENTS

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1 INTRODUCTION

This document contains a further description of the breakwater alternatives which were determined to be lucrative in the longlist. In this short list, the minimum required breakwater dimensions will be determined; it is assumed that each alternative breakwater concept is technically feasible when applying enough fortifications if necessary. The more fortifications are required the more expensive the higher the breakwater costs. Based on the dimensions of each alternative a construction price of each alternative will be estimated.

The construction costs for each alternative will be compared with the construction costs of the rock armour reference design. If the alternative breakwater turns out to be more less than approximately 30 % cheaper than the reference breakwater (€ 25,000,000.-, a boundary condition set by the contractor) the alternative will not be optimized further and will not be put in the MCA, since the main purpose of the alternative breakwater is to develop a low cost alternative breakwater (see project plan).

To estimate the construction costs of the alternatives, the material costs are multiplied by three factors: The construction factor, the complexity factor and the indirect costs contractor factor. The costs included in each of these factors are described in chapter 3, multiplication factors.

Note: The costs described in this document are rough estimations. The actual cost may vary from -23% to +32%, depending on the alternative (see chapter 3.4, accuracy costs determination). The costs of the alternative that turns out to be the most favourable alternative in the MCA will be determined with more certainty.

A second criterion is the durability of the alternative. If the durability of an alternative breakwater concept is less than 10 years, the breakwater will not be optimized further. This boundary condition is set by the contractor. He assumes that the alternative breakwater concept is not lucrative if the construction lifetime appears to be less than 10 years.

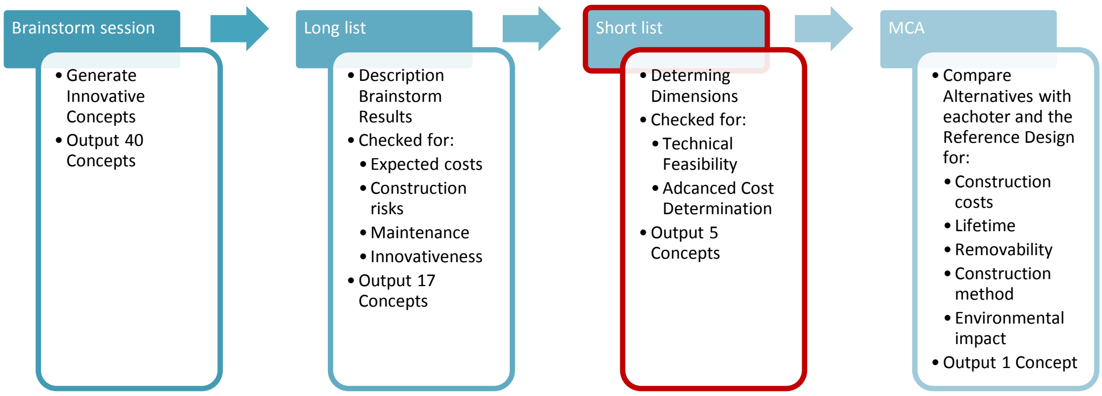


Figure 1: Process for determining "Best Alternative"

2 ALTERNATIVE BREAKWATER CONCEPTS

2.1 Geo-containers or -tubes

2.1.1 Introduction

The geo-container or –tubes breakwater consist of geotextile sandbags which have been filled with sand. The difference between geo-containers and geo-tubes is the size of the bags. The geo-containers are smaller (0.5 – 10 m^3) and more cubical. The geo-tubes are bigger (100 – 750 m^3) and more oblong (Geonet). Because the geo-containers are smaller, more geotextile per cubic meter is needed. Therefore the geo-containers are more expensive than the geo-tubes.

There are two different breakwater layouts of using geo-containers or -tubes in a breakwater, both of these layouts can be performed using geo-containers or tubes:

- A breakwater construction completely constructed of geo-containers or –tubes.
- A core of sand with an armour layer of geo-containers or -tubes.

For each of these four layouts, the material costs will be determined, in order to give a good estimation of the most favourable layout.

2.1.2 Dimensions

Main breakwater dimensions
The maximum water depth is set to 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design). The dimensions of the construction are based on the dimensions of the rock armour layer reference design, which results into a crest height of 5 meters and a crest width of 3.6 meters (see reference design). The total breakwater height will be 15.8 meters (water depth + crest height).

The expected transmission coefficient through the geo-container breakwater is expected to be less than the transmission through the reference design breakwater. It is assumed that the geo-textile and fill have a smaller porosity percentage than the rock of the reference design. For this reason, the transmission coefficient of the geo-container breakwater is expected to be equal to or less than 0.1. This results into a required breakwater length of approximately 600 meters, see the transmission-length diagram in the basis of design.

Dimensions cross section
According to (Geonet), the maximum slope which can be achieved when using geo-containers or –tubes is approximately 1:4, a schematic drawing of the containers or –tubes can be seen in figure 2. This slope is only applicable when constructing the entire breakwater with geo-tubes or containers. If a sand core is applied before placing the geo-containers or –tubes, the slope of the breakwater will be less steep. According to (Geonet), a maximum slope of 1:20 can be performed, when spraying the sand on the

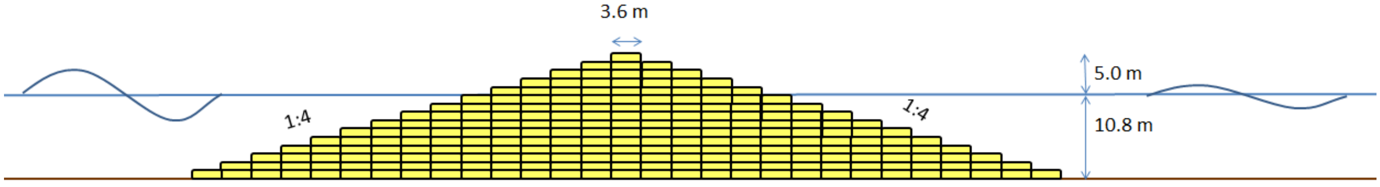


Figure 2: Schematic cross section geo-container/tubes breakwater

surface level using a hopper. This 1:20 slope will be covered with a geo-tube or geo-container armour layer.

Required material

For the entire geo-container and geo-tube, a geo-container/-tube volume of 1071 m^3 per running meter is required, in order to achieve a slope of 1:4 on both sides of the breakwater. Since the required length of the breakwater is approximately 600 meters, a total volume of 642,600 m^3 geo-tubes or -containers is required.

The sand core geo-container/-tubes armour layer consists of two different volumes, the volume of the sand core and the volume of the armour layer. The required volumes can be determined by calculating the entire volume of the breakwater minus the required sand core volume. According to (TenCate, 2007), the diameter of a geo-tube or -container is 3 meters; it is assumed that one layer of geo-tubes or -containers will fulfil the required armour layer thickness. When applying slopes of 1:20, the total volume of the sand is 3329 m^3 per running meter, the total volume of the container is 1711 m^3 per running meter (based on a simple surface calculation). The total required amount of material for a 600 meter breakwater is 1,997,363 m^3 sand and 1,026,637 m^3. In figure 3 a schematic drawing of the sand core breakwater can be seen.

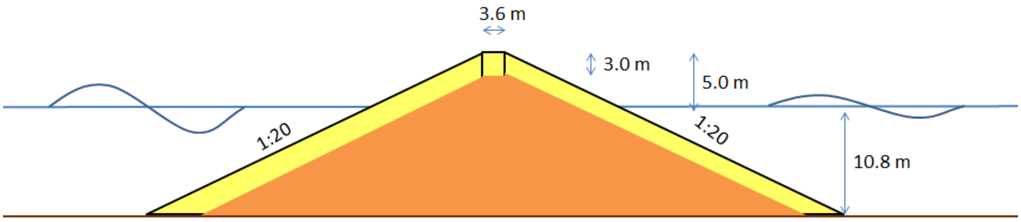


Figure 3: Sand core geo-container/tubes armour layer

2.1.3 Lifetime

The lifetime of geo-containers and –tubes is not yet known, since they are a relative new solution. However, the Road and Hydraulic Engineering Division in the Netherlands has tested some samples of the polypropylene and polyamide geotextiles functioning for 30 years under block revetments on the Dutch sea dikes. The tensile strength had only decreased about 10% and the hydraulic functioning was still satisfactory. In the past couple of years the technology of geo-synthetics has improved considerably. It may be assumed that the lifetime of the geo-containers and –tubes is at least 40 years. (Pilarczyk, 2000).

2.1.4 Material costs

Unit prices
The unit prices of the materials used in the material costs calculation can be seen in table 1. The costs per square meter geo-container are higher than the costs of the geo-tubes. This is caused by the difference in size of the materials. Per square meter geo-container, more geotextile is required, which is more expensive than the sand fill.

Material	Unit	Unit price	Source
Geo-tubes	€/m^3	26.16	(Fowler & Trainer, 1997)
Geo-container	€/m^3	29.43	(Fowler & Trainer, 1997)
Sand	€/m^3	5.00	(Stive, 2015)

Table 1: Unit prices geo-tube/container breakwater

Total material costs
In table 2 the material costs of the four different breakwater layouts can be seen. The most lucrative breakwater is the breakwater which is completely performed in geo-tubes. Due to this costs argument, only the geo-tube breakwater configuration will be taken into account in the next chapter (multiplication factor).

	Geo-tube	Geo-container	Geo-tube+ Sand	Geo-container+ Sand
Geo-tube (m^3)	642,600		1,026,637	
Geo-containers (m^3)		642,600		1,026,637
Sand (m^3)			1,997,362	1,997,362
Unit price				
Geo-tube (€/m^3)	€ 26.16	€ 26.16	€ 26.16	€ 26.16
Geo-containers (€/m^3)	€ 29.43	€ 29.43	€ 29.43	€ 29.43
Sand (€/m^3)	€ 5.00	€ 5.00	€ 5.00	€ 5.00
Material costs				
	€ 16,810,416	€ 18,911,718	€ 36,843,642	€ 40,200,745

Table 2: Material costs per breakwater configuration

2.1.5 Construction risks

The most critical construction risk of the geo-containers/tubes is to achieve the required sand density in the geo-tubes. If this density will not be achieved the construction is assumed to be not stable due to possible settlements. A second construction risks is the placement accuracy of the geo-tubes or –containers especially when the water depth increases, (Allsop, 1998). The most common placing method for the geo-tubes is dropping them out of a split barge, due to currents caused by waves and tides, the geo-tubes may be placed not accurate.

2.2 Refraction islands

2.2.1 Introduction

The refraction islands solution consists of a couple of islands which are constructed using the same rock layer build up as the rock armour layer reference design.
Huge slopes are required if the islands only consists of sand, see chapter sandwich breakwater, thereby the heads of the islands may erode due to environmental impacts, which will lasts in maintenance, which is not preferred. For this reason, only islands consisting of a rubble mount protection will be applied in this chapter. It is assumed that the waves will refract towards the islands caused by the decreasing water depth. The refraction will be created by applying a slope between the island gaps. In this chapter, the relation between the island gaps and the island diameter will be determined, whereupon the savings of the breakwater will be determined.

2.2.2 Dimensions

Main breakwater dimensions
The main dimensions of the refraction islands are set equal to the dimensions of the rock armour layer reference design, since the extreme environmental storm boundary conditions for both of the breakwater alternatives are equal. The extreme water depth at the breakwater location is 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design), the required crest height 5 meters and the crest width is 3.6 meters (see reference design). The front slope of the islands will be 1:3 and the rear slope 1:1.5 (same as the rock armour layer reference design).

Dimensions gaps
The relationship between the space between two islands (A) and the length of the island including the gap (B), compared to the transmission coefficient is assumed to be:

$$C_t = \frac{A}{B}$$

this formulae is based on the transmission coefficient through a vertical pile breakwater (see figure 4) it is assumed that this rule of thumb can be applied, due to the reflecting effect of the islands, if this reflecting effect will not occur, the transmission through the gaps may be higher (see chapter 2.3, vertical piles). Besides this transmission through the island gaps, a transmission through the rock islands will occur. The occurring transmission coefficient is expected to be equal to the transmission coefficient of the reference design, because the islands consist of the same material as the rubble mound breakwater. This means the transmission coefficient of the islands is: $C_t = 0.2$.

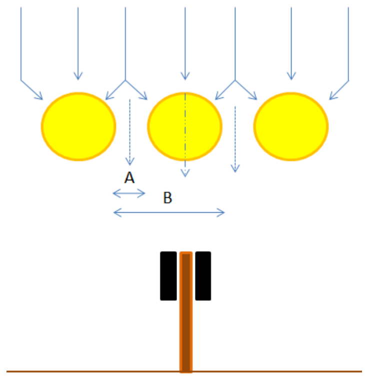


Figure 4: Transmission through islands

The maximum allowed transmission + overtopping coefficient at the lee side of the breakwater is 0.3. If the transmission coefficient becomes higher than 0.3, the operational limits cannot be guaranteed, even if the breakwater length is extended (see diffraction and transmission diagram in the basis of design).

The total number of gaps has to be an equal number, if the number of gaps is not equal, the gap between the islands will be exactly in front of the jetty. There is a risk of waves which will not refract towards the islands and will head straight forward to the jetty. In order to prevent this phenomenon, the middle refraction island will be situated in front of the jetty.

Based on the energy of the waves, the relation between the transmission coefficients of the islands and the gap is equal to:

$$E_{t,tot} = E_{t,trans} + E_{t,gap}$$

$$\frac{1}{8} \times \rho_{wat} \times g \times H_{t,tot}{}^{2} = \frac{1}{8} \times \rho_{wat} \times g \times H_{t,trans}{}^{2} + \frac{1}{8} \times \rho_{wat} \times g \times H_{t,trans}{}^{2}$$

$$(H_s \times C_{t,tot})^2 = (H_s \times C_{t,trans})^2 + (H_s \times C_{t,gap})^2$$

$$C_{t,tot} = \sqrt{C_{t,tran}{}^{2} + C_{t,gap}{}^{2}}$$

Where:

E	Wave energy	
H_t	Wave height caused by transmission	
H_s	Significant incoming wave height	
$C_{t,tot}$	Total transmission coefficient	Max 0.3
$C_{t,tran}$	Transmission coefficient through islands	0.2
$C_{t,gap}$	Transmission coefficient through gaps	Variable

The maximum allowed transmission coefficient through the breakwater gaps is equal to 0.22. In order to define the most lucrative gap dimensions, each gap transmission coefficient between 0.1 and 0.22 will be taken into account.

In figure 5 and 6, the greatest possible gap can be seen, a gap which is completely from the crest to the subsoil. The slope between the gaps is set to 1:3, which is equal to the front slope of the breakwater. This slope is determined to be stable in the reference design. The maximal gap between two islands is set to 94.8 meters. This length is required in order to achieve a 1:3 slope with a breakwater height of 15.8 meters.

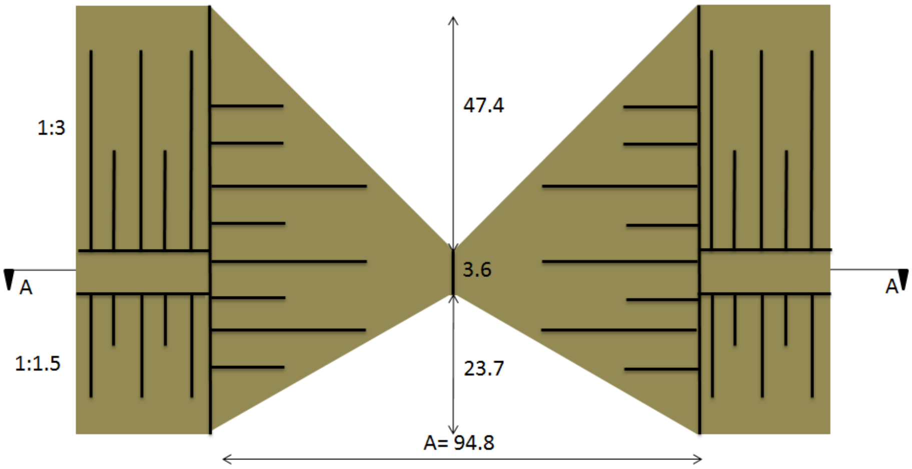


Figure 5: Top view gap

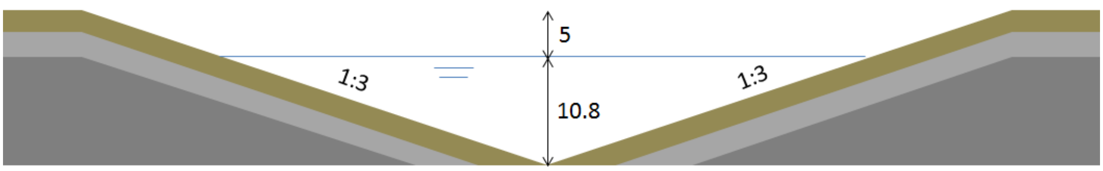


Figure 6: Cross section A-A gap

Material savings

When calculating the material savings for each variable in the transmission coefficient (C_t), the following savings per gap will be achieved. The C_t is set as a variable in this calculation. The required number of gaps is set to 2. The savings per gap are determined using a simple surface calculation, based on figure 5 and 6.

C_t gap	B (m)	A (m)	Savings gaps (m^3)	Nr. gaps	C_t tot	Extra bw. length (m)
0.01	Na	Na	Na	Na	0.21	0
0.02	Na	Na	Na	Na	0.21	0
0.03	Na	Na	Na	Na	0.21	0
0.04	Na	Na	Na	Na	0.21	0
0.05	Na	Na	Na	Na	0.21	0
0.06	302	18	1,679	2	0.21	0
0.07	303	21	2,638	2	0.22	3
0.08	303	24	3,867	2	0.22	3
0.09	303	27	5,428	2	0.22	3
0.1	304	30	7,436	2	0.23	5
0.11	305	34	9,816	2	0.23	5
0.12	305	37	12,635	2	0.24	5
0.13	305	40	15,953	2	0.24	5
0.14	306	43	20,093	2	0.25	8
0.15	306	46	24,587	2	0.25	8
0.16	318	51	33,306	2	0.26	32
0.17	334	57	45,962	2	0.27	64
0.18	334	60	54,410	2	0.27	64
0.19	357	68	77,752	2	0.28	111
0.2	393	79	119,973	2	0.29	182
0.21	393	83	138,596	2	0.29	182
0.22	431	95	208,649	2	0.3	300

Table 3: Material savings and additions per C_t coefficient

Length extension

The more transmission through the gaps is accepted, the higher the transmission coefficient, the more breakwater length is required. The length of the reference design breakwater is determined to be 604 meters. Due to the extra transmission trough the breakwater gap, a lower tolerance of diffraction is allowed, this will last into an extra breakwater length (see transmission-length diagram in the basis of design).

2.2.3 Lifetime

The lifetime of the refraction islands is equal to the lifetime of the rock armour layer reference design. The minimal lifetime of the refraction islands is 50 years. For a further determination of the lifetime of the breakwater, see the reference design.

2.2.4 Material costs

Unit prices
For this cost price calculation it is assumed that the entire breakwater savings will be of quarry run, since the top two layers are still required in the gaps its slope.

Material	Unit	Unit price	Source
Quarry run	€/m³	€ 40.-	(Stive, 2015)
Breakwater	€/m¹	€ 31,855.-	Reference design

Table 4: Unit price refraction islands breakwater

Total material costs
When taking the total savings of quarry run (qr), minus the extra breakwater length costs, the total savings can be established.

Ct gap	Savings qr (m^3)	Savings qr (€)	Extra bw length (m)	Extra bw length (€)	Savings
0.01	Na	Na	0	€ 0	Na
0.02	Na	Na	0	€ 0	Na
0.03	Na	Na	0	€ 0	Na
0.04	Na	Na	0	€ 0	Na
0.05	Na	Na	0	€ 0	Na
0.06	1,679	€ 67,165	0	€ 0	€ 67,165
0.07	2,638	€ 105,501	3	€ 0	€ 22,677
0.08	3,867	€ 154,684	3	€ 82,824	€ 71,860
0.09	5,428	€ 217,119	3	€ 82,824	€ 134,295
0.1	7,436	€ 297,431	5	€ 82,824	€ 138,154
0.11	9,816	€ 392,634	5	€ 159,277	€ 233,356
0.12	12,635	€ 505,390	5	€ 159,277	€ 346,113
0.13	15,953	€ 638,140	5	€ 159,277	€ 478,863
0.14	20,093	€ 803,715	8	€ 159,277	€ 555,243
0.15	24,587	€ 983,465	8	€ 248,472	€ 734,993
0.16	33,306	€ 1,332,237	32	€ 248,472	€ 306,492
0.17	45,962	€ 1,838,482	64	€ 1,025,744	€ -206,635
0.18	54,410	€ 2,176,419	64	€ 2,045,118	€ 131,301
0.19	77,752	€ 3,110,081	111	€ 2,045,118	€ -413,128
0.2	119,973	€ 4,798,905	182	€ 3,523,209	€ -1,005,152
0.21	138,596	€ 5,543,836	182	€ 5,804,057	€ -260,220
0.22	208,649	€ 8,345,964	300	€ 5,804,057	€ -1,197,918

Table 5: Total savings per C_t coefficient

The total material costs of the reference design breakwater are calculated to be € 19,431,803.- (see reference design). When the material costs of the reference breakwater are reduced by the material savings caused by the breakwater gaps, the total material of the refraction islands breakwater is € 18,696,809.-.

Total savings		
Reference design breakwater costs	€	19,431,803.-
Savings	€	734,993.-
Breakwater costs		-
Total costs	€	18,696,809.-

Table 6: Material costs refraction islands

2.2.5 Complexities

The current breakwater has been calculated using a very positive starting point. The first positive assumption is the required amount of armour and filter layer, when constructing these layers in a slope instead of straight forward, more rocks are required per running meter. The second positive starting point is the slopes between the gaps. These slopes may be less steep or may have to be performed rounded, in order to make these slopes comparable to a breakwater head. Thereby, a numerical model has to be made, in order to define the wave behaviour at the lee side of the breakwater.

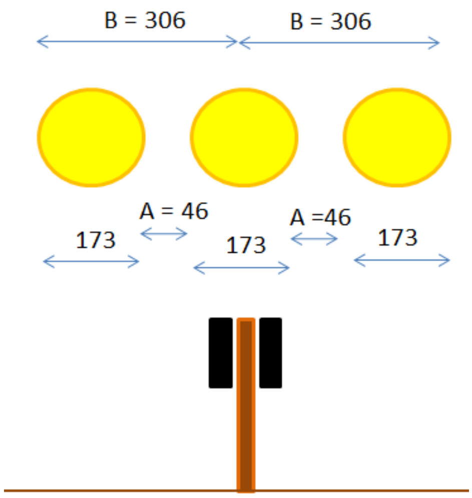


Figure 7: Dimensions islands and gaps

2.3 Vertical piles

2.3.1 Introduction

The vertical piles breakwater concept consists of one or multiple pile rows constructed of vertical piles. The vertical pile row can be constructed using steel, wood, concrete or bamboo. In this chapter the dimensions for steel and wooden piles are determined. The materials bamboo and concrete will not be taken into account in this chapter. It is assumed that both of the materials are not able to withstand the occurring horizontal wave load forces without huge fortifications and therefore will become too expensive when applied.

2.3.2 Dimensions

Main breakwater dimensions

The required height, from the top of the subsoil to the top of the construction is set to 10.8 m (CD+ HHWS + surge + sea level rise, see basis of design)+ 3.4 m (required crest height according to the vertical wall overtopping calculation, mentioned in appendix II) = 14.2 meters. The breakwater length is dependent of the transmission though the breakwater, the higher the transmission coefficient, the more breakwater length is required (see transmission/length diagram in the basis of design).

Dimensions cross section

The required pile dimensions have been determined in appendix II: schematic calculation vertical piles. This calculation is based on the calculation mentioned in the Manual for structural hydraulic engineering (Dr. S. van Baars, 2003) and the shore protection manual (CERC, 1984). In figure 8 the required dimensions of the wooden and steel piles can be seen. The mass of a single steel pile is 12 ton and the volume of a single wooden pile is 2.8 m^3, based on some simple surface calculations.

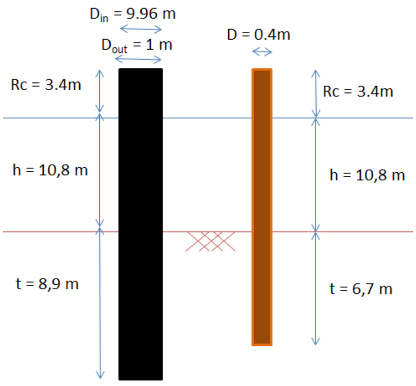


Figure 8: Length of vertical piles

Required vertical piles

As a rule of thumb, the transmission coefficient through the vertical pile breakwater can be determined using the following equation:

$$C_t = \frac{A}{B}$$

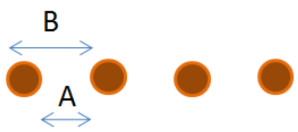


Figure 9: Relation A and B

Where A is the spacing between two piles and B is the diameter of the pile plus the spacing between two piles, see figure 9. However, though the effect of diffraction around the vertical piles, the C_t value may be higher than mentioned in the rule of thumb. According to (Clifford & John, 1986), the actual transmission though the piles breakwater may be 3 times higher than determined using the rule of thumb, this conclusion is based on the results of a practical test. For this reason, it is assumed that a single vertical pile row is not able to guarantee the operational wave conditions, if the piles are constructed with spacing. The piles have to be constructed almost against each other in order to achieve the operational limits. If the spacing between two piles becomes too small, it is assumed that the piles cannot be constructed properly, due to the placing tolerances which are required in order to construct this vertical pile breakwater.

When increasing the number of vertical pile rows with 100%, the wave transmission will averagely be decreased with 18%, according to (Weele, 1965). This percentage in transmission reduction is based on practical tests.

The total required distance A and B can be determined using the above mentioned reduction and multiplication factors. When calculating the required $C_t = \frac{A}{B}$, for both the wooden and steel vertical piles, with a transmission coefficient from 0.01 to 0.3, the relationship between the pile diameter and the pile spacing will be determined.

Summarized: the total required pile row length can be determined by multiplying the C_t factor, used for calculating the relationship between the pile diameter and the spacing, with 3 (Clifford & John, 1986). This applied C_t factor has to be decreased with 18% for each 100% of added pile rows (Weele, 1965).

According to a feasibility study to a vertical pile breakwater for Maasvlakte II (P.J. Eversdijk, 1997), a minimum of 4 pile rows is required, in order to achieve a certain transmission factor, even if the pile rows are constructed without any spacing, for this reason 4 pile rows will be applied. The four pile row breakwater will increase the number of pile rows with 100% two times, so a reduction on the required transmission reduction is set to 2 times 18% (Weele, 1965).

The transmission factor, which results after these multiplications, will determine the required breakwater length, based on the transmission-length diagram, mentioned in the basis of design.

In appendix II, a table with the variation between the transmission coefficients between 0.01 and 0.3 is to be seen. The principle of this calculation is based on a 4 row pile breakwater. A summary of the table mentioned in the appendix can be in table 7, this table contain the most advantageous pile row dimensions.

	A (m)	B (m)	L required (m)	Piles required
Wood	0.05	0.45	609	1,350
Steel	0.14	1.14	609	540

Table 7: Required piles per pile row

2.3.3 Lifetime

The lifetime of the steel piles is depending on the thickness of the walls and the corrosion rate of the steel. According to (British Standard, 2000), the maximum corrosion rate of steel is 0.17 mm/side/year. To guarantee a certain lifetime, the wall thickness of the steel piles has to be increased. For the steel piles the wall thickness has to be increased with: 0.17*2*50= 17mm.

The wood in the vertical pile breakwater is Merbau wood. Merbau wood has a lifetime up to 10 to 25 years according to (Matbase).

2.3.4 Material costs

Unit prices
The steel price is based on the unit price of steel of March 2015. The wood price is assumed after consultation with experts from RHDHV.

Material	Unit	Unit price	Source
Wood	€/m^3	€ 950.-	(Stive, 2015)
Steel	€/ton	€ 1,100.-	(WSDOT, 2015)

Table 8: Unit price vertical piles

Total material costs
When multiplying the required amount of material with the unit prices of the different materials, the total material costs of the wooden vertical pile breakwater appears to be the most lucrative vertical pile breakwater material, for this reason, only the wooden vertical pile breakwater will be taken into account in chapter 3, multiplication factors.

Single piece	Steel	Wood
Weight (ton)	12	
Volume (m^3)		3
Piles required (row)	540	1350
Required rows	4	4
Required amount	25935	15268
Material costs	Steel	Wood
Unit price	€ 1.100	€ 950
Material costs	€ 28.529.029	€ 14.504.733

Table 9: Material costs vertical piles

2.3.5 Complexities

The most complex part of constructing the wooden vertical pile breakwater is the spacing between two vertical piles; this spacing is only 50 centimetres. If a single pile will be constructed under a small angle the piles may hit each other. This risk is taken into account by determining the multiplication factors, see chapter 3.3.3, multiplications for the vertical piles.

2.4 Pyramid of piles

2.4.1 Introduction

The pyramid of piles breakwater consists of horizontal positioned tubes stacked on top of each other in order to create a pile pyramid. The tubes or piles used in the pyramid of piles breakwater may be massive or hollow, depending on the construction material of the piles or tubes. The different materials investigated for the pyramid of piles breakwater are:

- Steel
- High-density polyethylene (HDPE)
- PVC
- Wood (Merbau)
- Concrete
- Bamboo

2.4.2 Dimensions

Main breakwater dimensions

The maximum water depth is set to 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design). The required crest height and width of the construction are based on the dimensions of the rock armour layer reference design; a crest height of 5 meters and a minimum crest width of approximately 3.6 meters (see reference design). The total breakwater height will be 15.8 meters (water depth + crest height).

The expected transmission coefficient through the pyramid of piles breakwater is assumed to be very low. The piles or tubes itself are not water-permeable, therefore the waves transmitted through the breakwater will have to flow through the small clearances between the piles or tubes, if there are any clearances at all. Therefore, the transmission coefficient of the pyramid of piles breakwater is expected to be maximum 0.1. A maximum transmission coefficient of 0.1 results in a required breakwater length of approximately 600 meters, according to the transmission-length diagram in the basis of design.

Dimensions cross section

The dimensions of the cross section of the pyramid of piles breakwater are depending on the dimensions of the used piles or tubes. The minimum required dimensions of the pyramid of piles breakwater are shown in figure 10. The slope of the structure has to be as steep as possible, to reduce the required amount of piles or tubes. This means a slope of √(3):1 is applied. See figure 11.

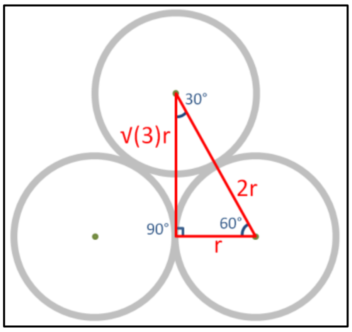


Figure 11: Determining the slope of the breakwater

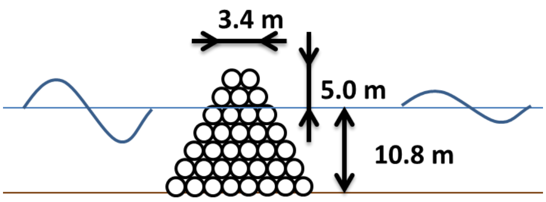


Figure 10: The minimum required dimensions of the pyramid of piles breakwater

Required material

As mentioned in the introduction of the pyramid of piles breakwater, the materials of which the piles or tubes are made are: Steel, HDPE, PVC, Wood, Bamboo or Concrete. The buoyancy of these materials is calculated to check of the materials will sink or will float. It is desired that the tubes or piles sink so no extra weight has to be added to increase the gravitational force of the construction to make the pyramid stable. Table 10 shows the buoyancy of 1 m^3 of material determined using Archimedes' principle.

Materials:			Steel	HDPE	PVC		Wood (Merbau)		Concrete	Bamboo	
Buoyancy force	F_{buo}	N	9975,6	9975,6	9975,6	9975,6	9975,6	9975,6	9975,6	9975,6	9975,6
Gravitational force	F_g	N	76284	9487	12714	14181	6846	8802	23472	2934	3912
Resulting force	F_{res}	N	66308	-489	2738	4205	-3130	-1174	13496	-7042	-6064
Does it sink?			Yes	NO	Yes	Yes	NO	NO	Yes	NO	NO

Table 10: Buoyancy of 1 m3 of (massive) material

As can be seen in table 10, the materials steel, PVC and concrete can be used without applying extra weight. The materials that will float can also be used when extra weight or some sort of anchor is applied. The application of extra weight is very expensive, because a lot of extra weight is needed to reach the required level of gravitational force for the construction. The required gravitational force for a stable breakwater construction with the form of the pyramid of piles breakwater is high, comparable with a traditional rubble mound breakwater, since the rubble mount construction achieves its stability from gravitational forces. Therefore, the solutions that will float will not be taken into account any further.

The resulting gravitational force of 1 m^3 of PVC is small in comparison to the resulting gravitational force of 1 m^3 of steel or concrete. The resulting force of 1 m^3 of steel is almost 16 times as big and the resulting force of 1 m^3 of concrete is more than 3 times as big. The resulting gravitational forces of the materials when applied in tubes are depending on the tube diameter and the wall thickness of the tubes. Steel and PVC are applied in tubes with relative small wall thicknesses in comparison to the wall thickness of concrete tubes. Thereby, the diameters of the concrete and steel tubes can in general be higher than the diameter of PVC tubes. Therefore, the resulting gravitational force of the concrete or steel tubes will in general be way higher than the resulting gravitational force of PVC tubes. However the resulting gravitational force of 1m^3 of steel is almost 5 times as high as the resulting gravitational force of 1 m^3 of concrete, the gravitational force of the steel tubes is expected to be in the same range as the gravitational force of the concrete tubes, due to the thinner walls of the steel tubes in comparison to the concrete tubes.

