Decision Support Method for the Initial Selection of a Breakwater Alternative in Dutch Inland Waterways

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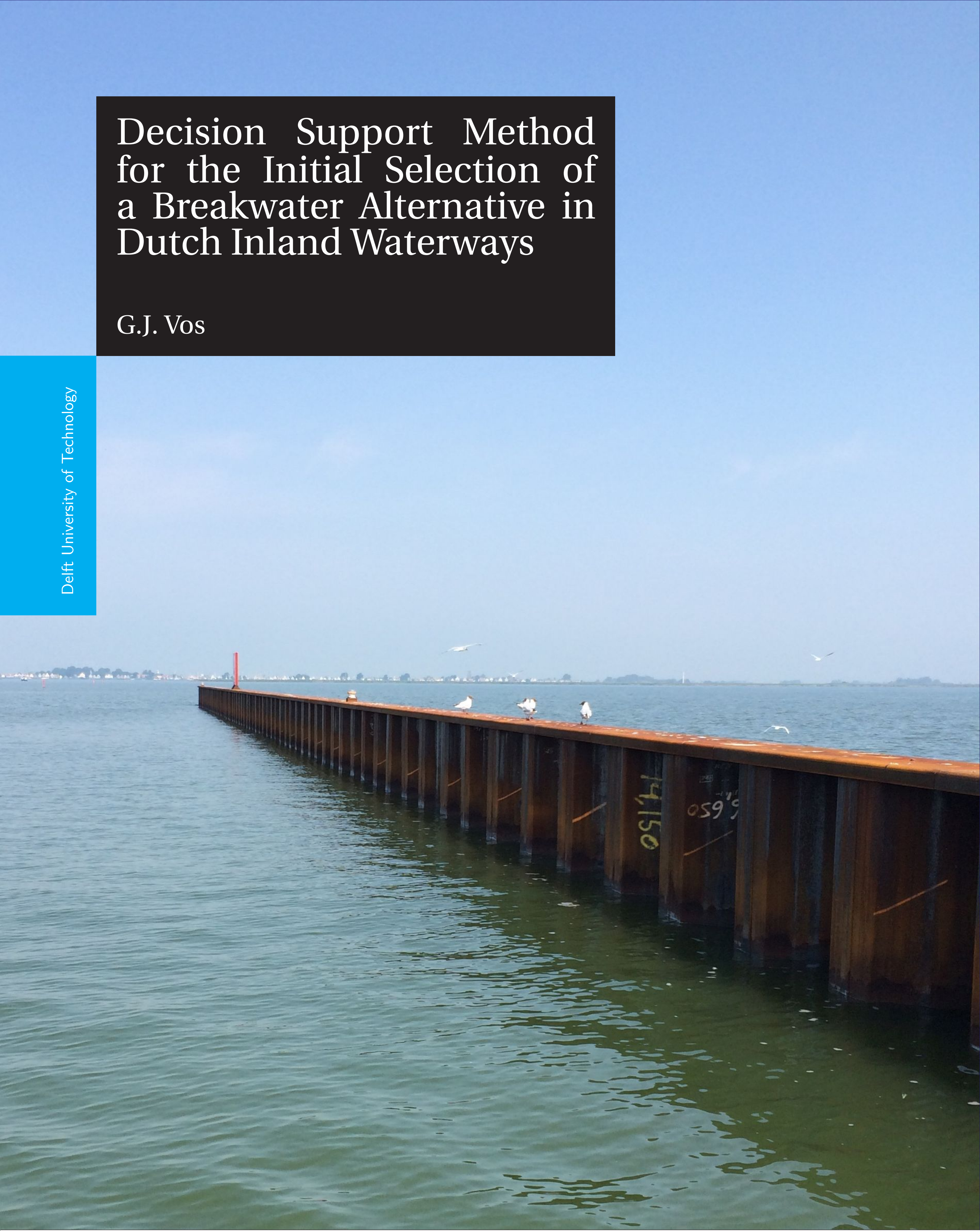

TU Delft Delft University of Technology

Hakkers CONSTRUCTIEF IN WATERBOUW

Challenge the future

Decision Support Method for the Initial Selection of a Breakwater Alternative in Dutch Inland Waterways

by

G.J. Vos

in partial fulfilment of the requirements for the degree of

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Thesis committee

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"Effiency is doing things right,
effectiveness is doing the right things."
(Peter Drucker, 1909-2005)

Abstract

Many breakwater projects start with the selection phase which consists of the consideration of alternatives. These breakwaters are required to protect ports, banks and habitat areas from significant wave action. Moreover, upcoming Design & Construct contracts drive contractors to perform the consideration of alternatives and design themselves. Therefore, knowledge about the design of breakwaters is highly preferred in those cases. This leads to less of outsourced work and reduction in engineering time.

In 2013, contractor Hakkers designed and constructed a small-scale breakwater to protect a marina in IJburg, Amsterdam. The constructed breakwater was built in inland waterways where the magnitudes of the boundary conditions are limited (for example, small water depths and wave heights). Compared to coastal areas with more significant boundary conditions, this leads to less detailed structures without crown wall or multi-layers, to breakwaters with smaller dimensions and to other breakwater alternatives (e.g., synthetic and reef breakwater).

The goal was to develop a comprehensive method for a quick selection of a breakwater alternative in Dutch inland waterways. The method had to provide knowledge about breakwater designs and support a factual assessment of breakwater alternatives. Furthermore, the EMVI (Economical Most Beneficial Registration) tender approach would be included as well, since it is more frequently applied to civil engineering projects. This implies that the focus is on an effective breakwater and an efficient design procedure.

In the literature study it is found that the experience in the design and construction of breakwaters developed by trial-and-error, which is only true for large-scale breakwaters in coastal areas. In fact, the scope of inland waterways in The Netherlands is considered in which lakes and rivers as optional locations are distinguished. In these water systems the defined ‘breakwater’ and ‘dam’ structure are found. The design of these structures fit in the general design process where firstly promising alternatives are obtained and secondly a decision is made under consideration of technical assumptions and predicted impacts. This process can be advanced by parametric design which implies that variables are standardized. Thereby, variables in the hydraulic aspects and construction elements can be quickly elaborated.

In this thesis a decision system method (DSM) is developed which enables designers to systematically compare multiple breakwater alternatives. It consists of straightforward guidelines in sixteen steps including a ‘pre-selection’ and ‘cost-based selection’. The pre-selection criteria are requirements, contracts, laws, permits and regulations, and pre-limiting conditions (location and purpose, and unfavourable boundary conditions). In this phase the considered breakwater alternatives are qualitatively assessed which results in a quick insight. As a result, in the subsequent steps a smaller number of breakwater alternatives is investigated which saves time of engineering. In these steps the structural performances, dimensions and cost estimates are considered. What is more, the dimensions in height and width enable a material, labour and equipment cost estimates. Including an EMVI discount, the cost-based selection can be performed on the pre-selected breakwaters. The breakwater which meets the functional requirements, show structural competence and is the lowest in costs shall be recommended to develop in further design phases.

From the results it can be concluded that the DSM meets the goal and supports the choice of the most effective breakwater in Dutch inland waterways. The efficiency of the method is found in a quick procedure, which is a result of selection tables and classification of variables. On the other hand, the most effective breakwater is assured by a comprehensive assessment which is due to the incorporation of extra aspects (for example, tender approach, contract, legislation and pre-limiting conditions) besides the requirements and costs. This is an advantage compared to other methods which consider only the requirements and/or costs. Also a factual consideration is established by excluding weighing factors and subjective scores. Apart from this, two case studies have been elaborated, which show that the governing aspects are the functional requirements and the costs estimates. These also provide first insight in the construction costs. The method also anticipates to EMVI fictitious discounts as a selection criterion.

PREFACE

This thesis finalises my master degree in hydraulic engineering. As a master student with the specialisation hydraulic structures, I preferred to graduate in the field of breakwaters as this type of structure had caught my attention. After contacting the company Hakkers, a broad graduation topic was created, which was both interesting as challenging to me. The initial idea changed quickly in the thesis preparation phase. This idea was to perform an extensive study on the selection of a breakwater alternative for a single project. Later on, this request was transformed to the development of a generic method for the consideration of breakwater alternatives. In other words, a large puzzle was planned in which oversight was most important.

I would like to say a special word of thanks to the thesis committee. They have been supporting in many ways to bring this thesis to a higher level of expertise. It has been a privilege to extensively discuss the contents and the works with them to bring it to the point of success. Moreover, they kept me enthusiastic by their own enthusiasm. First of all, I would like to show my gratitude to Bas Jonkman, who has been willing to be the chairman of the committee. He has been able to give me constructive criticism and smart insights to obtain a coherent research. Secondly, I appreciate the shared expertise in breakwaters of Henk Jan Verhagen, who has been my first daily supervisor. He has been guiding me from the point of no objective to the point of achieving the goal. Likewise, I would like to say thanks to my second daily supervisor, Peter van der Linde. He has been helpful in providing structural expertise, construction know-how and cost considerations. This has been a practical contribution to the thesis and my knowledge. It has also been a privilege to work at the company Hakkers BV. Thirdly, I am grateful for both Coen Kuipers and Leon Hombergen as committee members, who were involved in the entire process and provided the finishing touch. I would like to say thanks to Coen's professional insights on breakwaters and his clear ideas on the created method. Frequent meetings with him provided me with the necessary improvements. This has been similar to conversations with Leon, who is specialized in construction processes and contracts. He subjected the developed method regularly to the practical use of it for designers and contractors, which has been an important and unmistakable aspect.

Also the help of family, friends and colleagues of the TU Delft and Hakkers has been appreciated. Especially, a great thanks to my parents, who have been encouraging me continuously. I would also like to mention that the colleagues at the TU Delft and company Hakkers have been supporting as both comrades and experts. It has been great to work with them. Finally, I would like to thank the readers of my thesis. They helped me to finalize the text, which will be an advantage for the upcoming reader. For the latter, I hope you will enjoy reading and implementing the method.

G.J. Vos
Delft, February 2016

CONTENTS

1

INTRODUCTION

For more than 4000 years, breakwaters have been used to dissipate wave energy. The name 'breakwater' originates from the function of the structure: to break the motion of water in waves. The reduction or elimination of wave action is achieved by wave breaking and counteraction of waves by breakwaters consisting of rocks, concrete blocks or piles, for instance. Historically, numerous errors and failures of these structures occurred due to a lack of know-how. As a result, various studies were carried out to evolve knowledge about the engineering and construction.

In 2013, contractor Hakkers was awarded to design and construct a small-scale breakwater of sheet piles in front of a navigation lock in IJburg, Amsterdam. During storm conditions vessels were unable to enter the navigation lock from the lake IJmeer. This was due to a design wave height of approximately 1 m. The concerned lake is a typical inland waterway with limited water depths, fetch lengths and wave heights. Accordingly, the design conditions and sheet pile dimensions should not be compared to large coastal breakwaters. These structures are normally complex with details (e.g., additional crown walls, heavy toe structures and multiple layers of varying stone sizes) and have to sustain higher loads. In contrast, small-scale breakwater in inland waters will have similar design rules, but the boundary conditions are in a smaller order of magnitude. These are characterized by smaller water depths (up to 4 m) and wave heights (up to 2 m), for example. Moreover, other breakwater alternatives are taken into account as well.

In early phases of breakwater designs a decision have to be made for a conceptual alternative. Such a decision requires knowledge to be gathered, design steps to be listed and optional breakwater alternatives to be chosen. Many unknowns are present and a method to obtain quick results and a broad study is not present yet. Therefore, in this thesis a method is developed which focuses on the selection of a breakwater alternative in inland waterways. The developed decision support method (DSM) provides an approach which systematically considers, selects and neglects breakwater alternatives. The multiple design steps enable quick, easy and extensive decision making which result in the most effective breakwater and an efficient design process.

This thesis consists of the following contents. In Chapter 2, a description is provided of the IJburg breakwater project, which contains the problem about the consideration of breakwater alternatives. Subsequently, the goal to develop a DSM is defined to which the research questions and hypotheses are formulated. Prior to, the general background information is provided in Chapter 3. This defines the frame of reference, which includes breakwaters of different scales and features of design processes. In Chapter 4 the method is developed, which consists of multiple design steps. Subsequently, the DSM is tested in two case studies, which can be found in Chapter 5. In Chapter 6 the conclusions and the hypotheses are discussed. The last chapter consists of the recommendations, which can be found in Chapter 7.

2

PROBLEM STATEMENT AND RESEARCH QUESTIONS

In this chapter the breakwater nearby the marina of IJburg is considered. The area of the project is described wherein the local issues led to the involvement of contractor Hakkers BV. The location is broadly described in Section 2.1. Section 2.2 speaks about the consideration of alternatives. In this study a problem has been revealed, which is described in Section 2.3. The problem led to a goal defined in Section 2.4. These are the introduction to formulate the research questions in Section 2.5, which give direction to the literature study and overall research. In Section 2.6 hypotheses are proposed to the presumed outcomes of the questions.

2.1. LOCATION ANALYSIS

IJburg is a region of Amsterdam which is build on islands in the lake IJmeer (Figure 2.1). In 1965 the architects Van den Broek and Bakema had the idea to construct a city in the nearby lake (Gemeente Amsterdam Stadsdeel Oost, 2014). Therefore, these man-made islands should provide a living area, mainly for inhabitants of Amsterdam.

Figure 2.1: IJburg Overview
(Google. (2014). *Google Maps*. Retrieved from https://maps.google.nl/. Accessed on September 9, 2014.)

In 1980, the ideas of the architects were implemented by the Municipality Council of Amsterdam, which resulted in creation of several islands. As a result, IJburg consist of Steigereiland, Rieteiland, Middeneiland, Centrumeiland and Haveneiland (Figure 2.2).

Figure 2.2: IJburg Islands Overview
(Google. (2014). *Google Maps*. Retrieved from https://maps.google.nl/. Accessed on September 9, 2014.)

One of the showpieces of Haveneiland is the IJburg marina. Sailing boats and small vessels (up to 12 meter in length) can enter the small harbour by using the navigation lock in the Bert Haanstrakade (Figure 2.3). One can distinguish the berth places and the boat passage. Moreover, two guide works of approximately 40 m and 60 m are present, extending the lock chamber at both ends. These structures provide slackening and wait opportunities for vessels. Apart from this, the lock chamber itself is approximately 40 m by 16 m and can be closed by two horizontal sliding doors. Within the vessel passage, a two lane road and movable overpass have been integrated.

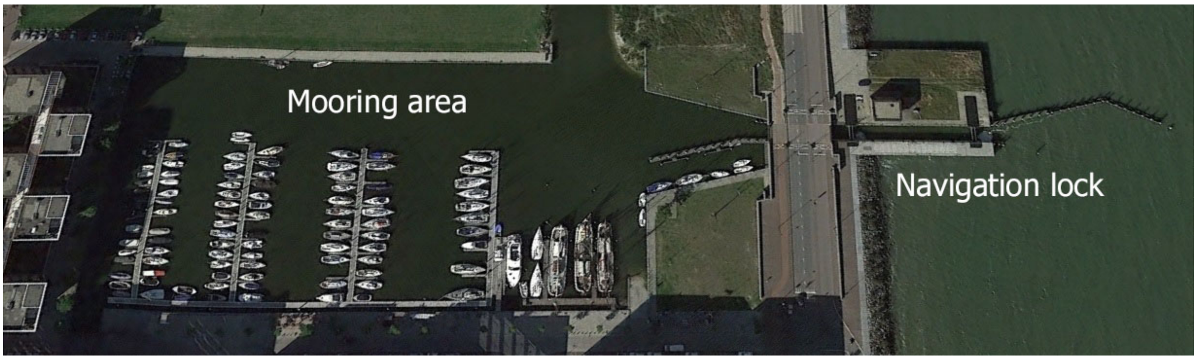


Figure 2.3: IJburg Marina and Navigation Lock
(Google. (2014). *Google Maps*. Retrieved from https://maps.google.nl/. Accessed on September 9, 2014.)

2.2. REFERENCE PROJECT

During severe wind conditions, waves can prevent vessels from entering the navigation lock. Consequently, the Municipality of Amsterdam requested measures by means of a hydraulic structure to increase the accessibility and navigability. The advised structure to take into account was a detached breakwater (Timmermans and Bemmelen, 2013).

The contractor of the works would be responsible for the design, construction and 15-years of maintenance. The company should be performance-focused, professional and qualified, in order to reduce the risk associated with the project and enhance the quality. After the public tender Hakkers was awarded with the project. In addition, the tender was valued by the principles of the Best Value Procurement, which prescribed criteria about the optimization of the finances and the project risks. Shortly after the selection phase and pre-award phase (Haghgoo Daryasari, 2013a,b), all required boundary conditions (for example, ice thickness and wave heights) were determined. Accordingly, in the study of the breakwater alternatives, six types of breakwater were considered (Staphorsius, 2013), namely:

1. Rubble mound breakwaters (e.g. quarry stone, tetrapods and granite blocks);
2. Caisson breakwater (made of reinforced concrete);
3. Floating breakwater (made of reinforced concrete or other materials);
4. Pile breakwater (made of wood);
5. L-wall breakwater (made of concrete);
6. Vertical wall breakwater (made of sheet piles or (permeable) panels on piles).

Using a trade-off matrix, including the requirements and client's conditions, the alternatives were qualitatively analysed. Based on the results the vertical wall breakwater was chosen to design further into preliminary design (Figure 2.4).

Figure 2.4: Marina and Breakwater
(Google. (2014). *Google Maps*. Retrieved from https://maps.google.nl/. Accessed on September 9, 2014.)

The two vertical wall alternatives were quantitatively and structurally studied. It concerned prefab concrete plates and steel piles (Hooijschuur, 2013). Due to significant ice loading, the prefab concrete wall was disapproved. Contrarily, the steel piles were chosen, because these were able to resist the design load. The final design eventually consisted of a steel sheet pile wall with a U-profile capping beam (Figure 2.5).