It is expected that extra weight has to be added to the PVC tubes due to the lower resulting gravitational force. Thereby, more tubes of PVC are needed to create the required pyramid due to the smaller diameter. Since PVC a more expensive material than concrete and steel, when comparing the unit price provided by several PVC suppliers with the unit prices of concrete and steel, and concrete and steel can better withstand the outdoor environment than PVC, the material PVC will not be taken into account any further.

The materials left for the pyramid of piles breakwater concept are: steel and concrete. For these materials, the largest most common diameters are applied, because: the bigger the diameter, less tubes are needed, the less material is needed, the lower the costs are.

The used steel tubes are 3000 mm in diameter. This is the largest common used diameter for steel tubes according to (voorbij funderingstechniek, 2014), these tubes are often used as foundation material. Since the steel tubes are not used as foundation piles, the thickness of the walls of the tubes can be limited. The thickness of the steel piles is determined by the corrosion rate of the steel tubes due to the salt water. The maximum erosion of steel in the splash zone is 0.17 mm/side/year according to (British Standard, 2000). The required thickness for the walls of the tubes can be estimated for a certain lifetime (25 years in this case): 25*0.17*2=8.5mm. However, with a wall thickness of 8.5 mm, the walls will be completely eroded after 25 years. Therefore, the wall thickness is assumed to be 15 mm.

The used concrete tubes are just over 3600 mm in diameter. This is the largest common used diameter for pre-fabricated concrete tubes according to (Con Cast Pipe, 2014), these tubes are often used as sewer or drainage pipes. The thickness of the walls of the concrete tubes is approximately 280 mm according to the same reference.

With these tubes, the required amount of tubes for the breakwater cross section can be determined. The vertical centre to centre distance between two layers is equal to √3 times the radius of the tube (see figure 11). So the amount of vertical layers required can be calculated with:

$$\text{nr of vertical layers} = \frac{h_{\text{required}} - 2 * \text{radius}}{\sqrt{3} * \text{radius}} + 1$$

The required amount of tubes horizontal next to each other on the top layer can be calculated by dividing the required crest width by the tube diameter. Every layer downwards has one more tube than the layer above. The total amount of required tubes per cross section can be determined.

The steel tubes pyramid consists of 27 tubes and the concrete tubes pyramid consists of 15 tubes. Figures 12 and 13 are cross sections of the pyramid of piles.

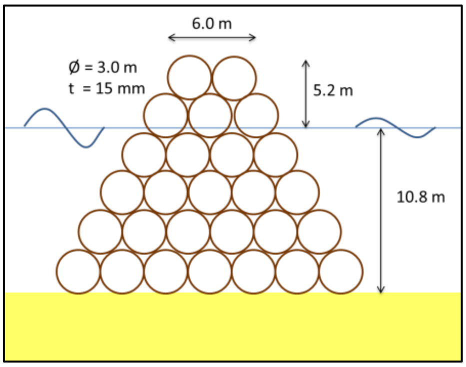


Figure 12: Cross section of the steel tubes pyramid breakwater

The steel tubes have a usage of material of 0.141 m^3/m and the concrete tubes have a usage of material of 3.110 m^3/m. This leads to the total material usage per m breakwater as shown in table 11.

	Total usage of material per meter breakwater
Steel tubes	3.8 m^3 / m
Concrete tubes	46.7 m^3 / m

Table 11: Total usage of material per meter breakwater

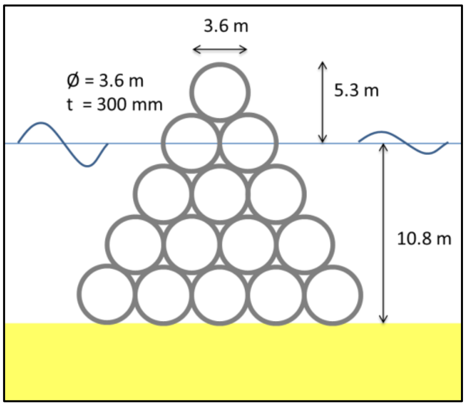


Figure 13: Cross section of the concrete tubes pyramid breakwater

2.4.3 Lifetime

The lifetime of the steel tubes is expected to be at least 25 years, see required material in chapter 2.4.2. If the concrete tubes are constructed properly, there is enough concrete cover for the reinforcements and the tubes are handled with care during construction, the lifetime of the concrete tubes is expected to be at least 50 years.

2.4.4 Material costs

Unit prices

The unit prices of the materials used in the material costs calculation can be seen in table 12.

Material	Unit	Unit price	Source
Steel	€/ton	1,100	(Washington State Department of Transportation, 2015)
Concrete	€/m^3	600	(Stive, 2015)

Table 12: Unit prices pyramid with piles breakwater

Total material costs

In table 13 the total material costs of the two pyramid of piles breakwaters can be seen. The most lucrative breakwater is the concrete pyramid of piles breakwater. Therefore only the concrete pyramid of piles breakwater will be taken into account in the next chapter (multiplication factor).

	Steel pyramid of piles	Concrete pyramid of piles
Steel tubes (m^3/m)	3.8	-
Concrete tubes (m^3/m)	-	46.7
Length required (m)	600	600
Steel (m^3)	2,280	-
Steel (ton)	17,784	-
Concrete (m^3)	-	28,020
Unit price		
Steel (€/ton)	€ 1,100	€ 1,100
Concrete (€/m^3)	€ 600	€ 600
Material costs		
	€ 19,551,846.-	€ 16,794,954.-

Table 13: Material costs for the steel pyramid of piles and the concrete pyramid of piles

2.4.5 Complexities

The biggest construction risk of the pyramid of piles is the stability of the individual tubes. Due to the wave impact the waves, it is expected that the tubes can start rocking. The rocking of the tubes will lead into failure. This phenomenon can be prevented by connecting the tubes during construction. However, making the connection between the tubes will introduce additional construction costs.

A second construction risk is the stability of the construction as a whole. If the construction does not have enough gravitational force, the construction may start to slide towards the jetty or may turn over. This is not desired.

2.5 Jacket breakwater

2.5.1 Introduction

When constructing a breakwater based on the shape of an offshore jacket, the breakwater is able to withstand relatively big forces. If the ocean facing side of the jacket is closed, almost no wave transmission will occur through the breakwater. The jacket construction can be performed using different kinds of materials, like bamboo, wood, steel and concrete. The materials considered in this study are: wood (Merbau) and steel. Concrete will not be taken into account in this chapter, since a concrete, almost similar, solution will be described in chapter 2.6, slab with support. Bamboo will not be investigated because the material floats (see chapter 2.4, pyramid of piles); the material can be applied in the jacket breakwater but the weight of the construction has to be increased drastically in order to make the construction stable. Due to the extra weight required, the total costs of the construction are expected to be higher than the reference design.

2.5.2 Dimensions

Main breakwater dimensions

It is desirable to reduce the overtopping as much as possible. However, the calculation formulas can only be applied up to an overtopping discharge of 1.00E-02 m^3/s/m. It is assumed this specific overtopping discharge has little to none influence on the wave height on the lee side of the breakwater. The required crest height for the jacket breakwater can be determined with a maximum specific overtopping discharge of 1.00E-02 (for the service limit state). The crest height depends on the angle of the ocean side panel of the jacket breakwater. Table 14 shows the required crest height, according to appendix III.

	Angle	Required Crest Height (in m)
Vertical	NA	3.40
Battered	10:1	3.70
Battered	5:1	4.20

Table 14: Required crest height for a specific overtopping discharge of 1.00E-02 m^3/s/m

With a lower crest height, fewer materials are needed and so the costs can be reduced. Therefore the chosen angle of the ocean side panel is vertical.

It is expected that the front wall of the jacket breakwater designs can be built in such a way that the resulting transmission coefficient of the total construction is equal to or less than 0.1.This results in a required length of approximately 600 meter, see the transmission-length diagram in the basis of design.

Dimensions cross section
The dimensions of a cross section of the jacket construction are shown in figure 14.

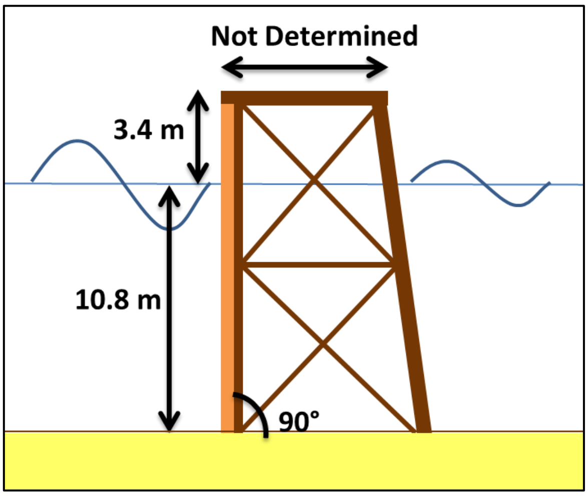


Figure 14: Dimensions of the jacket breakwater

Required material

Steel jacket breakwater
The ocean wall of the steel jacket breakwater is exposed to corrosion due to the salt water. According to (British Standard, 2000), the maximum corrosion of steel is 0.17 mm/side/year. This means that when the lifetime of the construction has to be 50 years, the ocean wall must have a minimum thickness of 0.17*2*50=17 mm on top of the required thickness of the front wall due to the wave impact forces. The thickness of the front wall needed due to the wave impact forces is assumed to be at least 8 mm. Therefore, the total thickness of the ocean wall is set tot 17+8=25 mm. The required height of the front wall is determined by the maximum water depth (HHWS+SLR+Surge, 10.8 meters) and the required crest height. This means that a minimum height of 10.8+3.4=14.2 meters is required. The required amount of steel for the front wall can be calculated: 14.2*0.025= 0.355 m^3 per meter breakwater.

The amount of steel required for the supporting structure (the jacket itself) is based on a jetty construction designed by Royal HaskoningDHV for the project Sohar Bulk Jetty in Oman. The total amount of steel required for the jetty construction was approximately 0.52 m^3 per running meter jetty. However, the wave conditions for the Sohar Bulk Jetty were less severe and the forces on a jetty construction are lower than the forces expected on the jacket breakwater construction. Therefore it is assumed that the total amount of steel required for the supporting structure of the jacket breakwater is approximately twice the total amount of steel required for the jetty construction in Oman.

Table 15 shows a summary of the required material for the steel jacket breakwater.

	Amount of steel required per meter breakwater (m^3/m)	Total amount of steel required (m^3)
Ocean wall	0.355	213
Supporting construction	1.04	624
Total	1.395	837

Table 15: Required amount of steel for the steel jacket breakwater

Wooden jacket breakwater

The thickness of the ocean facing side of the wooden jacket breakwater is determined to be 0.3 m. The required height of the front wall is determined by the maximum water depth (HHWS+SLR+Surge, 10.8 meters) and the required crest height. This means that a minimum height of 10.8+3.4=14.2 meters is required. The required amount of wood for the front wall can be calculated: 14.2*0.3= 4.26 m^3 per meter breakwater.

The wall/support ratio is assumed to be equal to the wall/support ratio of the steel jacket construction $\left(\frac{1.04}{0.355} = 2.93\right)$. This results into a support structure material usage of $2.93 * 4.26 = 12.48$ m^3/m.

Table 16 shows a summary of the required material for the steel jacket breakwater.

	Amount of wood required per meter breakwater (m^3/m)	Total amount of wood required (m^3)
Ocean wall	4.26	2,556
Supporting construction	12.48	7,488
Total	16.74	10,044

Table 16: Required amount of wood for the wooden jacket breakwater

Foundation of the jacket breakwater

A foundation is needed to guarantee the stability of the construction. It is assumed that both the steel and the wooden jacket breakwater construction require the same foundation. The costs of the foundation are assumed based on (Dorreman, Balgstuwen gevuld met lucht en/of water, 1997).

2.5.3 Lifetime

The steel jacket breakwater is designed for a lifetime of 50 years, see chapter 2.5.2. The wood in the wooden jacket breakwater has a lifetime up to 10 to 25 years according to (Matbase).

2.5.4 Material costs

Unit prices

The unit prices of the materials used in the material costs calculation can be seen in table 17.

Material	Unit	Unit price	Source
Steel	€/ton	1,100	(Washington State Department of Transportation, 2015)
Wood (Merbau)	€/m^3	950	(Stive, 2015)
Foundation	€/m	23,000	(Dorreman, Balgstuwen gevuld met lucht en/of water, 1997)

Table 17: Unit prices pyramid with piles breakwater

Total material costs

In table 18 the total material costs of the two jacket breakwaters can be seen. The most lucrative breakwater is the steel jacket breakwater. Therefore only the steel jacket breakwater will be taken into account in the next chapter (multiplication factor).

	Steel jacket breakwater	Wooden jacket breakwater
Steel (m^3)	837	-
Steel (ton)	6,528.6	-
Wood (m^3)	-	10,044
Foundation (m)	600	600
Unit price		
Steel (€/ton)	€ 1,100	€ 1,100
Wood (€/m^3)	€ 950	€ 950
Foundation (€/m)	€ 23,000	€ 23,000
Material costs		
	€ 20,981,460.-	€ 23,341,800.-

Table 18: Material costs for the steel pyramid of piles and the concrete pyramid of piles

2.5.5 Complexities

The width of the construction is not determined yet. The material usage of the supporting structure is based on a jetty construction designed by RHDHV for the project Sohar Bulk Jetty in Oman. When the construction is calculated in detail it might be possible that the assumed amount of material needed for the supporting structure turns out to be not enough. In that case the total material costs of the construction will increase. However, it is expected that the assumed amount of material needed for the supporting structure is enough to create a strong enough support structure.

2.6 Slab with support

2.6.1 Introduction

The slab of support breakwater consists of prefabricated concrete elements with a backfill of filled containers or geo-bags. The purpose of the slab with support breakwater is to reduce the required materials and reduce the construction time and therefore reduce the total costs of the breakwater construction. The same principia as the jacket breakwater applies to the slab with support breakwater when the front wall of the construction is positioned vertical.

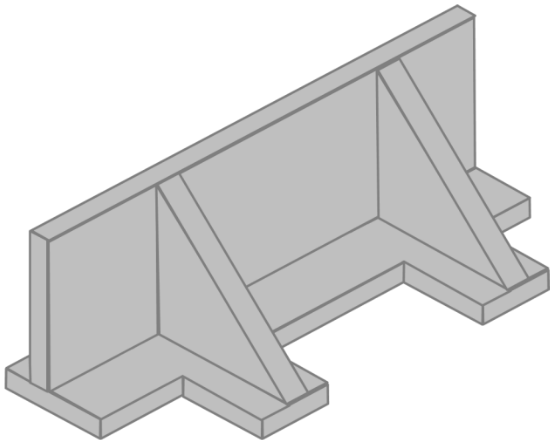


Figure 15: Impression of the slab with support elements

2.6.2 Dimensions

Main breakwater dimensions

It is desirable to reduce the overtopping as much as possible. However, the calculation formulas can only be applied up to an overtopping discharge of 1.00E-02 $m^3/s/m$. It is assumed this specific overtopping discharge has little to none influence on the wave height on the lee side of the breakwater. The required crest height for the slab with support breakwater can be determined with a maximum specific overtopping discharge of 1.00E-02 (for the service limit state). The crest height depends on the angle of the front wall of the construction. Table 19 shows the required crest height, according to appendix III.

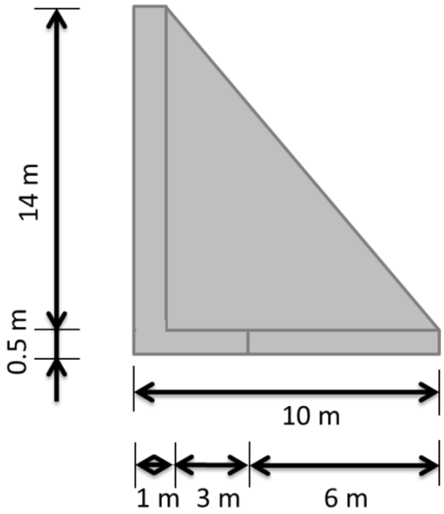
	Angle	Required Crest Height (in m)
Vertical	NA	3.40
Battered	10:1	3.70
Battered	5:1	4.20

Table 19: Required crest height for a specific overtopping discharge of 1.00E-02 $m^3/s/m$

With a lower crest height, fewer materials are needed and so the costs can be reduced. Therefore the chosen angle of the front wall is vertical.

The front wall of the concrete elements is non-water-permeable. It is assumed that the elements can be placed close enough to each other to guarantee that the total transmission coefficient of the construction is equal to or less than 0.1. This results in a required length of approximately 600 meter, see the transmission-length diagram in the basis of design.

Dimensions of the elements
The dimensions of the prefabricated elements are assumptions, no structural analysis are performed yet. The elements have to be designed as small as possible with as less concrete as possible. If the construction becomes too big, huge equipment is required in order to lift the concrete construction. An impression of the designed prefabricated elements is shown in figure 15. The assumed dimensions of the elements are shown in figure 16.



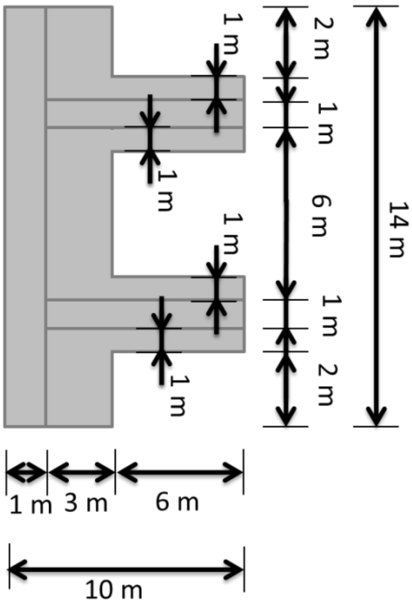


Figure 16: Dimensions of the elements

Backfill
Due to the severe wave conditions, it is expected that the prefabricated elements are pushed towards the jetty during high waves or storms or that the elements will be turned over due to the high wave force on the front wall. Therefore, a backfill is needed. The options investigated for the backfill are geo-containers or (shipping) containers (1 TEU) filled completely with sand. A second positive effect of applying a backfill is a reduced amount of concrete. The backfill will add weight to the slab construction. The extra weight applied by the backfill does not have to be performed in concrete.

Required material

Concrete elements

With the dimensions shown in figure 16, the total volume of reinforced concrete per element can be calculated, see table 20.

Front wall		
Height (m)	14	
Thickness (m)	1	
Length (m)	14	x
Volume (m^3)	196	
Ground slab		
Thickness (m)	0.5	
Width (m)	4	
Length (m)	14	x
Volume ground slab (m^3)	28	
Extended ground slab		
Thickness (m)	0.5	
Width (m)	6	
Length (m)	3	
Nr. of extended ground slabs (Pcs)	2	x
Volume extended ground slabs (m^3)	18	
Support		
Height (m)	14	
Width (m)	9	
Thickness (m)	1	
Nr. of supports (Pcs)	2	x *
Volume of supports ($m^\$$)	126	
Total volume of element		
Total volume (m^3)	368	

Table 20: Calculation of the total required amount of concrete

* The volume calculated by multiplying all the support dimensions in the table is divided by 2 because the support is triangular instead of rectangular.

The required number of elements can be determined: 600/14=42.9 → this means a total of 43 elements are needed.

The total amount of concrete needed can be calculated: 43*368=15,824 m^3.

Geo-containers

It is assumed that the space between the supports is filled with geo-containers. The volume of geo-containers can be calculated based on the dimensions of the elements shown in figure 16: 0.5*14*9*12=756m^3 per element. This results in a total volume of geo-containers of 756*43=32,508 m^3.

Containers with sand fill

It is assumed that 6 containers between the supports of one element and 6 containers between the supports of two elements will suffice, see figure 17 for the lay-out of the solution. The total amount of containers required can be calculated: 2*6*43=516 containers. The containers are completely filled with sand. The containers of 1 TEU have an internal volume of 33.14 m^3, see appendix I for the calculation of the internal volume. The total amount of sand fill required can be determined: 516*33.14=17,100 m^3.

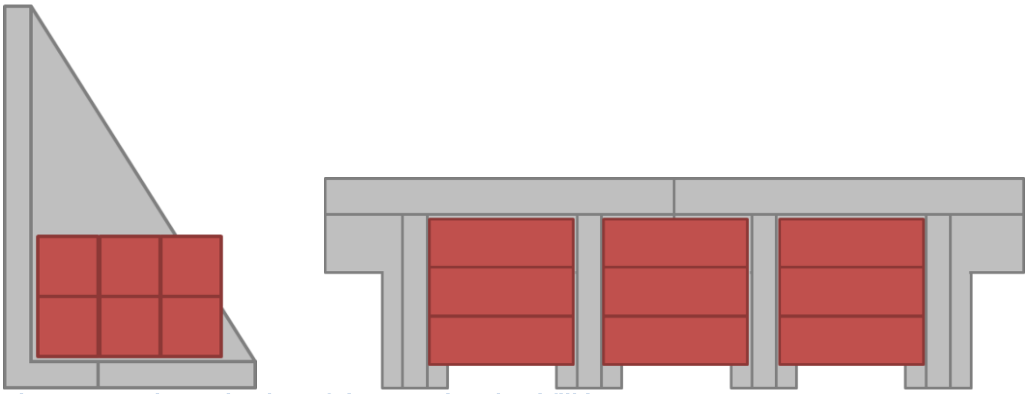


Figure 17: Schematisation of the container backfill layout

2.6.3 Lifetime

Lifetime of the concrete elements

If the concrete elements are constructed properly, there is enough concrete cover for the reinforcements and the elements are handled with care during construction, the lifetime of the concrete elements is expected to be at least 50 years.

Lifetime of containers

The lifetime of an unprotected container wall is only 2 years. However, when they are placed in the backfill of the construction, the only function of the containers is to create extra gravitational force for the total construction. So if the containers will corrode away, the construction will still suffice, as long as the (majority of the) sand remains in place. Thereby, the lifetime of the bottom of the container is much longer than the lifetime of the walls of the container, at least 25 years. Therefore, the sand in the rear six containers still contributes to the gravitational force of the construction.

Lifetime of the geo-containers

The lifetime of geo-containers and –tubes is not yet known, since they are a relative new solution. However, the Road and Hydraulic Engineering Division in the Netherlands has tested some samples of the polypropylene and polyamide geotextiles functioning for 30 years under block revetments on the Dutch sea dikes. The tensile strength had only decreased about 10% and the hydraulic functioning was still satisfactory. In the past couple of years the technology of geo-synthetics has improved considerably. It may be assumed that the lifetime of the geo-containers and –tubes is at least 40 years. (Pilarczyk, 2000).

2.6.4 Material costs

Unit prices
The unit prices of the materials used in the material costs calculation can be seen in table 21.

Material	Unit	Unit price	Source
Concrete	€/m^3	600	(Stive, 2015)
Geo-container	€/m^3	29.43	(Fowler & Trainer, 1997)
Containers	€/pcs	1,000.-	Various second hand shops
Sand	€/m^3	5.-	(Stive, 2015)

Table 21: Unit prices slab with support breakwater

Total material costs
In table 22 the total material costs of the two slab with support breakwater designs can be seen. The most lucrative breakwater is the slab with support breakwater with a containers filled with sand backfill. Therefore only this solution will be taken into account in the next chapter (multiplication factor).

Slab with support	Geo-container backfill	Container with sand backfill
Concrete (m^3)	15,824	15,824
Geo-containers (m^3)	32,508	-
Containers (Pcs)	-	516
Sand (m^3)	-	17,100
Unit price		
Concrete (€/m^3)	€ 600	€ 600
Geo-containers (€/m^3)	€ 29.43	€ 29.43
Containers (€/pcs)	€ 1,000	€ 1,000
Sand (€/m^3)	€ 5	€ 5
Material costs		
	€ 10,451,110.-	€ 10,095,900.-

Table 22: Material costs for the slab with support breakwater with geo-container backfill or with containers filled with sand backfill.

2.6.5 Complexities

Due to the wave impact forces, the construction may start to slide or to turn over. This is not checked yet. Thereby, a filter layer may have to be applied to reduce the chance of liquefaction of the subsoil.

2.7 Bellows structure

2.7.1 Introduction

The bellows structure consists of a rubber wrap filled with air. This construction has been investigated by a graduation student of the technical university of Delft (Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997). The dimensions and construction lay-out are based on his graduation report.

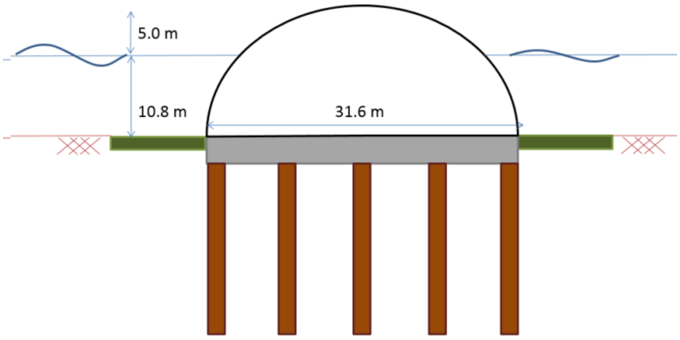


Figure 18: Schematic bellows construction

2.7.2 Dimensions

Main breakwater dimensions
The schematic dimensions of the bellows breakwater are situated in figure 18. The water depth at the breakwater location is 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design). In order to avoid the effect of overtopping at the lee side of the breakwater, during operational conditions, a crest height of 5 meters is required. The required crest height is based on the required crest height of the reference design, since the bellows structure exists of a similar slope.
It is assumed that no transmission will occur through the breakwater ($C_t = 0$), this results in a required breakwater length of approximately 600 meters, see the transmission-length diagram in the basis of design.

Dimensions cross section
The width of the bellows structure is 2 times the total height of the structure, according to (Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997). This means that the total width of the bellows structure is 31.6 m. Besides this rubber blanked, a foundation and a soil protection are required. Both of these construction parts are required over the entire length breakwater (600 meters). The soil protection is required in order to prevent the subsoil from eroding. Eroding will last into an unstable foundation construction and must therefore be prevented.

Required material
The required amount of rubber wrap per running meter can be calculated:

$$Surface\ rubber = 0.5 \times 2 \times \pi \times r_{bellow} = 49.5\ m^2/m$$

The total required rubber surface is 49.5 X 600 (breakwater length) = 29,700 m^2.

The total required soil protection and breakwater foundation have a length of 600 meters.

2.7.3 Lifetime

According to the report (Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997), the durability of the construction is approximately 50 years.

2.7.4 Material costs

The material cost, mentioned in table 23 have been calculated using the graduation report of (Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997).

Unit prices

Material	Unit	Unit price	Source
Foundation	€/m^2	€ 23,000.-	(Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997)
Rubber blanket	€/m^1	€ 550.-	(Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997)
Soil protection	€/m^1	€ 1,000.-	(Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997)

Table 23: Unit prices bellows breakwater

Total material costs
The total material costs of the bellows structure can be calculated by multiplying the required amount on construction material with the unit prices of the different material. The total construction costs of the bellows breakwater can be seen in table 24.

Material	Price	Unit	Amount	Unit2	Total
Foundation	€ 23,000	m1	600	m1	€ 13,800,000.-
Rubber blanket	€ 550	m2	29,688	m2	€ 16,328,427.-
Soil protection	€ 1,000	m1	600	m1	€ 600,000.-
Total					€ 30,728,427.-

Table 24: Material costs bellows structure

2.7.5 Complexities

The most complex part of the bellows construction, is to create the foundation of the breakwater which are occurring at the project location. Thereby, the rubber blanket has to be constructed whit in this foundation. The construction method for constructing the bellows structure will be optimized if the bellows structure turns out to be the most favourable breakwater concept.

2.8 Car wreckages

2.8.1 Introduction

The car wreckage breakwater alternative consists of car wreckages which will be stacked in the form of a pyramid. In this alternative breakwater investigation a distinction will be made between uncompressed cars and compressed cars. The compressed cars will be compressed by a car compressor, in order to decrease the height of the car, and herewith increase the stability of the cars. According to the National highway traffic safety administration (NHTSA), average car dimensions are determined as shown in table 25.

Mass car	1,200 kg
Length car	4.75 m
Width car	2.00 m
Height car, uncompressed	1.75 m
Height car, compressed	0.50 m

Table 25: Dimensions of cars

In the analysis of the car wreckages breakwater, only the compressed cars will be taken into account. It is assumed that the transmission through an uncompressed cars breakwater will be higher than the maximum allowed transmission through the breakwater. This is caused by the huge gaps within the uncompressed cars, due to the removed car windows and the irregularities of a car.

The car breakwater will be compared using two different car setups. A distinction between cars orientated parallel to the coastline and cars orientated perpendicular to the coastline is made.

2.8.2 Dimensions

Main breakwater dimensions
The water depth at the project location is set to 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design). The required height of the breakwater is assumed to be equal to the crest height of the reference design breakwater (5 m). The total required breakwater height is therefore 15.8 metres.

The required breakwater length is set to 600 meters, the required breakwater length is based on an estimated transmission coefficient equal to or lower than 0.17, see transmission length diagram in the basis of design.

Dimensions cross section

According to the van der Meer grading formulae mentioned in the reference design, the required nominal gradation mass (M_{50}) for a 1:3 slope is 4,450 kg, with a nominal diameter (D_{n50}) of 1.2 meter. Since the nominal diameter of a single car is larger than the nominal diameter of a single armour stone, the car will be exposed to more suction forces than the rock unit. Thereby the mass of the average car is lower than the required mass. When varying with the slope of the breakwater in the formulae of van der Meer, the minimum required slope is 1:9 with a nominal mass equal to 1,200 kg. The van der Meer formulae can be seen in the reference design report.

In figure 19, the required cross section is schematized. When dividing the required breakwater height with the height of a single car, 32 stacked cars are required. The required parallel orientated cars can be determined by dividing the required length by the width of a single car. The required perpendicular orientated cars can be determined by dividing the required length by the length of a single car.

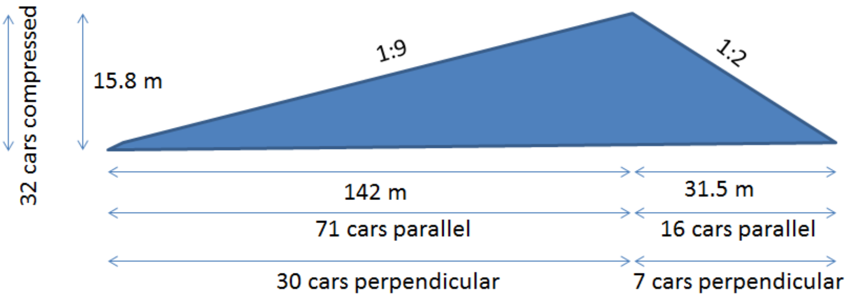


Figure 19: Required cars per breakwater layout

Required material

The total required cars can be determined by calculating the surface of the triangle (figure 19) and multiply this amount with the required cars in the longitudinal direction of the breakwater. In formulae:

$$Required\ cars = \frac{Length\ breakwater}{Length/Widht\ car} \times Nr.cars\ cross\ section$$

$$Nr.cars\ cross\ section = 0.5\ \times Nr\ cars\ in\ lengt\ \times Nr\ stacked\ cars$$

The total required cars for the parallel and the perpendicular orientation can be seen in table 26.

	Parallel	Perpendicular
Cars required	175,832	177,600

Table 26: Number of cars required

2.8.3 Lifetime

The compressed car units can be schematised as steel bars, with a high porosity. The steel parts of the compressed cars will corrode during the breakwater its lifetime. Thereby, deformations of the breakwater are expected caused by the wave forces, since the cars will not be attached to each other. Due to this corrosion and these deformations, the lifetime of the breakwater is expected to be 20 years (based on a rough assumption).

2.8.4 Material costs

Unit prices
The unit price of a scrap car is estimated to be €150.- per unit, this price is based on the average purchasing price of scrap car dealers. In order to prevent oil and fuel leftovers polluting the ocean, a fee for cleaning the cars is implemented. The fee for this cleaning is set to €100.- per car. This price is based on the average price which cleaning companies charge for cleaning the cars

Material	Unit	Unit price	Source
Car	€/unit	€ 150.-	Scrap car dealers
Car cleaning	€/unit	€ 100.-	Cleaning companies

Table 27: Unit price car wreckage breakwater

Total material costs
When multiplying the total required cars per breakwater layout with the costs for the car wreckages and the car cleaning. The parallel car layout appears to be the most lucrative breakwater layout. The total material costs are calculated to be € 43,958,000.-.

	Parallel	Perpendicular
Required cars	175,832	177,600
Car costs	€ 150.-	€ 150.-
Cleaning costs	€ 100.-	€ 100.-
Total costs	€ 43,958,000.-	€ 44,400,000.-

Table 28: Total material costs car wreckage breakwater

2.8.5 Complexities

There are no big complexities expected when constructing the car wreckages breakwater. The cars are able to be put in the water independently, which requires no big construction equipment.

If this breakwater appears to be the most lucrative breakwater alternative, an extended calculation of the weight and the diameter of the cars have to be performed. The car may be performed with a gentler slope, if the suction caused by the wave forces at the cars appears to be too high.

2.9 Tires

2.9.1 Introduction

The tires breakwater consists of a couple of tires stacked on top of each other in the form of a pyramid. In order to make this breakwater feasible, it is assumed that there are enough tires available in the surroundings of the project location, because many tires are required in order to achieve the required dimension of the tire pyramid breakwater.

2.9.2 Dimensions

Main breakwater dimensions
The required crest height is 5 meter, which is equal to the crest height of the reference design. It is assumed that this height is required in order to reduce the influence of overtopping at the lee side of the breakwater to zero. This crest height, in combination with the water depth of 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design) makes the total required breakwater height 15.8 meters.

The required breakwater length is set to 600 meters. The required breakwater length is based on an estimated transmission coefficient equal to or lower than 0.17, see transmission length diagram in the basis of design.

Dimensions cross section
The schematised tire dimensions which will be taken into account for the tire breakwater can be seen in figure 20. The average dimensions for this tire have been determined on basis of the offer of different tire suppliers.

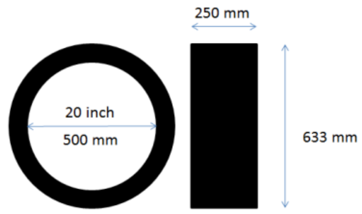


Figure 20: Dimension tire

The density of tire rubber is approximately 1020 kg/m^3 to 1270 kg/m^3 according to (Geosyntec Consultants, 2008). The density of steel-belted tires is usually higher than glass-belted tires. It is assumed that the tires used are steel-belted. Therefore, the used density is 1270 kg/m^3. The underwater weight of a single tire is 250 kg/m^3, this value is determined by reducing the relative density of tires with the relative density of the seawater (1020 kg/m^3).

In order to achieve a stable breakwater construction, a huge cross section of tires is required in order to achieve enough gravitational forces. These gravitational forces will prevent the breakwater from sliding towards the jetty. It is assumed that a 1:10 front and rear slope will provide enough gravitational forces for the tire breakwater construction. This breakwater slope is based on a quick and dirty assumption, if the tire construction appears to be the most favourable breakwater concept, these dimensions will be optimized and calculated further. A schematic drawing of the tire breakwater can be seen in figure 21.

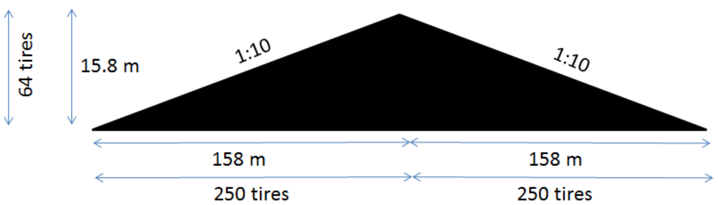


Figure 21: Schematic cross section tire breakwater

The total required breakwater height is 15.8 meters, the average height of a single tire is 0.25 meters, which results in a 64 high tire stacking. The length, resulting from a 1:10 slope is 158 meters, which results into a ground layer of 2 times 250 tires (tire width 0.633 meters).

Required material
The total required cars can be determined by calculating the surface of the triangle (figure 21) and multiply this amount with the required cars which are required into the breakwater its length. In formulae:

$$Required\ tires = \frac{Length\ breakwater}{Width\ tire} \times Nr.tires\ cross\ section$$

$$Nr.cars\ cross\ section = 0.5\ \times Nr\ tires\ in\ lengt\ \times Nr\ stacked\ tires$$

In total 9,600,000 tires are required in order to construct the tires breakwater.

2.9.3 Lifetime

It is assumed that rubber is not affected by salt water. For this reason, the durability of a single tire in salt water is assumed to be at least 50 years, which is equal to the durability of the reference rubble mound breakwater.

2.9.4 Material costs

Unit prices

The unit price of a single tire is set to € 2.-. According to different second hand tire shops, the costs of a second hand tire are approximately € 10.-. However, the tires which are applied in the tire breakwater do not have to fulfil any requirements which are required in order to achieve a certain safety on the roads. Thereby, a massive amount of tires is required. For these reasons, the costs of a scrap tire is set to € 2.-.

Total material costs
When multiplying the total required tires with the costs of a single tire, the total material costs of the tire breakwater are set to € 19,200,000.-, see table 29.

Required tires	9,600,000
Tire costs	€ 2.-
Total costs	€ 19,200,000.-

Table 29: Material costs tire breakwater

2.9.5 Complexities

The most complex part of the tire breakwater is the tire connection which is required in order to be sure that the tires stays into position during the entire lifetime of the breakwater. It is assumed that a single tire is not able to last the wave forces, and will be separated from the breakwater. For this reason, some fortifications are required in order to achieve a stable construction. The costs of these fortifications will be taken into account in chapter 3, multiplication factors.

2.10 Container pyramid

2.10.1 Introduction

The container pyramid consists of containers, which have been stacked on top of each other, in order to create a breakwater. The most critical part of the container pyramid is the container wall, due to corrosion there may occur some gaps in the container walls. If the gaps in the breakwater its walls become too high, the transmission through the breakwater may become unacceptable. According to (3werf, 2000), the end of a container lifetime will be achieved when the containers are not able to be removed out of the water at one piece. A container wall is 1.6 millimetres thick and will end its lifespan when the wall thickness is equal to 0.85 millimetres. According to (3werf, 2000) the durability of a container in salt water is as shown in table 30.

Zone	Durability (years)
Splash zone	3
Tidal zone	2
Low water zone	2
Below water zone	8

Table 30: Durability container (3werf, 2000)

In order to increase the lifetime of the container pyramid breakwater, two different protection methods will be investigated in this chapter, a breakwater coating and a steel shield protection.

2.10.2 Dimensions

Main breakwater dimensions
The transmission through the container pyramid breakwater is expected to be nihil. According to the transmission length diagram in the basis of design, the required breakwater length is approximately 600 meters (based on a transmission coefficient of 0).

The required crest height is assumed to be equal to the crest height of the reference breakwater, since the same front and rear slopes are applied. The total height of the breakwater is herewith 15.8 meters (water depth of 10.8 m below water level, CD + HHWS + surge + sea level rise, see basis of design, and 5 meter crest height).

Container dimensions
The dimensions of a single container unit are as shown in table 31.

	Length (m)	Width (m)	Height (m)	Maximum filled weight (kg)
1 TEU	6.058	2.438	2.591	30,480
2 TEU	12.192	2.438	2.591	30,480

Table 31: Dimensions sea containers (Hapag-Lloyd)

The maximum filled weight of the 2 TEU containers is equal to the maximum filled weight of the 1 TEU containers, the 2 TEU containers will not be taken into account, since buoyancy forces are an issue. If a 1 TEU container is filled with its maximum fill weight but no water is put into the container, it will float due to the buoyancy force. For this reason, the containers have to be filled with water and sand in order to prevent the container from floating.

In appendix I, a schematic calculation of the buoyancy forces at a single container and the required amount of containers per container layout variation can be found. This calculation concludes that the containers have to be filled with water, in order to prevent floating. Thereby, the maximum allowed weight in sand will be added to the container, in order to increase the gravitational forces and herewith the stability of the single containers.
A single container will not remain standing when filled with the maximum allowed payload. If the maximum container weight will be exceeded, the container bottom will collapse when the container is lifted. When the containers are filled with more weight, a more complex construction method has to be developed. Developing a more complex construction method is more expensive than a regular construction method; therefore only the maximum allowed filled weight will be taken into account.

Dimensions cross section

The container pyramid breakwater consists of stacked containers. The slopes of the container pyramid breakwater are designed as steep as the slope of the reference rubble mound breakwater with a rock armour layer. The containers at the front slope will be constructed at a slope of 1:3. The rear slope will be constructed at a slope of 1:1.5.

The container pyramid breakwater will be performed by investigating three different containers layouts. The first layout consists of containers which are constructed parallel to the coast. The second layout consists of containers constructed perpendicular to the coast. The third layout consists of a combination between layout 1 and layout 2, the front slope will be constructed using containers perpendicular to the coast and the rear slope will be constructed using containers parallel to the coast.

Container protection

<u>Coating</u>

When coating the container pyramid breakwater, the lifetime of the breakwater will increase. In this chapter, three different coating approaches will be taken into account. The first approach is coating each container completely both the internal container surface as the external container surface. This approach requires a lot of container coating. The required coating for a single container is approximately 75 m^2.

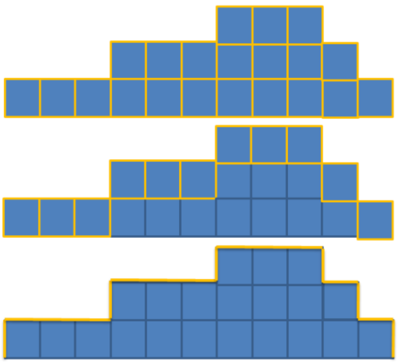


Figure 22: Coating approaches

In order to reduce the required coating, a second coating approach is taken into account. In this approach only the external containers are coated (both the internal as the external container surface) since these containers are most influenced by the salt ocean conditions. If the external wall is protected from corrosion, the formation of gaps in the external wall will be postponed so the exceedance of the maximum allowed transmission coefficient will be deferred. The second coating approach requires less coating than the first approach.

A third coating approach is achieved by only coating the ocean facing container walls. This requires less coating than the first or the second approach, but is assumed to be as effective as both of them. If the external container walls will not rust away, the maximum allowed transmission coefficient will not be exceeded.

The required coating calculations are to be found in appendix I: schematic calculation container pyramid. This appendix determines the required coating for the three coating approaches and the three container layouts.

Steel shield

A second approach for increasing the breakwaters lifetime is by welding a steel plate at the ocean facing container walls and roofs. These steel plates will protect the breakwater as long as the container frames lifetime. The steel shield breakwater is considered in two different steel shield layouts: a layout where all of the external containers are covered and a layout where only the ocean facing slope of the container pyramid breakwater is covered. It is assumed that both of the steel shield layouts are able to guarantee a breakwater lifetime equal to the lifetime of the container frame.

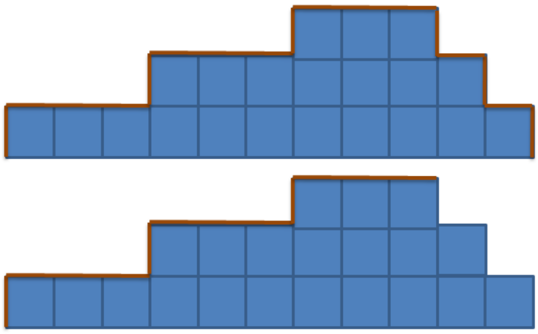


Figure 23: Entire and ocean facing steel coating

Required material

The required amount of containers, coating and steel for all of the possible container pyramid layouts can be seen in table 32. An extended quantity determination can be found in appendix I: schematic calculation breakwater pyramid.

Layout	Required containers	1 (m^2)	2 (m^2)	3 (m^2)	4 (ton)	5 (ton)
Parallel	7,730	572,020	167,850	104,600	4,085	2,675
Perpendicular	8,860	655,640	200,000	114,200	4,460	2,300
Perp. + Parra.	6,660	492,840	145,100	92,450	3,610	2,300

Table 32: Required amount of breakwater material

Where:

1: Required coating: coating all containers
2: Required coating: coating only ocean facing containers
3: Required coating: coating ocean facing walls
4: Required weight of steel: complete steel shield
5: Required weight of steel: ocean facing steel shield

Breakwater reinforcement

According to appendix I, chapter strength of the containers, the container will collapse, if the containers are not stacked centrically. Due to the swell waves at the project location, it is assumed to be impossible to stack the containers centrically. Thereby, the first containers have to be constructed at the subsoil, which is minimally 9.5 meter below the still water level. For this reason, a container reinforcement has to be applied, in order to allow the containers to be stacked eccentrically. The costs for the container protection will be taken into account in chapter 3, multiplication factors, since the required dimensions for this container reinforcement are not known yet.

2.10.3 Lifetime

The lifetime of the container construction without any protection is two years, after these two years, the transmission though the breakwater is assumed to be too high. When applying the container coating, the lifetime of the container walls will be extended. According to experts of RHDHV, the minimum lifetime of the container walls, when applying a container coating is minimally 10 years.
When applying a steel container protection, the lifetime of the containers can be set equally to the lifetime of the container frame, it is assumed that the container frame will minimally last for 20 years in the tidal zone. This assumption has to be verified during an eventual further design.

2.10.4 Material costs

Unit prices
The unit price of second hand shipping containers vary from €1,500.-/pcs to €3,000.-/pcs. However, the containers used in the container pyramid breakwater are not required to comply with the regular rules for shipping containers because the containers are used in a different way. Thereby, there are a lot of containers needed (more than 500), so they can be bought in by bulk. Therefore, the unit price of the containers for the backfill is estimated to be €1,000.-/pcs.
The costs of the container fill and the container coating have been determined, based on the experience of RHDHV. The steel price is based on the unit price of steel of March 2015. In table 33 the applied unit prices can be seen.

Material	Unit	Unit price	Source
Container	€/unit	€ 1,000.-	Second hand shops
Container fill (sand)	€/m^3	€ 5.-	(Stive, 2015)
Coating	€/m^2	€ 25.-	(Stive, 2015)
Steel	€/ton	€ 1,100.-	(WSDOT, 2015)

Table 33: Unit prices container pyramid

Total material costs
In total, 15 different container solutions have been determined. The first column of table 34 indicates the layout of the container, the next 5 columns describe the way of protection. The total material costs have been determined by multiplying the required coating and the required amount of containers with the unit prices mentioned in table 33.

The most lucrative breakwater, according to table 34, is the breakwater which is constructed both perpendicular as parallel with a steel shield applied at the ocean facing walls, this container pyramid breakwater will be taken into account in chapter 3, multiplication factors.

Note: These material costs excludes the required container reinforcements.

Protection	1 (m^2)	2 (m^2)	3 (m^2)	4 (ton)	5 (ton)
Parallel	568,658	167,580	104,599	4,085	2,042
Perpendicular	651,787	199,151	114,167	4,458	2,229
Perp.+ para	489,943	145,058	92,447	3,611	1,805
Containers	**1**	**2**	**3**	**4**	**5**
Parallel cont.	7730	7730	7730	7730	7730
Perp. cont.	8860	8860	8860	8860	8860
Perp. +para cont.	6660	6660	6660	6660	6660
Container fill (sand)	11	11	11	11	11
Material costs	**1**	**2**	**3**	**4**	**5**
Parallel	€ 22.358.184	€ 12.331.226	€ 10.756.712	€ 12.635.215	€ 10.388.474
Perpendicular	€ 25.626.586	€ 14.310.689	€ 12.186.090	€ 14.235.339	€ 11.783.630
Perp. + para	€ 19.263.325	€ 10.641.191	€ 9.325.909	€ 10.986.370	€ 9.000.555

Table 34: Material costs container pyramid

1 Coating all containers
2 Ocean facing containers completely coated
3 Only ocean facing walls coated
4 Steel shield: ocean and lee side facing walls
5 Steel shield: only ocean facing walls

2.10.5 Complexities

The most complex part of the container pyramid, are the required welding works at the quay. Before the containers can be taken to the construction site, the container reinforcements and the steel shield have to be welded at the containers. These welding works will be performed at the quay, because of the more stable environmental conditions. The total costs for applying the welding works and the container protection, will be taken into account in a multiplication factor in chapter 3.

2.11 Container wall with struts

2.11.1 Introduction

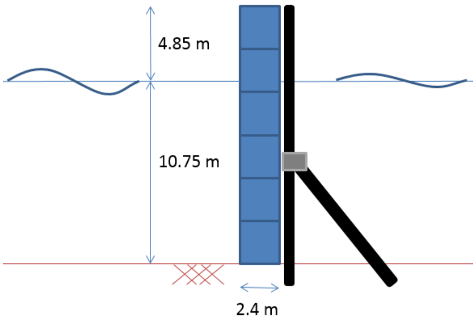


Figure 24: Simplified schematisation wall with struts

The container wall breakwater solution consists of a container wall supported by a steel supporting construction. When constructing the containers stacked on each other, there are fewer containers required in comparison with the container pyramid. In order to guarantee the container walls from crushing out of the container frame. According to ISO 1496-1, freight containers – specification and testing (ISO, 1990), the maximum allowed force at a container wall 135 kN/container wall surface (2.5 x 6 meters). According to appendix IV, the occurring force per container wall is 252 kN (2.5 x 6 meters). The maximal occurring force at the container walls is higher than the maximum allowed force, for this reason, the container walls have to be fortified.

2.11.2 Dimensions

Main breakwater dimensions

The required length of the breakwater construction is approximately 600 meters, according to the transmission length diagram in the basis of design (there is no transmission through the breakwater expected, since the construction exists of closed container walls). The required crest height above the highest water level is assumed to be approximately 3.4 meters, based on the overtopping calculation found in appendix III. The water depth during the highest water level is 10.8 meters (CD + HHWS + surge + sea level rise, see basis of design).

Dimensions cross section

The dimensions of a single container can be seen in table 35 (Hapag-Lloyd).

	Length (m)	Width (m)	Height (m)	Maximum filled weight (kg)
1 TEU	6.058	2.438	2.591	30,480

Table 35: Dimensions container

The minimal height of the container wall is 14.2 meters. In order to achieve this height, 14.2/2.6 = 6 containers have to be stacked on top of each other in order to achieve the minimum breakwater height. In figure 24, a schematic cross section of the container wall with struts can be seen. The dimensions of the steel supporting construction have to be determined during a potentially further research.

Required material

In total an amount of 600 containers is required for the entire breakwater length. Each container will be filled with 10.65 m^3 of sand, in order to increase the total gravitational force of the containers. If this 10.65 m^3 sand fill is exceeded, the container floor will collapse when the container is lifted, see appendix I. In total 10.65 x 600 (number containers) = 6,400 m^3 sand fill is required.

$$Nr\ containers = \frac{Length\ breakwater}{Length\ single\ container} \times Containers\ per\ cross\ section$$

$$Nr\ containers = \frac{600}{6} \times 6 = 600$$

The supporting steel construction, which has to protect the containers from overturning, requires the same dimension as mentioned in the jacket breakwater concept. The steel protection has been defined to be the most favourable support, according to the jacket breakwater alternative. In total 1.040 m^3/m steel is required in order to achieve the steel supporting construction. When multiplying this amount with the relative density of steel (7.8 ton/m^3) and the breakwater length (600 meters), a total 4,867 ton steel is required for the supporting construction.

When only applying a steel supporting construction, the total surface of the construction in contact with the subsoil is very small. In order to prevent the construction from sliding caused by the horizontal wave loads and a small friction surface, a foundation will be taken into account when determining the material costs of the container wall breakwater. This foundation is required over the entire breakwater length (600 meters).

In order to prevent the container walls from breaking out of the container frame, a steel support is required inside the container. When welding a x-construction against the container walls, it is assumed that the container walls are able to withstand the extreme environmental wave conditions. The total length of 1 steel bar is:

$$\sqrt{2.5^2(H\ container) + 6^2(L\ container)} = 6.5\ m$$

In total 13 m length of steel construction is required in order to achieve the x-construction in front of the container walls. For this alternative analysis, is assumed that the diameter of a steel construction bar is 10 cm (based on a quick and dirty estimation). The total required steel per container protection is herewith 0.1 m^3 steel per container fortification. In figure 25, the schematic dimensions of the container fortification can be seen. This fortification will only be applied at the ocean facing container wall. In total 0.1 x 600 (nr containers) = 60 m^3 steel in required. When multiplying this amount with the relative density of steel (7.8 ton/m^3), a total amount of 485 ton steel is required.

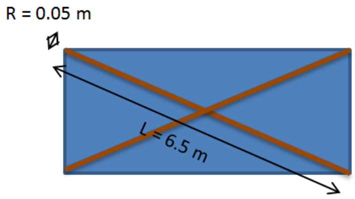


Figure 25: Schematic container protection

According to (3werf, 2000), the lifetime of the container wall is only 2 years at the tidal zone. In order to increase the container lifetime, a coating will be applied. The main purpose of the coating is to prevent gaps in the breakwater walls, which will cause transmission through the breakwater. For this reason, each container will be coated both internal as external. Each container requires 74 m^2 coating (see chapter 2.11, container pyramid). In total 74 x 600 (breakwater length) = 44.400 m^2 coating is required.

2.11.3 Lifetime

The lifetime of the container construction without any protection is two years, after these two years, the transmission though the breakwater is assumed to be too high. When applying the container coating, the lifetime of the container walls is extended. According to experts of RHDHV, the minimum lifetime of the container walls, when applying a container coating is minimally 10 years.

2.11.4 Material costs

Unit prices
In table 36 the unit prices of the used construction material can be seen. The material costs of the containers are explained in chapter 2.10, container pyramid. The costs of the container fill and the container coating have been determined, based on the experience of RHDHV. The steel price is based on the unit price of steel of (WSDOT, 2015). The applied foundation is assumed to be as expensive as the foundation construction which is required beneath the bellows structure.

Material	Unit	Unit price	Source
Container	€/unit	€ 1,000.-	Second hand shops
Container fill (sand)	€/m^3	€ 5.-	(Stive, 2015)
Coating	€/m^2	€ 25.-	(Stive, 2015)
Steel	€/ton	€ 1,100.-	(WSDOT, 2015)
Foundation	€/m^1	€ 23,000.-	(Dorreman, Balgstuwen gevuld mel lucht en/of water, 1997)

Table 36: Unit prices container wall breakwater

Total material costs
By multiplying the total required amount of materials with the unit prices of the materials, the total material costs mentioned in table 37 are determined. The total material costs of the container wall with struts breakwater is calculated to be € 21,422,200.-.

Material	Required	Unit price	Total
Containers	600 units	€ 1,000.-	€ 600,000.-
Contain fill	6,400 m^3	€ 5.-	€ 32,000.-
Coating	44,400 m^2	€ 25.-	€ 1,100,000.-
Support	4,867 ton	€ 1,100.-	€ 5,353,700.-
Container protection	485 ton	€ 1,100.-	€ 533,500.-
Foundation	600 m	€ 23,000.-	€ 13,800,000.-
Total			€ 21,422,200.-

Table 37: Total material costs container wall with support

2.11.5 Complexities

The most complex part of constructing the container wall with struts breakwater is that the containers have to be connected to each other before they are placed into the water, since exactly centric stacking is assumed to be not possible due to the swell waves and the water depth. A second complexity is the required container fortification, the exact required amount of fortification have to be determined during further research, which may result in higher container fortification costs per container.

2.12 Old ships

2.12.1 Introduction

The idea of the old ships breakwater is to sink some old ships in front of the jetty to provide shelter. There are a lot of types of ships which may be eligible for the use as breakwater. However, in former research performed by RHDHV, the most beneficial ship to sink is a cape size bulk carrier (Bosman, 2015).

2.12.2 Dimensions

Cape size bulk carriers have a length of approximately 275 to 325 meters. The beam of a cape size bulk carrier is approximately 45 to 60 meters. The draft of a cape size bulk carrier is approximately 17.5 to 21.5 meters. Since 2005, it is required that cape size bulk carriers have double hulls. Applying smaller ships results in the requirement of more ships. It is assumed that the total costs of the small ships will be higher than the total costs of the sunken cape size bulk carriers.

The required length of the “old ships breakwater” is approximately 600 m (transmission and overtopping are estimated to be nihil), see the transmission length diagram in the basis of design. This means that a total of two cape size bulk carriers are required.

Cape Size Bulk Carriers	
Length	275 to 325 meters
Beam	45 to 60 meters
Draft	17.5 to 21.5 meters

Table 38: Properties cape size bulk carriers

2.12.3 Lifetime

The maximum corrosion rate for steel in the splash zone of salt water is 0.17 mm/side/year according to (British Standard, 2000). The hull of an ocean going ship is between 9 and 25 mm thick. The 25 mm thick sections are in the splash zone.

The lifetime of the hull can be calculated:

$$\frac{25}{0.17 * 2} = 73.5 \; years$$

The average scrapping age of cape size bulk carriers is approximately 20 to 25 years. Therefore, the expected lifetime of the “old ships breakwater” is 73.5-25=48.5 years.

However, the hull of the ship will break down earlier due to the wave impact on the hull and the other external forces on the ship during its lifetime like the loading and unloading of, for example, ores or coal. If the old ships breakwater turns out to be the most favourable in the MCA, the lifetime of the old ships breakwater will be investigated in more detail.

2.12.4 Costs

According to (Bosman, 2015), the costs to sink a single cape size bulk carrier on the project location are €10,000,000.-. These costs include the purchase of the old ship, the stripping and cleaning of the ship, the transportation of the old cape size bulk carrier to the sink location and the sinking process itself.

The total costs for the old ships breakwater are:

$$2 * €10{,}000{,}000.- = €20{,}000{,}000.-$$

2.12.5 Complexities

There are no expected complexities when constructing the old ships breakwater.

2.13 Vertical tubes with band

2.13.1 Introduction

The idea of the "*vertical tubes with band or slab*" breakwater is of vertical steel tubes with a slightly deformable but non-water-permeable rubber slab in between (for example, a conveyor band), see figure 26.

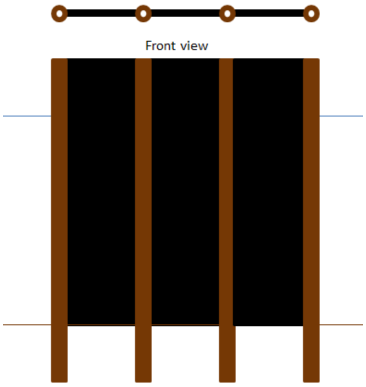


Figure 26: Impression of the vertical tubes with band or slab concept

2.13.2 Dimensions

Main breakwater dimensions

It is desirable to reduce the overtopping as much as possible. The calculation of the required crest height of the structure can be found in appendix III. The calculation formulas can only be applied up to an overtopping discharge of 1.00E-02 m^3/s/m. The required height, to guarantee that the maximum overtopping discharge of 1.00E-02 m^3/s/m is not exceeded, is 3.40 meters, see jacket breakwater. The maximum water depth at the breakwater location is a combination of CD+HHWS+SLR+Surge. The maximum water depth is 10.8 meters. This means that the total required height of the construction is 14.2 meters.

The transmission of the construction is estimated to be nihil and it is assumed that the specific overtopping discharge has little to none influence on the wave height on the lee side of the breakwater. The required length of the breakwater is determined using the transmission-length diagram in the basis of design. The required breakwater length is approximately 600 meters.

The width of the slabs is determined by the conveyor band manufacturer. According to (Dunlop Conveyor Belting, 2011) and (Challange), the maximum width of conveyor band that is manufactured is 2,200 mm. This means that the width of a slab of conveyor band is 2.2 meters.

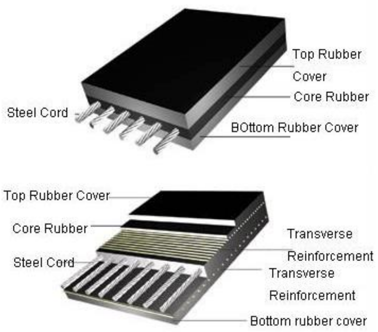


Figure 27: Reinforced rubber conveyor band. (QinmgDao HuiZhong Rubber Co. , Ltd.)

Required material

The chosen material for the slab in between the piles is a conveyor bands. The conveyor belts are reinforced with steel strings so it can resist high tensile stresses. See figure 27.

Normally, the steel reinforcement is placed in longitudinal direction. However for this application of the conveyor band, the steel reinforcements have to be placed in transverse direction, since tension in the horizontal direction will occur.

Dimensions cross section

The length of the steel piles is depending on the diameter and thickness of the piles and the forces on the construction. Table 39 shows a couple of piles that can be used for the construction and the required length of the steel piles. The investigated pile diameters vary from 1.0 meter to 3.0 meter, because piles smaller than 1.0 meter are expected to be not strong enough and piles with an diameter of 3.0 meter are the largest most common used diameter for steel tubes according to (voorbij funderingstechniek, 2014). See appendix IV for the determination and calculation of the pile possibilities.

Diameter [m]	Wall thickness [mm]	Required length in soil [m]	Total length [m]	Nr. of piles required [pcs]
1.0	99	11.64	25.8	189
1.5	57	11.68	25.9	164
2.0	43	11.71	25.9	144
2.5	37	11.73	25.9	129
3.0	33	11.75	25.9	117

Table 39: pile possibilities

See figure 28 for an overview of dimensions the construction.

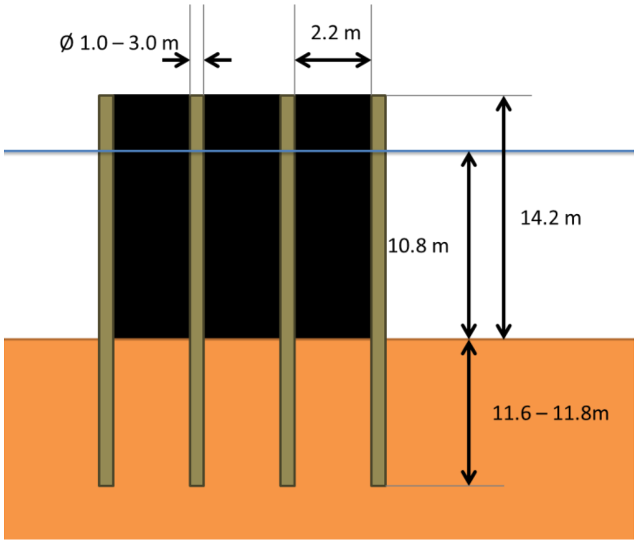


Figure 28: overview of the dimensions of the construction

With the dimensions of the piles and the rubber band known, the required amount of material, for a 600 m breakwater, can be determined for each of the vertical tubes with band breakwater designs, see table 40.

Variants: Diameter [m]	Required amount of steel [m^3]	Required amount of rubber band [m^2]
1.0	1,366.4	5,873.1
1.5	1,097.6	5,092.1
2.0	986.0	4,467.3
2.5	956.5	3,998.7
3.0	932.1	3,623.8

Table 40: Required amount of materials for the different vertical tubes with band breakwater designs

2.13.3 Lifetime

The lifetime of the conveyor belt is expected to be longer than the lifetime of the vertical steel tubes. The lifetime of the vertical steel tubes is minimally 50 years, see appendix IV.

2.13.4 Material costs

Unit prices

The unit price of conveyor belts vary a lot due to the different types of reinforcement used. A standard steel cord conveyor belt unit price lies between €5 and €100 per meter, depending on the manufacturer, the width of the belt and the required strength. When the width of the conveyor belt is taken into account the unit price is approximately €40 to €60 per square meter. However, the steel cord reinforcements in the belt used in the vertical tubes with band breakwater have to be placed in transverse direction instead of the longitudinal direction, which is normal. Therefore it is expected that the total costs to manufacture the required belts is much higher than the manufacturing costs for standard steel cord conveyor belts. Therefore the unit price of the steel cord conveyor belt used in the vertical tubes with band breakwater will be higher than the unit price of standard steel cord conveyor belts. It is expected that the unit price of the belt used in the vertical tubes with slab breakwater is approximately €200.- per square meter.

The unit prices of the materials used in the material costs calculation can be seen in table 41.

Material	Unit	Unit price	Source
Steel	€/ton	1,100.-	(Washington State Department of Transportation, 2015)
Conveyor band	€/m^2	200.-	Based on conveyor band manufactures

Table 41: Unit prices vertical tubes with band breakwater

Total material costs

In table 42 the total material costs of the vertical tube with band breakwater designs can be seen. The most lucrative breakwater is the one with tubes with a diameter of 3.0 meters. Therefore only the vertical tubes with band breakwater with tubes with a diameter of 3.0 meters will be taken into account in the next chapter (multiplication factor).

Diameter (m)	1.0	1.5	2.0	2.5	3.0
Steel (m^3)	1,366.4	1,097.6	986.0	956.5	932.1
Steel (ton)	10,657.9	8,561.3	7,690.8	7,460.7	7,270.4
Conveyor belt (m^2)	5,873.1	5,092.1	4,467.3	3,998.7	3,623.8
Unit price					
Steel (€/ton)	€ 1,100	€ 1,100	€ 1,100	€ 1,100	€ 1,100
Conveyor belt (€/m^2)	€ 200	€ 200	€ 200	€ 200	€ 200
Material costs					
	€13,046,481	€10,538,670	€9,452,644	€9,103,400	€8,816,404

Table 42: Material costs for the steel pyramid of piles and the concrete pyramid of piles

2.13.5 Complexities

The connection between the rubber slab and the pile is not designed yet. An example of how the connection can be designed is the Berliner-wall principia. The rubber slab will be mounted in a steel frame, which will be shoved in between two H-profiles. For this solution the piles have to be redesigned as H-profiles. However, the material costs of the H-profiles are expected to be in the same order of magnitude as the material costs of the tubes. The connection between the rubber slab and the pile will not be investigated before the MCA. If the vertical tube with band or slab breakwater turns out to be the best alternative in the MCA, the connection between the rubber slab and the pile will be investigated and designed in more detail.

In a standard steel cord conveyor belt, the steel cords run in the longitudinal direction of the conveyor belt. However, for the use of the conveyor belt in the vertical tubes with band breakwater concept, the steel cords have to run in the transverse direction of the conveyor belt. This may be a challenge for the manufactures when constructing the conveyor belt.

A risk during construction of the vertical tubes with band breakwater is the placement tolerances of the vertical piles. When they are placed too far apart, or too close to each other, it may be difficult to construct the conveyor belt in between them.

2.14 Sandwich breakwater

2.14.1 Introduction

The idea of the sandwich breakwater is to lock an artificial sand coast in place by placing it in between two fixed constructions. These fixed constructions can be for example: sheet piles, a container wall or a combined-wall. See figure 29 for an impression.

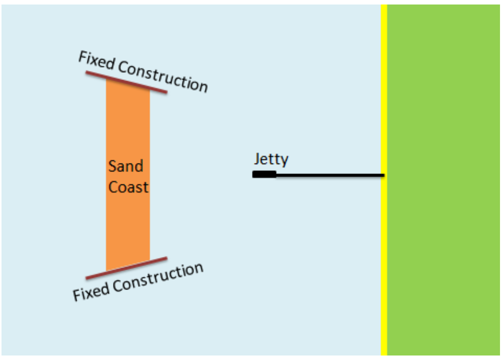


Figure 29: Impression of the sandwich breakwater

2.14.2 Dimensions

Main breakwater dimensions
The ocean side of the artificial sand coast will act like a normal sand coast, the slope of the sand under water will move towards the equilibrium profile. The average steepness of the ocean side slope is approximately 1:90 according to (Global Security). The crest height is determined to 5 meters above HHWS+SLR+surge, based on the height of the reference design. After consultation of an expert on breakwaters and coasts of RHDHV (Groenewegen, 2015), it is assumed that the crest requires a minimum width of 50 meters and that a rear slope steepness of 1:50 will suffice. The steepness of the slopes of the construction is approximately 1:5 from 1 meter above HHWS+SLR+Surge and above.