Figure 2.5: Sheet Pile Breakwater of IJburg

2.3. PROBLEM DESCRIPTION

From a *practical* point of view, the determination of the boundary conditions and the further design procedure did not fit within the works of contractors in general. Traditionally, contractors received RAW (Consent Registration Works) tenders. Thus, the design process belonged to engineering and consultancy companies. Nowadays, more D&C (Design & Construct) contracts are appropriated to contractors. The purpose of these contracts is to give contractors the opportunity to fit the design to his equipment, in order to have the best economical solution. Moreover, the D&C contracts require the development of a design with the determination of the boundary conditions and requirements as starting point. In the majority of the design requests of contractors, the boundary conditions are still provided by the client. If this is not the case, contractors are more likely to outsource the determination of the boundary conditions to an engineering consultant. Thus, contractors would have had to gain and apply knowledge simultaneously to obtain similar results as the engineering consultant for a design in the same amount of time. On the other hand, when designers of contractors are available and there is well-developed knowledge in the field of expertise, it pays off to prepare the design internally. This is also the case in the field of breakwaters. Yet, a method and guideline containing knowledge for the design of breakwaters is absent. Starting point is the phase of consideration of breakwater alternatives and the selection of the most effective breakwater. Accordingly, due to a lack of knowledge and design steps, there is a high probability of inefficiency in the design process.

From a *scientific* point of view, it can be concluded that there is no clear distinction between large-scale breakwaters in coastal areas and relatively simple breakwaters in inland waterways. Design manuals like States Army Corps of Engineers; Coastal Engineering Research Center (1984), CIRIA, CUR, CETMEF (2007) and various PIANC/Marcom reports, mainly discuss the breakwaters of high technical level. These breakwaters are designed for relatively large water depths and wave heights, respectively 7-15 m and 6-9 m in the port of Scheveningen (Pilarczyk, 2000), for instance. This includes breakwaters with multiple filter layers, various materials, varying slopes, toe structure and/or crown wall. As a consequence, smaller breakwaters in inland waterways (characterised by relatively small water surfaces and limited water depths) are designed by a more extensive approach and with irrelevant details, resulting in high engineering and construction costs, according to CUR (2000). Moreover, a document containing various inland breakwaters and a method to assess these structures in a quick manner is not developed yet. As a result, there is a high probability of an inefficient design process, which may lead to a less cost-effective structure.

2.4. GOAL DEFINITION

The goal is to develop a method for the initial selection of a breakwater alternative in Dutch inland waterways. Therefore, the method will consist of factors of influence and multiple breakwater alternatives which both show affection with inland waters. The most effective breakwater alternative can be recommended accordingly.

The method shall be simplified to perform the assessment in an extensive, yet quick and simple to use manner. In other words, the various factors of influence should be clustered and the interaction with the breakwater alternatives should be known. Moreover, unfavourable breakwaters will be eliminated in an early stage in order to save time in the subsequent steps which consider dimensions and costs per breakwater alternative.

The factors of influence are roughly: start-up documents, legislation, boundary conditions, the requirements, the EMVI criteria, design aspects (e.g., load-structure interaction), materials, construction works. The mentioned design aspects should take into account international accepted design guidelines, engineering judgement, simple numerical models and rules of thumb. In this matter, globally accepted results are obtained in the correct order of magnitude by rather simple computations.

The method will consist of clear and straightforward design steps which result in an efficient elaboration. Therefore, the document will be written in such a way that conclusions can be drawn systematically. Given the limited time and money of a contractor, this guideline will supports decision making in short-term based on a broad and well-justified study. Additionally, the above explained selection phase is preparatory to the sketch design and/or conceptual design phase.

2.5. RESEARCH QUESTIONS

Based on the goal definition, the following main-research question has been formulated:

What is an efficient method for the selection of the most effective breakwater alternative in Dutch inland waterways?

The question suggests information and knowledge about breakwaters to be collected. This is market out by frame of reference which consists of the consideration of breakwater alternatives and the inland waterways conditions. The research question also implies a method, which can be implemented in a limited amount of time and considers sufficient alternatives. The result is saving costs related to the time of design, the type of structure and the phase of construction.

The words *effective* and *efficient* are essential in which effectiveness can be defined in two different ways. A breakwater can be effective in functionality and cost. In other words, it should meet the aim of the client and the contractor by fulfilling the requirement and providing the lowest cost. On the other hand, there is efficiency, which means that a breakwater design is accomplished in a minimum amount of time and with the least effort. Moreover, effectiveness and efficiency are closely related. In this matter, the preferred results are obtained in limited time. The latter implies less design costs, which is referred to as *cost-effective*.

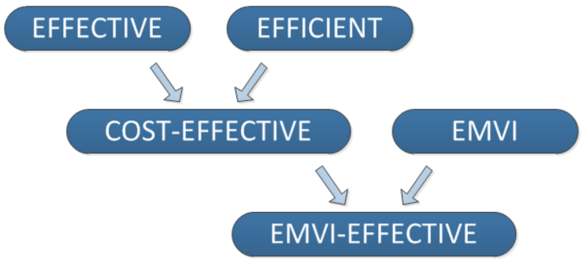


Figure 2.6: Effectiveness, Efficiency, Cost-effectiveness and EMVI-effective

Nowadays, *EMVI* tendering is frequently applied to civil engineering projects by clients. The projects are characterized by formulated EMVI criteria, which can obtain an EMVI score. Typical criteria are, for example, ecology, durability or noise. The score of these criteria is translated to a fictitious discount, which is subjected to the total project cost. When a designer is able to predict the fictitious discount, the most expensive structure can correspond to the lowest total costs. This is an example of *EMVI-effective*.

The main-research question is divided into the following sub-research questions:

What are suitable breakwater alternatives when the requirements, criteria, type of contract and tender approach are provided?

How to consider the governing boundary and initial conditions in order to reject breakwater alternatives?

How are the breakwater alternatives and characteristic approaches adapted to the classified design conditions?

How do the equipment and construction methods contribute in the selection?

What is the most favourable breakwater alternative in terms of meeting requirements and costs?

The questions imply that multiple criteria have an effect on the breakwater selection. Insight, meaning, approach and weighing of these factors will be clarified.

2.6. HYPOTHESES

As a result of the method that originates from the research questions, it is expected that:

- *the DSM will result in a different breakwater alternative than the chosen IJburg breakwater, which consists of steel sheet piles.*
- *the choice of a breakwater alternative will be different when a cost estimate is provided besides the requirements.*
- *compared with previous performed considerations of breakwater alternatives, the DSM provides the same results in less time.*

Higher project costs can be avoided by the gained time and the work done in-house, but also by the dimensions of the breakwater alternatives and the consideration of the construction costs. For example, the costs of construction (e.g. material, time and effort) of a sheet pile wall could exceed that of a block wall, while comparing these alternatives exclusively by the requirements of the client would result in the sheet pile wall as the best option.

To conduct a study of breakwater alternatives, research has to be carried out. The limited special circumstances of a project do not outweigh the well-known design steps in the majority of the work. Since all the research of the design steps is done to conduct a set of guidelines, a significant amount of time is gained.

3

General Background

This chapter consists of general knowledge to define the research scope. In Section 3.1 the history of the design of large-scale breakwaters is considered. Subsequently, typical breakwaters and Dutch inland waterways are discussed in Section 3.2. In Section 3.3 the position of the method in the design process is shown. The chapter is concluded by a description of the advantage of parametric design in Section 3.4.

3.1. History of Large-scale Breakwaters

This section considers the development in knowledge of large-scale breakwaters in coastal areas. As a result, the type of structure, the experience in design and construction of breakwaters, and unknown inland breakwaters are clarified. These form the basis of this research.

Breakwaters are hydraulic structures used as measure of protection. These structures primarily protect port areas from wave attack where vessels enter, manoeuvre and berth. Beaches (with valuable habitat) which are threatened by significant erosion could also be protected. Besides the ports and beaches, breakwaters can also be utilized to prevent significant siltation in waterways.

Figure 3.1: Masonry Breakwater, Dover
(Geograph. (2010). *Southern Breakwater (East End), Dover.* Retrieved from http://www.geograph.org.uk/. Accessed on September 23, 2014.)

In history, many failures of breakwaters occurred due to the principle of trial-and-error as a design method.

Yet, 2000BC Egyptians constructed a masonry breakwater of stones in Alexandria (Takahashi, 2002). The Greeks were also working on breakwaters consisting of mounds of rubble, which can be seen along the Greek coast. In contrast, the Romans came up with monolithic structures of concrete.

Due to European trade overseas, breakwaters became popular and necessary in the 19th century. For example, England was proceeding with rubble mound and monolithic breakwaters. When a monolithic breakwater survived with structural problems, the Englishmen started to combine these two in deep water conditions (Figure 3.1). The first composite breakwater was created in Dover (1844). Failures still occurred due to instability of the berm by significant wave action.

The engineers in France tried to solve the instability issues. They decided to continue with milder slopes above seawater level, heavier concrete blocks and smaller stones in the core. The first breakwater to include these features was constructed in Marseille (1845). It was a French achievement, but it required a lot of heavy armour material.

In Italy they found the solution in a composite breakwater (half rubble mound and half monolithic with a berm) for deep water conditions in the 20th century. Unfortunately, these structures could not resist the breaking waves, which were created by the rubble mound berm. At that time, the International Association for Hydraulic Research (IAHR) was founded to investigate the various failure mechanisms of breakwaters. However, all this research was focussed on large breakwaters and for heavy wave attack.

Years later, the French came up with the idea of interlocking to replace the rubble mound structures (1949). As a result, many types of concrete blocks were developed (e.g. Tetrapod, Dolos, etc.) by designers and companies. Since 1945 technology have been evolving which resulted in more stable structures and more know-how about breakwaters.

Currently, breakwaters are designed by both conservative as well as improved design rules, and still many design rules are reconsidered. Obviously, the experience in the design, the construction and the maintenance of the breakwaters plays a major role. For example, brick breakwaters are not constructed any more, because these are expensive to construct due to the intensive labour. Contrarily, more composite breakwaters (e.g., caisson plus rubble mound) have been applied and many new concrete blocks are designed.

The description of the history shows the development of knowledge and experience of breakwater designs over the years. Also the focus on the large coastal structures is emphasized. Thus, inland breakwaters have not yet been recognized as a separate group, although in this thesis they will be considered as such. These smaller breakwaters can be designed in accordance with the same design rules, but are more simplified structures and have other alternatives.

3.2. SMALL-SCALE BREAKWATERS IN INLAND WATERWAYS

As mentioned above, there is a difference between large coastal breakwaters and small inland breakwaters. In this section, the characteristics of inland waterways, inland breakwaters and river groynes are discussed. This is done in order to understand the local conditions and type of structures considered in this research.

3.2.1. INLAND WATERWAYS

Inland waters can globally be divided into: lakes, rivers, canals and creeks (CUR, 2000). Lakes are characterized by relatively large surface areas of water and with a certain fetch length that can be affected by wind. In case of rivers and canals significant wind-waves are less present due to a small fetch length, but vessel-generated waves and discharges should be taken into account. Creeks are smaller open channels in which less transport of water is observed. While waves are not present, flow velocities can be adjusted by obstacles (e.g., loose rock). A less relevant, but more extensive distinction of surface waters in The Netherlands is found in Table 3.1. The English equivalents are shown accompanied by the Dutch terms.

Table 3.1: Types of Surface Waters (CUR, 2000)

River Systems	**Functional Waters**
Wells (Bronnen)	Drinking Pools (Drink Poelen)
Brooks (Beken)	Urban Waters (Stadswateren)
Small Rivers (Kleine Rivieren)	Ditches (Sloten)
Rivers (Rivieren)	Streams (Weteringen/Vaarten)
	Canals (Kanalen)
	Harbours (Havens)
Stagnant Waters	**Brackish Waters**
Ponds (Vennen/Pingoruines)	Natural/Digged Pools (Dobben)
Whirls (Wielen/Kolken)	Terrain Layer Dike (Inlagen)
Outdated River Branche (Oude Rivierarm)	Creeks (Kreken)
	Sand, Gravel, Clay Pits (Zand-, Grind-, Kleigaten)
	Ditch Holes (Petgaten)
	Lakes and Puddles (Meren en Plassen)

Rivers and lakes are the surface waters considered in this thesis. It is assumed that small waters like canals and creeks do not require breakwaters.

3.2.2. INLAND BREAKWATER TYPES

In the Dutch inland waters, four types of breakwaters can be recognised, which are affected by loads from waves and flows. These go by the names: breakwaters, dams, groynes and sills (CUR, 2000). While breakwaters dissipate wave energy only, dams, groynes and sills carry both wave and flow forces. For example, dams prevent waves to enter the sheltered area, where also habitat area lays (Figure 3.2). What is more, dams and sills do not have a direct hinterland, but are often constructed in front of banks. Hence, the structures have still surface water on both sides.

Figure 3.2: Inland Waterway Breakwater
(Heuvelman Ibis. (2010). *Aanleg oeverbescherming Reitdiep*. Retrieved from http://www.heuvelman-ibis.nl/. Accessed on November 27, 2014.)

In this thesis only the breakwaters and dams are considered. These are both referred to as breakwaters.

3.2.3. RIVER GROYNES

River groynes are found along the major channels, where navigation is enabled. The function of these structures is to guide and change direction of a river. Meanwhile sufficient water depth is guaranteed in the main channel and the banks are protected from erosion. The impact of flow velocities is the main concern during the design and construct phase.

Since the hydronamic processes of river groynes deviate from breakwaters, these structures are not included in the thesis.

3.3. DESIGN PROCESS OF BREAKWATERS

In this section, the phase of the method in this thesis is compared with the general design process of civil engineering structures. Also the various design levels and the aspects to include in breakwater designs are discussed.

3.3.1. DESIGN LEVEL

The design products can globally be divided into Concept Design (CD) at 'macro level', Preliminary Design (PD) at 'meso level' and Detailed Design (DD) at 'micro level'. A absolute distinction between the contents of the mentioned products is not made in practise. Consequently, the contents strongly depend on the registration manual of the client or the choice of the designer.

Figure 3.3: Position of the Phase of Selection

This thesis lays within the phase of selection (Figureß 3.3). It is assumed that the CD can be started after the breakwater alternative is selected. Especially in the phase of selection, optimizations are not relevant, which is in contrast to the further detailing in PD and FD. Therefore, rules of thumb and conservative design rules suffice to do the selection.

3.3.2. DESIGN SEQUENCE

A project is initiated with stating the goals of the object and investigating the feasibility. The complete procedure is visualized in Figure 3.4. In addition, in case of multiple design phases, 'iteration' is inevitable, while complex projects would require 'subsystems' to be investigated.

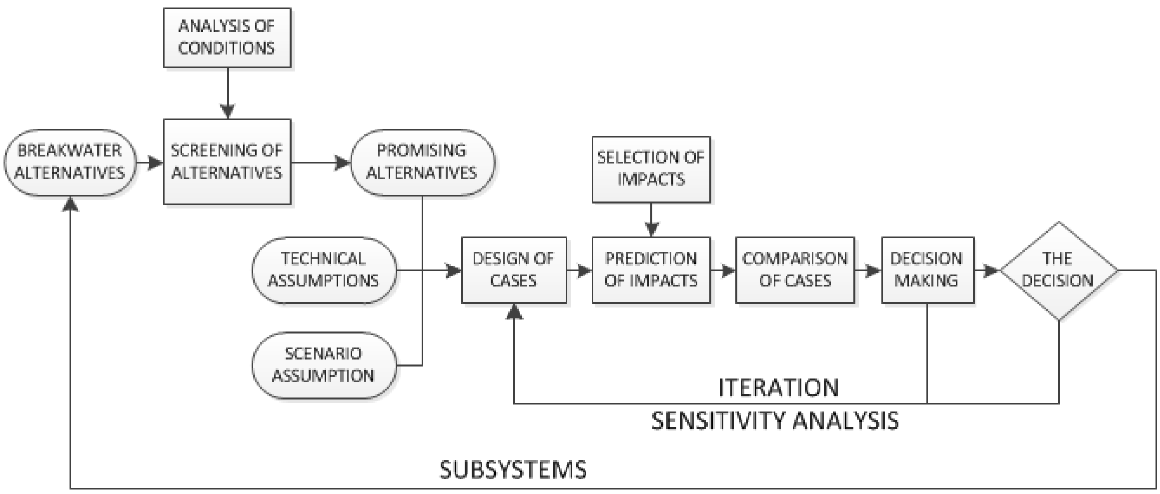


Figure 3.4: General Design Process

The pattern of the flow chart is close to the design sequence provided by Roozenburg and Eekels (1995). This book distinguishes the *analysis*, which includes the definition of the problem, the goal, but also the requirements and the criteria posed by the client. This is followed by the *synthesis* of the alternatives. Numerous alternatives are developed and considered in the *simulation* which consists of design computations. An *evaluation* of the results leads to a *decision* about the promising alternatives for further design.

Takahashi (2002) wrote specifically about the selection criteria for breakwaters and developed the following list. In the screening of alternatives and the decision making these topics should be considered.

1. Importance of breakwaters;
2. Layout of breakwater;
3. Environmental conditions;
4. Utilization conditions;
5. Executive conditions;
6. Costs of construction;
7. Construction terms;
8. Available construction materials;
9. Maintenance;
10. Demolishing.

This thesis follows the process from 'breakwater alternatives' until 'the decision' from the figure. Iteration is not applied and subsystems are not recognized, because these will not become relevant until after the selection phase.

3.4. PARAMETRIC DESIGN

For designs consisting of elements, parametric design can be an useful tool to obtain a quick analysis. Various materials and other variables will be discussed in this thesis. Therefore, in this section, the relevance of parametric design is discussed.

More increasingly designers use parametric design to come up with quick solutions. Instead of using elements with individually varying variables, n parametric design elements with standardized variables are used to describe an object (Tolman et al., 2001). Thus, for particular objects, sizes, colors, strengths, stiffness's, behaviour, etc. are fixed. For example, sheet pile profiles are provided with fixed dimensions and structural properties, such as, AZ12-770 and AZ13-770. These AZ-profiles have approximately ten variables, which are known and do not have to be mentioned specifically. When parametric design was not implemented ten variables would still fluctuate. Subsequently, the design would require more time to determine the properties of the unique profiles. So, complex objects are becoming simplified, standard objects, which shortens the time span of the design phase. What is more, it makes designs more reliable and less costly, since the costs are reduced due to the suppliers mass-production of these standard objects which are used in many projects.