Due to the mild slopes of the construction, the centre line of the breakwater (in longitudinal direction) lies approximately 550 meter further seaward than the centre line of the reference design. Therefore the diffraction of the incoming waves around the breakwater has to be re-determined. It is assumed that the overtopping and transmission of the incoming waves over and through the construction is nihil. With the transmission and overtopping known, the required length can be determined in the same way as the required length is determined for the reference design. The required length of the construction is approximately 1140 meters.

Dimensions cross section
Figures 30 and 31 show the dimensions of the cross section.

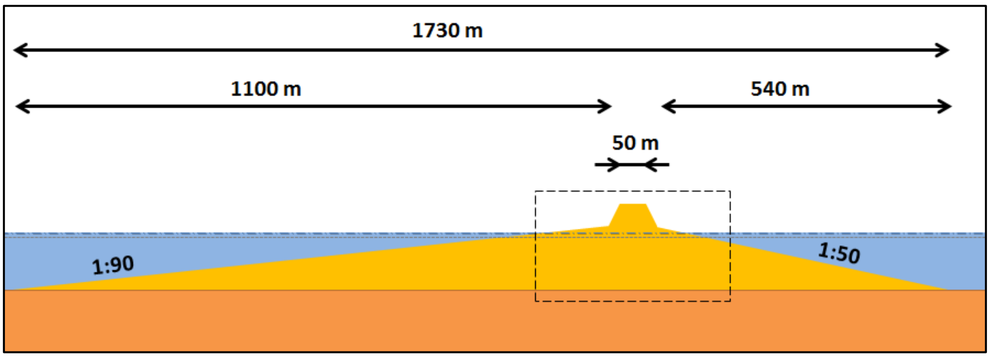


Figure 30: Cross section – overview

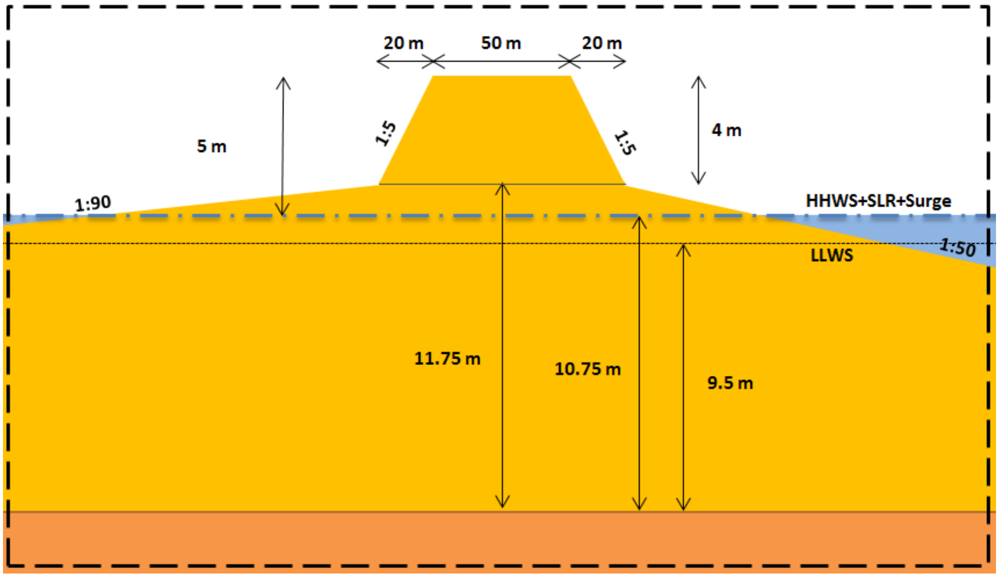


Figure 31: Cross section - centre part

Required material
With the dimensions of the cross section the total required amount of sand per meter breakwater can be determined. The total required amount of sand per meter breakwater is approximately 10,900 m^3/m. The total required amount of sand for the whole breakwater can be determined: 10,900 * 1,140 = 12,426,000 m^3.

2.14.3 Lifetime

The lifetime of the construction is depending on the lifetime of the fixed constructions. When the fixed constructions have a lifetime of 50 years, the lifetime of the complete construction is expected to be also 50 years.

2.14.4 Material costs

Unit prices
The unit prices of the materials used in the material costs calculation can be seen in table 43..

Material	Unit	Unit price	Source
Sand	€/m^3	5.00	(Stive, 2015)

Table 43: Unit prices sandwich breakwater

Total material costs

In table 44 the material costs of the sandwich breakwater can be seen. The total costs of the sand are already twice as high as the total costs of the reference breakwater. Therefore, the fixed constructions are not designed and thus are not included in the costs.

	Sandwich breakwater
Sand (m^3)	12,426,000
Unit price	
Sand (€/m^3)	€ 5,00
Material costs	€ 62,130,000

Table 44: Material costs per breakwater configuration

2.14.5 Complexities

The fixed constructions on the heads of the breakwater are not designed, because the material costs of the sand coast of the breakwater are already almost twice as much as the total costs of the reference design. If the fixed constructions on the heads of the breakwater are designed, the material costs will rise even further.

2.15 Enclosed waste

2.15.1 Introduction

The concept of the enclosed waste breakwater is to lock a big waste product like containers or car wreckages (compressed or uncompressed) between two rows of piles. Every waste product element is surrounded by 4 piles because otherwise one of the waste elements can be pushed out of the construction due to the wave impact forces. Figure 32 shows an impression of the concept. The piles can be made from different materials. The materials steel and wood will be taken into account.

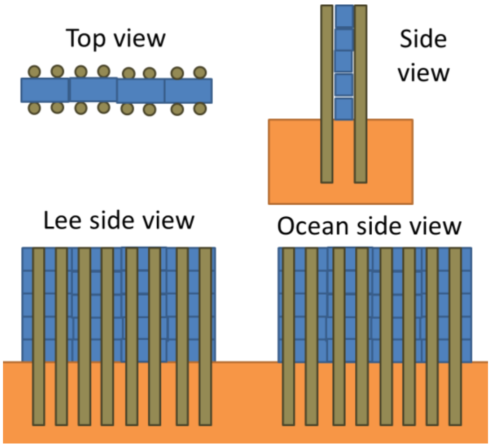


Figure 32: Impression of the enclosed waste breakwater

2.15.2 Dimensions

Main breakwater dimensions
It is desirable to reduce the overtopping as much as possible. However, the calculation formulas can only be applied up to an overtopping discharge of 1.00E-02 m^3/s/m. The required crest height for the enclosed waste breakwater can be determined with a maximum specific overtopping discharge of 1.00E-02 (for the service limit / the operational conditions). The required crest height is 3.40 meter, according to appendix IV. The maximum water depth at the breakwater location is a combination of HHWS+SLR+Surge. The maximum water depth is 10.8 meters. This means that the total height of the construction has to be 14.2 meters.

The transmission of the construction is estimated to be equal to or less than 0.1. It is assumed that the specific overtopping discharge has little to none influence on the wave height on the lee side of the breakwater. The required length of the breakwater is determined using the transmission-length diagram in the basis of design. The required breakwater length is approximately 600 meters.

Container dimensions
The dimensions of a single container are showed in table 45, based on (Hapag-Lloyd).

	Length (m)	Width (m)	Height (m)	Maximum filled weight (kg)
1 TEU	6.058	2.438	2.591	30,480
2 TEU	12.192	2.438	2.591	30,480

Table 45: Dimensions sea containers (Hapag-Lloyd)

Since buoyancy is an issue and the 1 TEU and 2 TEU containers have the same maximum filled weight, the 2 TEU containers will not be taken into account.

Height of the construction

The required height of the construction is 14.2 meter and the height of a container is 2.591 meter. The total required stacked containers can be determined: 14.2 / 2.591 = 5.48 → this means a total of 6 layers of containers are needed. The total height of the construction will therefore be: 6 * 2.591 ≈ 15.55 meter.

Required containers

Per breakwater cross section, 6 containers are required. The containers have a length of 6.058 meter and the total required breakwater length is approximately 600 meters. With this summary, the total amount of containers can be calculated:

$$Required\ containers = \frac{Length\ breakwater}{Length\ container} \times Nr.containers\ cross\ section$$

$$= \frac{600}{6.058} \times 6 \approx 600\ containers\ (100\ stacks\ of\ 6\ high)$$

Container fill

To increase the gravitational force, the containers are filled partly with sand. The empty weight of the 1 TEU containers is 2,250 kg according to (Hapag-Lloyd). The containers are filled to their maximum filled weight, which means a total of 30,480-2,250=28,230 kg of fill material is applied per container. Sand, with a density of 1,900 kg/m^3, is chosen as the fill material. This means that a total of 28,230/1,900=14.9 m^3 sand is applied per container as fill material. The total amount of fill material can be calculated: 14.9*600=8940m^3.

Container coating

The containers have to be coated to increase their lifetime, see chapter 2.15.3, lifetime of the enclosed waste breakwater. The total area of container that has to be coated can be calculated:

$$A_{coating} = (l * w + 2 * w * h + 2 * l * h) * 2 =$$

$$(6.058 * 2.438 + 2 * 2.438 * 2.591 + 2 * 6.058 * 2.591) * 2 = 117.6\ m^2\ per\ container$$

The total amount of coating can be calculated: 117.6*600=70,560m^2.

Car wreckage dimensions

Only the compressed cars will be taken into account. It is assumed that the transmission through an uncompressed cars breakwater will be higher than the maximum allowed transmission through the breakwater. This is caused by the huge gaps within the uncompressed cars, due to the removed car windows and the irregularities of a car.

The dimensions of an average compressed car wreckage are as show in table 46, according to the National highway traffic safety administration (NHTSA).

Mass car	1200 kg
Length car	4.75 m
Width car	2 m
Height car	0.5 m

Table 46: Average car wreckage dimensions

Height of the construction
The required height of the construction is 14.2 meter and the height of a car wreckage is 0.5 meter. The total required stacked car wreckages can be determined: 14.2 / 0.5= 28.4 → this means a total of 29 layers of car wreckages are needed. The total height of the construction will therefore be: 29 * 0.5 ≈ 14.5 meter.

Required car wreckages
Per breakwater cross section, 29 car wreckages are required. The car wreckages have a length of 4.75 meter and the total required breakwater length is approximately 600 meters. With this summary, the total amount of car wreckages can be calculated:

$$Required\ car\ wreckages = \frac{Length\ breakwater}{Length\ car\ wreckage} \times Nr.car\ wreckages\ cross\ section$$

$$= \frac{600}{4.75} \times 29 \approx 3{,}683\ car\ wreckages\ (127\ stacks\ of\ 29\ high)$$

Wooden pile dimensions

Number of piles on the lee side of the breakwater
The wood used for the wooden piles is Merbau wood. Merbau wood has a maximum allowed flexural strength of 20 N/mm^2 when used as construction material according to (Blok, 2006). According to (Meier, 2014), the trunk diameter of merbau trees is between 1.2 and 1.5 meter.

The maximum occurring moment in the wooden piles on the lee side of the enclosed waste breakwater is calculated according the same theory as described in appendix IV. The values of the maximum occurring moment in the piles (with a diameter of 1.5 meter) are shown in table 47.

	Containers	Car wreckages
M_{max}	9,054.546 kNm	6,862.555 kNm

Table 47: The maximum occurring moment in the wooden piles with a diameter of 1.5 meter when applied in the enclosed waste breakwater.

The maximum moment that the Merbau piles can withstand when the piles have a diameter of 1.5 meter can be determined with the formula:

$$M_{max} = \sigma_{max} * S_m$$

Where $\sigma_{max}$is the maximum allowed flexural strength, and S_m is the section modulus of the pile.

The maximum moment can be calculated:

$$M_{max} = \sigma_{max} * \frac{\pi * d^3}{32} = 20 * 10^3 * \frac{\pi * 1.5^3}{32} = 6{,}626.8\ kNm$$

It can be concluded that the Merbau piles will fail in the situation schematised in figure 32. To make the construction strong enough, there are at least 3 piles per stack of car wreckages or containers needed on the lee side of the construction.

Required amount of wood

With the same method as used above (and using the same principia as described in appendix IV), the minimum required pile diameters and the depth in the soil are calculated for the wooden piles for both the container enclosed waste breakwater and the car wreckages enclosed waste breakwater. The results of these calculations are shown in table 48.

Enclosed waste	Containers	Car wreckages
Min. pile diameter (m)	1.5	1.4
Depth in Soil (m)	4.70	4.32

Table 48: Minimum required pile diameter and depth in soil of the wooden piles for the enclosed waste breakwater

With the number of required piles per stack of waste material and the number of stacks of waste known the total number of piles can be calculated. Table 49 shows the calculation.

Waste material	Containers	Car wreckages	
Piles needed at the ocean side	2	2	
Piles needed at the lee side	3	3	+
Total piles needed per stack of waste	5	5	
Amount of waste (stacks)	100	127	x
Total number of piles required	500	635	

Table 49: Determination of the required amount of wooden piles

Since the depth of the piles, the height of the constructions, the minimum required pile diameters and the total number of piles required are known, the total amount of wood required can be determined, see table 50.

Waste material	Containers	Car wreckages	
Depth of the piles (m)	4.70	4.32	
Height of the construction (m)	15.55	14.5	+
Total pile length (m/pcs)	20.25	18.82	
Minimum required pile diameter (m)	1.5	1.4	*
Volume of wood per pile (m^3)	35.78	28.97	
Total number of piles required	500	635	x
Total volume of wood required (m^3)	17,890	18,397	

Table 50: Determination of the total volume of wood required

* The volume of wood per pile is calculated with the surface area of the pile times the total pile length: $\pi * \left(\frac{d}{2}\right)^2 * l$

Steel pile dimensions

General
The steel used for the steel piles has a maximum flexural strength of 355 N/mm^2 and has a density of 7.8 ton/m^3. The thickness of the walls of the piles is set to 20 mm to reduce the complexity of the calculation. 20 mm is a regular wall thickness for steel tubes according to (ArcelorMittal). The thickness of the steel tubes will be increased to compensate for the occurring corrosion of the steel. This extra steel is explained in chapter 2.15.3, lifetime of the enclosed waste breakwater.

Required amount of steel
With the same method as used for the wooden piles (and using the same principia as described in appendix IV), the optimal pile diameters and the depth in the soil are calculated for the steel piles for both the container enclosed waste breakwater and the car wreckages enclosed waste breakwater. The results of these calculations are shown in table 51.

Enclosed waste	Containers	Car wreckages
Optimal pile diameter (m)	1.4	1.3
Depth in Soil (m)	5.60	5.16

Table 51: Minimum required pile diameter and depth in soil of the steel tubes for the enclosed waste breakwater

With the number of required piles per stack of waste material and the number of stacks of waste known the total number of piles can be calculated. Table 52 shows the calculation.

Waste material	Containers	Car wreckages	
Piles needed at the ocean side	2	2	
Piles needed at the lee side	2	2	+
Total piles needed per stack of waste	4	4	
Amount of waste (stacks)	100	127	x
Total number of piles required	400	508	

Table 52: Determination of the required amount of steel piles

Since the depth of the piles, the height of the constructions, the optimal pile diameters, the wall thickness of the piles and the total number of piles required are known, the total amount of steel required can be determined, see table 53.

Waste material	Containers	Car wreckages	
Depth of the piles (m)	5.60	5.16	
Height of the construction (m)	15.55	14.5	+
Total pile length (m/pcs)	21.15	19.66	
Optimal pile diameter (m)	1.4	1.3	
Wall thickness (m)	0.0234	0.0268	* **
Volume of steel per pile (m^3)	2.14	2.11	
Total number of piles required	400	508	
Density of steel (ton/m^3)	7.8	7.8	x
Total ton of steel required (ton)	6,677	8,361	

Table 53: Determination of the total amount (ton) of steel required

* The volume of steel per pile is calculated with the surface area of the pile times the total pile length: $\left(\pi * \left(\frac{d}{2}\right)^2 - \pi * \left(\frac{d-2t}{2}\right)^2\right) * l$

** The thickness of the walls is increased to compensate for the corrosion of the steel. This extra steel is explained in chapter 2.15.3, lifetime of the enclosed waste breakwater.

2.15.3 Lifetime

Lifetime of containers

The lifetime of an unprotected container wall is only 2 years, for this reason the containers have to be protected with a coating. When constructing the enclosed waste breakwater, each container has to be coasted both internal as external, in order to guarantee a lifetime of at least 10 years.

Lifetime of car wreckages

Since there is never done any research on the lifetime of compressed car wreckages in salt water, a lifetime has to be assumed. After consultation with various experts of RHDHV, the expected lifetime of the compressed car wreckages is set to 20 years.

Lifetime of wooden piles

The wood in the enclosed waste breakwater is Merbau wood. Merbau wood has a lifetime up to 10 to 25 years according to (Matbase).

Lifetime of steel piles

The lifetime of the steel piles is depending on the thickness of the walls and the corrosion rate of the steel. According to (British Standard, 2000), the maximum corrosion rate of steel is 0.17 mm/side/year. To guarantee a certain lifetime, the wall thickness of the steel piles has to be increased. For the steel piles in combination with the containers the wall thickness has to be increased with: 0.17*2*10=3.4 mm. For the steel piles in combination with the car wreckages the wall thickness has to be increased with: 0.17*2*20=6.8 mm.

2.15.4 Material costs

Unit prices
The unit prices of the materials used in the material costs calculation can be seen in table 54.

Material	Unit	Unit price	Source
Containers	€/pcs	1,000.-	Various second hand shops
Container coating	€/m^2	25.-	(Stive, 2015)
Car wreckages	€/pcs	250.-	Consultation with several colleagues of RHDHV
Sand	€/m^3	5.-	(Stive, 2015)
Wood	€/m^3	9500.-	(Stive, 2015)
Steel	€/ton	1,100.-	(Washington State Department of Transportation, 2015)

Table 54: Unit prices enclosed waste breakwater

Total material costs
In table 55 the total material costs of the enclosed waste breakwater designs can be seen. The less expensive breakwater layout is the one with the steel piles and container wreckages as enclosed waste. However, these containers may require container wall fortifications in order to prevent the container walls from breaking out of the container, which will last into too high operational conditions. The difference between the container and car wreckages enclosed waste breakwaters its costs in approximately €400,000.-. Thereby the expected lifetime of a container is only 10 years, while the lifetime of a compressed car is set to 20 years. Due to the required fortifications to prevent the walls from breaking out of the frame and the longer lifetime of the car wreckages, the car wreckages breakwater will be taken further into account in chapter 3, multiplication factors.

Piles	Wood		Steel	
Waste material	Containers	Car wreckages	Containers	Car wreckages
Material				
Containers (Pcs)	600	-	600	-
Container coating (m^2)	70,560	-	70,560	-
Car wreckages (Pcs)	-	3,683	-	3,683
Sand (m^3)	8,940	-	8,940	-
Wood (m^3)	17,890	18,397	-	-
Steel (ton)	-	-	6,677	8,361
Unit price				
Containers	€ 1,000	€ 1,000	€ 1,000	€ 1,000
Container coating	€ 25	€ 25	€ 25	€ 25
Car wreckages	€ 250	€ 250	€ 250	€ 250
Sand	€ 5	€ 5	€ 5	€ 5
Wood	€ 950	€ 950	€ 950	€ 950
Steel	€ 1,100	€ 1,100	€ 1,100	€ 1,100
Material costs				
	€ 19,406,178	€ 18,397,590	€ 9,754,109	€10,106,525

Table 55: Material costs for the enclosed waste breakwater designs

2.15.5 Complexities

When there is some spare space between the piles and the enclosed waste material, the waste can start to quiver and even so the complete construction can start to resonate. Therefore it is important that there are no spaces left between the waste material and the steel piles or that some sort of damping is installed.

2.16 Replacing core

2.16.1 Introduction

When replacing the breakwater core with a waste material like containers, tires or cars, the total breakwater costs may be reduced. An expensive part of a breakwater is the breakwater core, due to the huge surface of required rock per running meter. This chapter estimates the cost reductions when replacing the breakwater core with an alternative core material. The layout of the breakwater is equal to the rock armour layer reference breakwater, according to this layout an amount of 431 m^3 quarry run per running meter is required. This chapter assumes that the entire breakwater core can be replaced by the alternative materials; containers, tires or cars, without applying any extra armour layer material. If this concept will be determined as the best solution, a more accurate calculation will be performed.

2.16.2 Dimensions

Main breakwater dimensions
The main dimensions of the alternative core breakwater will be equal to the dimensions of the reference design breakwater. This breakwater has a length of 600 meters, a height of 15.8 meters (10.8 meters CD + HHWS + surge + sea level rise and a 5 meter high crest height). The front slope is set to 1:3 and the rear slope to 1:1.5. In this reference design, the required amount of quarry run is set to 431 m^3/m, which is equal to a total amount of 431 x 600 (breakwater length) = 258,600 m^3. For an explanation of the dimensions determination, see the reference design.

Required core material
Containers
The containers will be constructed parallel orientated to the coast, when constructing the containers parallel to the coast, a steeper slope can be achieved, which will last into less containers. The container core has to replace 431 m^3/m quarry run. The volume of a container per running meter is equal to 2.5 x 2.5 x 1 = 6.25 m^3/m. The total required containers per cross section is herewith equal to 431/6.25 = 68 containers per cross section. The total required amount of containers can be determined using the following formulae:

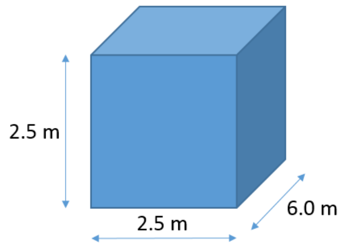


Figure 33: Dimensions container

$$Nr\ containers = \frac{Length\ breakwater}{Length\ single\ container} \times Containers\ per\ cross\ section$$

In total 6735 containers are required in order to replace the quarry run core with containers.

Tires

The dimensions of an average tire are determined as see in figure 34. The volume of a single tire is equal to 0.0786 m^3. The tires have to replace 431 m^3 of quarry run, which is equal to 431/0.0786 = 5483 tires per running meter. The total required tires can be determined by:

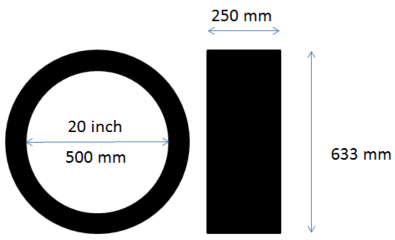


Figure 34: Average dimensions tire

$$Nr\ tires = \frac{Length\ breakwater}{Length\ single\ tire} \times Tires\ per\ cross\ section$$

A total of 3,285,600 tires are required in order to replace the quarry run core with old car tires.

Cars

The dimensions of an average car are shown in table 56 (NHTSA). In the analysis of the car wreckages breakwater, only the compressed cars will be taken into account. It is assumed that the transmission through an uncompressed cars breakwater will be higher than the maximum allowed transmission through the breakwater. This is caused by the huge gaps within the uncompressed cars, due to the removed car windows and the irregularities of a car. Therefore, only the compressed car will be taken into account.

Mass car	1200 kg
Length car	4.75 m
Width car	2.00 m
Height car	1.75 m
Height compressed car	0.5 m

Table 56: The dimensions of an average car

The cars will be constructed parallel orientated to the coast, when constructing the cars parallel to the coast, a steeper slope can be achieved, which will last into less cars. The car core has to replace 431 m^3/m quarry run. The volume of a car per running meter is equal to 0.5 x 2.0 x 1 = 1.0 m^3/m. The total required cars per cross section is herewith equal to 431/1 = 431 cars per cross section. The total required amount of cars can be determined using the following formulae:

$$Nr\ cars = \frac{Length\ breakwater}{Length\ single\ car} \times Cars\ per\ cross\ section$$

A total of 54,316 cars are required in order to replace the quarry run core with old cars.

2.16.3 Lifetime

The lifetime of an unprotected container is set to 2 years, according to (3werf, 2000), see chapter 2.10, container pyramid.

The compressed car units can be schematised as steel bars, whit a high porosity. The steel parts of the compressed cars will corrode during the breakwater its lifetime. Thereby, deformations of the breakwater are expected caused by the wave forces. Due to this corrosion and these deformations, the lifetime of the breakwater is expected to 20 years (based on a rough assumption).

It is assumed that rubber is not affected by salt water. For this reason, the durability of a single tire in salt water is assumed to be at least 50 years, which is equal to the durability of the reference rubble mound breakwater.

2.16.4 Material costs

Unit prices

The unit price of the quarry run can be found in the reference design. The unit price of the container and the container fill can be found in chapter 2.10, container pyramid. The unit price of the tires can be found in chapter 2.9, tires. The unit price of the car and car cleaning can be found in chapter 2.8, car wreckages.

Material	Unit	Unit price	Source
Quarry run	€/m^3	40.-	Reference design
Container	€/unit	1,000.-	Second hand shops
Container fill	€/m^3	5.-	(Stive, 2015)
Tire	€/unit	2.-	Second hand shops
Car	€/unit	150.-	Second hand shops
Car cleaning	€/unit	100.-	Cleaning companies

Table 57: Unit costs replaced core breakwater

Total material costs

The total material costs of the reference breakwater are equal to € 19,431,803. - see reference design, this is including the quarry run core. The total material costs for the armour layer is equal to € 19,431,803 - €40 (cost quarry run) x 258,600 m^3 (volume quarry run) = € 9,087,803.-. The costs for the rock armour layer is set to € 9,087,803.-.

When adding the material costs of the core replacing to the armour layer material costs, the total material costs can be calculated, see table 58.

	Containers	Tires	Cars
Armour layer	€ 9,087,803.-	€ 9,087,803.-	€ 9,087,803.-
Containers	6735		
Tires		3285600	
Cars			54316
Material costs			
	€ 15.860.014,86	€ 15.662.539,86	€ 22.670.339,86

Table 58: Construction costs replaced core breakwater

The total material costs of the tire and the container core, are quite close to each other. However, fortifications for both of the constructions are required (see chapter 2.10, container pyramid and chapter 2.9, tires). It is assumed that the multiplication factor applied to cover these fortifications is equal for both the container and the tire replacements. Thereby, the lifetime of a tire is assumed to be longer than the lifetime of a container. For the above mentioned reasons, only the tire core breakwater will be taken into account in chapter 3, multiplication factors.

2.16.5 Complexities

The biggest unknown for the tire core breakwater is the extra required amount of amour layer material. Due to the gaps in the tires, some extra armour layer material may be required, caused by the losses of rock in these gaps. Thereby, the tires may subsidence through the weight of the armour layer. These two construction risk will be added into a multiplication factor in chapter 3.

2.17 Piles and tires

2.17.1 Introduction

The piles and tire breakwater alternative is a combination of vertical piles encased by old car tires. The purpose of the car tires is to reduce the required amount of vertical piles. When less vertical piles are required, the total construction costs of the breakwater are estimated to be reduced. The piles can be constructed using steel or wood. The materials bamboo and concrete will not be taken into account in this chapter. It is assumed that both of the materials are not able to withstand the occurring horizontal wave load forces without huge fortifications and will become too expensive for this reason.

2.17.2 Dimensions

Main breakwater dimensions

The required height, from the top of the subsoil to the top of the construction is set to 10.8 m (CD + HHWS + surge + sea level rise, see basis of design) + 3.4 m (required crest height according to the vertical wall overtopping calculation, mentioned in appendix II) = 14.2 meters. The breakwater length is dependent of the transmission though the breaker, the higher the transmission coefficient, the more breakwater length is required (see transmission/length diagram in the basis of design).

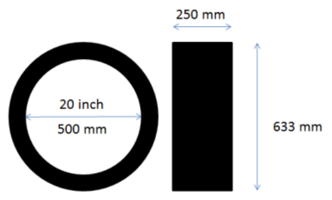


Figure 35: Tire dimensions

Dimensions tire

There are many different tire sizes available, which results into an uncertainly when determining the dimensions of a single tire. The dimensions of the tires used are schematised in figure 35. These dimensions are assumed to be the average tire dimensions.

The tire and piles breakwater will enable some water to flow though the breakwater, caused by the gaps between two tires. When a higher tolerance of variation in tire dimension is permitted, more waves will be able to flow through the breakwater caused by more gaps in the wave reducing construction.

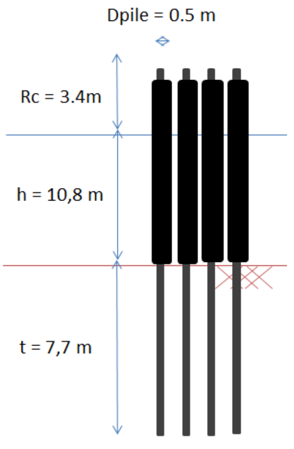


Figure 36: Schematic pile and tire breakwater

Dimensions piles

When determining the required pile length, the required piling depth of the vertical piles is set to 7.7 meters, according to the calculation mentioned in appendix II. The diameter of the piles is set equal to the diameter of the average tire (0.633), since the piles have to withstand the forces which occur at the entire piles. The maximum pile diameter is set to 0.5 meters, this diameter is equal to the internal diameter of the tire, in practice, the diameter of the piles will have to be reduced, in order to achieve a certain placing margin of the tires over the piles.

When checking the piles at bending stresses, the wooden pile with a diameter of 0.5 meters, are not able to withstand the occurring bending forces which are occurring at the construction, for this reason the wooden piles will not be taken into account further in this design.

When determining the required wall thickness of the steel piles, using the maximum bending stress of steel (sigma), the minimum required wall thickness is equal to 0.1 meters.

Required piles

When constructing the vertical piles and tire breakwater, a small spacing between the entire pile and tire diameter is required in order to be sure that each single tire fits around the piles, without hitting the tires of the pile next to it. This minimal required spacing between the piles including the tires is set to 0.02 meters. This will last into a maximal tire diameter of 650 mm.

When tires with the same dimensions are applied and it is assumed that 10% of the entire vertical wall consists of gaps. These gaps are caused by rounding of the tires and by irregularities of the tire stacks.

When summing both of these two irregularities up, the total expected transmission through the piles and tires breakwater is assumed to be equal to, based on the rule of thumb mentioned in the vertical piles breakwater concept:

$$C_t = \frac{A}{B} + Irreculatitities\ tires = \frac{0.02}{0.65} + 0.1 = 0.13$$

According to the chapter vertical pile pyramid, the real occurring transmission through the breakwater is approximately three times higher than the transmission which has been determined using the rule of thumb. The total estimated transmission though the breakwater is herewith 0.39. The maximum allowed transmission though the breakwater, according to the transmission length diagram in the basis of design is set to 0.3. In order to achieve this transmission coefficient of 0.3, multiple pile rows have to be performed.

Each 100% pile row multiplication will result into a 18% lower transmission coefficient. When applying two pile rows, the total transmission coefficient is 0.32, which is higher than 0.3, for this reason, 4 pile rows are required in order to construct the vertical pile and tire breakwater, the transmission coefficient for this breakwater is 0.26, when reducing the transmission factor a second time.

The required breakwater length with a transmission factor of 0.26, is set to 640 meters, according to the transmission length diagram mentioned in the basis of design. The total required piles per row is equal to 640/0.65 (pile width + placing margin) = 985 piles.

Required steel and tires

The weight of a single steel pile is 2.6 ton (external diameter 0.5 meter, internal diameter 0.48 meter and a pile length of 21.9 meters, multiplied with the relative density of steel 7.8 ton/m^3).

The total pile height above the subsoil level is 14.2 meters; the height of a tire is 0.25 meters. When dividing the required height with the height of a tire, 57 stacked tires per pile are required, in order to fill the entire pile with tires. When multiplying this amount of tires with the total required piles per pile row, 56.145 tires per pile row are required.

2.17.3 Lifetime

The lifetime of the steel piles is depending on the thickness of the walls and the corrosion rate of the steel. According to (British Standard, 2000), the maximum corrosion rate of steel is 0.17 mm/side/year. To guarantee a certain lifetime, the wall thickness of the steel piles has to be increased. For the steel piles the wall thickness has to be increased with: 0.17*2*50= 17mm.

It is assumed that rubber is not affected by salt water. For this reason, the durability of a single tire in salt water is assumed to be at least 50 years, which is equal to the durability of the reference rubble mound breakwater.

2.17.4 Material costs

Unit prices
The unit price of a single tire is set to € 2.-. According to different second hand tire shops, the costs of a second hand tire are approximately € 10.-. However, the tires which are applied in the tire breakwater do not have to fulfil any requirements which are required in order to achieve a certain safety on the roads. For this reason, the costs of a scrap tire is set to € 2.-.

The steel price is based on the unit price of steel of (WSDOT, 2015).

Material	Unit	Unit price	Source
Tire	€/unit	€ 1,000.-	Second hand shops
Steel	€/ton	€ 1,100.-	(WSDOT, 2015)

Table 59: Unit prices piles and tires breakwater

Total material costs
In table 60, the total material costs of the piles and tires breakwater can be seen.

Required material	
Weight pile (ton)	2.6
Piles required (row)	985
Tires required	56,145
Required rows	4
Material costs	
Unit price tire	€ 2
Unit price steel	€ 1,100.-
Material costs	€ 11,845,715.-

Table 60: Material costs piles and tires

2.17.5 Complexities

The most complex part when constructing the piles and tires breakwater, is putting the tires over the vertical piles, due to the low relative density of the tires, the tires will not sink very fast. Thereby, if the tires will turn over when sinking to the bottom, the tires will stick to the vertical pile and will not sink any further.

A second complexity when constructing this breakwater is the required tire separation. In order to reduce the transmission through the breakwater, the tires requires as less irregularities as possible, in order to avoid gaps in the tire breakwater.