This thesis is inspired by the idea of obtaining and using both dependent as independent standard variables. This is found in the classification of relevant variables. In other words, the boundary conditions, design considerations and the dimensions of a breakwater have multiple fixed values, which are enclosed in classes with a certain range. Note that, that the materials are completely standardized (for instance: stone classes, placed blocks, caissons, etc.).

4

Decision Support Method for Breakwater Selection

This chapter describes the decision support method (DSM) to consider breakwater alternatives in Dutch inland waterways. The method supports a designer in making a choice for one or more breakwater alternative(s). In the following sections, the implementation of the method is discussed. Multiple sections are clarified by intermezzo's which provide examples of the application of the method.

4.1. Overview

An overview of the method contents is shown in Figure 4.1. First of all, design steps 2 to 7 have to be completed to perform the pre-selection of the breakwater alternatives. Secondly, the pre-selected breakwaters are subjected to design steps 9 to 15. This will result in a reselection which is mainly based on cost estimates. Finally, a decision can be based on these two selections.

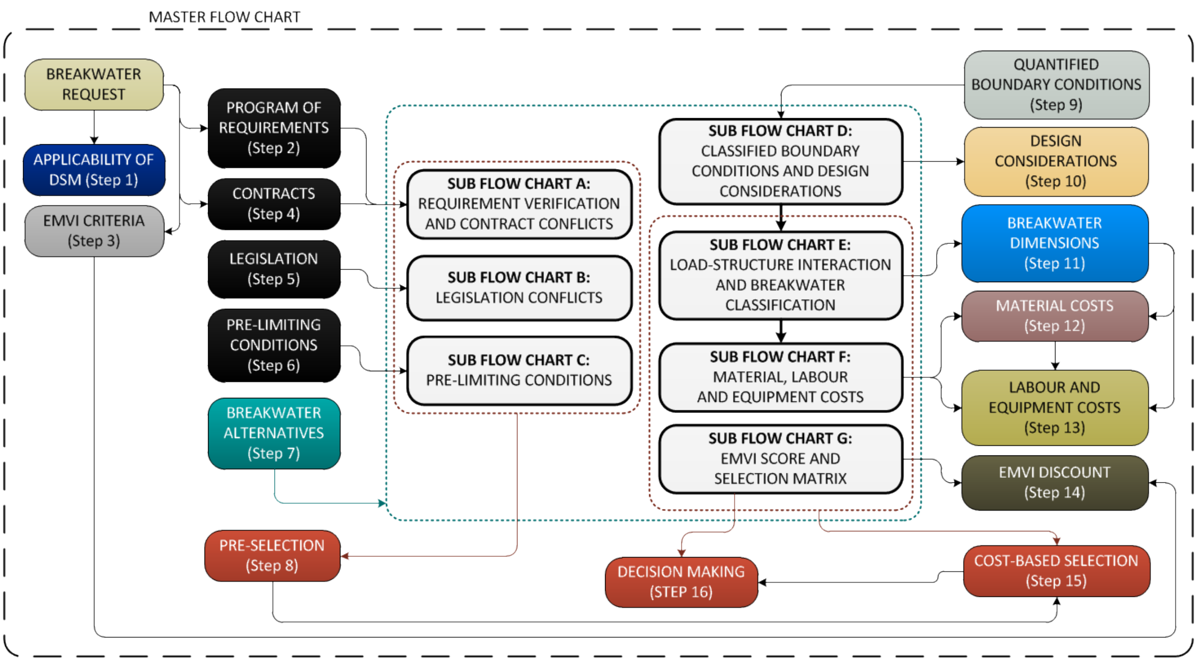


Figure 4.1: Overview of Decision Support Method

The applicability of the DSM is considered in *Step 1*. It determines if the method can be implemented without adjustments or supplements. It is also meant to prevent that a designer is planning to apply the method while he is guided by a different objective (for example, preliminary design of breakwater) than the DSM is designed for, the selection of a breakwater alternative.

A designer starts with gathering all the requirements from the client which are considered in *Step 2*. The client includes these in the registration guide, the contract document and/or via the meetings. Before the contract is signed, the tender document from the designer is assessed. The choice to consider the tender approaches (e.g. EMVI, BVP) is underpinned in *Step 3*. Apart from this, the design and construct projects are all subjected to a contract. The two contracts to take into account are discussed in *Step 4*. Due to the public locations, projects are subjected to multiple laws, permits and regulations which are controlling the Dutch inland waters. The legislation is discussed in *Step 5*. Additional pre-limiting conditions are enclosed in *Step 6*. These consist of limitations originating from the locations characteristics, purpose of the breakwater and expected boundary conditions. The optional breakwater alternatives are discussed in *Step 7*. An intermediate step is found in *Step 8*, which consist of the pre-selection of feasible breakwaters. This step contains the qualitative consideration, which contains the impact of the requirements, type of contract, legislation and pre-limiting conditions. The advantage is that a smaller amount of breakwater alternatives will continue in the structural approach, dimensioning process and cost estimation.

The engineering phase provides the opportunity to verify the structural performance and direct costs. Therefore, in *Step 9* and *Step 10* the relation of the governing boundary conditions and relevant design considerations are discussed. These can be referred to as an assessment of hydraulic influences and impacts. In this context, the strength and stability of the breakwater elements are essential, which is referred to as ‘hydraulic fulfilment’. The result is that the breakwater alternatives can be dimensioned and provided with a material cost estimate, which is explained in *Step 11* and *Step 12*. The construction equipment and construction costs are considered in *Step 13*, which takes into account the amounts of materials and the size of the breakwater. Also an EMVI discount estimate is included in *Step 14*. Subsequently, a cost-based selection can be accomplished in *Step 15* which consists of the material costs, construction costs and EMVI discounts. In *Step 16* a final decision on one or more breakwaters shall be supported by the results of the pre-selection and the cost-based selection.

One can observe multiple design steps and flow charts which are connected. These flow charts deliver information to complete the task in each design step. To perform a quick implementation of the DSM, a master flow chart is developed providing an overview of the design steps. It can be used as a guide to implement the method in an easy manner. The flow chart is enclosed in Appendix L **Master Flow Chart: Overview of the Decision Support Method**. In addition, the sub flow charts contain results of the various design steps and provide input for the estimation of the boundary conditions, structure dimensions and cost estimates.

The remainder of this chapter will describe the steps from the DSM.

4.2. STEP 1: APPLICABILITY OF DSM

Prior to the implementation of the method, the preferred structure of the client has to be identified. This is to prevent misuse of the method and to foresee complications in an early stage. For example, when a retaining wall is requested, an breakwater is not considered, which declares the method not applicable. A designer should consider a validity range for which the DSM is suitable:

1. Phase: Selection
2. Subject: Consideration of alternatives
3. Structure: Breakwaters
4. Location: Inland waterways in the Netherlands
5. Request: Design (and construct)
6. Tender: EMVI-based (optional)

The decision support method is a guide to choose between multiple breakwater alternatives. For the reason that the phase of selection is under consideration, rough calculations by rules of thumb are realized instead of detailed engineering and optimization. Thus, for PD and FD computations, this method is not applicable. Contrarily, the calculations of the method could support a CD.

Continue with Step 2 when agreed with the method background and content limitations.

Intermezzo 'Applicability of DSM for port in North Sea'

Along the North Sea coast a new port area is planned. The port requires two breakwaters to realize the entrance for vessels. In the validity range, the criterion of inland waterways is not met. As a consequence, the DSM is declared not applicable to this project. The main cause is that the magnitude of multiple boundary conditions are probably exceeded. For instance, wind velocities, vessel dimensions, water depth, flow velocities, fetch length and wave heights can be higher than the method provides for.

4.3. STEP 2: LIST OF REQUIREMENTS

A breakwater alternative is feasible when it fulfils the formulated requirements. Therefore, it is important to obtain the list of requirements.

The requirements originate from the client, the designer and the stakeholders (Wentzel et al., 2005), which are partly included in the contract document and tender guideline. The choice for a particular breakwater alternative mainly depends on the functional, performance and technical requirements, which are either mandatory or negotiable. A thorough analysis tends to provide a clear objective of the structure. References to a collection of relevant requirements for breakwaters are shown in Table 4.1.

Table 4.1: Reference to Requirements

Description	**Reference**	**Example**
Functional Requirements	Appendix A.2	The structure reduces/prevents wave action to enhance shipping.
Performance Requirements	Appendix A.3	The structure reduces the significant wave height to 0.30 m.
Technical Requirements	Appendix A.4	The structure should have fenders.
Stakeholder Conditions	Appendix A.5	The structure incorporates mooring opportunities.

Systems engineering is a method to consider requirements in both complex and small projects. It is effective when solutions are not directly visible and where multiple disciplines are active (US DoD Systems Management College, 2001). The method enables a systematic approach to develop and verify the list of requirements. To support this, the requirements should be understandable, unambiguous, comprehensive, complete and concise. The common constraint, as a result of the many clients and stakeholders, is that the requirements could conflict. Due to the method a well-considered choice for an appropriate solution can be made. In this method, the requirements are recognised, listed and systematically verified per breakwater alternative, according to system engineering.

Now, the list of requirements can be developed. The provided requirements in the references can be applied, but are indicative. A designer can choose to neglect and/or add requirements. Additionally, the client can be consulted to find out the mandatory and negotiable requirements. Ordinarily, the functional requirements are mandatory.

Continue with Step 3 when the relevant requirements are considered.

Intermezzo 'List of requirements for small port along river'

Many small ports in The Netherlands are located along rivers. These rivers are used by recreational and transport vessels. For a harbour along the Merwede in Brabant, a detached breakwater is preferred in front of the entrance to interrupt the progression of vessel-generated waves. This is equivalent to requirement F1 (Appendix A.2). What is more, the breakwater should not block the entrance, which is enclosed in requirement F10, and on the breakwaters inner-side mooring facilities are provided. The mooring opportunities are conform requirement F11.

4.4. STEP 3: EMVI CRITERIA

The formulated EMVI criteria can provide a fictitious discount which can reduce the total costs of a breakwater alternative significantly. Consequently, the criteria should be listed and scored. The score will predict the discount.

In the phase of tendering, various tender approaches can be inflicted by the client. These approaches can shift the focus from lowest price to an expensive design with maximum fictitious discount (e.g. BVP and EMVI). The most common tender approaches are enclosed in Table 4.2. In Appendix B these are discussed extensively.

Table 4.2: Tender Approaches and Reference

Abbreviation	**Definition**	**Reference**	**Component of Assessment**
BVP	Best Value Procurement	Appendix B.1	Excluded
EMVI	Economical Most Beneficial Registration	Appendix B.2	Included
CO2 Performance Ladder	n/a	Appendix B.3	Excluded
LP	Lowest Price	n/a	Included

The EMVI and LP tender approach are considered in the DSM. As a matter of fact, EMVI also focusses on the LP, but this includes a fictitious discount (see Section 4.15). BVP is not part of the assessment, because it considers the functioning of the organisation and the criteria are ill-defined. Therefore, a discount estimation is difficult to predict. Furthermore, the CO2 Performance tender approach becomes part of the EMVI criteria list, which goes by the name 'CO2 Ambition Level'. In conclusion, LP and EMVI are considered in the assessment of the breakwater alternatives. The typical EMVI criteria in Table 4.3 can be considered.

Table 4.3: EMVI Criteria and Reference

EMVI Criteria	**Reference**
System Quality	
Durability	
Innovation	
Ecological Impact	
CO2 Ambition Level	Appendix B.2
Hindrance	
Noise	
Risks	
Life Cycle Cost	

Now, the list of EMVI criteria can be developed. The provided requirements in the references can be applied, but are indicative. A designer can choose to neglect and/or add EMVI criteria. In Step 14 of the DSM, the EMVI discount will become part of the cost-based selection.

Continue with Step 4 when the EMVI criteria are determined.

Intermezzo 'EMVI criteria for bank protection in Beulakerwijde'

The manager of the Beulakerwijde in The Netherlands requests a bank protection, which is detached from the bank. Besides the recreational purpose of the lake, the location is also a natural habitat area. Therefore, the client prescribes four environmental EMVI-criteria including a maximum discount. The criteria are defined as: the breakwater should be durable (€120,000), ecologically sound (€190,000), low in CO2 emission (€70,000) and limited in noise disturbance (€90,000). In total a perfect design would score a discount of €470,000.

4.5. STEP 4: CONTRACTS

The relevant contracts consider the design, construction and maintenance. The experience of the contractor and breakwater characteristics are considered to conclude a breakwater alternative is favoured. For example, when a contractors has no experience in construction of an innovative breakwater, this alternative has more risk than others. Therefore, the type of contract is affecting the choice for a breakwater alternative.

These contracts for civil engineering projects can generally be divided into the design, construct and combination contracts, which are regularly subjected to the UAV-2005 regulations in The Netherlands (Rijkswaterstaat, 2014c). The relevant contracts are listed in the Table 4.4. What is more, when contracts are taken into account, it can be said that the procedure of tendering (for instance, non-public tender, competitor dialog, tender below threshold) is not affecting the choice for a breakwater alternative. Thus, the tender procedure is not part of the DSM.

Table 4.4: Design and Construction Contracts and Reference

Abbreviation	**Definition**	**Reference**	**Component of Assessment**
D&B / D&C	Design & Build / Design & Construct	Appendix C.1	Included
DB&M	Design, Build & Maintain	Appendix C.2	Included
DBF&M	Design, Build, Finance & Maintain	Appendix C.3	Excluded
DBFM&O	Design, Build, Finance, Maintain & Operate	Appendix C.4	Excluded
E&C	Engineering & Construct	Appendix C.5	Excluded
RAW	Consent Registration Works	Appendix C.6	Excluded

In Appendix C the type of contracts are discussed including the reason for expulsion. The D&B and DB&M are reasonable contracts to consider. On the contrary, the DBFM&O contract is left out of the assessment since it includes the operation of a breakwater, which is a rather unclear activity. In fact, when the static structure is positioned, it is constantly operational without human interference, in principle. Therefore, the payment of the structure and the maintenance are adding more value to the breakwater selection, which are found in the DB&M and DBF&M contracts. DBF&M and DBFM&O contain a financial plan for the payback of the investment. However, it will not affect the choice of a breakwater alternatives and is out of scope for this thesis. The E&C contract is neglected for the reason that the type of structure is already chosen. This makes the function of the DSM ineffectual. Apart from this, the RAW is not part of the method, because it only considers the construction phase which follows after the design phases.

Now, the applied contract can be chosen. The designer can consider the lessons learned from experience in the design and the construction of a particular breakwater in a D&B contract. On the other hand, a DB&M contract also takes into account the required maintenance and experience in the repair works.

Continue with Step 5 when the type of contract is chosen.

Intermezzo 'Contract for breakwater with mooring facilities' (Preview)

An engineering company chose to elaborate a tender document for a mooring structure as a port extension. The recommended structure is a breakwater designed to withstand wave impact and provide a sheltered zone for moored vessels. For this project, the client provides a DB&M contract to the winning company. It can be concluded from practice that there is sufficient experience in the design of rubble mound breakwaters. Also certainty is found in the estimation of the amount of maintenance over the lifetime of the structure. In contrast, the block wall breakwater is difficult to maintain when settlements occur, which is a relevant threat. The reef ball breakwaters are rarely designed, which provides a lack of experience in designing this breakwater. What is more, the required maintenance cannot be accurately estimated. Therefore, rubble mound is a most preferred as compared with the block wall and the reef ball breakwater.

4.6. STEP 5: LEGISLATION

Particular legislation could obstruct the construction of a breakwater alternative. For example, excavation activities in contaminated subsoils could imply that permits are not granted and regulations on water quality are not met. Therefore, the relevant legislation should be examined.

The project location and construction activities in surface waters are subjected to laws, regulations and permits to prevent a negative impact on surrounding, environment and society. Various waters in the Netherlands are subjected to laws and regulations, which frequently prescribe permits. Nowadays, most of the permits are covered by the Omgevingsvergunning (Permit of Surroundings). A reference to common Dutch legislation for breakwater projects is included in Table 4.5.

Table 4.5: Reference to Laws and Permits

Description	Reference	Example
Laws	Appendix D.1	Scheepvaartverkeerswet (Law of Maritime Traffic)
Permits	Appendix D.2	Bouwvergunning (Permit of Construction)
Regulations	Appendix D.3	Besluit Bodemkwaliteit (Decree Soil Quality)

Now, the relevant legislation can be selected. The provided legislation in the references can be applied, but are indicative. Thus, the designer can also choose to supplement these lists. New legislation can be sorted based on engineering judgement. It is recommended to consult juridical experts. Additionally, based on the expectation of conflicts between breakwater and permit, one can choose to leave out certain breakwater alternatives, because conflicts could lead to additional works and costs or construction plans delays.

Continue with Step 6 when the relevant laws, permits and regulations are determined.

Intermezzo 'Legislation of Natura2000 area' (Preview)

In the middle of a Natura2000 area, a lake is situated. The banks are rich of vegetation (mainly cane-vegetation), but damaged due to large waves. Therefore, a breakwater is requested parallel to the banks. For the construction phase, specific legislation is applicable, namely Wet van Flora and Fauna (Law of Flora and Fauna) and the Natuurbeschermingswet (Law of Nature Protection). Prescribed by the Board of Directors also the decision Vogel- en Habitatrichtlijn (Birds and Habitat Regulation) should be taken into account. Noise generated by driving piles, which is required for the sheet pile and wooden piled breakwater, could make it difficult to comply with the legislation.

4.7. STEP 6: PRE-LIMITING CONDITIONS

The pre-limiting conditions consist of the location, purpose and boundary conditions. Based on engineering judgement breakwater alternatives can be neglected or incorporated. Hence, the essential differences in the pre-limiting conditions are defined. For example, the floating type of breakwaters are not applicable when ice loads have to be considered.

Breakwaters can be found in lakes and rivers which is the distinction of the location conditions. The major difference in the location are the governing loads, which are flow velocities in rivers and ice and waves in lakes. In rivers, groynes should be considered. It is advised to use other guidelines to perform the consideration of groyne alternatives, for the reason that the method only recommends optional breakwater types. Select one of the following options:

- Lake breakwaters;
- River groynes.

Breakwaters can be constructed to protect large ports or less economical environments. Larger permanent structures are preferred for ports, while less profitable areas can be protected by smaller structures. Therefore, a distinction is created between profitable and non-profit locations. Choose between the following purposes:

- Small ports and mooring facilities;
- Swimming areas, banks, habitats.

Most designers have an idea of the boundary conditions at a certain location which can be described qualitatively. Unfavourable conditions, as formulated below, may affect the applicability of certain breakwaters. Select the relevant boundary conditions:

- Large waves (e.g., wave height of 1.5 m);
- Large water depths (e.g., water depth of 3 m);
- High flow velocities (e.g., flow velocity of 1 m/s);
- Ice (e.g., ice thickness of 0.1 m);
- Weak subsoil (e.g., subsoil consisting of light clay, modulus of vertical subgrade reaction of 0.045);
- Earthquakes (e.g., vertical acceleration of 10 cm/s^2).

The designer can also choose to supplement the conditions. These can be sorted based on engineering judgement.

Continue with Step 7 when the pre-limiting conditions are determined.

Intermezzo 'Pre-limiting conditions for lay-by facilities in Sneekermeer' (Preview)

A new location for lay-by facilities is planned along the Sneekermeer. This implies that a typical lake area is considered with a small port or mooring opportunities. Reef ball and piled breakwaters are less attractive as permanent structures. Moreover, it is assumed that the lake has a small water depth and small wave heights, due to a short fetch length. The flow velocities can only be generated by yachts and small sailing vessels. Therefore, these are assumed to be limited. For this typical Dutch lake, ice loads should be taken into account. The subsoil is assumed be weak (clay or organic sand) and earthquakes are neglected. The weak subsoil of the bed is not preferable for gravity based structure, such as caisson, block wall and gabion breakwaters.

4.8. STEP 7: BREAKWATER ALTERNATIVES

This design step specifically considers optional breakwater alternatives which are applicable to inland waterway conditions. These should be functional, profitable and feasible. In further design steps, the breakwater alternatives will be subjected to assessment criteria from the previous steps. Moreover, dimensions and cost estimates will be provided as well.

Inland and coastal breakwaters differ at certain aspects. For example, many details of large coastal breakwaters can be neglected in inland breakwaters. These could be detailed toe structures, crown walls, multi-layered structures. Also entire breakwater alternatives are ignored for the reason of complexity and high costs. In Figure 4.2 the breakwaters are shown which are applicable to inland waterways.

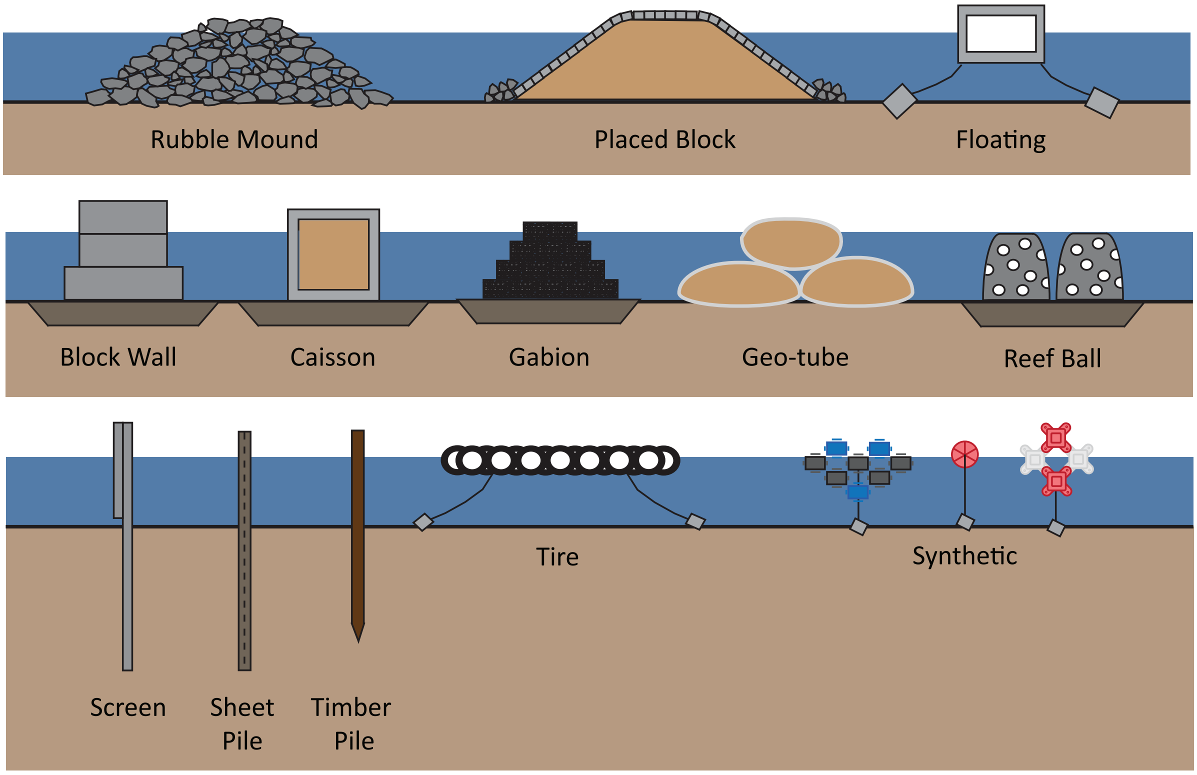


Figure 4.2: Schematization of Optional Breakwater Alternatives

The breakwater alternatives can be clustered in types, which consists of mound, monolithic, composite and special breakwaters. Short descriptions of these are enclosed in Appendix G. Mound breakwaters consist of large independent elements, which are placed in a hill-shape. In contrast, monolithic breakwaters are referred to as a single mass structures. What is more, a composite breakwater is a combination of these two which results in an economical design. The special types are floating breakwaters, for instance. Within the types considered, numerous breakwater alternatives can be listed. Table 4.6 consists of optional (readable) and infeasible (strike-through) breakwaters.

Table 4.6: Breakwater Types and Alternatives

Mound Breakwaters	**Monolithic Breakwaters**	**Composite Breakwaters**	**Special Breakwaters**
Rubble Mound	Sheet Pile	~~Horizontal Composite~~	Floating
Placed Block	Caisson	~~Vertical Composite~~	Timber Pile
~~Concrete Block~~	Block Wall		Tire
	~~L-wall~~		Reef Ball
	~~Masonry~~		Gabion
			Screen
			Geotube
			Synthetic
			~~Pneumatic~~
			~~Plate~~

The majority of the infeasible breakwaters are only profitable in deep waters (around and above 10 m) with large wave heights (above 3.5 m) where stones larger than 10 ton have to be applied. Typical breakwaters are the concrete block, the horizontally composite and the vertically composite breakwater. Also the horizontal plate breakwater is only feasible in coastal zones, where the breaking of large waves is required. What is more, the masonry breakwater is found to be a difficult structure to construct and maintain. Including the less known pneumatic breakwater and unstable concrete L-wall breakwater, these breakwater alternatives are excluded from the DSM.

A description of the optional and not feasible breakwaters is enclosed in respectively Appendix I and Appendix J. This section also provide the breakwater interaction with the hydraulic boundary conditions and EMVI-criteria. This lead to dimensions, material costs and construction costs, which is also discussed.

Now, the breakwater alternatives can be chosen. A designer can choose to neglect and add breakwaters to the list. New breakwater alternatives should fit the format and can be sort out based on engineering judgement.

Continue with Step 8 when the list of breakwater alternatives is completed.

Intermezzo 'Breakwater types and reasons for elimination' (Preview)

In certain regions rubble is rarely found and beforehand it is known that the shipping costs will be high. In that case, the rubble mound breakwater could be left out. Additionally, when experience and knowledge is lacking for further design phases in case of tire and reef ball breakwaters, for instance, these could be neglected. Also when the supply of steel for reinforcement and sheet piles is limited, the price of steel increases. A designer could neglect this breakwater alternative for that reason.

4.9. STEP 8: PRE-SELECTION

In this step the first selection of the considered breakwater alternatives takes place. This will save engineering time in the subsequent design steps, as a result of a smaller number of breakwaters considered.

The pre-selection contains the requirements, the contracts, the legislation and the pre-limiting conditions, which are considered in the previous design steps. These items are verified per breakwater alternative and are systematically verified in the references found in Table 4.7.

Table 4.7: Optional Breakwater Alternatives with Reference

Description	Reference
Rubble Mound	Appendix I.1
Placed Block	Appendix I.2
Sheet Pile	Appendix I.3
Caisson	Appendix I.4
Block Wall	Appendix I.5
Floating	Appendix I.6
Piled	Appendix I.7
Tire	Appendix I.8
Reef Ball	Appendix I.9
Gabion	Appendix I.10
Screen	Appendix I.11
Geotubes	Appendix I.12
Synthetic	Appendix I.13

Subsequently, flow charts are developed which show the interaction between a breakwaters and aspects. This is done in order to comment on the applicability of a breakwater. The references to the relevant flow charts are provided in Table 4.8. Experienced users of the DSM can use these flow charts to do a quick implementation and assessment.

Table 4.8: Reference to Sub Flow Charts (A to C)

Reference	Description
Appendix M	**Sub Flow Chart A: Requirement Verification and Contract Conflicts**
Appendix N	**Sub Flow Chart B: Legislation Conflicts**
Appendix O	**Sub Flow Chart C: Pre-limiting Conditions**

By using green, red and yellow shapes in the several flow charts, a breakwater alternative can respectively cooperate, conflict or be situation dependent. The verification is performed by engineering judgement. The findings can be embedded in the pre-selection scheme (Table 4.9) and will lead to the first breakwater alternatives to be eliminated from the process.

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBES	SYNTHETIC
I1	REQUIREMENTS													
I2	CONTRACT													
I3	LEGISLATION													
I4	PRE-LIMITING CONDITIONS													
I5	FEASIBILITY													

Table 4.9: Pre-selection Scheme

The advantage is that the neglected breakwaters are not dimensioned and estimated in costs. However, a breakwater alternative can be eliminated, but still be favoured due to the low total costs in Step 15. So, a designer should consider adjustments to these breakwaters in order to fulfil the pre-selection criteria.

Continue with Step 9 when the first breakwater alternatives are apostatized.

4.10. STEP 9: QUANTIFIED BOUNDARY CONDITIONS

The boundary conditions of a project determine the impacts on the breakwaters, which will be investigated in the following design steps. Therefore, the relevant boundary conditions should be mapped out. These consist of wind, water depth, flow velocity, waves, ice, subsoil and earthquakes. The visualizations of these is presented in Figure 4.3 and point out that their presence can not be ignored.

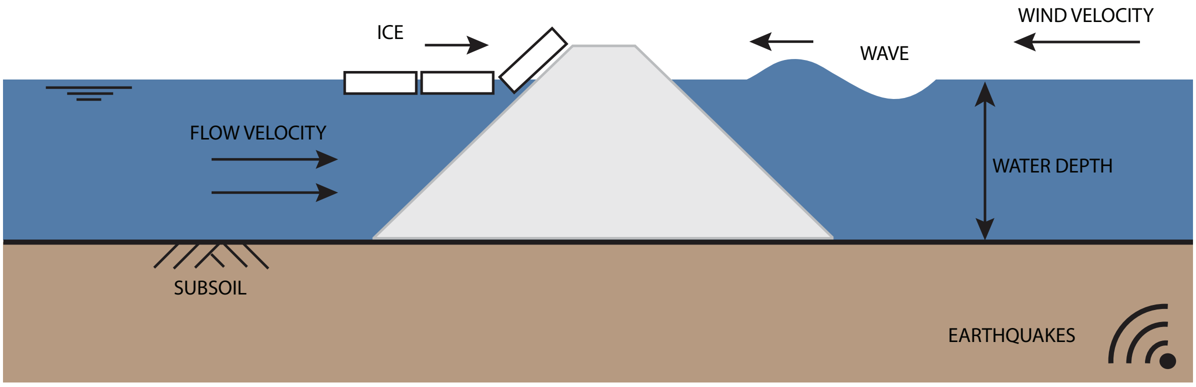


Figure 4.3: Schematization of Boundary Conditions

The boundary conditions vary in composition and magnitude, but are limited due to the shallow water of inland waterways. A designer should take into account the following maximum values.

(a) Wind velocities up to 30 m/s;
(b) CEMT vessels with a draught up to 4.5 m (in deep water);
(c) Water depth up to 4 m;
(d) Flow velocities up to 2 m/s (at the point of origin);
(e) Fetch lengths up to 30 km;
(f) Wave heights up to 2 m (at the location of the structure);
(g) Ice thickness up to 0.4 m;
(h) All subsoils;
(i) Earthquakes up to 100 cm/s^2.

Adjustments due to higher magnitudes can be incorporated, but could result in additional calculations. Therefore, it is recommended to fit the deviating parameters to the maximum values provided and to prevent underestimation for the reason of time. For example, an actual fetch length of 50 000 m can be underestimated by the maximum fetch length of 30 000 m. In addition, when other types of boundary conditions are preferred to be included, these can be added. In this matter, new design formulas should be provided and additional computations carried out.

By performing a location analysis, a designer can obtain the magnitudes of the boundary conditions. As mentioned before, the ranges are limited, which enables the boundary conditions to be classified in a limited number of fixed values. Consequently, the elaboration is uncomplicated and quickly performed. Computations are not required, since all the results are shown. A designer can fit the boundary conditions from surveys into the classification and then submit these to Table 4.10. For instance, a measured flow velocity of 78 m/s can be matched to the classified flow velocity of 1 m/s. Descriptions, typical magnitudes, survey techniques and classifications of the boundary conditions are enclosed in Appendix E.

Boundary conditions are both provided as computed. The computed boundary conditions are the vessel-generated flow velocities and the waves developed by vessels and wind. A designer can read the results of the computations and use the classification to fill out the table. For example, an International Vessel can have a bow thruster developing a flow velocity of 0.65 m/s at the axle.

Table 4.10: Summary of Design Boundary Conditions

Boundary Conditions	Magnitude	Units	Reference
Wind Velocity	...	m/s	Appendix E.1
Type of Vessel	...	–	Appendix E.2
Vessel Properties	...	*Length* (m)	Appendix E.2
	...	*Beamwidth* (m)	
	...	*Draught* (m)	
Water Depth	...	m	Appendix E.3
Flow Velocity from Discharge	...	m/s	Appendix E.4
Flow Velocity from Return Flow Velocities	...	m/s	Appendix E.4
Flow Velocity from Main Propeller	...	m/s	Appendix E.4
Flow Velocity from Bow Thruster	...	m/s	Appendix E.4
Type of System	...	–	Appendix E.5
Fetch Length	...	m	Appendix E.5
Wave Height by Wind	...	*Wave Height* (m)	Appendix E.5
	...	*Wave Length* (m)	
	...	*Wave Period* (s)	
Wave Height by Vessel	...	*Wave Height* (m)	Appendix E.5
	...	*Wave Length* (m)	
	...	*Wave Period* (s)	
Ice thickness	...	m	Appendix E.6
Subsoil	...	–	Appendix E.7
Modulus of Subgrade Reaction	...	N/mm^3	Appendix E.7
Earthquake	...	cm/s^2	Appendix E.8

A designer can choose to leave out boundary conditions when these are negligible or not present. Moreover, other boundary conditions can be sort out based on engineering judgement and fitted into the performed computations.

In Appendix P **Sub Flow Chart D: Classified Boundary Conditions and Design Considerations** the classification is shown including a reference to the required appendices. The purpose of this flow chart is to have an overview of all factors. Also when various boundary conditions are known beforehand, an elaboration is possible without consulting the report. For instance, high design wind velocities of 16.8 m/s have been prescribed by a client. In the flow chart one can find 15 m/s as closest classified design wind. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Continue with Step 10 when the boundary conditions are determined.

Intermezzo 'Boundary conditions from convoy vessel on the River Rhine'

The River Rhine enables larger inland vessel to transport cargo. The dominant vessel is one vessel with 6 convoys, which is the largest vessel found on inland waterways. Within the classification of design vessels, type Convoy Vessel (2) is the closest to the governing vessel. The dimensions are 200 m in length, 33 m in beamwidth and 4.5 m in draught. The properties will lead to a flow velocity of the main propeller of 1.2 m/s, a flow velocity of a bow thruster of 0.75 m/s, a return flow velocity of 1 m/s (assumed) and a vessel generated-wave height of 0.2 m (distance to structure is 10 m and water depth is 4 m).

4.11. STEP 10: DESIGN CONSIDERATIONS

The boundary conditions in Step 9 interact with the various breakwater alternatives. This interaction is referred to as design considerations and determines both the final predicted dimensions and functionality of the structure. The most important impacts and consequences are shown in Figure 4.4. One can distinguish the action and position of wind, waves, ice and flows. The wind conditions will develop a water level rise, or wind set-up, and wind-generated waves. What is more, the behaviour of waves can result in wave reflection, wave run-up, wave overtopping and wave transmission.

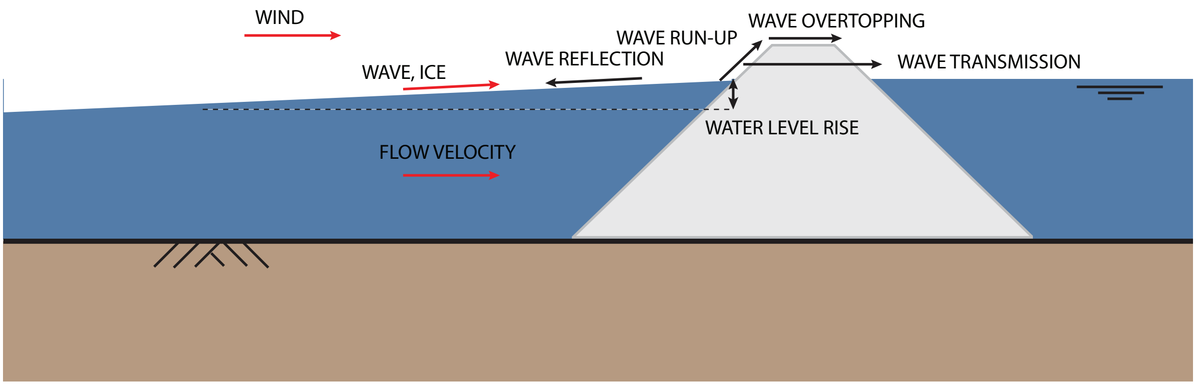


Figure 4.4: Schematization of Design Considerations

From the boundary conditions in Table 4.10, the interaction with sloped and walled structures can be determined. These are the design considerations which are computed by using various design rules. For instance, an ice thickness of 0.3 m results in a computed ice force of 248 kN/m. One does not have to calculate, but only read the correct values which saves time. The Table 4.11 should be completed with values from the classification only. The classified values are enclosed in Appendix F.