3 MULTIPLICATION FACTORS

3.1 Introduction

The total costs of building a (alternative) breakwater consist of more costs than the material costs alone. To estimate the total costs of the breakwater alternatives, the total material costs are multiplied by three different factors: the construction factor, the complexity factor and the indirect costs contractor factor. The costs each of these factors covers is explained in the next three paragraphs. These multiplication factors are only designed in order to achieve a certain representation of the entire construction costs. Each factor has a bandwidth of +/- 0.1, but cannot be lower than 1.

Construction factor
The construction factor is a compensation for:
- The costs for the used equipment.
- The costs for the placement or installation of construction parts.
- The costs for transportation of construction parts.
- The costs for preparation work on construction parts at the quay.
- Other constructing related costs.

Complexity factor
The complexity factor is a compensation for:
- Complex construction parts which are expected to cost more than calculated.
- Construction parts of which the costs are unknown.

Indirect costs contractor factor
The indirect cost contractor factor is a compensation for the contractors cost for:
- The survey costs.
- Engineering costs.
- Construction site and site office costs.
- Some unexpected expense.
- Profits.
- Other contractor related costs.

The indirect costs contractor factor is in general set to 1.35 after consultation with (Stive, 2015). Only the indirect costs contractor of the old ships is different, the reason why is explained in chapter 3.3.12.

3.2 Construction costs alternatives

Table 61 shows an overview of the material costs of the alternatives, the applied compensation factors and the total construction costs. In chapter 3.3 the chosen multiplication factors are explained per breakwater alternative.

	Alternative	Material costs	Construction factor	Complexity factor	Indirect costs contractor factor	Total costs
1	Geo-containers	€ 16.810.416	1.1	1.2	1.35	€ 29,956,161
2	Refraction islands	€ 18,696,810	1.3	1.2	1.35	€ 39,375,482
3	Vertical Piles	€ 14,504,733	1.3	1.2	1.35	€ 30,546,968
4	Pyramid of piles	€ 16,794,954	1.3	1.2	1.35	€ 35,370,173
5	Jacket breakwater	€ 20,981,460	1.2	1	1.35	€ 33,989,965
6	Slab with support	€ 10,095,900	1.2	1.1	1.35	€ 17,990,894
7	Bellows structure	€ 30,728,428	1.2	1.1	1.35	€ 54,758,059
8	Car wreckages	€ 43,958,000	1.1	1	1.35	€ 65,277,630
9	Tires	€ 19,200,000	1.2	1.2	1.35	€ 37,324,800
10	Container pyramid	€ 9,000,555	1.3	1.8	1.35	€ 28,432,752
11	Container wall with struts	€ 21,422,158	1.3	1.2	1.35	€ 45,115,064
12	Old ships	€ 20,000,000	1	1	1.1	€ 22,000,000
13	Vertical tubes with band	€ 8,816,404	1.4	1.2	1.35	€ 19,995,604
14	Sandwich breakwater	€ 62,130,000	1.1	1.2	1.35	€ 110,715,660
15	Enclosed waste	€10,106,525	1.3	1.2	1.35	€ 21,284,342
16	Replacing core	€ 15,662,540	1.3	1.2	1.35	€ 32,985,309
17	Piles and tires	€ 11,845,715	1.3	1.2	1.35	€ 24,947,075

Table 61: Overview of the material costs, compensation factors and the total costs of the alternatives

3.3 Multiplications per breakwater concept

3.3.1 Geo-containers

Construction factor (1.1)
The construction factor for the geo-containers breakwater is set to 1.1, because of the relative easy placement of the geo-tubes. The geo-tubes are placed using a ship or a barge that can open its hull to release the geo-tubes.

Complexity factor (1.2)
The complexity factor of the geo-containers breakwater is set to 1.2, because of the difficulty to achieve the required density in the geo-tubes and the tolerances of placement of the geo-containers.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.2 Refraction islands

Construction factor (1.3)
The construction factor for the refraction islands is set to 1.3, because of the difficulty to create the islands. The refraction islands are constructed in the same way as the reference design. Therefore the same construction factor is applied.

Complexity factor (1.2)
The complexity factor of the refraction islands is set to 1.2, because the calculations for the refraction islands are calculated using very positive starting points.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.3 Vertical Piles

Construction factor (1.3)
The construction factor for the vertical piles breakwater is set to 1.3, because of the piling of the vertical piles and the very close placement of the piles.

Complexity factor (1.2)
The complexity factor for the vertical piles breakwater is set to 1.2, because of the very severe construction tolerances. The risk that one of the piles is constructed under only a small angle is very present.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.4 Pyramid of Piles

Construction factor (1.3)
The construction factor for the pyramid of piles is set to 1.3, because the piles have to be connected together. Making the connection between the piles is labour-intensive. Besides, connecting the piles together means that the complete pyramid has to be placed in one piece instead of tube by tube. In order to do so, cranes with very big lifting capacity are required. Thereby, probably a jacked up vessel is required to place the pyramid due to the swell waves at the project location.

Complexity factor (1.2)
The complexity factor of the pyramid of piles is set to 1.2, because of the complexity of connecting the piles together in combination with the risk of sliding or overturning.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.5 Jacket Breakwater

Construction factor (1.2)
The construction factor of the jacket breakwater is set to 1.2, because the jacket construction has to be welded together. Thereby, the jacket construction has to be transported to the project location and placed in the swell waves. The unit price of the foundation required, includes the application of the foundation, therefore these costs are not included in the construction factor.

Complexity factor (1.0)
The complexity factor for the jacket breakwater is set to 1.0, because there are no risks that are expected to increase the total cost significantly.

Indirect costs contractor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.6 Slab with Support

Construction factor (1.2)
The construction factor for the slab with support breakwater is set to 1.2, because of the required equipment for placing the elements and the prefabrication of the elements. However, the backfill is placed after the elements, so the elements already have a wave braking effect and thus relative simple vessels can be used for placing the containers.

Complexity factor (1.1)
The complexity factor for the slab with support breakwater is set to 1.1, because of the risk of liquefaction due to the placement of the pre-fabricated concrete elements and the risk that the elements may start to slide or to turn over.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.7 Bellows Structure

Construction factor (1.2)
The construction factor for the bellows structure is set to 1.2, because of the required construction method for the foundation and the construction of the rubber blanket within this foundation

Complexity factor (1.1)
The complexity factor for the bellows structure is set to 1.1, because of the chance of failure of the rubber blanket.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.8 Car Wreckages

Construction factor (1.1)
The construction factor for the car wreckages breakwater is set to 1.1, because the car wreckages can be placed directly in the water. There is no special jacked up vessel required, the car wreckages can just be thrown in the water and they will sink to their position.

Complexity factor (1.0)
The complexity factor for the car wreckages breakwater is set to 1.0, because there are no risks that are expected to increase the total cost significantly.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.9 Tires

Construction factor (1.2)
The construction factor for the tires breakwater is set to 1.2, because of the tire connection required. The tire connection has to be strong enough to last the entire lifetime of the breakwater. Applying this connection is very time consuming and labour intensive.

Complexity factor (1.2)
The complexity factor for the tires breakwater is set to 1.2, because the stability of the entire construction, it may be too light and may start to slide towards the jetty. Thereby, there is a chance the tire connection will fail.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.10 Container Pyramid

Construction factor (1.3)
The construction factor for the container pyramid breakwater is set to 1.3, because it is very difficult to place the containers is their exact position in the swell waves. To place the containers, a jacked up vessel is needed in the swell waves. Even when a jacked up vessel is used, it is assumed very difficult to place the containers in their exact location.

Complexity factor (1.8)
The complexity factor for the container pyramid breakwater is set to 1.8, because of the fortifications needed in the containers. When the containers are fortified for a unit price of €1,000.-/container, the total construction costs (with a complexity factor of 1.0 instead of 1.8) will be equal to the current calculated construction costs.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.11 Container Wall with Struts

Construction factor (1.3)
The construction factor for the container wall with struts breakwater is set to 1.3, because the containers have to be connected to each other before they are placed in the water, since centric stacking is assumed to be not possible in the swell waves. This results in a big construction, which is quite hard to handle.

Complexity factor (1.2)
The complexity factor of the container wall with struts breakwater is set to 1.2, because of the uncertainty in the required container protection. The exact amount of required protection is not determined yet, only a rough estimation is made. The exact amount of required protection may be higher than the estimated value.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.12 Old Ships

Construction factor (1.0)
The construction factor for the old ships breakwater is set to 1.0, because all the relative construction costs are taken into account in the material costs, according to (Bosman, 2015).

Complexity factor (1.0)
The complexity factor for the old ships breakwater is set to 1.0, because there are no risks that are expected to increase the total cost significantly.

Indirect costs contractor factor (1.1)
The indirect costs contractor factor for the old ships breakwater is set to 1.1 instead of 1.35 as for the rest of the alternatives. This is done because the contractor only has survey costs and profits to be taken into account, all the other costs are covered in the material costs of the old ships breakwater. Thereby, there are no construction site and site offices costs at all.

3.3.13 Vertical Tubes with Band

Construction factor (1.4)
The construction factor for the vertical tubes with band breakwater is set to 1.4, because of the piling of the vertical tubes, the construction of the conveyor belt with steel cord in transverse direction instead of the longitudinal direction and the connection of the band with the piles.

Complexity factor (1.2)
The complexity factor for the vertical tubes with band breakwater is set to 1.2, because of the risk of placement tolerances of the vertical piles. When they are placed too far apart, or too close to each other, it may be difficult to construct the conveyor belt in between them.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.14 Sandwich Breakwater

Construction factor (1.1)
The construction factor for the sandwich breakwater is set to 1.1, because of the application of the enormous amount of sand.

Complexity factor (1.2)
The complexity factor for the sandwich breakwater is set to 1.2, because of the unknown costs of the fixed constructions. The costs of the fixed constructions are not known because they are not designed yet due to the high material costs of the sand alone.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.15 Enclosed Waste

Construction factor (1.3)
The construction factor for the enclosed waste breakwater is set to 1.3, because of the piling of the vertical piles. There are a lot of piles required, so the transport costs will be relative high. Besides, the piles and enclosed waste have to be placed in the swell waves, which is almost not feasible without the use of a jacked up vessel.

Complexity factor (1.2)
The complexity factor for the enclosed waste breakwater is set to 1.2, because of the risk of placement tolerances of the vertical piles. When they are placed too far apart, the complete construction can start to resonate. When the vertical piles are place to close to each other, the enclosed waste doesn't fit between the piles.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.16 Replacing Core

Construction factor (1.3)
The construction factor for the replacing core alternative is set to 1.3, because the same construction method is used as for the reference design.

Complexity factor (1.2)
The complexity factor for the replacing core alternative is set to 1.2, because it is expected that some of the filter and armour layer material will "disappear" in the gaps in the tires and because the tires may be compacted due to the load of the filter and armour layer.

Indirect costs contractor factor (1.35)
The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.3.17 Piles and Tires

Construction factor (1.3)
The construction factor for the piles and tires breakwater is set to 1.3, because of the piling needed for the vertical piles and the sorting of the tires by size. The piling in the swell waves is quite difficult work, therefore probably a jacked up vessel is required. The sorting of the tires by size is a very labour intensive work.

Complexity factor (1.2)
The complexity factor for the piles and tires breakwater is set to 1.2, because of the placing tolerances of the piles and the risks of the sinking tires. The risk that one of the piles is constructed under only a small angle is very present. Thereby, the tires sink very slowly because of the low relative density. When one of the tires sinks a bit under an angle, the tire may get stuck and the pile will not be completely filled with tires.

Indirect costs contractor factor (1.35)

The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

The indirect costs contractor factor is set to 1.35 after consultation with (Stive, 2015), like described in chapter 3.1, introduction. This is a compensation for the survey costs, the engineering costs, the construction site and site offices costs, some unexpected expenses and profits.

3.4 Accuracy costs determination

To estimate the accuracy of the costs determination, all the multiplication factors are increased and decreased by 0.1 (the factors are not decreased if that results in a factor smaller than 1). Table 62 shows an overview of the bandwidth of the costs determination.

	Alternative	Calculated construction costs in €	Factors +0.1		Factors -0.1*	
			Construction costs in €	Difference in %	Construction costs in €	Difference in %
1	Geo-containers	€ 29,956,161	€ 38,025,161	+27%	€ 23,114,322	-23%
2	Refraction islands	€ 39,375,482	€ 49,340,881	+25%	€ 30,849,736	-22%
3	Vertical Piles	€ 30,546,968	€ 38,277,991	+25%	€ 23,932,810	-22%
4	Pyramid of piles	€ 35,370,173	€ 44,321,884	+25%	€ 27,711,675	-22%
5	Jacket breakwater	€ 33,989,965	€ 43,505,057	+28%	€ 28,849,508	-15%
6	Slab with support	€ 17,990,894	€ 22,836,926	+27%	€ 13,881,863	-23%
7	Bellows structure	€ 54,758,059	€ 69,507,704	+27%	€ 42,251,588	-23%
8	Car wreckages	€ 65,277,630	€ 84,135,612	+29%	€ 54,947,500	-16%
9	Tires	€ 37,324,800	€ 47,049,600	+26%	€ 29,040,000	-22%
10	Container pyramid	€ 28,432,752	€ 34,715,139	+22%	€ 22,951,414	-19%
11	Container wall with struts	€ 45,115,064	€ 56,533,074	+25%	€ 35,346,560	-22%
12	Old ships	€ 22,000,000	€ 29,040,000	+32%	€ 20,000,000	-9%
13	Vertical tubes with band	€ 19,995,604	€ 24,928,383	+25%	€ 15,759,323	-21%
14	Sandwich breakwater	€ 110,715,660	€ 140,538,060	+27%	€ 85,428,750	-23%
15	Enclosed waste	€ 21,284,342	€ 25,741,095	+25%	€ 16,094,281	-22%
16	Replacing core	€ 32,985,309	€ 41,333,443	+25%	€ 25,843,191	-22%
17	Piles and tires	€ 24,947,075	€ 31,260,841	+25%	€ 19,545,429	-22%

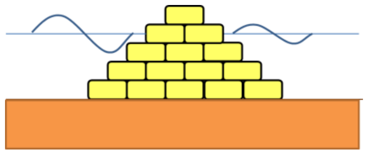
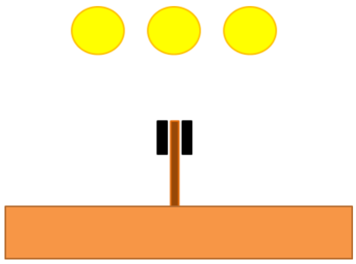
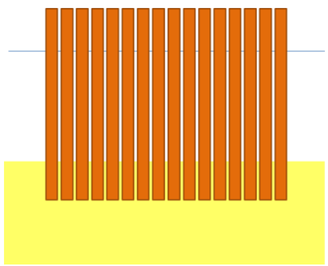
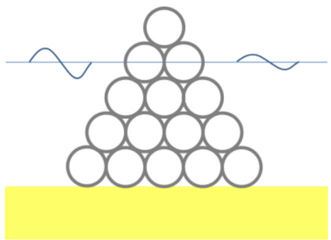
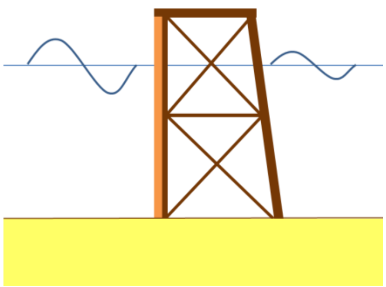
Table 62: Bandwidth of the costs determination

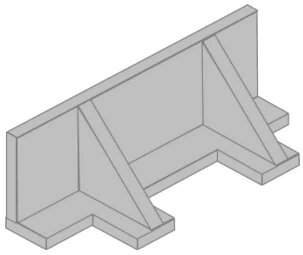
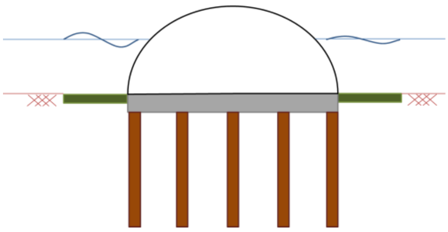
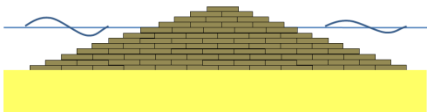
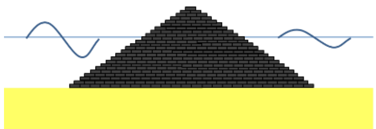
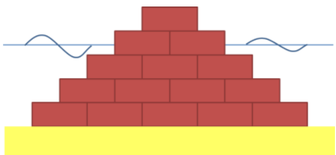
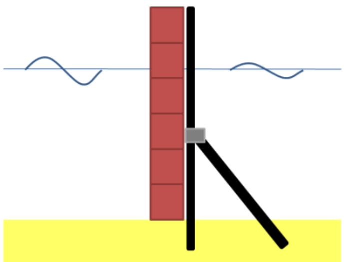
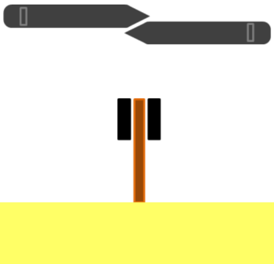
* The factors are not decreased if that results in a factor smaller than 1.

From table 62 can be concluded that the total bandwidth of the costs are between +32% and – 23%. However, the ranges of the costs are different and smaller for each separate alternative. For the alternative that turns out to be the most favourable in the MCA, the costs will be determined more accurate.

4 CONCLUSION

Table 63 shows an overview of the alternatives of the short list. The alternatives will proceed to the Multi Criteria Analysis (MCA) if the total construction costs are lower than €25,000,000.-.

Pictogram	Alternative		Construction costs	Minimum lifetime	Proceed to MCA
	1	Geo-containers	€ 30,000,000.-	≤ 40 years	NO
	2	Refraction islands	€ 39,400,000.-	≤ 50 years	NO
	3	Vertical piles	€ 30,500,000.-	± 20 years	NO
	4	Pyramid of pile	€ 35,400,000.-	≤ 50 years	NO
	5	Jacket breakwater	€ 34,000,000.-	± 50 years	NO

Pictogram	Alternative		Construction costs	Minimum lifetime	Proceed to MCA
	6	Slab with support	€ 18,000,000.-	≤ 25 years	YES
	7	Bellows structure	€ 54,800,000.-	± 50 years	NO
	8	Car wreckages	€ 65,300,000.-	± 20 years	NO
	9	Tires	€ 37,300,000.-	≤ 50 years	NO
	10	Container pyramid	€ 28,400,000.-	≤ 20 years	NO
	11	Container wall with struts	€ 45,100,000.-	≤ 10 years	NO
	12	Old ships	€ 22,000,000	± 48.5 years	YES

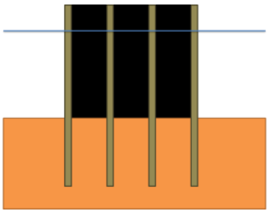
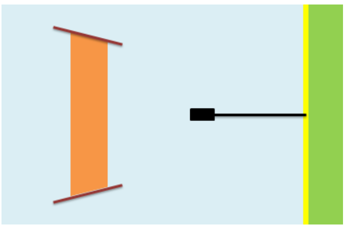
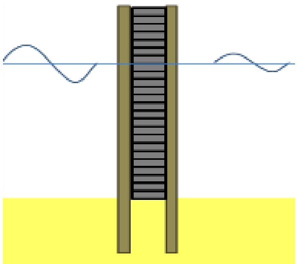
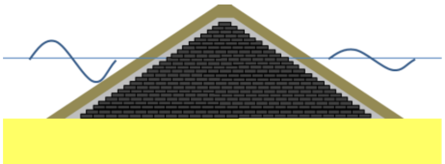
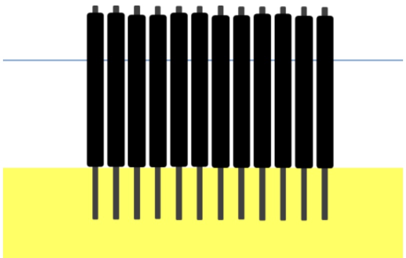
Pictogram	Alternative		Construction costs	Minimum lifetime	Proceed to MCA
	13	Vertical tubes with band	€ 20,000,000.-	± 50 years	**YES**
	14	Sandwich breakwater	€ 110,700,000.-	Variable	**NO**
	15	Enclosed waste	€ 20,500,000.-	≤ 10 years	**YES**
	16	Replacing core	€ 33,000,000.-	≤ 50 years	**NO**
	17	Piles and tires	€ 25,000,000	± 50 years	**YES**

Table 63: overview of the alternatives

When the alternatives that will not proceed to the MCA are filter out, the alternatives shown in table 64 remain.

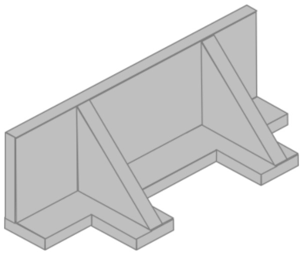
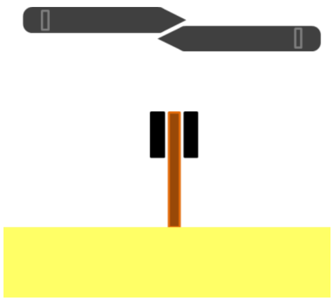
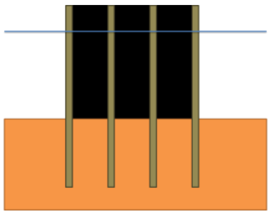
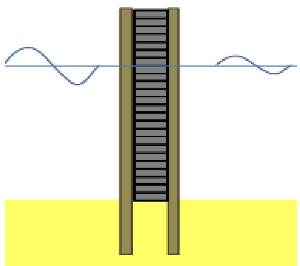
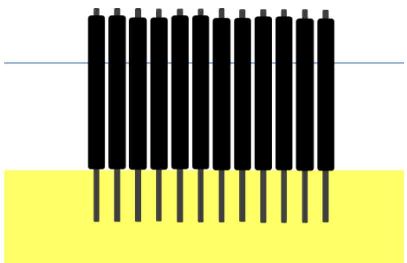
Pictogram	Alternative		Construction costs	Minimum lifetime	Proceed to MCA
	6	Slab with support	€ 18,000,000.-	≤ 25 years	**YES**
	12	Old ships	€ 22,000,000	± 48.5 years	**YES**
	13	Vertical tubes with band	€ 19,300,000.-	± 50 years	**YES**
	15	Enclosed waste	€ 20,500,000.-	≤ 10 years	**YES**
	17	Piles and tires	€ 25,000,000.-	± 50 years	**YES**

Table 64: The alternatives that proceed to the MCA

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6 APPENDIX I: SCHEMATIC CALCULATION CONTAINER PYRAMID

6.1 Introduction

In this appendix, the buoyancy force of a single container will be checked, with and without fill of sand or water. After this buoyancy calculation, the required amount of containers is determined, using breakwater slopes comparable to the reference design rubble mound breakwater. Thereby, the required amount of container protection is schematically calculated. Finally, the strength of the bottom layer of containers is checked. The following container dimensions are taken into account for determining the above dimensions, see table 65

Boundary	Description	Value
L_{con}	Length container	6.06 m
W_{con}	Width container	2.44 m
h_{con}	Height container	2.59 m
M_{emty}	Weight empty container	2,250 kg
M_{max}	Maximum weight + fill container	30,480 kg
$M_{fill,max}$	Maximum weight fill	28,230 kg
h_{ref}	Water level reference level	9.5 m
h_{sls}	High water level operational conditions	10.25 m
h_{uls}	High water level normative storm conditions	10.75 m
ρ_{wat}	Density of seawater	1020 kg/m^3
ρ_{sand}	Density of sand	2650 kg/m^3
g	Gravitational constant	9.78 N/kg

Table 65: Boundary conditions calculation

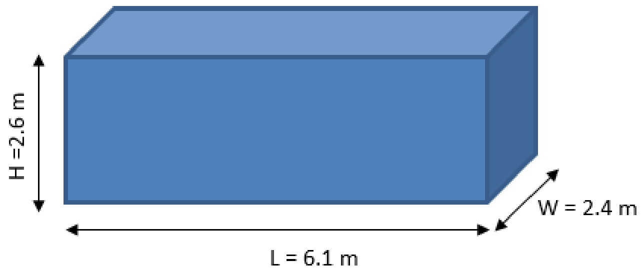

Figure 37: Dimensions container

6.2 Buoyancy force container

The buoyancy force of a single container can be determined using the following formula:

$$F_{buo} = V_{con} \times \rho_{wat} \times g$$

Where:

F_{buo}	Buoyancy force of the container	N
V_{con}	Volume of the container (L x W x h)	38.3 m^3
ρ_{wat}	Density of seawater	1020 kg/m^3
g	Gravitational constant	9.78 N/kg

$$F_{buo} = 381{,}742\ N = 381.7\ kN$$

The gravitational force of the container can be calculated using the following formulae:

$$F_g = M \times g$$

Where:

F_g	Gravitational force container	N
M_{emty}	Mass empty container	2,250 kg
M_{max}	Maximum weight container + fill	30,480 kg
g	Gravitational constant	9.78 N/kg

$$F_{g,Empty} = 22{,}005\ N = 22.0\ kN$$

$$F_{g,Filled} = 298{,}094\ N = 298.1\ kN$$

The buoyancy force of a single container is 381.7 kN, while the gravitational container forces are 22.0 kN for an empty container and 298.1 kN for a filled container. For this reason it can be concluded that a container will start floating for both the empty as the filled situation.

A single container will not remain standing when filled with the maximum allowed payload. If the maximum container weight will be exceeded, the container bottom will collapse when the container is lifted. When the containers are filled with more weight, a more complex construction method has to be developed. Developing a more complex construction method is more expensive than a regular construction method; therefore the maximum allowed filled weight will be taken into account.

When filling the container with water, instead of air, the gravitational forces of a single container will increase. The maximum fill weight is equal to 28.230 kg. When filling the container with sand, having a relative density of 2650 kg/m^3 an amount of 10.65 m^3 sand is required in order to achieve the fill weight. The internal dimensions of the container are shown in table 66.

Boundary	Description	Value
$L_{con,in}$	Inside length container	5.895 m
$W_{con,in}$	Inside width container	2.350 m
$h_{con,in}$	Inside height container	2.392 m

Table 66: Internal dimension container

The internal volume is equal to 33.14 m^3, which makes the volume of internal air equal to 33.14-10.65 = 22.48 m^3. When filling the empty area with water, the extra container weight caused by the water will be equal to 22.48 x 1020 (relative density water) x 9.78 (gravitational constant)/1000 = 225 kN per container. The total weight of a single container filled with water is equal to 523 kN. The buoyancy force is 382 kN, so the container will sink if it is filled with sand and water.

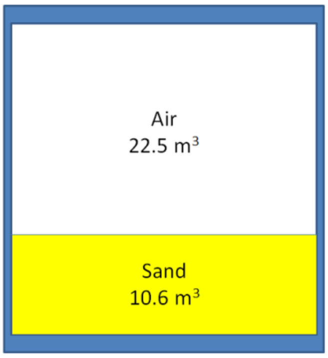

Figure 38: Air/Sand distribution container

6.3 Required containers

6.3.1 Introduction

The container pyramid will be checked using three different container layouts, a layout parallel to the coast, a perpendicular to the coast layout and a combination of perpendicular and parallel to the coast containers. The required crest height of the breakwater is based on the reference design rubble mound breakwater with a rock armour layer and set to approximately 5 meters. In total 6 container stacks are required in order to achieve the 10.75 meter water depth and approximately 5 meter crest height.

6.3.2 Parallel layout

The parallel layout consists of a 1:3 front slope and a 1:1 rear slope. In total 78 containers per cross section are required in order to perform the parallel container layout. The length of a single container is 6.06 meters. This container orientation requires the following amount of containers:

$$Required\ containers = \frac{Length\ breakwater}{Length\ container} \times Nr.containers\ cross\ section$$

$$Required\ containers = \frac{600}{6.06} \times 78 \approx 7730\ pieces$$

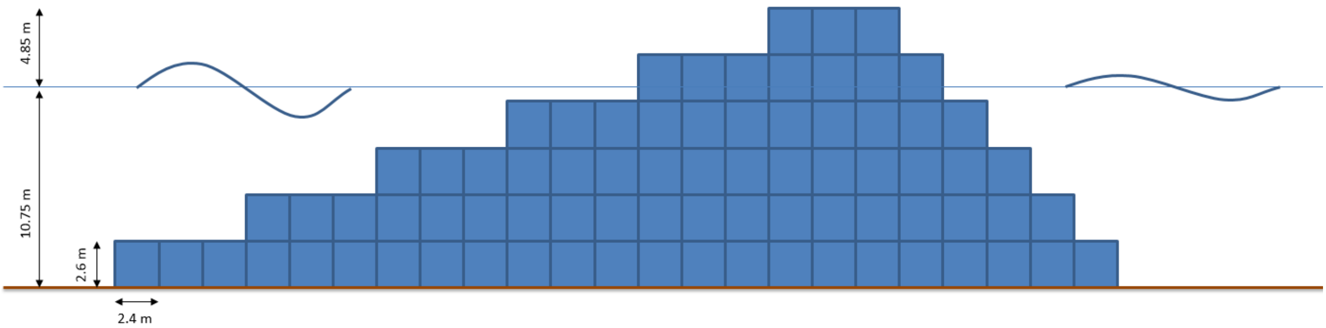

Figure 39: Parallel container layout

6.3.4 Perpendicular layout

The perpendicular container layout consists of a 1:2.5 front slope and a 1:2.5 rear slope. In total 36 containers per cross section are required in order to perform the parallel container layout. The width of a single container is 2.43 meters. This container orientation requires the following amount of containers:

$$Required\ containers = \frac{Length\ breakwater}{Width\ container} \times Nr.containers\ cross\ section$$

$$Required\ containers = \frac{600}{2.43} \times 36 \approx 8860\ pieces$$

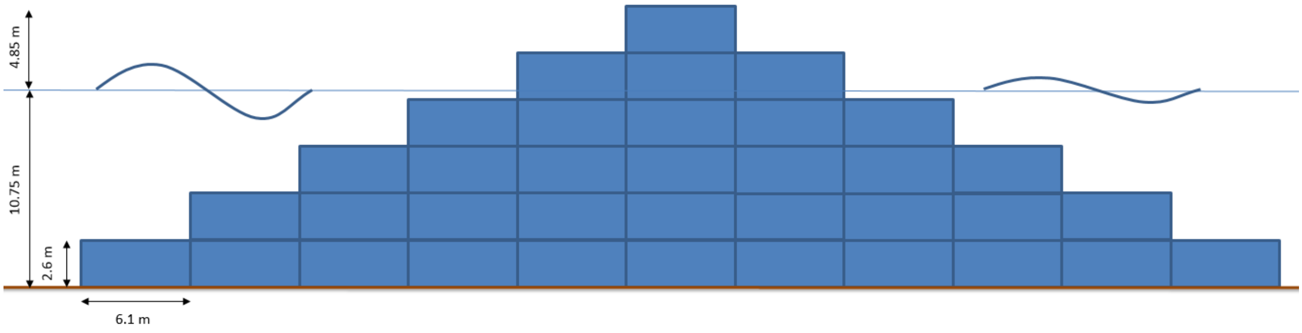

Figure 40: Perpendicular container layout

6.3.5 Perpendicular and parallel layout

The perpendicular and parallel container layout consists of a 1:2.5 front slope and a 1:1 rear slope. In total 21 perpendicular facing containers and 15 parallel facing containers per cross section are required in order to perform the parallel container layout. The width of a single container is 2.43 meters and the length of a single container 6.06. This container orientation requires the following amount of containers:

$$Required\ containers = \left(\frac{Nr.containers\ perp.}{Width\ \ container} + \frac{Nr.containers\ par.}{Length\ \ container}\right) \times L.breakwater$$

$$Required\ containers = \left(\frac{21}{2.43} + \frac{15}{6.06}\right) \times 600 \approx 6660\ pieces$$

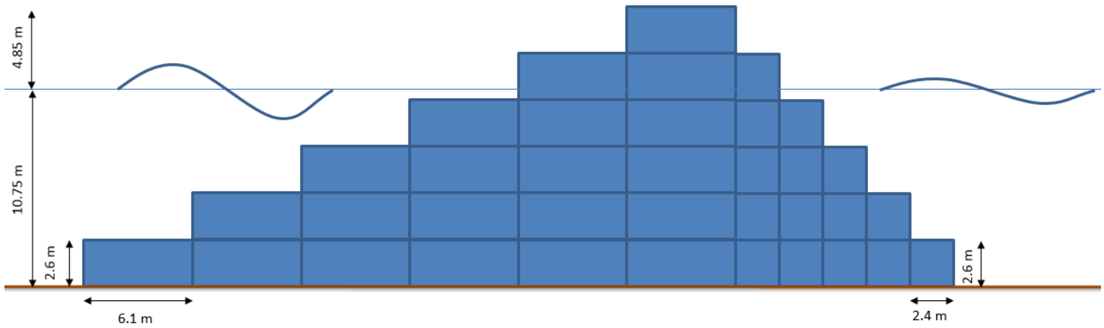

Figure 41: Perpendicular and parallel container layout

6.4 Protection

6.4.1 Introduction

In order to increase the lifetime of the container pyramid construction, some protection is required. In this chapter the required amount of coating and steel protection will be determined for different protection layouts.

6.4.2 Coating

<u>All containers coated</u>

This container coating solution will coat both the internal and external container surface, in order to guarantee a certain container lifetime. The surface of a single container

$$S_{container} = 2 \times L_{cont} \times W_{cont} + 2 \times L_{cont} \times H_{cont} + 2 \times H_{cont} \times W_{cont} = 37\ m^2$$

When coating both the internal as the external surface 74 m^2 coating per container is required. The required coating per container layout can be seen in table 67.