A designer can also use Appendix P **Sub Flow Chart D: Classified Boundary Conditions and Design Considerations** to obtain and oversee the classified design considerations. Some considerations depend on multiple variables. For example, wind set-up is a function of the water depth, the wind velocity and the fetch length. Since the results vary, a classification is made to obtain insight quickly. For instance, a water depth of 1 m, a wind velocity 20 m/s and a fetch length of 10 000 m will result in a wind set-up of 0.296 m (Appendix F.6). For the reason that the classification contains 0.2 m and 0.4 m, this value can either be under- or overestimated. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Table 4.11: Summary of Design Considerations

Design Consideration	**Magnitude on Slope**	**Magnitude on Wall**	**Units**	**Reference**
Flow Velocity from Discharge around River Groyne	...	...	m/s	Appendix F.1
Flow Velocity from Vessel around River Groyne	...	...	m/s	Appendix F.1
Distance Propulsion to Structure	...	...	m	Appendix F.1
Flow Velocity from Main Propeller on Structure	...	...	m/s	Appendix F.1
Flow Velocity from Bow Thruster on Structure	...	...	m	Appendix F.1
Water Level Rise from Wave	...	...	m	Appendix F.2
Wave Pressure	n/a	...	kN/m^2	Appendix F.2
Vessel Collision Force	n/a	n/a	kN	Appendix F.3
Ice Force	n/a	...	kN/m	Appendix F.4
Subgrade Reaction	...	...	N/mm^2	Appendix F.5
Wind Set-up	...	...	m	Appendix F.6

Once the design wave is determined, wave overtopping, wave transmission and wave reflection can be considered. These are regularly formulated requirements by the client and can be added to Table 4.12. For instance, a client requests a breakwater which provides access for pedestrians. Therefore, the maximum overtopping is 0.1 $l/s/m$ (Appendix F.9).

Table 4.12: Summary of Wave-Structure Interaction

Design Consideration	Magnitude on Slope	Magnitude on Wall	Units	Reference
Wave Run-up	...	n.a.	*m*	Appendix F.8
Wave Overtopping	...	...	*m*	Appendix F.9
Wave Transmission	...	...	*m*	Appendix F.11
Wave Reflection	...	...	*m*	Appendix F.12

A designer can choose to neglect design considerations when these are not relevant. Moreover, other design considerations can be sorted based on engineering judgement and fitted into the method.

Continue with Step 11 when the design considerations are determined.

Intermezzo 'Design Considerations'

One of the boundary conditions is the presence of the Convoy Vessel (2). The governing characteristics are a flow velocity of 1.2 m/s (main propeller) and a vessel generated-wave height of 0.2 m. The related design considerations are a wave run-up of 0.42 m for a rubble mound (45% slope angle), while this aspect cannot be considered for a vertical wall structure. In contrast, wave overtopping can be computed for both sloped as wall structures. A crest with a minimum free board of 0.1 m results in 0.12 and 0.17 l/s/m for respectively rubble mound and wall structure. When the requirement is a maximum allowed overtopping discharge of 1 l/s/m, the structure suffices and can even be lowered. The lowering will result in less costs.

4.12. STEP 11: BREAKWATER DIMENSIONS

The breakwater dimensions are required to obtain cost estimates in the subsequent design steps. One can observe that the heights and widths of a breakwater mainly depend on boundary conditions and design aspects. Also the structural performance should be considered, which determines whether a breakwater is stable. A reference is being made to this as 'hydraulic fulfilment'. It implies that a structures fails, suffices or requires improvements. This is will be enclosed in the cost-based selection of Step 15.

For the various breakwater alternatives, multiple elements and materials are considered. These are used as input for the computations of the element size and dimensions of the investigated breakwaters. The structural analysis is performed in a 2-dimensional environment (Figure 4.5). In other words, the alternatives are investigated in a cross-sectional view. As a consequence, the processes of refraction and diffraction are not considered.

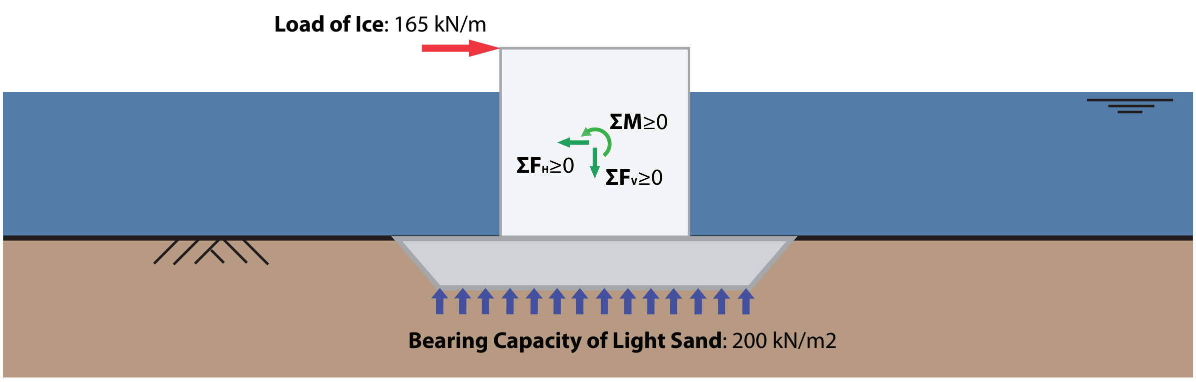


Figure 4.5: Scheme of Forces and Stability Verification (Example)

Waves, ice and flow velocities are considered as governing loads. This implies that vessel collision and wind are excluded in the load analyses. Moreover, steel structures are normally designed in front of breakwaters to carry vessel collision forces. In spite of this, the horizontal forces from wind are assumed to be negligible, and a water level difference between the sea side and sheltered zone will not occur. This latter is due to the permeability and relatively limited length of a breakwater.

Including the self-weight of the breakwater the horizontal, vertical and rotational stability can be verified. The general structural scheme is shown in Figure 4.5. Also an example of a horizontal line load by ice is drawn at the highest level of the breakwater. Since water levels are yet unknown, the most unfavourable scenario is chosen. What is more, an example of the bearing capacity pressure is included. The vertical force distribution over the footprint of the structure should be computed and compared with the allowed bearing capacity.

Various guidelines and codes prescribe safety factors to typical loads. The safety factors are a result of the uncertainty of the magnitude of the loads. Thus, a large variance results in a high safety factor. The Eurocode, NEN and ROK provide the following partial safety factors.

The partial safety factors on the mechanism are:

$$\gamma_{sliding} = 1.00$$
$$\gamma_{settlement} = 1.20$$
$$\gamma_{overturning} = 1.20$$

The partial safety factors on the loads are:

$$\gamma_{wave} = 1.35$$
$$\gamma_{ice} = 1.00$$

The partial safety factors on the materials are:

$$\gamma_{subgrade} = 1.20$$
$$\gamma_{steel-tension} = 1.00$$
$$\gamma_{wood-tension} = 1.30$$

This leads to adjusted forces for the classified wave heights and ice thickness's. To prevent too many computations and extensive tables with results, the wave heights are limited to four items instead of the nine classes considered for wave heights (Appendix E.5).

Table 4.13: Design Wave Forces with Safety Factor

Wave Height (m)	**Load** (kN/m)	**Load with Safety Factor (kN/m)**
0.5	2.5	3.4
1	7.5	10.1
1.5	15	20.3
2	25	33.8

Table 4.14: Design Ice Forces with Safety Factor

Ice Thickness (m)	**Load** (kN/m)	**Load with Safety Factor (kN/m)**
0.1	83	83
0.2	165	165
0.3	248	248
0.4	330	330

The structural performance and the required dimensions per breakwater alternative are determined in the references of Table 4.7. First of all, the crest height of the sloped and vertical structures are determined. The free board is dominated by either the wave run-up, wave transmission or wave overtopping. When wave overtopping requires the largest free board and the wave transmission is enclosed in a requirement, wave overtopping should be neglected. Thus, wave transmission becomes the governing factor. The general expressions to determine the crest height are as follows:

Sloped structure:

crest height	= water depth + wind set-up + free board
free board	= f(wave run-up, wave transmission or wave overtopping)

Vertical structure:

crest height	= water depth + wind set-up + water level rise by wave + free board
free board	= f(wave transmission or wave overtopping)

Floating structure:

dimensions	~ wave transmission

To draw conclusions about the vertical stability, the subgrade reaction force should be considered. The principle of distributed springs is applied. The allowed displacement is assumed to be 50 mm, and results in the allowed bearing capacities of the subsoils in Table F.16. For breakwaters with a pressure value on the subsoil that is higher than provided for the value provided, a designer can expect larger settlements.

Table 4.15: Design Modulus of Vertical Subgrade Reaction

Subsoil	k_s (N/mm^3)	f_{sub} **with Safety Factor** (kN/m^2)
Loam	0.001	58
Light Clay	0.003	113
Light Sand	0.005	200
Organic Sand	0.001	58
Peat	0.000	0
Clean Sand	0.002	63
Heavy Clay	0.005	225
Heavy Sand	0.060	2500

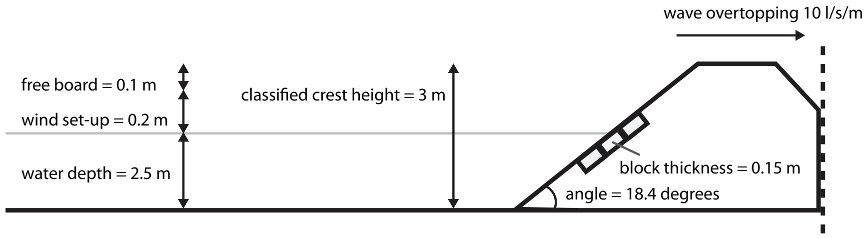
A designer can choose to follow the prescribed design approach. Mind that, over- or under-dimensioning should be considered due to the limited amount of classes.

In Appendix Q **Sub Flow Chart E: Load-Structure Interaction and Breakwater Classification** an overview of the breakwater classes, the characteristic dimensions and the sensitivity to various impacts are provided. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Continue with Step 12 when the dimensions per investigated breakwater alternative are determined.

Intermezzo 'Breakwater Dimensions'

For the protection of multiple private mooring facilities along the banks of the Westeinderplassen, Aalsmeer in the Netherlands, one of the optional breakwaters is the placed block. It is assumed that the water depth is 2.5 m, the design wind velocity is 20 m/s, the longest fetch length is 5000 m which result in a significant wave height of 0.5 m. The wave height results in a placed block thickness of 0.15 m. The angle of the slope is assumed to be 18.4 deg.



The crest height is a function of the water depth, wind set-up and wave run-up. The wind set-up originates from the tables and is around 0.06 m, which becomes 0.2 m in the classification. The wave run-up is 0.70 m, but is dominated by the requirement of an allowed wave overtopping of 0.1 l/s/m. The latter results in a free board of 0.1 m. Subsequently, the crest height of the placed block breakwater is (2.5+0.2+0.1=) 2.8 m which becomes 3 m in the classification.

4.13. STEP 12: MATERIAL COST ESTIMATE

In this section the material costs are investigated. These are part of the cost estimate in Step 15 and 'direct cost' in the SSK methodology (Figure 4.6).

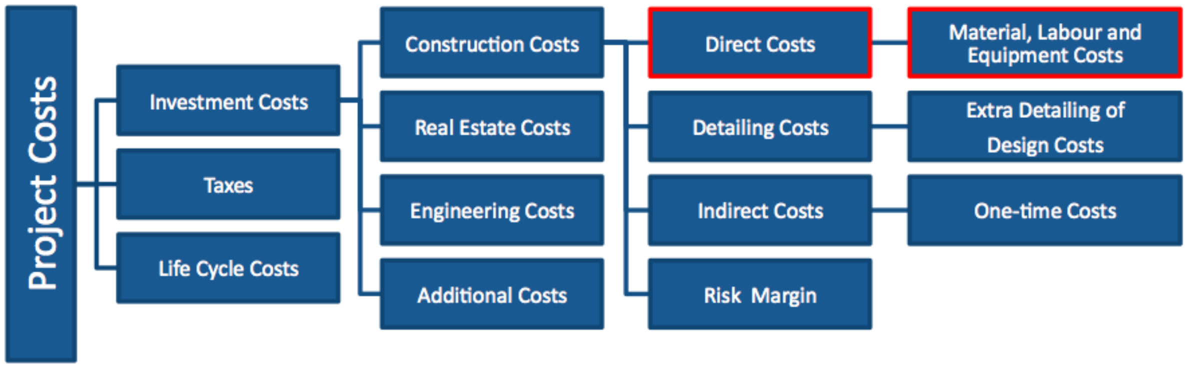


Figure 4.6: Standard Method of Cost Management (SSK)

The direct costs are the only costs considered. These predictable costs significantly vary per breakwater alternative and are therefore included. Other relevant cost items can be life cycle costs, risk margin, maintenance costs and demolition costs. The life cycle costs are not included in the price of the contractor to the client, but enclosed in a requirement or an EMVI criteria. When these costs are requested, it implies that the total project costs and maintenance costs should be estimated. These are rather subjective cost items and maintenance can be included in the requirements for a qualitative assessment. What is more, the risk margin can differ per breakwater alternative. Therefore, a designer can include a certain percentage of the investment costs or a fixed amount to cover the risks, for example. Regularly the risk margin is not a fixed percentage, but depends on the experience of the contractor of a breakwater alternative. In addition, a distinction can made between direct costs and commercial rates. The latter is characterized by a profit margin, which is out of the scope but considered in the type of contract, namely DB&M.

The breakwater alternatives have various materials like quarry stone, steel, concrete, wood and synthetics. The cost of material should be related to the unity costs including required quantities. For instance, steel has a high price compared to quarry stone per unit of weight, but a quarry stone breakwater requires more material that a sheet pile breakwater. Therefore, an analysis of the material costs per breakwater alternative pays off.

When the final cross-section is known, the unit price of the materials can be applied to a stretched meter of breakwater. An example is provided in Figure 4.7 where the total material costs consist of materials A and B.

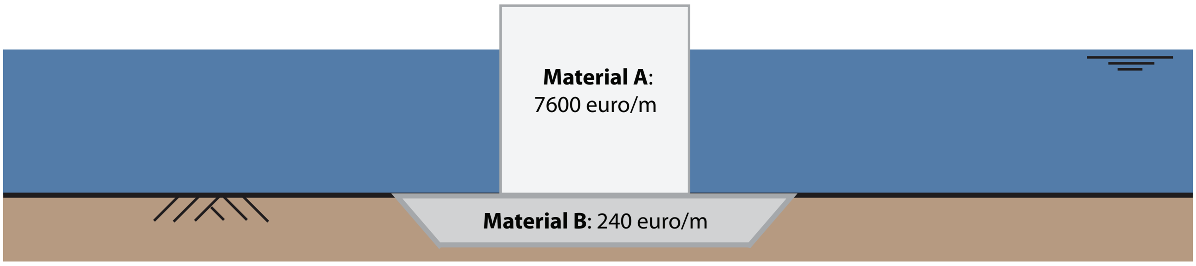


Figure 4.7: Schematization of Material Costs (Example)

The required information to perform the cost estimation is shown in the references shown in Table 4.7. The unity costs of the materials are first estimates. A designer should be aware that these values are indicative. Over time these values could change due to inflation and market forces. It is advised to verify the values in order to obtain realistic estimates.

In Appendix R **Sub Flow Chart F: Material, Labour and Equipment Costs** the cost of material per breakwater alternative is shown in an overview. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Continue with Step 13 when the material costs per investigated breakwater alternative are determined.

Intermezzo 'Material Cost Estimate'

The dimensions of the placed block breakwater are determined in the intermezzo of Step 8. From the tables provided, it can be said that the sand is 773 €/m, geotextile is 12 €/m and filter layer is 70 €/m. Because the slope is 18.4 deg and the crest is 3.5 m, the slope length becomes 23 m. The cost for the armour layer becomes 695 €/m. The cost of material is (773+12+70+695=) 1550 €/m.

4.14. STEP 13: LABOUR AND EQUIPMENT COST ESTIMATE

In this section the labour and equipment costs are studied. These are similar to the material costs part of the direct costs (Figure 4.6). These costs will be part of the cost-based selection in Step 15.

Various types of equipment are applicable per breakwater alternative. The first division is made by land-based and water-borne equipment. Examples of land-based equipment are: hydraulic excavators, hydraulic cranes, dump trucks and bulldozers. These tools can be applied to floating water-borne equipment. Dump vessels and barges are typical examples of water-borne equipment. Apart from this, breakwaters can have different orientations, namely; detached, non-detached, T-or L-head, immersed and submerged. A description is enclosed in Appendix H. For the construction costs, these orientations only make a difference for large coastal structures. In other words, non-detached rubble mound breakwaters enable both water-borne and land-based operations. In case of the small-scale breakwaters, only water-borne equipment is able to complete the works, because these structures are unable to carry dump trucks and cranes. Therefore, this equipment is not considered to be applied instead of water-borne equipment is taken into account.

The DSM only considers transport by vessels, since dump trucks cannot be used on the small breakwater. Moreover, it is complicated to determine the origin of the materials and transport distances. As a result, only the direct costs per breakwater alternative will be considered. These can be found in the references of Table 4.7.