Layout	Coating per container (m^2/unit)	Req. Containers (units)	Req. Coating (m^2)
Parallel	74	7,730	572,020
Perpendicular	74	8,860	655,640
Perp. + Para	74	6,660	492,840

Table 67: Required coating when all containers are coated

<u>External containers coated</u>

When coating all of the external containers, the amount of coating per layout required is shown in table 68. The total required coating per container is 74 m^2. The conversion of containers per cross section to total containers is equal to the formula mentioned in the chapter required containers.

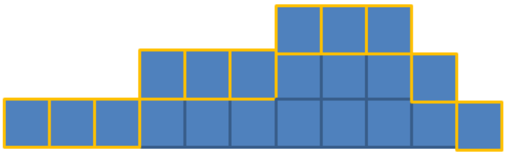

Figure 42: External containers coated

Layout	Coating per container (m^2/unit)	External containers (units/cont. length)	Total external containers (units)	Req. coating (m^2)
Parallel	74	23	2,280	167,850
Perpendicular	74	11	2,710	200,000
Perp. + Para.	74	6 perp. 5 para.	1,980	145,100

Table 68: Coating required per layout

Ocean facing container coating

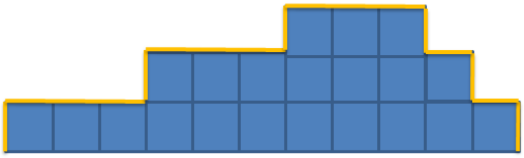
Figure 43: External container walls

The final coating layout will be performed by coating all of the external container walls, as seen in figure 43. The total required coating per container layout can be seen in table 69.
The required coating per container roof is equal to 30 m^2 (container length x width, roof will be coated both internal as external).
The required coating of a vertical wall, (parallel orientated container) is equal to 31 m^2 (container length x height, wall will be coated both internal as external).
The required coating of a vertical wall, (perpendicular orientated container) is equal to 13 m^2 (container width x height, wall will be coated both internal as external).

Layout	Nr roofs per cross section	Nr walls parallel per cross section	Nr walls perpendicular per cross section	Required coating per cross section (m^2)	Req. coating (m^2)
Parallel	23	12	0	1,060	104,600
Perpendicular	11	0	11	465	114,200
Perp. + Para.	6 + 5	5	6	560	92,450

Table 69: Coating required per layout

The required amount of coating per cross section is achieved by multiplying the number of walls and roofs by the required amount of coating per wall or roof. The conversion of containers per cross section to total containers is equal to the formula mentioned in the chapter required containers.

6.4.3 Steel plate

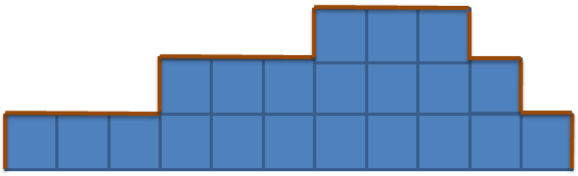
Figure 44: Steel shield protection

The steel plate has to prevent the container walls from corroding during the breakwaters lifetime. It is assumed that the breakwater frame has a lifespan of 20 years, so the steel shield has to prevent the container walls for a lifetime of 20 years. According to (Fontana, 1987) the corrosion speed of steel is approximately 0.4 mm/year in the splash zone. If a lifetime of 20 years is required, the total required steel thickness is approximately 0.4 mm/year x 20 years + 2 mm (safety) = 10 mm thick steel required.

Entire breakwater shield

When protecting the entire breakwater with a steel shield, the following amount of steel per breakwater layout is required (see table 70). The surfaces of a roof and a vertical wall can be seen in the chapter ocean facing container coating. The relative density of steel is 7800 kg/m^3. The steel plate thickness is 10 mm.

Layout	Nr roofs per cross section	Nr walls parallel per cross section	Nr walls perpendicular per cross section	Required steel per cross section (m^3)	Req. steel (m^3)	Weight steel (ton)
Parallel	23	12	0	5.3	525	4085
Perpendicular	11	0	11	2.3	575	4460
Perp. + Para.	6 + 5	5	6	2.8	465	3610

Table 70: Steel required per layout

Ocean facing steel protection

When only protecting the ocean facing containers with a steel coating, the following amount of steel per breakwater layout is required (see table 71). The relative density of steel is 7800 kg/m^3. The steel plate thickness is 10 mm.

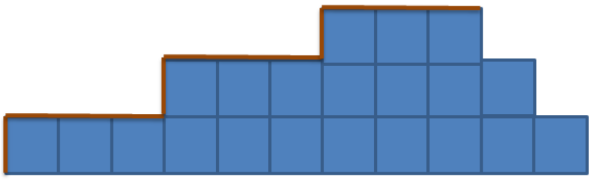

Figure 45: Ocean facing steel protection

Layout	Nr roofs per cross section	Nr walls parallel per cross section	Nr walls perpendicular per cross section	Required steel per cross section (m^3)	Req. steel (m^3)	Weight steel (ton)
Parallel	17	6	0	3	345	2675
Perpendicular	6	0	6	1.3	315	2300
Perp. + Para.	6	0	6	1.3	315	2300

Table 71:Steel required per layout

6.5 Strength of the containers

6.5.1 Introduction

During construction it is expected to be almost impossible to place the containers centric on top of each other (corners stacked). This is expected due to the swell wave conditions on the project location in combination with the water depth. The strength of the containers is therefore checked for the situation where the containers are acentric stacked.

It is expected that for the bottom two layers the inaccuracy of placement is at least 0.5 meters. The roof plate of the containers can withstand only very small loads, therefore only the strength of the beams of the frame is checked.

6.5.2 Determining load

The corners of the containers are strong and can withstand the load of several containers on top of them. However, when the containers are placed acentric, the forces have to be absorbed by the beams of the frame.

There are several ways of stacking containers, parallel or perpendicular. It is expected the vertical corner beams will push through the bottom of the containers when stacked parallel due to the weight of the top containers. The situation where the containers are stacked perpendicular to each other is expected to be more stable.

The strength of the frame is checked for the loading of a single container. This results in the situation shown in figure 46. Since the bottom of the containers is strong, it is assumed that a single top frame beam of a container of the bottom layer has to withstand the load of 1/6 of the weight of the top container.

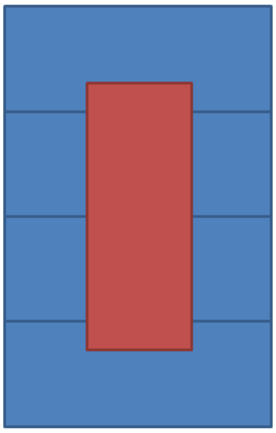

Figure 46: Situation of container loading (top view)

The gravitational force of a container filled with sand and water is 523 kN (see chapter 22.2). The buoyancy force on the same container in water is 382 kN (see chapter 22.2). Therefore the resulting force of the container in water is 523-382=141 kN.

1/6 of the weight of a container has to be absorbed by a single top beam of the frame of one of the bottom containers. This is 141/6=23.5 kN over a length of 2.6 meters (the width of the container), or 9.04 kN/m.

The total loading situation can be schematized to the situation shown in figure 47.

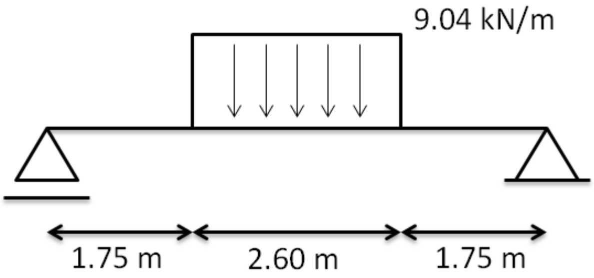

Figure 47: Schematization of the loading situation

6.5.3 Strength of the container

The top beams of the frame of the containers are made of Corten A steel. The Yield stress of Corten A steel is 355 N/mm^2. The dimensions of the top beams, according to (payam aghl, 2013) are shown in figure 48.

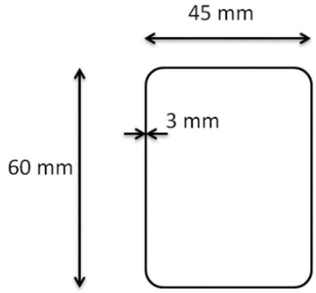

Figure 48: Dimensions of the top beams

The maximum occurring moment can be calculated:

$$M_{max} = \frac{qb}{4}\left(L - \frac{b}{2}\right) = \frac{9.04 * 2.60}{4}\left(6.1 - \frac{2.60}{2}\right) = 28.2kNm$$

To determine the stress in the beam, the section modulus has to be determined:

$$W = \frac{bh^3 - (b - 2w) * (h - 2w)^3}{6h} = \frac{45 * 60^3 - (45 - 2 * 3) * (60 - 2 * 3)^3}{6 * 60} = 9941.4\ mm^3$$

The maximum stress in the beam can be calculated:

$$\sigma_{max} = \frac{M_{max}}{W} = \frac{28.2 * 10^6}{9941.4} = 2836.6\ N/mm^2$$

The Yield stress of the material of the beam is 355 N/mm^2, therefore it can be concluded that the beam will fail.

It is assumed that the beams will fail for every stacking method except when the containers are stacked perfect centric. It is expected that the beams will fail, especially when taken into account that there will be more containers stacked and therefore the load will increase. Centric stacking is considered not possible in the swell waves, due to the swell waves and the water depth.

7 APPENDIX II: SCHEMATIC CALCULATION VERTICAL PILES

7.1 Introduction

In order to give a good prognosis of the total cost of the vertical pile breakwater, the pile dimensions have to be determined roughly. The diameter of the piles can be determined using the Manual for Structural Hydraulic engineering (Dr. S. van Baars, 2003) and the shore protection manual (CERC, 1984). The required amount of piles can be determined using diffraction and transmission models.

7.2 Required pile diameter

7.2.1 Wood

In order to determine the pile diameter, the following equations are taken into account. A boundary condition for the horizontal displacement is set to $w \leq 0.004L$, based on the sustainable limit state of buildings.

$$F_{max} = F_i + F_D = C_i \rho_{wat} g \frac{\pi}{4} D^2 H_s K_{im} + \frac{1}{2} C_d \rho_{wat} g D {H_s}^2 K_{dm}$$

$$M = F_i h S_i + F_D h S_D$$

Where:

F_{max}	Maximum occurring wave force	N
F_i	Maximum mass inertia force	N
F_D	Maximum drag force	N
M	Moment inside the pile at bottom surface	Nm
C_i	Mass inertia coefficient	2.0
C_D	Drag force coefficient	1.2
ρ_{wat}	Relative density water	1020 kg/m^3
g	Gravitational constant	9.78 N/s^2
D	Pile diameter	variable m
H_s	Significant wave height	3.3 m
K_{im}, K_{dm}, S_i, S_D	Coefficients based of the graphs of dean (H_b=0.78 h)	
H_b	Wave height when waves will start breaking	8.4 m
h	Initial water depth	10.75 m
K_{im}	Coefficient for determining F_{max} using CERC	0.32
K_{dm}	Coefficient for determining F_{max} using CERC	0.59
S_i	Coefficient for determining M using CERC	0.61
S_d	Coefficient for determining M using CERC	0.70

$$w = \frac{F_{max} L^3}{3EI}$$

Where:

w	Horizontal displacement with a maximum of $w = 0.004L$	mm
L	Length of the moment's arm ($L = h + 0.65\ t$) m	
E	Modulus of elasticity wood	1.8x10^4 N/mm^2
I	Moment of inertia	mm^4

$$I = \frac{\pi}{64} D^4$$

$$\gamma_{var} F_{max} = \frac{1}{24}\gamma' K_p {t_0}^2 (t_0 + 4D) - \frac{\gamma_{var} M}{t_0}$$

Where:

γ_{var}	Safety factor variable loads	1.5
γ'	Specific weight soil in water	10 kN/m^3
K_p	Passive soil pressure coefficient	1/3
t_0	Ad parameter in order to determine t	m
	$t = 1.2 t_0$	
t	Ramming depth of pile	m

Varying with the pile diameter (D), the following boundary of D can be set:
If D>0.4m, the boundary condition of $w \leq 0.004L$ cannot be guaranteed, summarized the maximum allowed pile diameter is equal to 0.4m, so the SLS boundary conditions can be guaranteed. When applying a pile diameter of 0.4 meter, the following values are to be found:

$F_{max} = 18\ kN$
$t = 6.7\ m$
$w = 0.06\ m$
$I = 1.89\ m^4$

7.2.2 Steel

The calculation of steel piles in comparable to the calculation of wooden piles. The only difference is the moment of inertia and the elasticity modulus. The values are equal to:

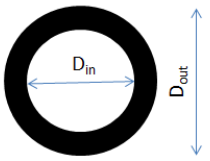

Figure 49: Internal and external diameter pole

$$I = \frac{\pi}{64}({D_{out}}^4 - {D_{in}}^4)$$

Where:

D_{out}	External pole diameter	
m		
D_{in}	Internal pole diameter (wall thickness 0.02 m)	D_{out} – 0.04 m

$$E = 2.1 * 10^5 \frac{N}{mm^2} = 2.1 * 10^8 \frac{kN}{m^2}$$

Applying the equations mentioned in chapter 7.2.1compared with the changes of chapter 7.2.2, the following values are determined:

$D_{out} = 1\ m$
$F_{max} = 55\ kN$
$t = 8.9\ m$
$w = 0.07\ m$
$I = 0.0067\ m^4$

7.3 Permitted bending stress

7.3.1 Wood

The unity check for checking the maximum occurring bending stress in defined into the following formulas. The maximum occurring moment will occur at the length X_m below bottom level. The maximum allowed bending stress is equal to 40 N/mm^2 (40*10^3 kN/m^2).

$$X_m{}^2(X_m + 3D) = \frac{6\gamma_{var}F_{max}}{\gamma' K_p}$$

Where:

X_m	Length under bottom level where the M_{max} occurs	m

$$M_{max} = \gamma_{var}(F_{max}X_m + M) - \frac{1}{24}\gamma' K_p X_m{}^3(X_m + 4D)$$

$$\sigma_{ocur} = \frac{M_{max}}{W}$$

σ	Occurring bending stress	kN/m^2
W	Resistance moment construction	m^3

$$W = \frac{I_{pile}}{.5\ D}$$

$$\sigma_{occur} < \sigma_{max}$$

According to the above formulas σ_{occur} = 38.5*10^3 kN/m^2. σ_{max} = 40*10^3 kN/m^2, which means that the pile will fulfil the boundary condition bending stress.

7.3.2 Steel

Using the same equations as seen in chapter 7.3.1, with the material input for steel, the maximum allowed bending stress can be calculated. The maximum allowed bending stress at the steel construction is equal to 235*10^3 kN/m^2.

According to the above formulas σ_{occur} = 57.5*10^3 kN/m^2. σ_{max} = 235*10^3 kN/m^2, which means that the pile will fulfil the boundary condition bending stress.

7.4 Number of vertical piles

In the tables 72 and 73, the most beneficial amount of vertical piles per pile rows can be seen. The wooden pile row required 1350 piles and the steel pile row 540 piles per pile row. A description of the assumptions applied in this table can be seen in chapter vertical piles.

T A/B	T occur	T multi	A wood	B wood	D wood	L req.	L/B wood	Piles wood
0,00	0,00	0,00	0,00	0,40	0,40	590,80	1477,00	1480,00
0,01	0,03	0,02	0,00	0,40	0,40	590,80	1462,23	1470,00
0,02	0,06	0,04	0,01	0,41	0,40	590,80	1447,46	1450,00
0,03	0,09	0,06	0,01	0,41	0,40	590,80	1432,69	1440,00
0,04	0,12	0,08	0,02	0,42	0,40	590,80	1417,92	1420,00
0,05	0,15	0,10	0,02	0,42	0,40	592,80	1407,90	1410,00
0,06	0,18	0,12	0,03	0,43	0,40	595,00	1398,25	1400,00
0,07	0,21	0,14	0,03	0,43	0,40	597,20	1388,49	1390,00
0,08	0,24	0,16	0,03	0,43	0,40	599,40	1378,62	1380,00
0,09	0,27	0,18	0,04	0,44	0,40	601,80	1369,10	1370,00
0,10	0,30	0,20	0,04	0,44	0,40	604,20	1359,45	1360,00
0,11	0,33	0,22	0,05	0,45	0,40	606,80	1350,13	1360,00
0,12	0,36	0,24	0,05	0,45	0,40	609,20	1340,24	1350,00
0,13	0,39	0,26	0,06	0,46	0,40	636,40	1384,17	1390,00
0,14	0,42	0,28	0,07	0,47	0,40	714,80	1536,82	1540,00
0,15	0,45	0,30	0,07	0,47	0,40	0,00	0,00	#N/A
0,16	0,48	0,32	0,08	0,48	0,40	0,00	0,00	#N/A
0,17	0,51	0,34	0,08	0,48	0,40	0,00	0,00	#N/A
0,18	0,54	0,36	0,09	0,49	0,40	0,00	0,00	#N/A
0,19	0,57	0,38	0,09	0,49	0,40	0,00	0,00	#N/A
0,20	0,60	0,40	0,10	0,50	0,40	0,00	0,00	#N/A
0,21	0,63	0,42	0,11	0,51	0,40	0,00	0,00	#N/A
0,22	0,66	0,44	0,11	0,51	0,40	0,00	0,00	#N/A
0,23	0,69	0,46	0,12	0,52	0,40	0,00	0,00	#N/A
0,24	0,72	0,48	0,13	0,53	0,40	0,00	0,00	#N/A
0,25	0,75	0,50	0,13	0,53	0,40	0,00	0,00	#N/A
0,26	0,78	0,52	0,14	0,54	0,40	0,00	0,00	#N/A
0,27	0,81	0,54	0,15	0,55	0,40	0,00	0,00	#N/A
0,28	0,84	0,56	0,16	0,56	0,40	0,00	0,00	#N/A
0,29	0,87	0,58	0,16	0,56	0,40	0,00	0,00	#N/A
0,30	0,90	0,61	0,17	0,57	0,40	0,00	0,00	#N/A

Table 72: Required piles wooden pile pyramid

T A/B	T occur	T multiple	A steel	B steel	D steel	L req.	L/B steel	Piles steel
0,00	0,00	0,00	0,00	1,00	1,00	590,80	590,80	600
0,01	0,03	0,02	0,01	1,01	1,00	590,80	584,89	590
0,02	0,06	0,04	0,02	1,02	1,00	590,80	578,98	580
0,03	0,09	0,06	0,03	1,03	1,00	590,80	573,08	580
0,04	0,12	0,08	0,04	1,04	1,00	590,80	567,17	570
0,05	0,15	0,10	0,05	1,05	1,00	592,80	563,16	570
0,06	0,18	0,12	0,06	1,06	1,00	595,00	559,30	560
0,07	0,21	0,14	0,08	1,08	1,00	597,20	555,40	560
0,08	0,24	0,16	0,09	1,09	1,00	599,40	551,45	560
0,09	0,27	0,18	0,10	1,10	1,00	601,80	547,64	550
0,10	0,30	0,20	0,11	1,11	1,00	604,20	543,78	550
0,11	0,33	0,22	0,12	1,12	1,00	606,80	540,05	550
0,12	0,36	0,24	0,14	1,14	1,00	609,20	536,10	540
0,13	0,39	0,26	0,15	1,15	1,00	636,40	553,67	560
0,14	0,42	0,28	0,16	1,16	1,00	714,80	614,73	620
0,15	0,45	0,30	0,18	1,18	1,00	0,00	0,00	#N/A
0,16	0,48	0,32	0,19	1,19	1,00	0,00	0,00	#N/A
0,17	0,51	0,34	0,20	1,20	1,00	0,00	0,00	#N/A
0,18	0,54	0,36	0,22	1,22	1,00	0,00	0,00	#N/A
0,19	0,57	0,38	0,23	1,23	1,00	0,00	0,00	#N/A
0,20	0,60	0,40	0,25	1,25	1,00	0,00	0,00	#N/A
0,21	0,63	0,42	0,27	1,27	1,00	0,00	0,00	#N/A
0,22	0,66	0,44	0,28	1,28	1,00	0,00	0,00	#N/A
0,23	0,69	0,46	0,30	1,30	1,00	0,00	0,00	#N/A
0,24	0,72	0,48	0,32	1,32	1,00	0,00	0,00	#N/A
0,25	0,75	0,50	0,33	1,33	1,00	0,00	0,00	#N/A
0,26	0,78	0,52	0,35	1,35	1,00	0,00	0,00	#N/A
0,27	0,81	0,54	0,37	1,37	1,00	0,00	0,00	#N/A
0,28	0,84	0,56	0,39	1,39	1,00	0,00	0,00	#N/A
0,29	0,87	0,58	0,41	1,41	1,00	0,00	0,00	#N/A
0,30	0,90	0,61	0,43	1,43	1,00	0,00	0,00	#N/A

Table 73: Required piles steel pile pyramid

In table 72 and 73, the columns are determined as follows.

T A/B is the transmission coefficient according to the rule of thumb. T is the transmission coefficient multiplied by 3. T multiple is the reduced transmission coefficient with two times 18% (2 times a 100% pile row increasing). A is the distance between two piles. B is the distance between two piles plus the diameter of a single pile. D is the diameter of a single pile. L is the required length, according to the transmission/length diagram, based on the T multiple occurring transmission coefficients. L/B is the required length of the breakwater divided by the distance between B. The last column describes the total required piles per pile row.

8 APPENDIX III: CALCULATION REQUIRED CREST HEIGHT VERTICAL AND STEEP WALLS

8.1 Introduction

To calculate the required crest height of a vertical or steep wall, an excel sheet is made with the formulas given in (Overtopping Manual, 2007). The required crest height is calculated for three situations:

- Vertical wall
- Battered wall (10:1)
- Battered wall (5:1)

This appendix contains screenshots of the excel sheet used, including the formula's, the input and the conclusions. To obtain the excel sheet, please contact one of the graduating students.

8.2 Vertical wall

8.2.1 Input (vertical wall)

Vertical wall:		
h*	0,2797615	h*=1.35(hs/Hm0)(2pi()hs/(g(Tm-1.0)^2)) [-]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
Hs	1,5	significant wave height [m]
g	9,78	gravitational acceleration [m/s2]
Tm-1,0	15,454545	spectral wave period = (Tp/1.1) [s]
Tp	17	peak wave period [s]
h* > 0.3 --> non-impulsive conditions		
h* < 0.2 --> impulsive conditions		
0.2 ≤ h* ≤ 0.3 --> both impulsive and non-impulsive conditions, the larger value assumed.		
q	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]

The mean overtopping discharge per metre structure width is assumed to be 1.00E-02 m^3/s/m because when it is chosen smaller, the formula may not be applied. However, 1.00E-02 m^3/s/m is equal to 1 L/s/m, which is not much. It is assumed that the contribution to the wave height on the lee side of the breakwater by a mean overtopping discharge of 1 L/s/m is nihil.

8.2.2 Non-impulsive conditions (vertical wall)

Non-impulsive conditions:		
Probalistic design:		
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-2.6\frac{R_c}{H_{m0}}\right)$$ valid for $0.1 < R_c/H_{m0} < 3.5$		
can be rewritten to:		
Rc	0,7854036	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-2.6) [m]
q	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,523602426
Rc	0,7854036	m
Deterministic design:		
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-1.8\frac{R_c}{H_{m0}}\right)$$ valid for $0.1 < R_c/H_{m0} < 3.5$		
can be rewritten to:		
Rc	1,1344719	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-1.8) [m]
q	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,756314615
Rc	1,1344719	m
Conclusion non-impulsive conditions:		
Rc	1,1344719	m

8.2.3 Impulsive conditions (vertical wall)

Impulsive conditions:		
Probalistic design:		
$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 1.5\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1} \quad \text{valid over } 0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$$		
can be rewritten to:		
Rc	2,7722687	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/1.5E-04)^(1/-3,1)/h**Hm0 [m]
q	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,517049408
Rc	2,7722687	m
Deterministic design:		
$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 2.8\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1} \quad \text{valid over } 0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$$		
can be rewritten to:		
Rc	3,3905974	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/2.8E-04)^(1/-3,1)/h**Hm0 [m]
q	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,632372448
Rc	3,3905974	m
Conclusion impulsive conditions:		
Rc	3,3905974	m

8.2.4 Conclusion (vertical wall)

Conclusion:		
Rc	3,4	m

8.3 Battered wall (10:1)

8.3.1 Input (battered wall, 10:1)

Battered wall (10:1):		
h*	0,2797615	h*=1.35(hs/Hm0)(2pi()hs/(g(Tm-1.0)^2)) [-]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
Hs	1,5	significant wave height [m]
g	9,78	gravitational acceleration [m/s2]
Tm-1,0	15,454545	spectral wave period = (Tp/1.1) [s]
Tp	17	peak wave period [s]
h* > 0.3 --> non-impulsive conditions		
h* < 0.2 --> impulsive conditions		
0.2 ≤ h* ≤ 0.3 --> both impulsive and non-impulsive conditions, the larger value assumed.		
qb	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
qv	7,69E-03	qvertical=qbatter/1,3

The mean overtopping discharge per metre structure width is assumed to be 1.00E-02 m^3/s/m because when it is chosen smaller, the formula may not be applied. However, 1.00E-02 m^3/s/m is equal to 1 L/s/m, which is not much. It is assumed that the contribution to the wave height on the lee side of the breakwater by a mean overtopping discharge of 1 L/s/m is nihil.

8.3.2 Non-impulsive conditions (battered wall, 10:1)

Non-impulsive conditions:		
Probalistic design:		
$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04 \exp\left(-2.6\frac{R_c}{H_{m0}}\right)$ valid for $0.1 < R_c/H_{m0} < 3.5$		
can be rewritten to:		
Rc	0,8511402	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-2.6) [m]
qv	7,69E-03	mean overtopping discharge per metre structure width [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,567426792
Rc	0,8511402	m
Deterministic design:		
$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04 \exp\left(-1.8\frac{R_c}{H_{m0}}\right)$ valid for $0.1 < R_c/H_{m0} < 3.5$		
can be rewritten to:		
Rc	1,2294247	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-1.8) [m]
q	7,69E-03	mean overtopping discharge per metre structure width [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,819616478
Rc	1,2294247	m
Conclusion non-impulsive conditions:		
Rc	1,2294247	m

8.3.3 Impulsive conditions (battered wall, 10:1)

Impulsive conditions:		
Probalistic design:		
$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 1.5\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1}$$ valid over $0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$		
can be rewritten to:		
Rc	3,0171107	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/1.5E-04)^(1/-3,1)/h**Hm0 [m]
q	7,69E-03	mean overtopping discharge per metre structure width [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,562714318
Rc	3,0171107	m
Deterministic design:		
$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 2.8\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1}$$ valid over $0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$		
can be rewritten to:		
Rc	3,6900491	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/2.8E-04)^(1/-3,1)/h**Hm0 [m]
q	7,69E-03	mean overtopping discharge per metre structure width [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,688222489
Rc	3,6900491	m
Conclusion impulsive conditions:		
Rc	3,6900491	m

8.3.4 Conclusion (battered wall, 10:1)

Conclusion:		
Rc	3,7	m

8.4 Battered wall (5:1)

8.4.1 Input (battered wall, 5:1)

Battered wall (5:1):		
h*	0,2797615	h*=1.35(hs/Hm0)(2pi()hs/(g(Tm-1.0)^2)) [-]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
Hs	1,5	significant wave height [m]
g	9,78	gravitational acceleration [m/s2]
Tm-1,0	15,454545	spectral wave period = (Tp/1.1) [s]
Tp	17	peak wave period [s]
h* > 0.3 --> non-impulsive conditions		
h* < 0.2 --> impulsive conditions		
0.2 ≤ h* ≤ 0.3 --> both impulsive and non-impulsive conditions, the larger value assumed.		
qb	1,00E-02	mean overtopping discharge per metre structure width (assumption) [m3/s/m]
qv	5,26E-03	qvertical=qbatter/1,9

The mean overtopping discharge per metre structure width is assumed to be 1.00E-02 m^3/s/m because when it is chosen smaller, the formula may not be applied. However, 1.00E-02 m^3/s/m is equal to 1 L/s/m, which is not much. It is assumed that the contribution to the wave height on the lee side of the breakwater by a mean overtopping discharge of 1 L/s/m is nihil.

8.4.2 Non-impulsive conditions (battered wall, 5:1)

Non-impulsive conditions:		
Probalistic design:		
$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-2.6\frac{R_c}{H_{m0}}\right)$		valid for 0.1 < R_c/H_{m0} < 3.5
can be rewritten to:		
Rc	0,946223	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-2.6) [m]
qv	5,26E-03	mean overtopping discharge per metre structure width [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,630815349
Rc	0,946223	m
Deterministic design:		
$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-1.8\frac{R_c}{H_{m0}}\right)$		valid for 0.1 < R_c/H_{m0} < 3.5
can be rewritten to:		
Rc	1,3667666	Rc=log(q/(0.04*SQRT(g*(Hm0)^3)))*(Hm0/-1.8) [m]
q	5,26E-03	mean overtopping discharge per metre structure width [m3/s/m]
g	9,78	gravitational acceleration [m/s2]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,911177727
Rc	1,3667666	m
Conclusion non-impulsive conditions:		
Rc	1,3667666	m

8.4.3 Impulsive conditions (battered wall, 5:1)

Impulsive conditions:		
Probalistic design:		
$\frac{q}{h_*^2\sqrt{gh_s^3}} = 1.5\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1}$ valid over $0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$		
can be rewritten to:		
Rc	3,4100115	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/1.5E-04)^(1/-3,1)/h**Hm0 [m]
q	5,26E-03	mean overtopping discharge per metre structure width [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,635993331
Rc	3,4100115	m
Deterministic design:		
$\frac{q}{h_*^2\sqrt{gh_s^3}} = 2.8\times10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1}$ valid over $0.03 < h_*\frac{R_c}{H_{m0}} < 1.0$		
can be rewritten to:		
Rc	4,1705827	Rc=((q/((h*^2)*SQRT(g*(hs^3))))/2.8E-04)^(1/-3,1)/h**Hm0 [m]
q	5,26E-03	mean overtopping discharge per metre structure width [m3/s/m]
h*	0,2797615	wave breaking parameter [-]
g	9,78	gravitational acceleration [m/s2]
hs	10,75	waterdepth [m]
Hm0	1,5	significant wave height calculated from the spectrum (=Hs) [m]
valid	Yes	0,777845702
Rc	4,1705827	m
Conclusion impulsive conditions:		
Rc	4,1705827	m

8.4.4 Conclusion (battered wall, 5:1)

Conclusion:		
Rc	4,2	m

8.5 Overview

Table 74 shows the overview of the calculations for a mean overtopping discharge per metre structure of 1.00E-02 $m^3/s/m$.

Overview: Valid values for Rc [m]	Non-impulsive prob.	Non-impulsive det.	impulsive prob.	impulsive det.
Vertical wall	Yes	Yes	Yes	Yes
	0,79	1,13	2,77	3,39
Battered wall (10:1)	Yes	Yes	Yes	Yes
	0,85	1,23	3,02	3,69
Battered wall (5:1)	Yes	Yes	Yes	Yes
	0,95	1,37	3,41	4,17

Table 74: Discharge per metre structure of 1.00E-02 $m^3/s/m$

Table 75 shows the overview of the calculations when the mean overtopping discharge per metre structure is reduced to 1.00E-03 $m^3/s/m$ (0.1 L/s/m).

Overview: Valid values for Rc [m]	Non-impulsive prob.	Non-impulsive det.	impulsive prob.	impulsive det.	Rc [m]
Vertical wall	Yes	Yes	No	No	#VALUE!
	1,36	1,97	Unvalid	Unvalid	
Battered wall (10:1)	Yes	Yes	No	No	#VALUE!
	1,43	2,06	Unvalid	Unvalid	
Battered wall (5:1)	Yes	Yes	No	No	#VALUE!
	1,52	2,20	Unvalid	Unvalid	

Table 75: Discharge per metre structure of 1.00E-03 m3/s/m

As can be seen in table 75, the formulas for the impulsive conditions may not be applied when the mean overtopping discharge per metre structure is reduced to 1.00E-03 $m^3/s/m$ (0.1 L/s/m).

9 APPENDIX IV: DETERMING PILE DIMENSIONS VERTICAL TUBES WITH BAND OR SLAB BREAKWATER

9.1 Introduction

The pile dimensions of the vertical tubes with band or slab breakwater are determined using Blum's method. Before the dimensions of the piles can be determined with Blum's method, the occurring wave forces have to be calculated and schematized. This schematization is done using the Extended Goda Formula.

9.2 Wave forces

9.2.1 Introduction

The wave forces are determined using the Extended Goda Formula. This equation is regularly used in order to determine the wave forces at constructions containing vertical walls.

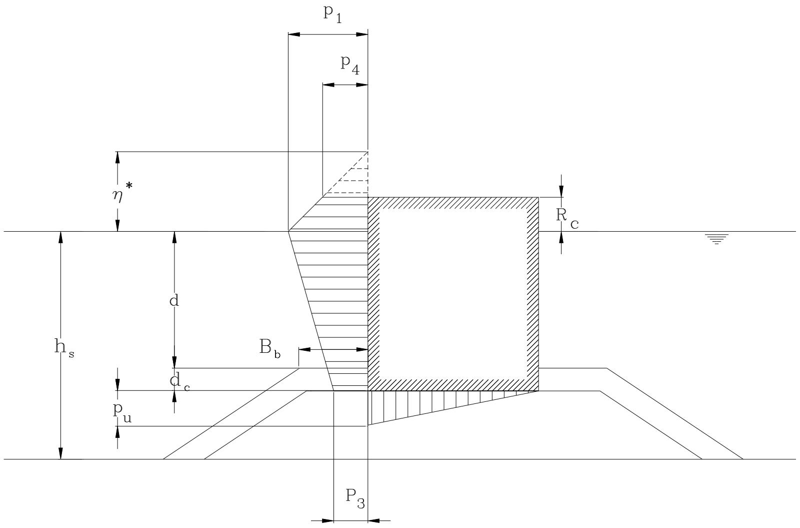

Figure 50: Occurring waves at construction (H. Outeraci, 1999)

$$\eta^* = 0.75\,(1 + \cos\beta)\lambda_1 H_s$$

$$P_1 = 0.5(1 + \cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha^* \cos^2\beta)\rho g H_s$$

$$P_3 = \alpha_3 P_1$$

$$P_4 = \alpha_4 P_1$$

$$P_u = 0.5(1 + \cos\beta)\lambda_3\alpha_1\alpha_3\rho g H_s$$

Where:

β	Angle of incidence of wave attack	[°]
H_s	Significant wave height	[m]
T_p	Peak wave period	[s]
L	Wave length	[m]
ρ_{wat}	Density water	[kg/m^3]
g	Gravitational constant	[m/s^2]
h	Initial water depth	[m]
d	Water depth in front of structure	[m]
d_c	height of rubble mound in front of the structure	[m]
R_c	Crest height (max 1.25 H_s)	[m]
λ_1, λ_2, λ_3	Multiplication factors, depends on the structure	[-]

The factors α_1, α_2, α_3, α_4 and α^* are determined by:

$$\alpha_1 = 0.6 + 0.5\left(\frac{\frac{4\pi h}{L}}{\sinh\frac{4\pi h}{L}}\right)^2$$

$$\alpha^* = max(\alpha_2, \alpha_|)$$

$$\alpha_2 = min\left(\frac{\left(1-\frac{d}{h}\right) * \left(\frac{H}{d}\right)^2}{3}, \frac{2d}{H}\right)$$

$$\alpha_3 = 1 - \left(\frac{d}{h}\right) * \left(1 - \frac{1}{\cosh\left(\frac{2\pi h}{L}\right)}\right)$$

$$\alpha_4 = 1 - \frac{{R_c}^*}{\eta^*}$$

$${R_c}^* = min(\eta^*, R_c)$$

$$\alpha_| = \alpha_{|0} * \alpha_{|1}$$

$$\alpha_{|0} = \frac{H}{d} \qquad H \leq 2d$$

$$\alpha_{|0} = 2 \qquad H > 2d$$

$$\alpha_{|1} = variable$$

$\alpha_{|1}$ can be found in a diagram on page 51 in (Takahashi, 2002).

9.2.2 Wave forces for design situation per meter breakwater

When the parameters of the design situation are used, the values for p_1, p_3, p_4 and p_u can be calculated. The parameters of the design situation are:

β	Angle of incidence of wave attack	90 [°]
H_s	Significant wave height	3.3 [m]
T_p	Peak wave period	18.6 [s]
L	Wave length	187 [m]
ρ_{wat}	Density water	1020 [kg/m^3]
g	Gravitational constant	9.78 [m/s^2]
h	Initial water depth	10.8 [m]
d	Water depth in front of structure	10.8 [m]
d_c	height of rubble mound in front of the structure	0 [m]
R_c	Crest height (max 1.25 H_s)	3.4 [m]
λ_1, λ_2, λ_3	Multiplication factors, depends on the structure	1, 1, 0 [-]

The results are shown in table 76.

Parameter	Value
P_1	16.8 kN/m^2
P_3	15.8 kN/m^2
P_4	0 kN/m^2
P_u	0 kN/m^2

Table 76: Results for p_1, p_3, p_4 and p_u

The calculated wave pressures are shown in figure 51.

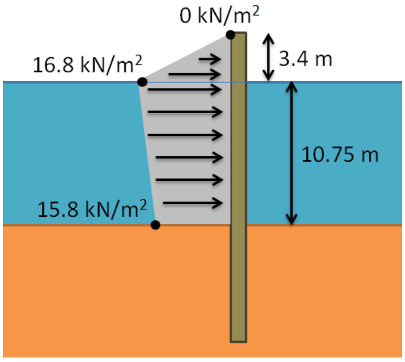

Figure 51: Calculated wave pressures per m breakwater

The wave pressures can be schematized to a line load in the longitudinal direction of the breakwater. This line load is calculated to be 203.8 kN/m and the point of application is 6.33 meter above the seabed. See figure 52.

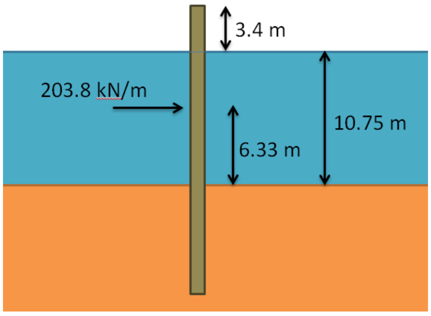

Figure 52: Schematized line load in longitudinal direction of the breakwater

9.3 Determining pile dimensions

9.3.1 Outer diameter and ramming depth of piles

The pile stability is checked using the Blum method, described in (Waterbouwkundige kunstwerken, 1987). The first formula used is:

$$\gamma_{var} F_{hor} = \frac{1}{24} \gamma' K_p {t_0}^3 * \left(\frac{t_0 + 4D}{t_0 + h}\right)$$

Where:

γ_{var}	Safety factor variable loads	1.5 [-]
F_{hor}	Occurring force on pile	[kN]
γ'	Specific weight soil in water	10 [kN/m^3]
K_p	Passive soil pressure coefficient	3 [-]
t_0	Parameter in order to determine t	[m]
D	Outer diameter of pile	[m]
h	Location of horizontal force related to seabed	6.33 [m]
t	Ramming depth of pile ($t = 1.2t_0$)	[m]

F_{hor} depends on the diameter of the pile:

$$F_{hor} = F_l * (w_b + D)$$

Where:

F_l	The occurring line load	203.8 [kN/m]
w_b	Width of the band	2.2 [m]
D	Outer diameter of pile	[m]

There are two unknown parameters: t_0 and D.

In order to calculate the ramming depth of the piles, logical values for D are assumed. The set values for D are: 0.5 m, 1.0 m, 1.5 m, 2.0 m, 2.5 m, 3.0 m and 3.5 m.

For these values of D, the occurring force on a single pile and the ramming depth of a single pile (t) can be calculated. The results of these calculations are shown in table 77.

D [m]	F_{hor} [kN]	t [m]
0.5	550.3	11.60
1.0	652.2	11.64
1.5	754.1	11.68
2.0	856.0	11.71
2.5	957.9	11.73
3.0	1059.8	11.75
3.5	1161.7	11.77

Table 77: Results for F_{hor} and t for set values of D

9.3.2 Maximum moment and location

The location of the maximum occurring moment is described by x_m. x_m can be determined by the formula:

$$x_m^2(x_m + 3D) = \frac{6 \cdot \gamma_{var} \cdot F_h}{\gamma' \cdot K_p}$$

Where:

x_m	Location of maximum moment (distance from seabed)	[m]
D	Outer diameter of pile	var. [m]
γ_{var}	Safety factor variable loads	1.5 [-]
F_h	Occurring force on pile	var. [kN]
γ'	Specific weight soil in water	10 [kN/m^3]
K_p	Passive soil pressure coefficient	3 [-]

The maximum occurring moment (M_{max}) can be calculated with the formula:

$$M_{max} = \gamma_{var} \cdot F_h \cdot (h + x_m) - \frac{1}{24} \cdot \gamma' \cdot K_p \cdot x_m^3(x_m + 4D)$$

Where:

M_{max}	Maximum occurring moment	[kNm]
γ_{var}	Safety factor variable loads	1.5 [-]
F_h	Occurring force on pile	var. [kN]
h	Location of horizontal force related to seabed	6.33 [m]
x_m	Location of maximum moment related to seabed	[m]
γ'	Specific weight soil in water	10 [kN/m^3]
K_p	Passive soil pressure coefficient	3 [-]
D	Outer diameter of pile	var. [m]

The location of the maximum moment (x_m) and the maximum moment (M_{max}) can be calculated for the set values of D and the accompanying values for F_{hor} and t. The results are shown in table 78.

D [m]	F_{hor} [kN]	t [m]	x_m [m]	M_{max} [kNm]
0.5	550.3	11.60	5.0	8,255
1.0	652.2	11.64	5.0	9,674
1.5	754.1	11.68	4.9	11,095
2.0	856.0	11.71	4.9	12,517
2.5	957.9	11.73	4.8	13,940
3.0	1059.8	11.75	4.8	15,364
3.5	1161.7	11.77	4.8	16,788

Table 78: Results for x_m and M_{max} for set values of D

9.3.3 Wall thickness

The required thickness of the walls of the piles can be calculated with the maximum moment and the yield strength of the steel piles. The yield strength of the steel is set to 355 N/mm^2. The relation between the yield strength and the maximum moment is:

$$\sigma_{max} = \frac{M_{max}}{S}$$

Where:

σ_{max}	Maximum yield strength of the steel used	355 [N/mm^2]
M_{max}	Maximum occurring moment	var. [kNm]
S	Section modulus	1.5 [-]

The section modules is determined by:

$$S = \frac{\pi(D^2 - (D - 2t_w)^2)}{32D}$$

Where:

S	Section modulus	1.5 [-]
D	Outer diameter of pile	var. [m]
t_w	Minimum required thickness of the wall	[mm]

The minimum required t_w can be calculated for the different values of M_{max} and the assumed value of σ_{max}. The results are shown in table 79.

D [m]	F_{hor} [kN]	t [m]	x_m [m]	M_{max} [kNm]	t_w [mm]
0.5	550.3	11.60	5.0	8,255	NA*
1.0	652.2	11.64	5.0	9,674	79
1.5	754.1	11.68	4.9	11,095	37
2.0	856.0	11.71	4.9	12,517	23
2.5	957.9	11.73	4.8	13,940	17
3.0	1059.8	11.75	4.8	15,364	13
3.5	1161.7	11.77	4.8	16,788	10

Table 79: Minimum required wall thickness (t_w)

***Note:** piles with a diameter of 0.5 meter can not be applied; With a wall thickness of 250mm (the pile is massive), the occurring bending stress in the pile is 717.5 n/mm^2, this is still almost twice as much as the yield strength of the material.

9.3.4 Extra wall thickness due to lifetime

Due to erosion an extra wall thickness is applied to the steel pipes to guarantee a lifetime of at least 50 years. According to (British Standard, 2000), the maximum erosion rate of steel is 0.17 mm/year/side. Which means an extra thickness of at least 50*0.17*2=17 mm is applied. The extra wall thickness applied is set to 20mm.

9.4 Summarization

The required dimensions, for a set value of D, are shown in table 80.

D [m]	F_{hor} [kN]	t [m]	M_{max} [kNm]	t_w [mm]	L [m]	Amount of steel per pile [m^3/pcs]
1.0	652.2	11.64	9,674	99	25.8	7.22
1.5	754.1	11.68	11,095	57	25.8	6.68
2.0	856.0	11.71	12,517	43	25.9	6.84
2.5	957.9	11.73	13,940	37	25.9	7.41
3.0	1059.8	11.75	15,364	33	25.9	7.97
3.5	1161.7	11.77	16,788	30	25.9	8.48

Table 80: Required dimensions for a set value of D

Due to the increasing size in pile diameter, fewer piles are needed in the 600 m long breakwater. Therefore, the total amount of steel needed can be lower while the amount of steel per pile is higher for bigger diameter piles. See table 81.

D [m]	Amount of steel per pile [m^3/pcs]	Nr. of piles needed [pcs]	Total amount of steel needed [m^3]
1.0	7.22	189	1364.58
1.5	6.68	164	1095.52
2.0	6.84	144	984.96
2.5	7.41	129	955.89
3.0	7.97	117	932.49
3.5	8.48	107	907.36

Table 81: Total amount of steel needed for a 600 m breakwater for a set value of D

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Appendix 7: Multi Criteria Analysis

Innovative Breakwater

16 June 2015

Final Report

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SUMMARY

A barge loading terminal is constructed at the west coast of Sumatra facing the Indian Ocean. Because the severe environmental conditions were underestimated, both during construction and operation, it was wrongly decided to omit the application of a protective dedicated breakwater. Moreover, the project budget was too limited to build a traditional breakwater.

For this reason, financially feasible alternative breakwater concepts have to be developed. These innovative concepts have to withstand the wave boundary conditions, which are occurring at the set location. The most favourable alternative, resulting from a Multi-Criteria-Analysis, will be designed in more detail. This choice will be based on the criteria: technical feasibility, costs and sustainability. For reference, a traditional breakwater solution will be developed as well, i.e. the Reference Design.

The purpose of this document is to filter the feasible and less expensive breakwater concepts, coming from the short list, to a most preferred breakwater alternative. This alternative will be optimized further during the next phase in the graduation process, the optimisation phase.

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CONTENTS

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1 INTRODUCTION

At this document, the alternative breakwater concepts will be compared with the reference rubble mound rock armour layer and with each other. The breakwater with the highest rating will be investigated further during the last phase of the graduation process.

The rating system in this multi criteria analysis will consist of a score between 1 and 5 (occasionally a score of 0 is applied for the reference design, if so, in the corresponding chapter is described why), which will result in 5 different ratings for each criteria. The criteria can be found in chapter 2. The following criteria will be taken into account into this MCA:

- Construction costs
- Construction lifetime
- Construction risks
- Removability alternative
- Environmental impact

It is known that the criteria construction costs, construction risks and removability alternative all can be translated costs and thus could be included in the construction costs. The criteria construction lifetime and removability have not been transformed to a currency in the foregoing alternative analysis phases. For this reason, these criteria will be taken into account as factors, in order to involve these unknown costs in the multi criteria analysis.

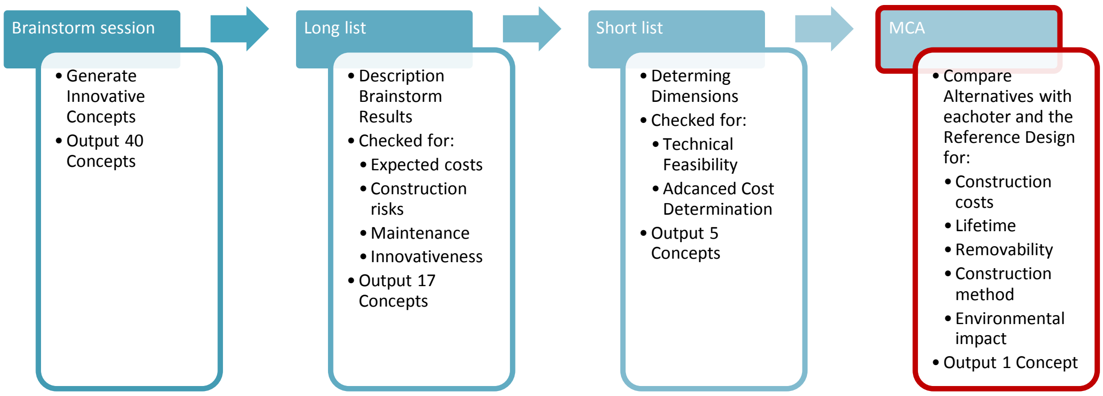


Figure 1: Process for determining "best alternative"

2 VARIANTS IN MCA

2.1 Reference design

In Figure 2, a schematisation of the reference design is shown. The dimensions of the reference design breakwater have been determined in the reference design. In Table 1 the costs and lifetime of the reference design are shown.

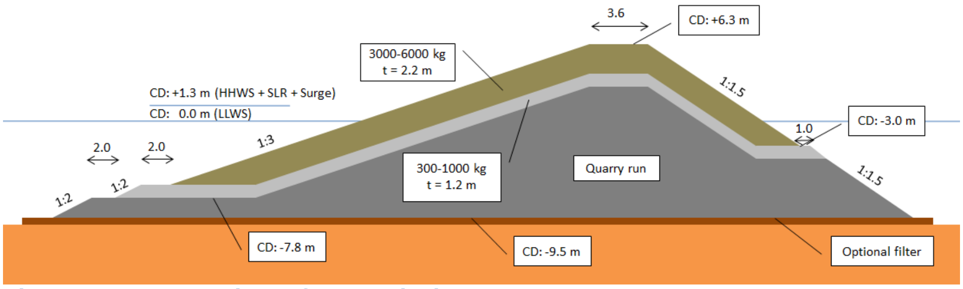


Figure 2: Cross section reference design

Name	Reference design
Costs	€ 37,500,000.-
Lifetime	At least 50 years

Table 1: Costs and lifetime reference design

2.2 Alternatives

In Table 2, the alternatives taken into account in the MCA are shown) including the total construction costs and the assumed lifetime'.

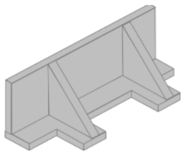
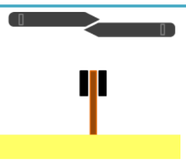
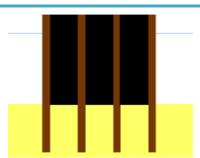
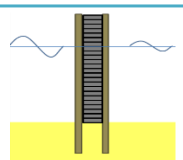
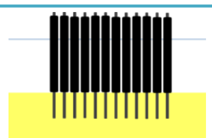
Pictogram		
	Name	**Slab with support**
	Costs	€ 18,000,000.-
	Lifetime	At least 25 years
	Name	**Old ships**
	Costs	€ 22,000,000.-
	Lifetime	Approximately 48½ years
	Name	**Vertical tubes with band**
	Costs	€ 20,000,000.-
	Lifetime	Approximately 50 years
	Name	**Enclosed waste**
	Costs	€ 20,500,000.-
	Lifetime	At least 10 years
	Name	**Piles and tires**
	Costs	€ 25,000,000.-
	Lifetime	Approximately 50 years

Table 2: Alternatives in MCA

3 CRITERIA

3.1 Introduction

In order to determine the best alternative, each alternative breakwater concept will be compared with the reference design and each other. For this comparison, a ranking consisting of five different ranking levels has been drawn. The levels taken into account in this MCA can be seen in Table 3.

Score	Rating
5	Very good
4	Good
3	Moderate
2	Poor
1	Very Poor

Table 3: Ratings used in MCA

In order to determine the best solution numerically, each score will result in a number of points. These points will be multiplied by the weighting factors, which are mentioned in chapter 4, weight factors.

3.2 Construction costs

The criterion of the construction costs is the most important, because it is the main purpose of the alternative breakwater (according to the project plan). The construction costs of the reference design are approximately € 37,500,000.-. The construction costs of the alternatives vary from approximately €18,000,000.- to € 25,000,000.- according to the short list (if the total costs of an alternative are higher than €25,000,000.- the alternative is filtered out in the short list), see Table 2. To compare these construction costs, the average, + or – 2.5%, of these two values is set to the average score (score of 3). This means that an average rating is scored by alternatives with construction costs between €20,500,000.- and €21,500,000.-, leading to the ranges include in Table 4.

An alternative will score a very poor rating (score of 1) if the construction costs of the alternative solution are more than €24,000,000.-. A very good score (score of 5) is awarded when the construction costs of the alternative are less than €18,000,000.-.

The ratings good (score of 4) and poor (score of 2) will fill the gaps between the criteria mentioned above.

Since the main purpose of the project is to develop a low-cost innovative breakwater. Each breakwater alternative which is taken into account in this MCA is less expensive than the reference breakwater design. In order to take these costs savings into account, the reference design breakwater will score a 0 for the criterion costs.

A summary of the breakwater construction costs rating can be seen in Table 4.

Score	Criteria
5	Total costs are ≤ € 18,000,000.-
4	€ 18,000,000.- < Costs < € 20,500,000.-
3	€ 20,500,000.- ≤ Costs ≤ € 21,500,000.-
2	€ 21,500,000.- < Costs < € 24,000,000.-
1	€ 24,000,000.- ≤ Costs < € 25,000,000.-
0	€ 25,000,000.- ≤ Costs (Reference Design)

Table 4: Score and criteria for breakwater costs

3.3 Construction lifetime

The minimal required lifetime of a breakwater alternative is set to 10 years. This required lifetime is shorter than the lifetime of the reference breakwater. The main purpose of the project is to invent a low-cost breakwater alternative, even if the lifetime of the breakwater will be reduced by this costs saving. The breakwater with a reduced lifetime can be applied on project locations which require a temporary breakwater protection. During the lifetime of the innovative breakwater, the client is able to save enough money for an eventual rebuild.
Due to the above reason, the criteria lifetime will be taken into account, since a longer lifetime is only a benefit of the alternative breakwater. The cost will not be multiplied with the required number of rebuilds, in order to achieve the same lifetime as the reference design.

The criteria breakwater lifetime will score a very good score (score of 5) if the lifetime is equal or higher than the lifetime of the reference design (the lifetime of the reference design is 50 years). If an alternative has a lifetime equal to half the lifetime of the reference design (25 years), the alternative will score moderate (score of 3), the moderate criteria has a deviation of 5 years upward and downward.

The alternative will score a very poor (score of 1) if the lifetime is equal to or less than 10 years. The criteria poor (score of 2) and good (score of 4) will fill the gaps between the criteria mentioned above.

A summary of the breakwater lifetime ratings can be seen in Table 5.

Score	Criteria
5	Lifetime is ≥ 50 years
4	30 < Years < 50
3	20 ≤ Years ≤ 30
2	10 ≤ Years < 20
1	Lifetime is ≤ 10 years

Table 5: Score and criteria for breakwater lifetime

3.4 Construction risks

The criterion construction risks is a criterion which compares the breakwater alternatives on the possible risks during the construction of the breakwater. The more risks may occur when construction the breakwater alternative, the lower the alternative will score at the criterion construction risks. The criterion construction risks is able to be expressed in costs as well, due to lack of time and the unknowns at the project location. Not all of the construction risks have been defined. For this reason, the criterion construction risks will be defined into a score instead of costs.

For the criteria construction method, the reference design is taken as an average construction method (score of 3). The construction of a rubble mound breakwater is assumed to be a proven construction method.

A very good score (score of 5) can be achieved if the construction requires less different construction materials (materials with equal sizes and weights) in comparison to the reference design breakwater. Thereby the alternative breakwater requires a smaller amount of materials than the reference design breakwater. The less materials will be applied, the less construction risks may occur.

The good score (score of 4) will be awarded if the alternative breakwater requires fewer units than the reference design breakwater, but the build-up of the construction is comparable to the reference design.

The very poor score (score of 1) will be awarded if the build-up of the construction complex, so a supporting construction is required during construction or a difficult connection method between two different breakwater units is required. If the construction requires a lot of handwork on-shore and vertical or foundation piles, the construction risks will increase as well.

The breakwater will score a poor rating (score of 2) if the construction requires only vertical piles, foundation piles or handwork on shore in order to achieve the desired construction.

A summary of the score criteria distribution for the criterion construction risks can be seen in Table 6.

Score	Criteria
5	Less breakwater units than reference design, easy to take units at location
4	Less breakwater units than reference design, construction build-up comparable to reference design.
3	Reference design breakwater or comparable
2	Lot of handwork at the quay is required Vertical piles or foundation piles required
1	A combination of handwork and vertical or foundation piles is required. Complex connection structure is required or a supporting structure during construction

Table 6: Score and criteria for construction method

3.5 Removability

The removability criterion evaluates the breakwater solutions at removability after the breakwaters lifetime. It is known that the removability criterion can be expressed in the total removability costs. Due to lack of time, the removability costs have not been defined for each breakwater alternative. For this reason, the criterion removability will be applied as factor, in order to allow a comparison of the different breakwater alternatives.

The removability of the breakwater concepts will be compared by the removability of the reference design breakwater. Constructions with a build-up comparable to the reference design breakwater will achieve an average score (score of 3).

The very good score (score of 5) will be assigned if only dredging is required in order to remove the breakwater after its lifetime.

The score good (score of 4) is assigned if the breakwater consists of less loose parts than the reference design. A requirement for this rating is that the loose parts can be removed in one piece.

The breakwater will gain a very poor score (score of 1) if the construction cannot be removed completely after its lifetime, due to a foundation which will last below surface level after removing the breakwater. If the construction has to be separated (cut to pieces) before it can be removed from the project location, the breakwater will earn a very poor score as well. This will primarily occur with steel constructions, which are not able to last many forces at the end of its lifetime, due to a lot of corrosion.

A breakwater will achieve a poor score (score of 2) if huge vertical forces are required in order to remove the construction. This force is particularly required when removing vertical piles.

A summary of the scores related to the criteria of removability can be seen in Table 7.

Score	Criteria
5	Only dredging required
4	Less loose parts than reference design
3	Removability is comparable to the reference design
2	Vertical piles which have to be removed by a huge vertical force
1	Construction cannot be removed completely, foundation will be left behind Huge corroded steel parts have to be separated before removing

Table 7: Score and criteria for removability

3.6 Environmental impact

The environmental impact criticises the origin of different construction materials. A material which has been recycled into the breakwater will achieve a higher score than a material which splotches the environment.

The breakwater will score a very good rating (score of 5) if the breakwater consists completely of recycled or reused building materials. This breakwater will reduce the amount of waste in the world.

The breakwater will score a good rating (score of 4) if the breakwater partly consists of recycled or reused building materials.

An average rating (score of 3) will be achieved if the building material is a natural material, like bamboo, rock, sand and/or certified wood.

A breakwater will be awarded with a poor score (score of 2) if the breakwater is constructed using a common (new) building material like concrete or steel.

The very poor score (score of 1) will be awarded if the building material might consist some fuel or oil leftovers or in some other way may impact the environment.

A summary of the scores related to the criteria environmental impact can be seen in Table 8.

Score	Criteria
5	100 % build of reused or recycled materials
4	Partly build of reused or recycled materials
3	Natural materials like bamboo (not treated), rock, sand or certified wood
2	Common building materials like concrete, steel or chemical treated bamboo.
1	Materials which used to consist of fuel or oil or may impact the environment in some other way

Table 8: Score and criteria environmental impact

4 SCORE TO ALTERNATIVES

4.1 Introduction

This chapter describes which alternative achieves which scores and why these scores are assigned to the alternatives.

4.2 Reference design

4.2.1 Costs & Lifetime

The costs of the reference design are estimated to be € 37,500,000.-. These costs are way higher than the construction costs of the alternatives, therefore the reference design scores a 0 for the rating cost criteria.

The estimated lifetime of the reference design breakwater is approximately 50 years, according to the rating criteria, the reference design breakwater will score a very good rating (score of 5) at the criteria lifetime. For an extended description of the construction costs and lifetime, see the document reference design.

4.2.2 Construction risks

The reference design will be constructed with different layers and sizes of rubble mound rock material. In order to place these rocks into position, some excavators are required; these excavators will be positioned at ocean proof barges. The most difficult part of constructing the reference breakwater is achieving the required breakwater slopes, these slopes requires certain accuracy. In order to guarantee the accuracy of the slopes, survey is required during the construction phase.

The alternative breakwater construction risks will be compared with the risks of the reference design breakwater. For this reason the reference design breakwater scores an average rating (score of 3) at the criterion construction risks.

4.2.3 Removability

The reference design breakwater can be removed in three different removal steps, one for each variable stone size. When removing the reference design breakwater, excavators and barges are required. The reference design breakwater will be rated with an average rating (score of 3) for the removability criterion. Each alternative concept will be compared with the removability of the reference breakwater, in order to achieve a fair comparison.

4.2.4 Environmental impact

The reference design is completely build of natural materials: Quarry run and rock armour stone. Therefore the reference design scores an average score (3) on the environmental impact.

4.2.5 Reference design criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
0	5	3	3	3

Table 9: rating reference design

4.3 Slab with support

4.3.1 Costs and lifetime

The total costs of the slab with support breakwater are calculated at € 18,000,000.-, which will rate the slab breakwater with a very good rating (score of 5) at breakwater costs.

The lifetime of the slab breakwater is approximately 25 years, which will give the breakwater concept an average rating (score of 3) for breakwater lifetime. An advanced description of the breakwater costs and lifetime can be found at the short list, chapter slab with support.

4.3.2 Construction risks

The slab with support construction consists of heave concrete elements which are prefabricated on-shore. These elements are transported towards the breakwater location. The breakwater has to be founded on a rock layer, so the bottom surface is smooth and equal. A backfill is required to make the construction stable. The heavy prefabricated concrete elements are expected to good installable, despite the weight of the elements. The rock filter layer is expected to be good installable. The backfill is placed in open container units, without a roof. The placement of the backfill is also expected to be very easy, since they are installed after the concrete elements at the lee side of the breakwater. Therefore, almost no construction risks are predicted when constructing the breakwater, so the alternative will score a very good score on the criterion construction risks (score of 5).

4.3.3 Removability

Removing the slab with support breakwater is comparable with the placement of the elements. Each elements can be removed separately. However, first the backfill has to be removed, which is quite difficult since it is expected that the containers will break when removed. When the backfill and heavy concrete elements are removed, the rock foundation layer can be removed. Therefore the slab with support breakwater construction scores an average rating (score of 4) for removability. It is assumed that removing the slab with support breakwater is less difficult than removing the reference design.

4.3.4 Environmental impact

The slab with support breakwater consists of concrete with used containers filled with sand as backfill. Since the main part of the slab with support construction are the concrete elements, the alternative is rated a poor score (score of 2) for environmental impact. The construction consists of more concrete in relation to the containers.

4.3.5 Slab with support criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
5	3	5	4	2

Table 10: rating slab with support

4.4 Old ships

4.4.1 Costs and lifetime

The costs of the old ship breakwater is estimated to be € 22,000,000.-. This amount of costs will last in a poor rating (score of 2) for the breakwater costs criteria.

The expected lifetime of the old ship breakwater is assumed to be 48.5 years. The lifetime of 48.5 years is rated with a good score (score of 4). The cost and lifetime determination can be found at the short list, chapter old ships.

4.4.2 Construction risks

The old ships breakwater construction only consists of two ships. It is assumed that taking the ships at the right place is not that difficult at all. If the ships are brought into its position, no huge difficulties during the construction phase are expected. Before the ships are able to be put into position, it has to be sure there is no oil or fuel left in the ship itself, this fact will be taken into account at the chapter environmental impact and the breakwater costs. The criteria construction risks will be rated with a very good rating (score of 5), since no big issues are expected when constructing the old ships breakwater.

4.4.3 Removability

After the lifetime of the old ships breakwater, the old ships are corroded due to the seawater. Due to this corrosion, it is assumed that the ships will collapse when they are removed at one piece. For this reason, the ships have to be separated in different parts in order to avoid the corroded ship hull from cracking. After separating the ship, the separated parts have to be removed by hoisting them out of the water or by letting the separated parts float. Removing the old ships breakwater is assumed to be more complicated than removing the reference breakwater. For this reason, the old ships breakwater will score a very poor rating (score of 1) for the removability criterion.

4.4.4 Environmental impact

The old ships may contain some oil leftovers, even though the ships are completely stripped and cleaned before submerging. However, the old ships breakwater concept is completely made of recycled materials. When the two aspects above are combined, the breakwater scores an average rating (score of 3) for environmental impact.

4.4.5 Old ships criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
2	3	5	1	3

Table 11: rating old ships breakwater

4.5 Vertical tubes with band

4.5.1 Costs and lifetime

The total costs of the vertical tubes with band or slab are estimated to be € 20,000,000.-. With this score, the vertical tube breakwater will score a good rating (score of 4) for construction costs. The estimated lifetime is set to 50 years, which will last in a very good score (score of 5) for lifetime. The advanced cost and lifetime determination can be found at the short list, chapter vertical tubes with band.

4.5.2 Construction risks

The method of attaching the band to the vertical tubes is not designed yet. However, it is expected that the connecting method is complex and require additional measurements. Due to the environmental conditions, it may be hard to achieve the specified tolerances to guarantee that the maximum allowable transmission is not exceeded.

The construction risks of the vertical tubes with band construction will be rated with a very poor rating (score of 1), caused by the little allowed tolerances of the construction and the in all probability required preparation works at the quay. This will last in huge placement risks.

4.5.3 Removability

When removing the vertical tube breakwater, the slab has to be removed first. The connection method is not designed yet. However, it is assumed that the slab can easily be removed from the vertical piles. Removing vertical piles requires huge vertical tension forces, and herewith equipment which is able to generate these forces. Due to the corrosion at the piles during the breakwaters lifetime, the piles might collapse when they are loaded with these vertical forces. It is assumed that removing the vertical tube breakwater is more complex than removing the reference breakwater. For this reason, the construction will achieve a poor rating (score of 2) for removability.

4.5.4 Environmental impact

The vertical tubes used in this alternative are made of steel. The band is made of rubber with steel reinforcements. Therefore the alternative scores a poor rating (score of 2) on environmental impact.

4.5.5 Vertical tubes criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
4	5	1	2	2

Table 12: rating vertical tubes

4.6 Enclosed waste

4.6.1 Costs and lifetime

The total construction costs for the enclosed waste breakwater is estimated to be € 21,300,000.-. This results in an average rating (score of 3) for breakwater costs. The lifetime of the breakwater is estimated to be 20 years. This will give this breakwater concept an average rating (score of 3) for breakwater lifetime. The advanced cost and lifetime determination is to be found at the short list.

4.6.2 Construction risks

The enclosed waste breakwater consists of car wreckages which are enclosed by vertical piles. Before the waste can be taken into its position, the vertical piles have to be constructed. This breakwater solution requires an extra construction step in order to create the enclosed breakwater. The enclosed waste breakwater requires both pile driving and excavating. While the reference design breakwater only needs excavating. Thereby, the excavating works at the enclosed waste breakwater have to be performed with more care, in order to prevent the vertical piles from damage. For this reasons the enclosed waste breakwater scores a poor rating (score of 2) for the criterion construction risks.

4.6.3 Removability

The enclosed waste breakwater has to be removed in two separated parts, the vertical piles and the waste between these piles both have to be removed. Removing the waste is assumed to be comparable to removing rubble mound. Removing vertical piles requires huge vertical forces and is hereby expected to be more complex than removing the rubble mound breakwater. When counting both of the above reasons, it can be concluded that removing the enclosed waste breakwater is more difficult than removing the reference breakwater. For this reason, the breakwater will score a poor rating (score of 2) for the criterion removability.