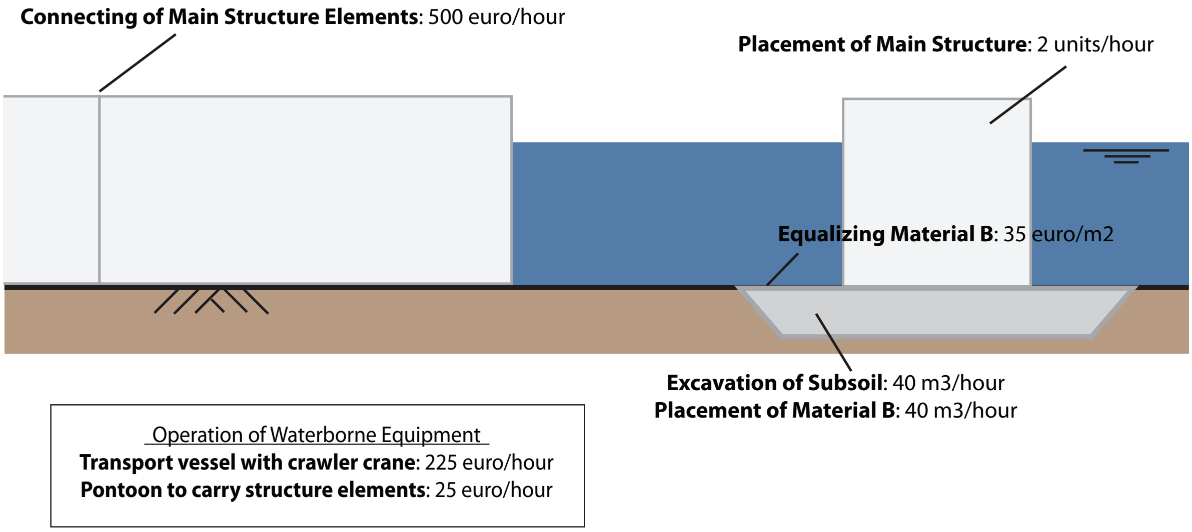


Figure 4.8: Schematization of Labour and Equipment Costs (Example)

In Appendix R **Sub Flow Chart F: Material, Labour and Equipment Costs** the construction costs per breakwater alternative are shown in an overview. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Continue with Step 14 when the construction costs per investigated breakwater alternative are determined.

Intermezzo 'Construction costs of placed block breakwater'

*The dimensions of the placed block breakwater are determined in the Intermezzo 'Breakwater Dimensions'. Besides the material costs, also the placement costs can be determined for a structure with a crest height of 3.5 m. The quantities are: sand of 129 m^3/m, geotextile of 23 m/m, filter layer of 2 $m3/m$ and place blocks of (23*0.15=) 79 $m3/m$. A vessel (110 €/$hour$) with excavator (140 €/$hour$, 35 €/$hour$ and 4 €/m^3) shall be used to place the sand, filter layer and placed blocks, which is (129+2+79=) 210 m^3/m. These cannot be expressed in quantities. Moreover, the armour layer mattress is placed with a special tool on the excavator, which can operate 6 units/hour. This is a quick construction method. For the determination of the placed block placement costs the quantity and excavator capacity is used, which provides a rather close estimate of the mattress. The excavator costs (210/4=) 52.5 €/m, which makes a time of construction of (52.5/140=) 0.375 $hours/m$ and results in (110*0.375=) 41.25 €/m vessel costs. The total placement costs are (52.5+41.25=) 93.75 €/m.*

4.15. STEP 14: EMVI DISCOUNT

The EMVI criteria, which are collected in Step 3 (Section 4.4), will result in a fictitious discount. A maximum discount per criterion is offered by the client. The more a breakwater meets a criterion, the more discount can be obtained. Moreover, the score per breakwater alternative is subjective. Accordingly, it is complicated to provide a factual quantification, which especially concerns the criteria about ecology and innovation. Plans to meet these criteria are often outsourced to specialists. The prediction of the EMVI discount can be included in Step 15.

Five EMVI scores are distinguished, which consist of a minimum and maximum discount. As chosen, score 1 is the lowest score and gets 20% of a criterion discount. In between, there are scores of 2 (40% of discount), 3 (60% of discount) and 4 (80% of discount). When the considered criterion is entirely fulfilled, the highest score of 5 (100% of discount) is achieved. The EMVI score per breakwater alternative can be found in the references of Table 4.7. The scores are determined based on engineering judgement. Since the material costs and cost of labour and equipment are expressed in costs per stretched meter, the length of the breakwater has to be estimated to obtain the EMVI discount in the correct units.

In Appendix S **Sub Flow Chart G: EMVI Score and Selection Matrix** the assumed score of the EMVI criteria per breakwater alternative is presented in an overview. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

Continue with Step 15 when the EMVI discount per breakwater alternative are determined.

Intermezzo 'EMVI discount of the placed block breakwater'

The requested breakwater from Intermezzo "EMVI Criteria" should be durable (€120,000), ecologically sound (€190,000), low in CO2 emission (€70,000) and reduced in noise disturbance (€90,000). A robust design of the placed block breakwater should results in a stable structure. Contrarily, in case of displacement of blocks, damage of the core and consequently the entire structure is likely to occur. Therefore, the durability is set to 3. Considering the ecology, it can be said that the water will be disturbed during the dumping of sand and rock. Moreover, it should be mentioned that excavation is not required for this type of breakwater. As a result, a score of 3 is attributed to ecological impact. Due to the manufacturing of concrete blocks and extensive construction works the CO2 ambition level of 2 is assumed. The noise disturbance based on the time of construction and intensity of the works is set 3.

EMVI criteria	Discount Score	Discount Factor	Total Discount
Durability	3	0.6	€120,000
Ecological	3	0.6	€190,000
CO2 emission	2	0.4	€70,000
Noise disturbance	3	0.6	€90,000

*A summation of the EMVI elaboration is shown in the table. The total discount of (0.6 * 120,000 + 0.6 * 190,000 + 0.4 * 70,000 + 0.6 * 90,000=) €268,000 is expected.*

4.16. STEP 15: COST-BASED SELECTION

In the previous steps the material costs, the construction costs and the EMVI discount are determined. These items together form the total direct costs, which can be compared to pre-selected breakwater alternatives. The cost-based selection is the last comparison, which supports the decision making.

Ordinarily, the project costs are determined by a Standard Method of Cost Management (SSK), which contain standard cost drivers. This method prescribes the total construction cost as a summation of the direct costs (labour, equipment, material), detailing (construction drawings), indirect costs (one-time costs, exploitation costs, general costs) and risk margin. The selection in the DSM is only based on the direct costs. In Table 4.16 should be completed with the findings from the design steps above.

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBES	SYNTHETIC
I6	HYDRAULIC FULFILMENT													
I7	COSTS OF MATERIAL (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I9	EMVI DISCOUNT (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I10	TOTALS (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 4.16: Cost-based Selection Scheme

In the first row of the table a not cost-based item is included. This is the hydraulic fulfilment, which provides information about the well-functioning or failure of a breakwater under the design load. When the breakwater is properly functioning a green shape is applied, the opposite holds for the red shape. Those breakwaters that are both able to withstand the loads, and low in costs, are options to be recommended.

Continue with Step 16 for the decision making.

4.17. STEP 16: DECISION MAKING

A decision for a certain breakwater alternative can be supported by the pre-selection (Step 8) and cost-based selection (Step 15). In fact, the pre-selection will be reselected in the cost-based selection. In this matter the most favourable breakwater alternative of both selections is chosen.

In the pre-selection the functional requirements are dominant, because these are normally not negotiable. Thus, the breakwater alternatives have to fulfil these requirements if these are to be considered in subsequent design steps. This is in contrast to the legislation, type of contract and pre-limiting conditions, which are determining the structure to be attractive or not. For the reason that costs are not known in this phase, more than one breakwaters may be selected.

The cost-based selection consists of a matrix with costs and hydraulic fulfilment. The latter contains a performance verification, which ensures that breakwaters are able to withstand the design loads. A failing breakwater does not have to be neglected, although improving the weak components of the structure will lead to higher cost. A designer should consider cost-efficient adjustments for breakwaters.

The pre-selected breakwater alternatives can be weighted based on the direct costs. Obviously, the breakwater alternative with the lowest costs will be recommended to further develop into a concept design or sketch design. It should be noticed that an expensive breakwater could be the lowest in cost due to the EMVI discount. Therefore, it is important to take into account the EMVI criteria.

In Appendix S **Sub Flow Chart G: EMVI Score and Selection Matrix** the table with pre-selection and cost-based selection per breakwater alternative is presented. Experienced users of the DSM can use this flow chart to do a quick implementation and assessment.

The decision can be made to recommend one breakwater alternative for tendering and further design.

4.18. REFLECTION ON DSM

This section consists of a reflection and a critical view to the developed method. The method is compared with other methods and the relevance of the design steps is described. Also the flow of information, the SSK cost items and the sequence of the design steps are considered.

4.18.1. COMPARISON WITH OTHER METHODS

The method in this report is developed with two selection phases, namely the pre-selection and the cost-based selection, which enables designers to quickly decide to neglect breakwater alternatives during the elaboration. This is a difference with the current methods (trade-off matrix and cost-based method), which have the selection at the end of the assessment. Instead of this, the pre-selection phase is built-in halfway the DSM and prior to the engineering part including cost-based selection. Thus, based on qualitative criteria the majority of the breakwaters are eliminated which saves time due to less alternatives to be considered. The engineering part mainly consists of dimensioning and cost estimates, and consumes a significant amount of time. Another advantage of the DSM compared to other methods is that the selection is not based on: requirements only (a), costs only (b), subjective weighing factors (c) and subjective negative or positive scores (d). This implies that the method is extensive and objective, since the DSM considers both requirements and costs, and subjective weighing factors on the criteria are not included. Furthermore, based on engineering judgement the types of contracts, legislations and pre-limiting conditions are considered. Compared to weight factor and score, this is a more factual assessment. For example, the experience of the contractor in design, construction and maintenance can be well estimated, when the contracts are considered. When legislation is taken into account, a prediction can be made (with experts) about any obstruction by particular permits or regulations. Moreover, the pre-limiting conditions show clear criteria which can be judged by hydraulic engineers. Therefore, making use of this method can significantly limit subjectivity. A disadvantage of this method is that innovative solutions are not included, although it could be the best option. Therefore, it is recommended to fit these into 'Step 7 Breakwater Alternatives' for an objective selection.

4.18.2. RELEVANT OF DESIGN STEPS

The relevance of the design steps is emphasized by their influence on the various breakwater alternatives. It can be concluded that the design steps are not equally important, but in some cases less interesting steps could be the point of attention. For example, when a project area with contaminated subsoil is considered, the permits have a major influence, while this is not the case for unpolluted subsoils. The design steps and the reasons to take them into account for each breakwater alternative, are listed below.

Step 1: Applicability of DSM is to prevent misuse of method.
Step 2: List of Requirements is to ensure fulfilment of the requirements.
Step 3: EMVI Criteria is to acknowledge the criteria for fictitious discounts
Step 4: Contracts is to include the design, construction and maintenance experience of the contractor.
Step 5: Legislation is to foresee any obstructions and delays in construction.
Step 6: Pre-limiting Conditions is to consider location, purpose and unfavourable boundary conditions.
Step 7: Breakwater Alternatives is to choose the optional structures.
Step 8: Pre-selection is the first selection based on the previous steps.
Step 9: Classified Boundary Conditions is to incorporate the quantified boundary conditions.
Step 10: Classified Design Considerations is to study and incorporate the load-structure interaction.
Step 11: Breakwater Dimensions is to investigate the stability and to support a cost estimate.
Step 12: Material Cost Estimate is to support the estimate of the direct costs.
Step 13: Labour and Equipment Cost Estimate is to also included in the estimate of the direct costs.
Step 14: EMVI Discount is to subtract from the direct costs.
Step 15: Cost-based Selection is to choose optional breakwaters.

Step 16: Decision Making is to decide which is the most effective breakwater(s).

4.18.3. SEQUENCE OF DESIGN STEPS

The sequence of the design steps for the cost-based selection deviates from those in the pre-selection phase. The pre-selection consists of the requirements (Step 2), EMVI criteria (Step 3), contracts (Step 4), legislation (Step 5) and pre-limiting conditions (Step 6), which the various breakwater alternatives are subjected to (Step 7). The sequence of Step 2 through 6 can change without having any influence on the assessment. So, these steps are not reciprocally dependent. However, projects are normally started with the requirements. Therefore, these can be considered in an early stage and display a dominant role. The EMVI criteria, type of contract, legislation, pre-limiting conditions could only suggest to incorporate or neglect a breakwater alternative, which can be an addition to conclusions based on the requirements. Due to the contract and requirements it is known that a breakwater should be designed. Therefore, the breakwater alternatives can only be considered in the last design step previous to the first selection (Step 8). In this phase, one could consider to perform a selection after each single design step. Surely, less breakwater alternatives per step decreases the time required for consideration.

In the cost-based selection, the sequence of Steps 9 through 13 is set and interdependent. Thus, the design considerations (Step 10) require known boundary conditions (Step 9) and can only be considered when the breakwater alternatives are taken into account. Subsequently, these three items determine the breakwater dimensions (Step 11). Only when the dimensions are known, a cost estimate of the materials can be performed (Step 12), labour and equipment (Step 13) can be performed. The dependency of the previous steps is in contrast with the EMVI discount (Step 14), which can be incorporated in one of the last steps prior to the second selection (Step 15). When all the previous steps are performed, a well-founded decision can be supported. The sequence of the design steps can not be changed, but a selection step can be inserted after the hydraulic fulfilment which implies that a smaller number of breakwater alternatives will remain to have a cost estimate completed.

4.18.4. FLOW OF INFORMATION

The method is extensive which could make it difficult to analyse all the considered information. Therefore, it is developed in such a way that information obtained via the design steps is to be overseen in the selection tables. These tables sum the considerations. This is a strength of the method and enables a designer to have a transparent assessment after investigating a significant amount of data. A scheme of the process and the amount of information is visualized in Figure 4.9.

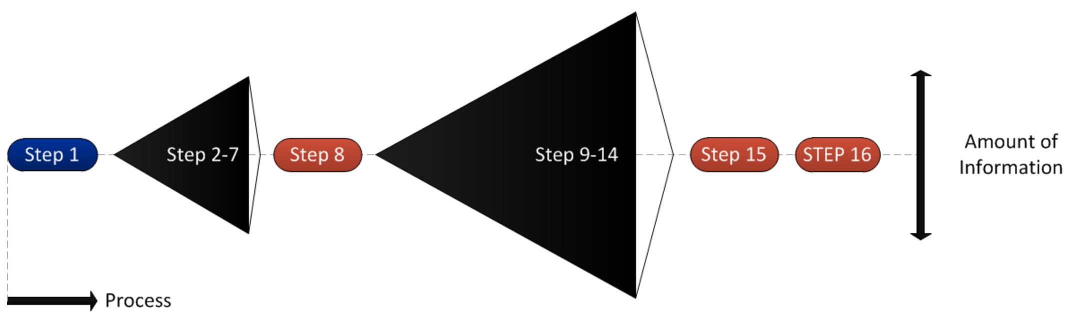


Figure 4.9: FLow of Information Scheme

4.18.5. CLASSIFICATION

In the method the boundary conditions, design considerations and dimensions are classified which means that fixed magnitudes of these are provided. The lists with classified variables within the boundary conditions are chosen based on the magnitudes that occur and a homogeneous distribution of values. Likewise, the classification of design considerations are determined including clustering of the computed results. For example, wind-generated wave heights of 1.35, 1.46, 1.52 and 1.67 m are clustered in a design wave height of 1.5 m. An advantage of the various classifications is that computations between boundary conditions and design considerations can be shown in tables. This enables designers to read the results and prevents large calculation sheet. These sheets are rather time consuming. The step between the design considerations and the breakwater dimensions is also simplified by classes. A limited amount of dimension classes does enable over-estimation and underestimation. When the classes are limited for all breakwater alternatives, a correct comparison can be made and quick assessment is enabled.

An important point of consideration is the amount of classified variables per aspect. Less classes would mean less results to read and a quicker method. Hence, the accuracy of the results would become an issue. On the other hand, more classes would increase the accuracy significantly, but the amount of results to be included in the document would increase. Considering both rationales, it is preferable to have more rather than less classes. To be able to work with the method, one could program a decision support system (DSS). In this manner, the amount of information is not restricted by a normal size document.

4.18.6. SSK COST ITEMS

The construction costs consist of the direct costs, detailing costs, indirect costs and risk margin (Figure 4.6). Only the direct costs are taken into account in the DSM, which is determined by objective unity cost of the material, equipment and labour. These fluctuate significantly among the breakwater alternatives and can be determined accurately. This is in contrast to the detailing costs (extra detailing of design) and the indirect costs which are respectively difficult to predict and approximately similar for all breakwater alternatives. Moreover, the risk margin can be increased for non-common breakwater alternatives and reduced for well-known breakwater alternatives. Therefore, such a criterion can be included in the method, but the unity costs or the percentage depend on the company. The direct costs consist of the material, labour and equipment costs. Currently, only the placement activities are incorporated, excluding the transport and mobilisation. These could also vary per breakwater alternative and location. For example, transport vessels and trucks can be deployed at different cost rates. Furthermore, when inaccessible project locations are considered, all transport should happen by vessels which provide either high or low transport costs. Besides the construction costs, the investment costs consist of real estate cost, engineering cost and additional cost. The real estate costs and additional costs are fixed cost, while the engineering costs are dependent of each breakwater alternative. The cost items are not quantified, but can be enclosed in the type of contract. In the D&B and DB&M contracts, the design experience of the contractor plays a role. A non-familiar breakwater alternative would require extra research and time of engineering. The relatively low engineering cost is unquantified in the method, but found in the type of contracts criterion in Step 4. The project costs consist of the investment costs, taxes and life cycle costs. In the life cycle costs, the maintenance aspect is enclosed. What is more, the maintenance costs is not included yet. Therefore, it is recommended to find a manner to take these costs into account.