4.6.4 Environmental impact

The enclosed waste alternative consists of steel piles with car wreckages in between them. The car wreckages may contain some oil or fuel leftovers. However, the car wreckages are recycled materials. When these aspects are combined, the enclosed waste alternative scores a poor rating (score of 2) for environmental impact.

4.6.5 Enclosed waste criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
3	3	2	2	2

Table 13: enclosed waste

4.7 Piles and tires

4.7.1 Costs and lifetime

The construction costs of the piles and tire construction are calculated to be approximately € 25,000,000.-. This results in very poor score (score of 1) for the construction costs criterion.

The expected lifetime of the construction is 50 years. This results in a very good score (score of 5) for the lifetime criterion. See the chapter piles and tires in the short list for the determination of the construction costs and construction lifetime.

4.7.2 Construction risks

Constructing the piles and tires breakwater is expected to be quite hard. The tires have to be sorted by size on-shore. When the tires are sorted, the exact location of the vertical piles has to be determined before they can be piled in place. When the piles are in place, the tires have to be placed over the piles by hand. Because all the different steps and a quite big amount of handwork required, the piles and tires breakwater scores a very poor rating (score of 1) for the construction risks criterion. Since the tires requires a lot of sorting and placing works, which may result in irregularities caused by human errors.

4.7.3 Removability

The removability of the piles and tires breakwater is expected to be quite hard as well. The tires have to be removed before the piles can be removed. Removing vertical piles requires huge vertical tension forces, and herewith equipment which is able to generate these forces. Due to the corrosion at the piles during the breakwaters lifetime, the piles might collapse when they are loaded with these vertical forces. It is assumed that removing the vertical tube breakwater is more complex than removing the reference breakwater. For this reason, the construction will achieve a poor (score of 2) rating for removability.

4.7.4 Environmental impact

The piles and tires breakwater is partly build using recycled materials, therefore the alternative scores a good rating (score of 4) for the environmental impact criterion.

4.7.5 Piles and tires criteria score

Costs	Lifetime	Construction risks	Removability	Environmental Impact
1	5	1	2	4

Table 14: piles and tires

5 WEIGHT FACTORS

The construction costs criterion is more important than the other criteria. Therefore weight factors are applied. The construction costs criterion is more important because the main purpose of this project is to select a low-cost, non-traditional breakwater solution for the specified location. The rest of the criteria are assumed to be equally important. The used weight factors are shown in Table 15.

Criterion	Weight factor
Construction costs	2
Construction lifetime	1
Construction method	1
Removability	1
Environmental impact	1

Table 15: Weight factors used

6 MCA

6.1 MCA with weight factors

The points scored per criterion per alternative are multiplied by the weight factors summed in chapter 5. The higher the value of the total score is, the better the alternative is. The maximum score which can be achieved is 30 points. Table 16 shows the total scores of the alternatives.

Design	Construction costs	Construction lifetime	Construction Risks	Removability	Environmental Impact	Total score
Reference design	0	5	3	3	3	14
Slab with support	5	3	5	3	2	23
Old Ships	2	3	5	1	3	16
Vertical Tubes with band	4	5	1	2	2	18
Enclosed waste	3	3	2	2	2	15
Piles and Tires	1	5	1	2	4	14
Weight factor	2	1	1	1	1	

Table 16: Total score of alternatives

6.2 Sensibility weight factors in MCA

When the weight factors are changed in the MCA, the total scores of the alternatives change. When for every criteria the same weight factor is applied (for example for every criteria a weight factor of 1), the total scores of the criteria are as shown in Table 17.

	Total score
Reference design	14 (x weight factor)
Slab with support	18 (x weight factor)
Old Ships	14 (x weight factor)
Vertical Tubes with band	14 (x weight factor)
Enclosed waste	12 (x weight factor)
Piles and Tires	13 (x weight factor)

Table 17: Total score of alternatives with same weight factor for every criterion

There are infinite combinations possible for the weight factors. However, the slab with support alternative scores the highest total score unless:

- The weight factor for the construction lifetime criterion is set to 3 or higher (with all the other weight factors set to 1).
- The weight factor for the environmental impact criterion is set to 4 or higher (with all the other weight factors set to 1).

6.3 Sensibility costs in MCA

According to the short list, the bandwidth of the costs estimation is -23% to + 36%. In order to determine the sensibility of the factor costs in the MCA, each costs price will be multiplied with -23% or +36%. In Table 18, the construction costs are shown, for the upper and lower uncertainty bandwidth.

Design	Costs	23% down	36% up
Reference design	€ 37,500,000.-	€ 28,875,000.-	€ 51,000,000.-
Slab with support	€ 18,000,000.-	€ 13,860,000.-	€ 24,480,000.-
Old Ships	€ 22,000,000.-	€ 16,940,000.-	€ 29,920,000.-
Vertical Tubes	€ 20,000,000.-	€ 15,400,000.-	€ 27,200,000.-
Enclosed waste	€ 20,500,000.-	€ 15,785,000.-	€ 27,880,000.-
Piles and Tires	€ 25,000,000.-	€ 19,250,000.-	€ 34,000,000.-

Table 18: Upper and lower limits construction costs

When translating the construction costs for the upper and lower limits of the construction costs to a score (criteria mentioned in chapter 3.2), the cost score criteria will change, see Table 19. When applying the same weight factors, as mentioned in chapter 5, it can be seen that the slab with support breakwater keeps the most favourable breakwater alternative.

Design	Score costs	Total score	Score costs - 23%	Total score -23%	Score costs + 36%	Total score +36%
Reference design	0	14	0	14	0	14
Slab with support	5	23	5	23	2	17
Old Ships	2	16	5	22	0	12
Vertical Tubes	4	18	5	20	0	10
Enclosed waste	3	15	5	19	0	9
Piles and Tires	1	14	4	20	0	12

Table 19: Score to alternatives upper and lower limits costs

7 CONCLUSION

The purpose of the project is to select and develop an alternative, low cost, non-traditional breakwater solution for the specified location. The most favourable alternative according to the MCA is the slab with support breakwater. Therefore this alternative will proceed to the next phase of the project.

In the next phase of the project the design of the slab with support breakwater will be optimized further. The dimensions of the prefabricated elements, the layout of the backfill, the construction method and the costs of the construction will be determined in more detail.

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Appendix 9: Documents for Rotterdam University of Applied Sciences

Innovative breakwater

16 June 2015

Final Report

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Document title	Appendix 9: Documents for Rotterdam University of Applied Sciences
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Status	Final Report
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Project name	Innovative Breakwater

Drafted by	Jesper van Grieken, Danny Janssen	
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Date/initials check	………………….	………………….
Approved by	Harry Dommershuijzen	
Date/initials approval	………………….	………………….

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CONTENTS

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1 INTRODUCTIE

Het reflectierapport geeft een overzicht van de onderlinge taakverdeling tussen de twee afstudeerstudenten. In hoofdstuk 2 is schematisch weergegeven welke student verantwoordelijk is voor welk deel van het verslag. In achtereenvolgens hoofdstuk 3 en 4 zijn de competenties van Jesper van Grieken en Danny Janssen weergegeven.

Er is, in tegenstelling tot de rest van het afstudeerrapport gekozen om dit reflectie rapport in het Nederlands op te stellen, omdat de aangereikte templates voor dit reflectierapport alleen in het Nederlands beschikbaar zijn.

2 OVERZICHT ACTIVITEITEN

2.1 Werkzaamheden per document

Bijlage		Jesper v. Grieken	Danny Janssen
1	**Project Plan**		
	Opstellen en specificeren project plan	50 %	50 %
2	**Basis of Design**		
	Opstellen en specificeren eisen golfbreker en omgevingscondities	50 %	50 %
3	**Reference Design**		
	Rock armour layer golfbreker		100 %
	Betonnen elementen golfbreker	100 %	
	Geotechnisch ontwerp	100 %	
	Lengte golfbreker	90 %	10 %
4	**Brainstorm sessie**		
	Opstellen presentatie en achtergrond informatie		100 %
	Overdenken aanpak brainstormsessie	50 %	50 %
5	**Long list**		
	Opstellen long list en beschrijven varianten	15 %	85 %
6	**Short list**		
	Uitwerken van de alternatieven	50 %	50 %
7	**MCA**		
	Opstellen, beoordelen en verslagleggen criteria voor de verschillende alternatieven	50 %	50 %
8	**Further design**		
	Bepalen dimensies en kosten slab with support golfbreker	50 %	50 %
Na	**Final report**		
	Opstellen en schrijven eindrapport	50 %	50 %

Opmerking: Alle documenten zijn door beide studenten nagekeken en aangepast. De percentages in bovenstaande tabel geven alleen in grove lijnen weer in hoeverre de gemaakte documenten in eerste instantie door de betreffende student zijn opgesteld en ingevuld, het nalezen en optimaliseren van deze documenten is hier niet in meegenomen.

Mijn ontwikkeling van competenties

Naam : **Jesper van Grieken**
Studentnummer : **0853476**

Competenties met Kerntaken	Beoordeling[1]	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
1 Initiëren 1.1 Signaleren en/of analyseren van (de behoefte aan) een civieltechnisch project in de gebouwde omgeving. 1.2 Ontwikkelen van een Programma van Eisen voor een te maken civiel technisch project.	B.1, B.3	1.1 In het **Project Plan** is onderbouwd waarom een golfbrekeroplossing benodigd is. 1.2 Het **Basis of Design** is een uitgebreid Programma van Eisen. Deze eisen en de aanvullende omgevingscondities zijn in samenwerking met de afstudeerpartner en de bedrijfsbegeleider opgesteld. Daarnaast zijn er door middel van literatuuronderzoek samen met de afstudeerpartner extra eisen opgesteld met betrekking tot het referentie ontwerp. Deze eisen zijn ook opgenomen in het **Basis of Design**.
2 Ontwerpen 2.1 Ontwerpen van oplossingsvarianten in de vorm van bv. schema's, tekeningen en/of berekeningen voor civieltechnische (deel)problemen. 2.2 Oplossingsvarianten beoordelen en de meest passende kiezen. 2.3 Inventariseren en verzamelen van gegevens	B.1, B.4,	2.1 In onder andere het **Reference Design**, de **Long List**, de **Short List**, en de **Breakwater optimization**, zijn verschillende schema's, tekeningen en berekeningen gemaakt. Een voorbeeld van deze activiteiten waar alle drie deze aspecten in terugkomen, is het ontwerp van de traditionele golfbreker met een bovenlaag van betonnen elementen; Dit ontwerp is berekend, geschematiseerd en getekend. 2.2 In de **Long List**, de **Short List** en het **MCA**, is er in stappen toegewerkt naar een meest passende oplossing. 2.3 In alle documenten zijn gegevens geïnventariseerd en verzameld. Het beste voorbeeld hiervan is het **Basis of Design**.

[1] De codes verwijzen naar de bijgevoegde rubric voor de beoordeling.

Competenties met Kerntaken	Beoordeling[1]	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
3 Specificeren 3.1 Schematiseren van de werkelijke situatie in een (vereenvoudigd) rekenkundig of fysisch model 3.2 Detailleren en/of berekenen en tekenen van een (deel van een) civieltechnische ontwerp 3.3 Contract-, begrotings- en vergunningsdocumenten opstellen en contractvorming organiseren en begeleiden	B.5	3.1 Van verschillende onderdelen zijn rekenkundige of fysische modellen gemaakt. Het beste voorbeeld hiervan is te vinden in het **Reference Design.** In het **Reference Design** zijn verschillende berekeningen gemaakt, daarnaast zijn er een D-Settlement en een D-Geo Stability model gemaakt om de zetting van de ondergrond en de stabiliteit van de taluds van de golfbrekers te bepalen. 3.2 In verschillende onderdelen van het project zijn er civieltechnische ontwerpen berekend en getekend, het beste voorbeeld hiervan is het **Reference Design**. In het **Reference Design** zijn zowel uitgebreide berekeningen als civieltechnische tekeningen gemaakt. 3.3 In het **Reference Design**, de **Short List** en de **Breakwater Optimization** zijn begrotingen gemaakt van de verschillende ontwerpen.
6 Monitoren, toetsen en evalueren 6.1 Hanteren van plan-do-check-act cyclus 6.2 Omgevingsbewust en maatschappelijk verantwoord handelen	A.2, B.1, B.2, B.3, B.5	6.1 Gedurende het gehele proces is er gewerkt volgens een plan-do-check-act cyclus. Door van te voren een goede planning op te stellen (planning in het **Project Plan**) is goed gepland, de taken zijn vrijwel allemaal volgens planning uitgevoerd. Gedurende het uitvoeren van deze taken is er elke week overleg geweest met de supervisor van RHDHV en meerdere keren per week met de medestudent om te controleren (checken) of de juiste weg nog bewandeld wordt. Indien dit niet het geval was, is er direct bijgestuurd (act). Aan het einde van elke taak is deze nog een keer gecheckt of het werk voldoet aan de verwachtingen en of de juiste aspecten behandeld zijn. Indien nodig zijn de verschillende onderdelen aangepast (check en act). Op deze manier zijn alle documenten behandeld. Alle documenten zijn onderling ook met elkaar gecheckt of er geen tegenstrijdigheden in voorkomen. 6.2 De **Long List** en de **Short List** zijn opgesteld aan de hand van de eisen die in het **Basis of Design** zijn vastgelegd. Uit deze eisen en de uitwerking hiervan in de twee genoemde documenten blijkt dat er omgevingsbewust en maatschappelijk verantwoord gehandeld is.

Competenties met Kerntaken	Beoordeling[1]	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
7 Analyseren en onderzoeken 7.1 Uitvoeren van onderzoek	A.1, A.2, B.2, B.3, B.4, B.5	7.1 Er is onderzoek gedaan naar de opgestelde eisen en omgevingscondities (**Basis of Design**) en naar de verschillende aspecten die komen kijken bij het ontwerp van een traditionele golfbreker (**Reference Design**). Daarnaast is er in de **Short List** onderzoek gedaan naar de verschillende materialen en materiaaleigenschappen om onder andere een goede inschatting te kunnen doen van de levensduur en de materiaalkosten van de verschillende constructies.
8 Communiceren en samenwerken 8.1 Verwoorden en verbeelden van informatie 8.2 Functioneren in teams	C.2, C.3, C.4	8.1 In (vooral) de **Short List** en de **Long List** zijn verschillende concepten beschreven en verduidelijkt met een illustratie. Daarnaast is er gedurende de **Brainstorm Session** een presentatie gehouden met daarna een brainstorm sessie om nieuwe concepten te genereren. Deze concepten zijn input geweest voor de eerder genoemde **Long List**. 8.2 Door met twee personen af te studeren is er bewezen dat er goed gefunctioneerd kan worden in een team. Daarnaast is er gedurende de **Brainstorm Session** in een team gezocht naar nieuwe concepten. Ook uit de **Breakwater Optimization** komt naar voren dat er goed gefunctioneerd kan worden in een team. Dit laatste is omdat er vanwege de beperkte tijd, hulp is geboden door verschillende collega's van RHDHV.
9 Management en ondernemen 9.1 Regie voeren over eigen leerproces 9.2 Projectmatig werken en processen aansturen	A.2, B.2, C.1, C.2	9.1 Gedurende het gehele afstudeerproces zijn er bijna dagelijks overleggen tussen de twee afstudeerstudenten geweest. Gedurende deze overleggen is er een verdeling gemaakt van de verschillende taken op zo'n manier dat ieder een zo evenwichtig mogelijke actielijst had en zoveel mogelijk diversiteit in taken had. 9.2 Tijdens de eerder genoemde overleggen zijn de verschillende taken verdeeld en zijn de verschillende processen gemanaged. Taken verwisselden vrijwel nooit van actiehouder, zo blijkt ook uit het **Reference Design**, dat de verschillende afstudeerders beiden hun eigen "projectje" hadden: de een de rots bekleding, de ander de xBloc bekleding.

4 COMPETENTIES DANNY JANSSEN

Mijn ontwikkeling van competenties

Naam : **Danny Janssen**
Studentnummer : **0853162**

Competenties met Kerntaken	Beoordeling	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
1 Initiëren 1.1 Signaleren en/of analyseren van (de behoefte aan) een civieltechnisch project in de gebouwde omgeving. 1.2 Ontwikkelen van een Programma van Eisen voor een te maken civiel technisch project.	B.1, B.3	Opstellen van het **basis of design**. Door de verschillende aspecten die benodigd zijn voor zowel de traditionele als de alternatieve golfbreker varianten te beschrijven. De geïnitieerde eisen en omgevingscondities zijn opgesteld door een wisselwerking van afstudeerpartner, de bedrijfsbegeleider en ik. Naast deze sessie, zijn er eisen door middel van een literatuuronderzoek opgesteld. Nadat de eisen geïnitieerd zijn de eisen gelijkwaardig over het projectteam verdeeld, om ze vervolgens uit te werken. De analyse van het probleem is terug te vinden in het **plan van aanpak.**
2 Ontwerpen 2.1 Ontwerpen van oplossingsvarianten in de vorm van bv. schema's, tekeningen en/of berekeningen voor civieltechnische (deel)problemen. 2.2 Oplossingsvarianten beoordelen en de meest passende kiezen. 2.3 Inventariseren en verzamelen van gegevens	B.1, B.4,	Binnen het project rapport zijn er kleine schematische (**long list)**tot verder uitgewerkte (**short list)** golfbreker alternatieven bedacht en beschreven. In zowel de long list als de short list is getest of de varianten voldoen aan het plan van eisen. Het **MCA** is opstelt aan de hand van secundaire wensen van de opdrachtgever (bedrijfsbegeleider) en onbekenden waardes binnen de alternatieven.

Competenties met Kerntaken	Beoordeling	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
3 Specificeren 3.1 Schematiseren van de werkelijke situatie in een (vereenvoudigd) rekenkundig of fysisch model 3.2 Detailleren en/of berekenen en tekenen van een (deel van een) civieltechnische ontwerp 3.3 Contract-, begrotings- en vergunningsdocumenten opstellen en contractvorming organiseren en begeleiden	B.	In het **reference design** heb ik een rock armour layer golfbreker ontworpen en doorgerekend met behulp van de CUR rock manual. Daarnaast zijn elk in de **short list** genoemde golfbreker varianten verder uitgewerkt en zijn de dimensies globaal bepaald, om een goed beeld te creëren van de verwachte constructie kosten per alternatief. In het **further design** is het meest voordelige golfbrekerconcept verder doorgewerkt, dit ontwerp is gecontroleerd door ervaren constructeurs van RHDHV.
6 Monitoren, toetsen en evalueren 6.1 Hanteren van plan-do-check-act cyclus 6.2 Omgevingsbewust en maatschappelijk verantwoord handelen	A.2, B.1, B.2, B.3, B.5	In het **further design** staat aangegeven op welke ontwerppunten de meest voordelige golfbreker geoptimaliseerd kan worden. Bij elke oplossingsvariant in de **long list** en de **short list** is nagegaan of de opgestelde varianten voldoen aan de eisen van de opdrachtgever. Als deze niet aan de eisen van de opdrachtgever voldoen, wordt het niet meegenomen.
7 Analyseren en onderzoeken 7.1 Uitvoeren van onderzoek	A.1, A.2, B.2, B.3, B.4, B.5	In het **final report** staat globaal aangegeven welke ontwerpstappen er zijn genomen en waarom. Daarnaast is er **project plan** een afstudeer proces gemaakt, die in grote lijnen is aangehouden. Binnen dit proces is alleen het aardbevingsontwerp en de hoofden van de golfbrekers niet meegenomen, wegens een verschuiving aan prioriteiten binnen het project. De variantenstudie koste meer tijd dan verwacht om het gewenste eindresultaat te behalen.

Competenties met Kerntaken	Beoordeling	Wat heb je gedaan? Waar is dat terug te vinden in het rapport?
8 Communiceren en samenwerken 8.1 Verwoorden en verbeelden van informatie 8.2 Functioneren in teams	C.2, C.3, C.4	De rapportage is ingevuld door gebruik te maken van tekst en zo veel mogelijk illustraties. Tijdens de verschillende fasen in het proces is er advies ontvangen van collega's van RHDHV. Door dit advies te bespreken en te verwerken met collega's is er ook aan de competentie werken in teams gewerkt.
9 Management en ondernemen 9.1 Regie voeren over eigen leerproces 9.2 Projectmatig werken en processen aansturen	A.2, B.2, C.1, C.2	Elke stap in het proces is van tevoren doorgesproken met mijn afstudeerpartner, zodat wij beide wisten welke stappen we in elke fase gingen nemen. Op deze manier is het afstudeer proces gedurende de gehele afstudeer periode gemanaged en zijn er zo min mogelijk onduidelijkheden ontstaan tijdens het afstuderen.

5 APPENDIX I: BEDRIJFSBEOORDELING JESPER VAN GRIEKEN

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HOGESCHOOL ROTTERDAM Instituut voor de Gebouwde Omgeving G.J. de Jonghweg 4-6 3015 GG Rotterdam	**BEOORDELING BEDRIJF AFSTUDEREN civAFS40** (in te vullen door de bedrijfsbegeleider)

Om tot een goede beoordeling van het afstudeerwerk te komen hoort de opleiding graag het oordeel van het bedrijf over een aantal beroepsvaardigheden/ competenties van de student.
Deze beoordeling wordt als advies meegenomen in de beoordeling van de afstudeerder op het aspect 'functioneren in het bedrijf'.

Naam student	**Jesper van Grieken** (samen met Danny Janssen)
Studentnummer	**0853476**
Opleiding	**Civiele Techniek**

Wat vindt u van de snelheid waarmee de student wegwijs is geworden in het bedrijf en het afstudeeronderwerp?

Goed

Wat vindt u van de aanpak die de student heeft gekozen? Was er aandacht voor de bedrijfscontext, een passende theoretische onderbouwing, een goede balans tussen hoofdlijnen en details?

Goede aandacht voor de bedrijfscontext; zowel geluisterd naar voorkeur van bedrijf hoe vertrouwelijkheid aan te pakken als naar de inhoudelijke wensen van bedrijf.
Niet altijd even bedachtzaam in de uitwerking; af en toe weer op het juiste pad moeten brengen (wordt dan uitstekend opgepakt).
Vanaf dag 1 de hoofdlijnen in de gaten gehouden. Hier en daar geplande details moeten laten vallen; overall goede detail accenten.

Heeft de student het proces goed gemonitord en tijdig bijgestuurd? Heeft hij het proces afgestemd met alle partijen?

Vooraf goede planning gemaakt en regelmatig bijgestuurd. Moest ongevraagd af en toe (kritische) sturing geven vanuit bedrijf. Door optimistische houding tijdens proces op eind van de rit wat tijdnood, maar grotendeels gecompenseerd door tomeloze inzet.

Wat is uw oordeel over de zelfstandigheid van de student? Heeft hij eigen initiatief genomen om contacten te leggen en te onderhouden? Om feedback te vragen? Om te informeren op een onderbouwende en overtuigende wijze?.

Zeer zelfstandig om, conform planning, voortgang te realiseren. Had meer om feedback tussendoor kunnen vragen; had dus kritischer kunnen zijn op eigen aanpak.

Wat vindt u van de samenwerking met de student, de wijze waarop hij communiceert, de wijze waarop hij met afspraken omgaat, met meningsverschillen omgaat?

Jesper is open in communicatie; vlot van de tongriem gesneden. Altijd positief en kan goed omgaan met druk en aanpassingen: wil het oplossen. Heeft (te) snel zijn mening klaar (verbeterpunt), maar luistert vaak wel in tweede instantie. (Te) terughoudend geweest met inschakelen van RHDHV collega's!

Bent u tevreden met het eindresultaat van de afstudeeropdracht? In hoeverre is het verrassend/vernieuwend? Is de voorgestelde oplossing acceptabel voor het bedrijf/organisatie en de beroepspraktijk? Welke elementen zijn bruikbaar voor uw bedrijf?

Tevreden met het eindresultaat. Afstudeerders hebben een vernieuwend concept ontwikkeld. Goed bruikbaar voor vervolgstudie en eventueel nadere uitwerking tbv een geinteresseerde klant.

Advies cijfer functioneren bedrijf: 7,8

Naam:	Bedrijf:	Handtekening:
R.J.H. Stive	**Royal HaskoningDHV**	

Datum: **15 juni 2015**

6 APPENDIX II: BEDRIJFSBEOORDELING DANNY JANSSEN

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HOGESCHOOL ROTTERDAM Instituut voor de Gebouwde Omgeving G.J. de Jonghweg 4-6 3015 GG Rotterdam	**BEOORDELING BEDRIJF** **AFSTUDEREN civAFS40** (in te vullen door de bedrijfsbegeleider)

Om tot een goede beoordeling van het afstudeerwerk te komen hoort de opleiding graag het oordeel van het bedrijf over een aantal beroepsvaardigheden/ competenties van de student.
Deze beoordeling wordt als advies meegenomen in de beoordeling van de afstudeerder op het aspect 'functioneren in het bedrijf'.

Naam student	**Danny Janssen** (samen met Jesper van Grieken)
Studentnummer	**0853162**
Opleiding	**Civiele Techniek**

Wat vindt u van de snelheid waarmee de student wegwijs is geworden in het bedrijf en het afstudeeronderwerp?

Goed

Wat vindt u van de aanpak die de student heeft gekozen? Was er aandacht voor de bedrijfscontext, een passende theoretische onderbouwing, een goede balans tussen hoofdlijnen en details?

Goede aandacht voor de bedrijfscontext; zowel geluisterd naar voorkeur van bedrijf hoe vertrouwelijkheid aan te pakken als naar de inhoudelijke wensen van bedrijf.
Niet altijd even bedachtzaam in de uitwerking; af en toe weer op het juiste pad moeten brengen (wordt dan uitstekend opgepakt).
Vanaf dag 1 de hoofdlijnen in de gaten gehouden. Hier en daar geplande details moeten laten vallen; overall goede detail accenten.

Heeft de student het proces goed gemonitord en tijdig bijgestuurd? Heeft hij het proces afgestemd met alle partijen?

Vooraf goede planning gemaakt en regelmatig bijgestuurd. Moest ongevraagd af en toe (kritische) sturing geven vanuit bedrijf. Door optimistische houding tijdens proces op eind van de rit wat tijdnood, maar grotendeels gecompenseerd door tomeloze inzet.

Wat is uw oordeel over de zelfstandigheid van de student? Heeft hij eigen initiatief genomen om contacten te leggen en te onderhouden? Om feedback te vragen? Om te informeren op een onderbouwende en overtuigende wijze?.

Zeer zelfstandig om, conform planning, voortgang te realiseren. Had meer om feedback tussendoor kunnen vragen; had dus kritischer kunnen zijn op eigen aanpak.

Wat vindt u van de samenwerking met de student, de wijze waarop hij communiceert, de wijze waarop hij met afspraken omgaat, met meningsverschillen omgaat?

Danny is een introvert persoon. Wordt verbaal overvleugeld door zijn medeafstudeerder Jesper. Luister goed, blijft rustig en werkt gestaag. Geeft maar af en toe wat feedback en/of tegengas; verbeterpunt. Is degelijk en levert volgens afspraak: "niet lullen maar poetsen". In een op een gesprekken loopt communicatie wat beter. (Te) terughoudend geweest met inschakelen van RHDHV collega's!

Bent u tevreden met het eindresultaat van de afstudeeropdracht? In hoeverre is het verrassend/vernieuwend? Is de voorgestelde oplossing acceptabel voor het bedrijf/organisatie en de beroepspraktijk? Welke elementen zijn bruikbaar voor uw bedrijf?

Tevreden met het eindresultaat. Afstudeerders hebben een vernieuwend concept ontwikkeld. Goed bruikbaar voor vervolgstudie en eventueel nadere uitwerking tbv een geinteresseerde klant.

Advies cijfer functioneren bedrijf: 7,8

Naam:	Bedrijf:	Handtekening:
R.J.H. Stive	**Royal HaskoningDHV**	
Datum: **15 juni 2015**		

7 APPENDIX III: PUBLICATIEVERKLARING

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HOGESCHOOL ROTTERDAM Instituut voor de Gebouwde Omgeving G.J. de Jonghweg 4-6 3015 GG Rotterdam	**VERKLARING** **M.B.T. PUBLICATIE VAN HET AFSTUDEERVERSLAG** (in te vullen door het bedrijf)

GEGEVENS AFSTUDEERVERSLAG		
Titel:		
Auteurs:	Naam	Studentnummer
	Jesper van Grieken	0853476
	Danny Janssen	0853162
Bedrijf:	Naam: Royal HaskoningDHV	
	Straat: George Hintzenweg 85	
	Postcode en plaats: 3009 AM Rotterdam	
	Telefoon: +31 88 348 90 00	
	Contactpersoon: Ronald J.H. Stive	
	Functie:Project Directeur Maritime & Waterways	
	E-mail:ronald.stive@rhdhv.com	

VERKLARING

Hierbij verklaar ik *(aankruisen wat van toepassing is)*

	Het bovenstaande afstudeerverslag moet in zijn geheel vertrouwelijk behandeld worden. Dit is ook aangegeven op het afstudeerverslag zelf.
X	Het bovenstaande afstudeerverslag mag gebruikt worden voor interne onderwijsdoeleinden.
X	Het afstudeerbedrijf gaat akkoord met plaatsing van het bovenstaande afstudeerverslag in de HBO-kennisbank (zie www.hbo-kennisbank.nl). *N.B. Wordt / is er door de studenten hiervoor een speciale publicatieversie gemaakt?* ***Publicatieversie uitbrengen ZONDER opname van Appendix A8, Breakwater optimization***
Ruimte voor opmerkingen en toelichting:	

Ondertekening :

Naam: **R.J.H. Stive**	Handtekening en stempel bedrijf:
Functie: zie boven	Datum **15 juni 2015**

Royal HaskoningDHV
Enhancing Society Together
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8 APPENDIX IV: TOESTEMMINGSFORMULIER STUDENT HBO-KENNISBANK

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Toestemmingsformulier juni 2014 (in te vullen door de student)

Toestemmingsformulier tot opname en beschikbaarstelling afstudeerscriptie in een digitale kennisbank

Datum	15-06-2015
Naam student	Jesper van Grieken Danny Janssen
Studentnummer	0853476 0853162
Naam instituut	IGO
Opleiding	Civiele Techniek
Afstudeerrichting	Bachelor
Titel	Innovative breakwater
Toestemming	Ja / ~~nee~~ Behalve appendix 8, Breakwater optimization
Emailadres	jespervangrieken@gmail.com janssendanny4@hotmail.com

Digitale kennisbank
De Hogeschool Rotterdam heeft een digitale kennisbank opgezet waarin de Hogeschool scripties die door studenten in het kader van hun studie aan de Hogeschool hebben geschreven, toegankelijk worden gemaakt voor derden. Hierdoor wordt het proces van creatie, verwerving en deling van kennis binnen het onderwijs mogelijk gemaakt en ondersteund.

De in de kennisbank opgenomen scripties worden toegankelijk gemaakt voor potentiële gebruikers binnen en buiten de Hogeschool. Om opname en beschikbaarstelling mogelijk te maken dient dit toestemmingsformulier.

Bij opname en beschikbaarstelling in de digitale kennisbank behoudt de student zijn of haar auteursrecht. Daarom kan hij of zij de toestemming tot het beschikbaar stellen van haar / zijn afstudeerscriptie intrekken.

Rechten en plichten student
De student verleent aan de Hogeschool kosteloos de niet exclusieve toestemming om zijn afstudeerscriptie op te nemen in de digitale kennisbank en om deze beschikbaar te stellen aan gebruikers binnen en buiten de Hogeschool. Hierdoor mogen gebruikers de afstudeerscriptie geheel of gedeeltelijk kopiëren en bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie en/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.

De student geeft de Hogeschool het recht de toegankelijkheid van de afstudeerscriptie te wijzigen en te beperken indien daar zwaarwegende redenen voor bestaan.

De student verklaart dat de stagebiedende organisatie dan wel de opdrachtgever van de afstudeerscriptie geen bezwaar heeft tegen opname en beschikbaarstelling van de afstudeerscriptie in de digitale kennisbank.

Paraaf:

Toestemmingsformulier januari 2009 (in te vullen door de student)

Verder verklaart de student dat hij of zij toestemming heeft van de rechthebbende van materiaal dat de student niet zelf gemaakt heeft om dit materiaal als onderdeel van de afstudeerscriptie op te nemen in de digitale kennisbank en aan derden binnen en buiten de Hogeschool beschikbaar te stellen.

Rechten en plichten Hogeschool

De door de student verleende niet-exclusieve toestemming geeft de Hogeschool het recht de afstudeerscriptie aan gebruikers binnen en buiten de Hogeschool beschikbaar te stellen.
De Hogeschool mag verder de afstudeerscriptie voor gebruikers binnen en buiten de Hogeschool vrij toegankelijk maken voor een gebruiker van de digitale kennisbank en mag deze gebruiker toestemming geven om de afstudeerscriptie te kopiëren en te bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie- en/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.
De Hogeschool zal ervoor zorgen dat vermeld wordt wie de schrijver(s) is/zijn van de afstudeerscriptie waarbij zij tevens aangeeft dat bij gebruik van de afstudeerscriptie de herkomst hiervan duidelijk vermeld moet worden. De Hogeschool zal duidelijk maken dat voor ieder commercieel gebruik van de afstudeerscriptie toestemming van de student nodig is.

De Hogeschool heeft het recht de toegankelijkheid van de afstudeerscriptie te wijzigen en te beperken indien daar zwaarwegende redenen voor bestaan.

Rechten en plichten gebruiker

Door dit Toestemmingsformulier mag een gebruiker van de digitale kennisbank de afstudeerscriptie geheel of gedeeltelijk kopiëren en/of geheel of gedeeltelijk bewerken. Gebruikers mogen dit alleen doen en de resultaten publiceren indien dit gebeurt voor eigen studie- en/of onderwijs- en onderzoeksdoeleinden en onder de vermelding van de naam van de student en de vindplaats van de afstudeerscriptie.

Naam: Jesper v. Grieken	Handtekening: D. JANSSEN Datum: 15-6-15