5

Case Studies

Two cases are defined to show the implementation of the DSM, which is discussed in Chapter 4. It concerns two distinct client requests and breakwater locations. The elaboration shows the effect of specific requirements, contracts, legislations and pre-limiting conditions. These items constitute the first breakwater selection based on feasibility. Subsequently, the boundary conditions and design considerations quantified and subjected to the various breakwater alternatives. As a consequence, an approximation can be provided of the breakwater dimensions, material costs and, labour and equipment costs. The final step is to take into account an EMVI discount and to recommend one or more breakwaters for a tender and/or preliminary design.

5.1. Case Study 1: IJburg Breakwater

The first case study involves the breakwater project in front of IJburg. An extensive description of the problem statement, the location analysis and the consideration of breakwater alternatives can be found in Section 2.1 and 2.2.

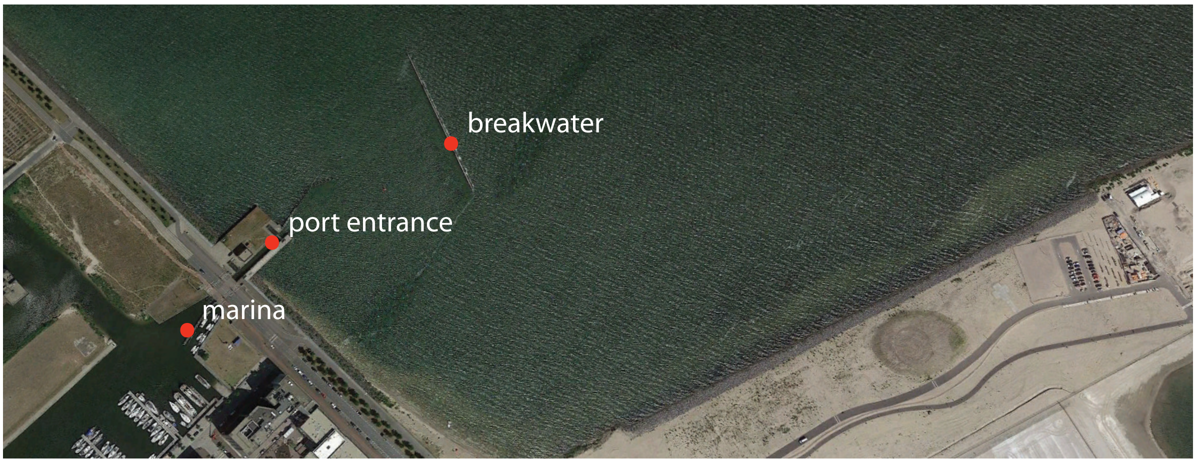


Figure 5.1: Topview of Location IJburg Case Study

In the following sections, the project is briefly described. The design reports provide the input for implementation of the DSM. Hence, recommendations are formulated for the choice of an optional breakwater alternative.

5.1.1. PROJECT DESCRIPTION

The IJburg breakwater project was requested to protect a small port area during storm conditions. An overview of the lake and IJburg is shown in Figure 5.1. A detached breakwater was planned to reduce the wave height and to enable recreational boats safely entering the ports entrance. The initial stage was to make a choice of a breakwater alternative.

5.1.2. DECISION SUPPORT METHOD IMPLEMENTATION

STEP 1: SYSTEM APPLICATION

The DSM applicability is discussed in Section 4.2. The validity range consist of requirements which should be satisfied to be able to apply the system. The following can be concluded:

1.	Phase: Selection	Satisfied
2.	Subject: Consideration of alternatives	Satisfied
3.	Structure: Breakwaters	Satisfied
4.	Location: Inland waterways in the Netherlands	Satisfied
5.	Request: Design (and construct)	Satisfied
6.	Tender: EMVI-based (optional)	Dissatisfied

The selection phase of the project is discussing the choice of a breakwaters alternative (1). These breakwater shall mainly be subjected to wave and ice loads (2). The IJmeer lake is a Dutch inland water (3). This will provide clear limits to the boundary conditions. Also the legislation is known. The requirement of Dutch inland waterway conditions implies that the considered boundary conditions can be incorporated without additional computations. What is more, the requirement about a design and construct contract is met, namely DB& M is considered. However, the EMVI tender phase (5) is not considered, because the prescribed tender procedure contains cost reduction factors originating from a BVP tender approach (Timmermans and Bemmelen, 2013).

STEP 2: LIST OF REQUIREMENTS

From the registration guideline and meetings with the client the requirements are defined. The design report (Staphorsius, 2013) provides the relevant functional requirements, operational requirements and stakeholder conditions.

The functional requirements are enlisted in Table 5.1. It is assumed that negotiation is plausible when breakwater alternatives are low in costs. In spite of this, when requirements are mandatory, breakwater alternatives are instantly repelled. The breakwater structure:

Table 5.1: Functional Requirements of IJburg Case Study

Code	Description
F1	reduces wave action to enhance shipping.
F6	shall be observable for navigation at all times.
F9	is able to be extended or to be shortened without removal of the expired structure.
F10	is not blocking the ship channel.
F11	shall enable mooring opportunities.
F12	shall not damage vessels.
F13	shall enhance the walk-ability.
F14	shall not provide facilities to stay overnight.
F15	shall not affect or interrupt the aquatic ecology.
F17	should have a sufficiently long lifetime.
F18	should not diminish the water quality.
F20	its construction should not hinder shipping, nearby traffic and local residents.
F21	should have the lowest investment costs.
F22	should have low life cycle costs.
F23	should support the requested aesthetics.
F24	should take limited space.
F25	should fit the local spatial shapes.

The operational requirements are detailed requirements, which are often mandatory. Two requirement are found (Table 5.2) which originate from the size of the vessels and an economical point of view of the client. The breakwater structure:

Table 5.2: Operational Requirements of IJburg Case Study

Code	Related Requirement	Description
O2	F1/F2/F3	reduces wind-generated waves to waves occurring at 4 Bft.
O4	F16	has a lifetime 30 years.

Stakeholder conditions are always negotiable. The client prefers a stationary position of the structure (Table 5.3). The breakwater structure:

Table 5.3: Stakeholder Conditions of IJburg Case Study

Code	Description
SC4	is fixed to one location.

STEP 3: EMVI TENDER APPROACH

In the BVP registration, which originates from the registration guideline, prescribed as costs reduction factors: performance dossier, risk dossier, opportunity dossier and interview. The EMVI criteria could fit in the opportunity dossier, which provide added value. Yet, the specific criteria and the amount of discount (or points) is dubious. Because of this, a fictitious discount is not included.

STEP 4: CONTRACT DEPENDENCY

The works prescribed by the client consisted of the design of a breakwater type structure, the realisation of the object and 15 years of maintenance (Timmermans and Bemmelen, 2013). This implies that a typical D&BM contract is considered.

STEP 5: RELEVANT LEGISLATION

The IJsselmeer and Markermeer are national water systems, which are essentially covered by the Waterwet. Contrarily, the Natura2000 and Habitat- and Vogelrichtlijn are only valid for the IJsselmeer, which implies that Natuurbeschermingswet is out of scope. The Natuurcompensatie is likewise neglected, since it is applicable to national parks, which is similar to a Natura2000 area. The laws to be considered in breakwater construction are shown in Table 5.4.

Table 5.4: Laws of IJburg Case Study

Code	Type of Law	English Translation
L1	Waterwet (Ww)	Law of Water
L2	Flora- en Faunawet	Law of Flora and Fauna
L3	Wet Bodembescherming (Wbb)	Law of Underground Protection
L4	Natuurbeschermingswet	Law of Nature Protection
L5	Wet Milieubeheer (Wm)	Law of Nature Preservation
L6	Scheepvaartverkeerswet (SVW)	Law of Vessel Traffic
L7	Ontgrondingenwet	Law of Excavation
L8	Natuurcompensatie	Compensation of Nature
L9	Kaderrichtlijn Water (KRW)	European Water Directive

Permits are prescribed by laws and are required for public projects. The location of these projects involve interest of civilians and state properties. Relevant permits for breakwater construction are shown in Table 5.5.

Table 5.5: Permits of IJburg Case Study

Code	Type of Permit	English Translation
P1	Omgevingsvergunning	Permit of Surroundings
P2	Milieuvergunning	Environmental Permit
P3	Watervergunning (WBR and KEUR)	Water Permit
P4	Watermelding	Water Notification
P5	Ontgronding	Excavation
P6	Besluit Uniforme Saneringen (BUS)	Decision Uniform Soil Restoration
P7	Saneringsvergunning	Permits of Soil Restoration
P8	Bergingsverzoek Slibdepot	Storage Request Silt Deposit
P9	Kabels en Leidingen Informatie Centrum (KLIC)	Centre of Information about Cables and Pipes
P10	Meldpunt Opbrekingen Openbare Ruimte (MOOR)	Contact Point of Public Area
P11	Bouwvergunning	Permit of Construction
P12	Sloopvergunning	Permit of Demolition

Regulations are extensions of laws and are specified by the board of directors of provinces and other municipalities. Breakwater designs and constructions should be in line with these decisions. The regulations in Table 5.6 are relevant.

Table 5.6: Regulations of IJburg Case Study

Code	Type of Regulation	English Translation
R1	Besluit Lozen Buiten Inrichtingen (BBI)	Decree Discharge Outside Interior
R2	Besluit Bodemkwaliteit (Bbk)	Decree Soil Quality
R3	Beheer- en Ontwikkelplan voor de Rijkswateren (BPRW)	Management and Development Plan of Waterways
R4	Activiteitenbesluit	Decree of Activities

STEP 6: PRE-LIMITING CONDITIONS

The Markermeer is a typical lake area. As a consequence, instead of a river groyne, a lake breakwater is considered. The inequality is found in the dominating load, which is flow velocity for a groyne and waves for a breakwater. The primary purpose of the breakwater is to protect a small yacht harbour instead of a natural reserve, bank or habitat area. This implies that an economical area is considered. Moreover, taking into account the various boundary conditions, it is expected that:

Large waves are present;
Large water depth is not present;
High flow velocities can be neglected;
Ice should be accounted for;
Weak subsoil is present;
Earthquakes can be neglected.

The fetch length in north-eastern direction is large, which results in significant wave heights due to high wind velocities. The water depth along the lake fluctuate from 2 to 4 m. Close to the banks, the water depth will decrease, which shall be the case in front of IJburg. What is more, high flow velocities are likely to occur in canals and nearby sailing vessels with large main propellers and large bow thrusters. But, the loads due to flow velocities will not outweigh the wave loads from yachts and sailing boats. In addition, ice loads are prescribed to retaining structures in lakes. Apart from this, the type of subsoil around Amsterdam is weak clay and extreme earthquakes are not likely to occur around Amsterdam.

STEP 7: BREAKWATER ALTERNATIVES

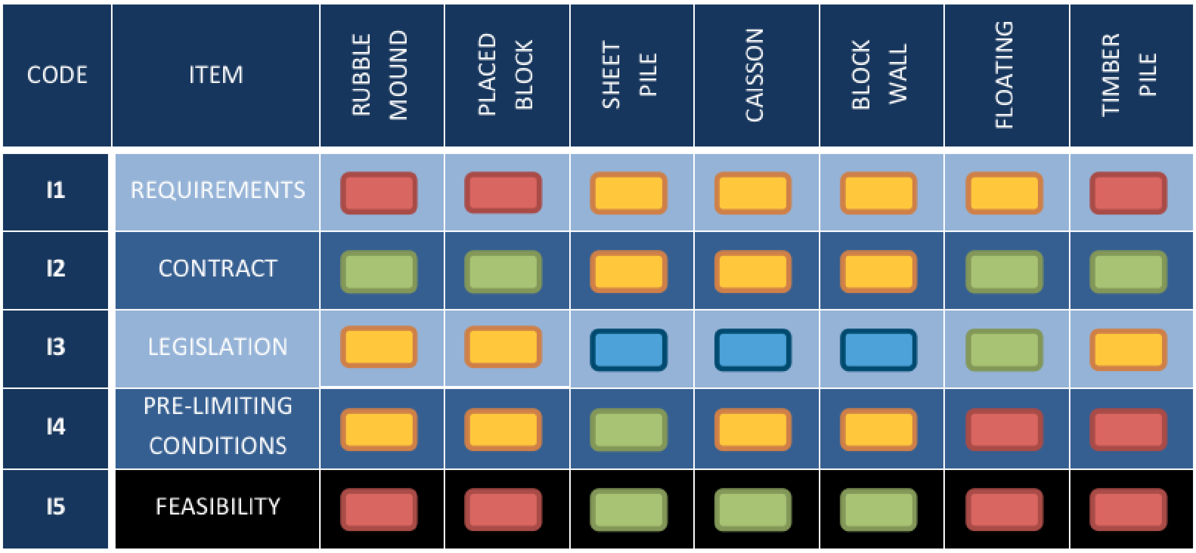
To perform a broad analysis, all optional breakwater alternatives are taken into account. The system provides the following breakwaters: rubble mound, placed block, sheet pile, caisson, block wall, floating, timber pile, tire, reef ball, gabion, screen, geotubes and synthetic. Moreover, these inland breakwaters do not have characteristics which instantly apostatizes them. For example, high shipping costs or abnormal prices of materials are not recognised.

STEP 8: PRE-SELECTION

In the previous steps the requirements, contracts, legislation and pre-limiting conditions are discussed, which are essential input for the pre-selection of several breakwater alternatives. The influence of these aspects for each breakwater is studied, which leads to recommendations of neglection and further examination of alternatives. The findings are shown in Table 5.7. In the table the color of the shapes indicate the conflict of a factor and breakwater alternative combination. The red shapes refer to 'Impossible or Not Preferable', the green shapes are 'Possible or Preferable' and the yellow shapes refer to 'Depends on Situation'. The blue shape implies many rules to meet and conflicts with certain permits.

The interaction of the breakwaters with the requirements and contracts is enclosed in Appendix M *Sub Flow Chart A: Requirement Verification and Contract Conflicts*. The requirement on enabling mooring opportunities (F11) and no damage to vessels (F12) are causing issues with the rubble mound, placed block, timber pile, tire, gabion, geotube and synthetic breakwater. While, the reef ball structure is not observable for navigation (F9). Contrarily, reducing wave action (F1) and sufficient lifetime (F17) are fulfilled by all breakwater alternatives. Numerous arguments can be provided to other requirements to take into account or neglect a breakwater.

Table 5.7: Pre-selection Scheme of IJburg Case Study



Apart from this, the DB& M contract requests attention for the experience in designing, construction and restoration of damage a particular breakwater. The design and maintenance of a rubble mound is relatively secure. In contrast, the repair works of sheet pile, caisson, block wall and screen breakwater are difficult and expensive. While the experience in designing of a reef ball, gabion and synthetic breakwater is regularly limited.

Laws, regulations and permits could also cause conflicts. The conflicts between legislation and breakwater alternatives are shown in Appendix N *Sub Flow Chart B: Legislation Conflicts*. The Law of Flora and Fauna is an obstruction to the noise generation of hammering activities of a sheet pile, pile and screen breakwater. Another issue occurs when structures penetrate or require removal of subsoil. Water quality could be decreased temporarily or polluted soils released, which is relevant for caissons, block walls, reef balls and gabions. For the mentioned breakwater alternatives, additional costs, delays and uncertainties in progress are expected.

The pre-limiting conditions as discussed, are studied per breakwater alternative in Appendix O *Sub Flow Chart C: Pre-limiting Conditions*. First of all, a lake area is considered. This implies that all breakwater alternatives are optional. Secondly, it is recommended to make a distinction between temporary or permanent solutions. As a port structure is requested, which has the risk of vessel collision, timber pile and reef ball breakwaters are less agreeable. Moreover, timber pile structure is sensitive to the fluctuation of weather conditions and generally have a low lifetime. In contrast, the reef ball breakwater has a rather high wave transmission and could not easily cope with high or fluctuating water levels due to the limited dimensions.

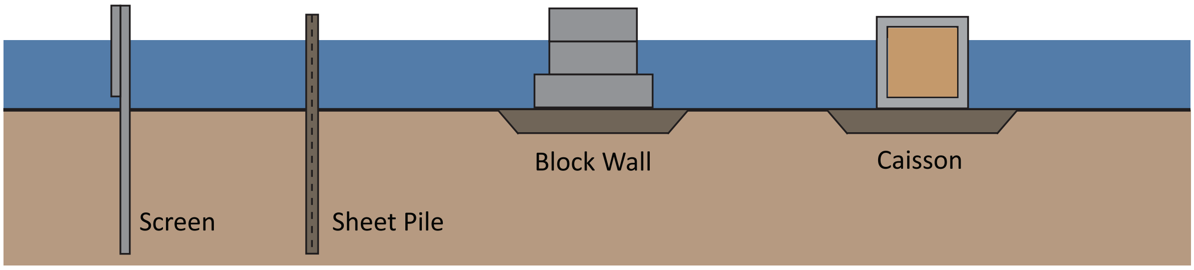


Table 5.8: Chosen Breakwater Alternatives after Pre-selection of IJburg Case Study

The boundary conditions are discussed and fit the expectations of the location. For example, relevant ice loads are a problem to the floating, timber pile, tire, reef ball and synthetic breakwaters. But the incorporation of stiff or weak subsoil conditions are unknown at this moment. This could be an advantage or risk to rubble mound, placed block, caisson, block wall and gabion breakwaters. Because settlements of these structure could allow weak spots to develop.

According to the analysis above, the sheet pile, caisson, block wall and screen breakwater seem feasible (Figure 5.8). These are recommended to continue with in the dimensioning and cost estimates.

In the case study all breakwater alternatives are taken into account in both selection phases. In this manner also insight in non pre-selected breakwaters is obtained. Nevertheless, the method is designed to incorporate <u>only</u> the pre-selected breakwaters for the subsequent design steps.

STEP 9: QUANTIFIED BOUNDARY CONDITIONS

The boundary conditions consist of wind, vessel, flow velocity, wave, ice, subsoil and earthquake data. These are derived from the original design report of the IJburg breakwater Haghgoo Daryasari (013b). The mentioned magnitudes are adapted to the classification of the boundary conditions, which are to be found in Table 5.9 and its references. These values are used for computations for the design considerations.

The probability of occurrence of the design storm is assumed 1/200 per year. This provides the maximum wind velocity, which should not lead to collapse of the breakwater. According to wind data from KNMI weather station at Lelystad, the design wind velocity is 20 m/s. What is more, a second design wind velocity is the maximum where the vessels should still be able to enter of the harbour. From experience the port manager knows that vessels are still sailing at wind conditions of 6 Bft, but these vessels can only enter the marina at wind conditions of 4 Bft. Consequently, the wave height generated at 6 Bft (approximately 12.3 m/s), has to be reduced to the wave heights generated at 4 Bft. Within the wind classification, this design wind velocity becomes 15 m/s.

The design vessel is between the 4 and 20 m in length. Therefore, a Small Yacht with a length of 15 m is assumed in the vessel classification. These are also able to sail in this region, since from a survey a water depth of 2.5 m (water level 0 m NAP and bed level -2.5 m NAP) is found. Because a lake is considered, a flow velocity from discharge of 0 m/s is expected. Exclusively vessels could generate flow velocities due to a return flow and main propeller flow velocities. In fact, the presumed recreational vessel does not have bow thrusters and return flow is neglected since to water displacement is insignificant compared to the larger vessels. Moreover, a large width of a river, where vessel sail far from the banks, will reduce the return flow velocity. A similar situation applies to lakes.

At the lake IJmeer, the fetch length is approximately 30 km. This can be concluded when a wind direction of 60 deg to the north is found. This draws a fetch line between IJburg and Lelystad. From rules of thumb a wind

Table 5.9: Boundary Conditions of IJburg Case Study

Boundary Conditions	**Magnitude**	**Units**	**Reference**
Wind velocity	15	m/s	Appendix E.1
(for design wind to design vessel conditions)			
Wind velocity	20	m/s	Appendix E.1
(for design storm to determine wave forces)			
Type of Vessel	Small Yacht	–	Appendix E.2
Vessel Properties	15	*Length* (m)	Appendix E.2
	5	*Beamwidth* (m)	
	2	*Draught* (m)	
Water Depth	2.5	m	Appendix E.3
Flow Velocity from Discharge	n/a.	m/s	Appendix E.4
Flow Velocity from Return Flow Velocities	0	m/s	Appendix E.4
Flow Velocity from Main Propeller	0.5	m/s	Appendix E.4
Flow Velocity from Bow Thruster	n/a.	m/s	Appendix E.4
Type of System	Largest Lake	–	Appendix E.5
Fetch length	30000	m	Appendix E.5
Wave Height by Wind	1	*Wave Height* (m)	Appendix E.5
(for design wind to design vessel conditions)	25	*Wave Length* (m)	
	4.0	*Wave Period* (s)	
Wave Height by Wind	1	*Wave Height* (m)	Appendix E.5
(for design storm to determine wave forces)	25.00	*Wave Length* (m)	
	4.0	*Wave Period* (s)	
Wave Height by Vessel	0.3	*Wave Height* (m)	Appendix E.5
	7.50	*Wave Length* (m)	
	2.2	*Wave Period* (s)	
Ice thickness	0.2	m	Appendix E.6
Subsoil	Light Clay	–	Appendix E.7
Modulus of Subgrade Reaction	0.045	N/mm^3	Appendix E.7
Earthquake	22	cm/s^2	Appendix E.8

velocity of 15 m/s, water depth of 2.5 m and fetch of 30 000 m leads to a significant wave height of 0.88 m. This could be explained by the fact that the wind velocity is increased from 12.3 m/s (wind from design report) to 15 m/s (classified design wind). For this reason the significant wave height from the classification will be 1 m. The computation by the formula of Young and Verhagen for a wind velocity of 20 m/s results in 1.18 m. For further calculations a classified design wave height of 1 m is considered, which prevents significant overestimation when higher values are chosen. Additionally, the wave and water depth ratio will prevent the breaking of waves, which result in lower wave heights. The maximum wave height generated by a Small Yacht vessel at a distance of 10 m to the breakwater is 0.28 m. This wave height is not considered due to the larger wind-generated wave height. In the classification of the flow velocities, this value would have become 0.3 m.

At the IJmeer a mean ice thickness of 0.2 m has been assumed from a trend. Over the years, the ice thickness is reduced from 0.4 m in the winter periods. Apart from this, also geotechnical data is required. From a geotechnical survey it has been found that from -2.5 m NAP to -9.0 m NAP there is layer of weak clay. The subsoil consist of the following layers:

-2.5	to -9.0	m NAP	Weak Clay
-9.0	to -10.0	m NAP	Peat
-10.0	to -14.0	m NAP	Sand Poor
-14.0	to -18.5	m NAP	Sand Loose with Clay Lumps
-18.5	to -25.0	m NAP	Strong Clay
-25.0	to -50.0	m NAP	Sand Compacted

As a result of the large Weak Clay layer, the classified Light Clay is chosen which is characterised by a modulus of vertical subgrade reaction of 0.045 N/mm^3. Assuming that the vertical force of the structure on the peat layer is sufficiently distributed, this would not cause additional settlements.

According to Crook (1996), IJburg is in Zone B of the earthquake zone division. This implies that horizontal acceleration of 22 cm/s^2 with an occurrence of 1/475 years. A short description of this acceleration is: 'Felt by most people; dishes rattle, some break'.

STEP 10: DESIGN CONSIDERATIONS

The various boundary conditions are classified and can be used for subsequent computations to determine the impact of flow velocities, wave heights, wave forces, ice forces and subgrade reactions (Table 5.10). Moreover, aspects on wave behaviour at a structure is considered by taking into account wave run-up, wave overtopping, wave transmission and wave reflection (Table 5.11).

Table 5.10: Design Considerations of IJburg Case Study

Design Consideration	**Magnitude on Slope**	**Magnitude on Wall**	**Units**	**Reference**
Flow Velocity from Discharge around River Groyne	n/a	n/a	m/s	Appendix F.1
Flow Velocity from Vessel around River Groyne	n/a	n/a	m/s	Appendix F.1
Distance Propulsion to Structure	4	4	m	Appendix F.1
Flow Velocity from Main Propeller on Structure	0.8	0.8	m/s	Appendix F.1
Flow Velocity from Bow Thruster on Structure	n/a	n/a	m	Appendix F.1
Water Level Rise from Wave	n/a	0.2	m	Appendix F.2
Wave Pressure (impact zone)	n/a	10	kN/m^2	Appendix F.2
Wave Pressure (at bed)	n/a	10	kN/m^2	Appendix F.2
Vessel Collision Force	n/a	n/a	kN	Appendix F.3
Ice Force	n/a	165	kN/m	Appendix F.4
Vertical Subgrade Reaction	0.14	0.14	$N/mm2$	Appendix F.5
Wind Set-up	0.4	0.4	m	Appendix F.6

Since river groynes are not considered, the impact of flow velocity due to a river discharge or return flow from vessels is out of scope. The vessel draught is smaller than the water depth, which enables them sail at the location of the breakwater. Taking into account a water depth of 2.5 m and a Small Yacht at a distance of 4 m to the structure, this result in a design flow velocity of 0.8 m/s. Within the classification, this becomes 0.75 m/s.

Considering waves in front of a vertical wall, a design wave of 1 m will result in a water level increase of 0.24 m. The classified water level rise is 0.2 m.The wave pressure at the impact zone is 9.8 kN/m^2 and at the bed 7.2 kN/m^2, when a wave height of 1 m and a water depth of 2.5 m is considered. The classification provides a value of 10 kN/m^2 for both pressures. This force is taken into account in contrast to the vessel collision force. Normally, guiding structures are designed to prevent damage to breakwaters, quay walls or jetties.

The classified ice thickness results in a design force of 165 kN/m. This will be the governing force acting on the breakwater. Apart from this, also the allowable vertical subgrade reaction pressure can be computed. For Light Clay, this becomes 0.14 N/mm^2.

Wind set-up also leads to a water level increase. A wind set-up of 0.36 m is computed with a fetch length of 30 000 m, a wind velocity of 20 m/s and a water depth of 2.5 m. In the classification, this becomes 0.4 m.

The largest wave height of 1 m is chosen to determine the wave-structure interaction. The wave run-up is computed for a slope of 34 deg of rough material and results in 1.38 m. In the next section, the allowed transmission of (0.4/1=) 0.4 is dominating the determination of the freeboard. Moreover, the run-up is related to the wave overtopping to support safety of pedestrians or vehicles. The breakwater is not designed for pedestrians. Owing to this, the wave run-up and wave overtopping are not considered. Additionally, the wave reflection is not considered. This could cause issues for vessels in front of the breakwater, when wave amplitudes increase and wave are in phase.

Table 5.11: Wave-Structure Interaction of IJburg Case Study

Design Consideration	Magnitude on Slope	Magnitude on Wall	Units	Reference
Wave Run-up	1	n/a	m	Appendix F.8
Wave Overtopping	n/a	n/a	m	Appendix F.9
Wave Transmission	0.4	0.4	m	Appendix F.11
Wave Reflection	n/a	n/a	m	Appendix F.12

STEP 11: BREAKWATER DIMENSIONS

The material quantities and construction duration highly depend on the dimension of the various breakwaters. A distinction is made between sloped, vertical and floating structures. According to the wave transmission ratio of 0.4, the sloped rubble structure require a freeboard of 1 m, while the vertical structure should have a freeboard of 0.5 m. For the floating breakwaters, the wave transmission should independently be analysed.

Sloped structure:

Crest height = water depth + wind set-up + freeboard
= 2.5 + 0.4 + 1 = 3.9 m
Freeboard = wave run-up, wave transmission or wave overtopping

Vertical structure:

Crest height = water depth + wind set-up + water level rise by waves + freeboard
= 2.5 + 0.4 + 0.2 + 0.5 = 3.6 m
Freeboard = wave transmission or wave overtopping

Floating structure:

dimensions ~ wave transmission = 0.4

The size of the breakwater elements depend mainly on the force acting on the structure. The horizontal forces considered originate from waves and ice. Ice forces are regularly significantly higher than wave forces and are frequently dominant. The loads including the required safety factors are:

Wave force = 10 kN/m
Ice force = 165 kN/m

Also the vertical stability should be taken into account by considering the vertical subgrade reaction of the subsoil. The magnitude of this pressure should be close to the exerted pressure on the subsoil. Otherwise, additional settlements, which are larger than 50 mm, are expected. The subgrade reaction including the required safety factor is:

Subgrade reaction 140 kN/m^2

The classified boundary and design considerations are impacts on the breakwaters. The breakwaters are subsequently analysed paying strict attention to Appendix I. In this appendix the required dimensions and structural performance are considered. An overview of breakwater considerations and classes is shown in Appendix Q *Sub Flow Chart E: Load-Structure Interaction and Breakwater Classification.* Extensive tables with information about the breakwater performance are left out and referred to.

Rubble Mound Breakwater: The design wave height during storm conditions is between 0.94 and 1.46 m, which corresponds with the nominal diameter of stone of respectively 417 and 646 mm. To prevent underestimation 646 mm is chosen. The critical flow velocity for this stone size is 1.75 m/s, which is above the classified flow velocity of 0.8 m/s. The classified dimensions of the rubble mound breakwater leads to a crest height of 4 m. This is similar to 'Rubble Mound 11' (RM11). The vertical pressure exerted to the subsoil is 115 kN/m^2, which is close to the allowed pressure. Settlements in centimetres are expected, which can be allowed. Ice loads are neglected for structures with a slope angle of 45 deg and less steep. Additionally, one could also chose the decrease the slope angle of 33 deg to be able to use smaller stone diameters and lower crest height. But the difference in volume is approximately similar or even larger.

Placed Block Breakwater: The design wave considered results in a required block thickness of 0.2 m, which is 'Placed Block 2'. Similar to the rubble mound breakwater, the crest level is at 4 m above bed level. The self-weight and footprint of the structure results in 44 kN/m^2, which is below the maximum allowed pressure.

Sheet Pile Breakwater: The classified crest heights for sheet piles vary with 2 m. The design crest height of 3.4 m is between 2 and 4 m. To obtain sufficient height, the highest crest height is chosen. The dominant ice force results in sheet pile type 'AZ 20-700'. This pile type has sufficient strength to resist the bending moment.

Caisson Breakwater: The caisson breakwater is approached as a vertical wall. The crest height is 4 m, which points to 'Caisson 2' in the classification. In the structural approach, the structure fails to resist the bending moment, but fulfils the friction requirement. Moreover, the exerted pressure on the subsoil is approximately 38 kN/m^2. Based on the bending moment requirement, the caisson breakwater is not advised. The caisson can cope with the ice force when the width is increased.

Block Wall Breakwater: A block wall height of 4 m is taken into account, which implies 'Block Wall 2' in the classification. The stability of the blocks relies on the upper block. 3 blocks with different widths are assumed. To withstand 165 kN/m in a rotational and horizontal force verification, the block with the largest width is required, which is 'Block 3'. Thus, the block wall will consist of two times 'Block 3'. This result in a vertical pressure of 341 kN/m^2, which exceeds the allowed subgrade reaction pressure. Large deformations and settlements of the structure are expected. Therefore, this breakwater alternative is not recommended.

Floating Breakwater: The floating breakwater performance is based on the wave transmission. To obtain a transmission of 0.4 with a wave height of 1 m, the Inter Boat Marina M4316 suffices. Ice loads and movement are not preferable, since it could increase the stresses in the concrete and it can displace the structure to areas where it can damage. It is possible to dimension the anchor system in the preliminary design phase to ice loads. This breakwater is not recommended, but with additional reinforcement in the concrete and anchor weight analysis this breakwater could be optional.

Timber Pile Breakwater: The minimum crest height of the piles will be at 4 m, which is similar to class 'Pile 2'. Reading the results of the compression and tensile stresses in the cross-section, it can be concluded that piles fail for any ice load. In contrast, wave transmission can be taken into account. For a wave height of 1 m the piles should be spaced with 0.1 m. Due to the failure by ice loads, this breakwater is excluded from the analysis.

Tire Breakwater: The tire breakwater is dimensioned based on the wave transmission. The design wave and transmission coefficient considered, lead to a tire breakwater with a larger width than 10 m. Therefore, this one is neglected based on the high labour time and costs. Also ice loads are not preferable for this breakwater alternative.

Reef Ball Breakwater: This system has a maximum crest height of 1.31 m, which is the 'Ultra Ball'. Due to the limited dimensions these structure are not applicable to water depths larger than 2 m. Therefore, this alternative will not affect waves or counter ice loads and will be dissuaded.

Gabion Breakwater: The crest height of the gabion breakwater shall be 4 m, which refers to 'Gabion 2'. Since the upper cages are connected, the bending moment and horizontal force resistance are increased. Still not all ice forces can be coped with. One could choose to connect all cages and increase the weight of the upper blocks, but the strength of these connection is uncertain. Due to bending moment failure, this alternatives is not advised.

Screen Breakwater: The crest height of the screen breakwater should also be above a water depth of 3.4 m, which result in 4 m within the classification. Ice and waves are both considered to observe respectively the strength and wave transmission. The wave transmission coefficient of 0.4 and a 1 m wave results in a porosity of 10 %. A plate of 20 mm thickness is assumed. Contrarily, the ice forces cannot be resisted by the TP508 tubular piles. Piles with a larger diameter could suffice, but will lead to higher material costs. Also the IPE300 support beam fails at the ice force. Thus, the screen breakwater is not recommended due to structural failure.

Geotube Breakwater: Within the geotube classification, 'Geotube 4' (crest height of 3.4 m) can fulfil the requested crest height of 3.9 m when a double layer of loose rock of 417 mm (nominal diameter) is applied. Accordingly, a crest height of 4.2 m is obtained. The vertical pressure is approximately 99 kN/m^2.

Synthetic Breakwater: The maximum size of the synthetic breakwater can attenuate wave heights up to 0.5 m. Since a wave height of 1 m is considered, this breakwater cannot be applied with sufficient effect. Moreover, the light weight structure and low strength synthetic elements are not applicable when ice loads are considered.

Table 5.12: Breakwater Classes of IJburg Case Study

Breakwater Alternative	**Classification**	**Performance**
Rubble Mound	Rubble Mound 11	
Placed Block	Placed Block 2	
Sheet Pile	AZ-20 700	
Caisson	~~Caisson 2~~	bearing capacity failure
Block Wall	~~Block Wall 2 and Block 3~~	bearing capacity failure
Floating	~~Inter Boat Marina M4316~~	attenuate waves, unable to withstand ice
Timber Pile	~~Pile 2~~	bending moment failure
Tire	~~Tire Breakwater 5~~	maximum size limit reached, unable to withstand ice
Reef Ball	~~Non~~	maximum water depth exceeded
Gabion	~~Gabion 2~~	rotational failure
Screen	~~TP508 and IPE300~~	bending moment failure
Geotube	Geotube 4	
Synthetic	~~Non~~	maximum wave height exceeded, unable to withstand ice

The optional breakwater alternatives obtained their dimensions and are studied on their structural performance. This leads to dissuasion of various breakwaters when their capacities are exceeded. The chosen breakwater classes and remarks on their performance are presented in Table 5.12.

STEP 12: MATERIAL COST ESTIMATE

The breakwaters obtain material costs by paying strict attention Appendix I and by following the chosen breakwater classes from Table 5.12. It is optional to extract the costs from Appendix R *Sub Flow Chart F: Material, Labour and Equipment Costs.*

Table 5.13: Breakwater Material Costs of IJburg Case Study

Breakwater Alternative	Value	Units	Remarks
Rubble Mound	1068	€/m	
Placed Block	1433	€/m	(=313+53+13+1054)
Sheet Pile	1280	€/m	
~~Caisson~~	>4579	€/m	
~~Block Wall~~	>3384	€/m	
~~Floating~~	>676	€/m	
~~Timber Pile~~	>1194	€/m	
~~Tire~~	>58	€/m	
~~Reef Ball~~	-	€/m	
~~Gabion~~	>1180	€/m	
~~Screen~~	>3024	€/m	(=1053+1895+76)
Geotube	1005	€/m	
~~Synthetic~~	-	€/m	

The type, the amount and the material costs are combined per chosen breakwater class. The computed costs per stretched meter is shown in Table 5.13. The values are indications and approximations. Thus, these could vary per company and period of time.

STEP 13: LABOUR AND EQUIPMENT COST ESTIMATE

The breakwaters obtain a labour and equipment costs by paying strict attention to Appendix I and by following the chosen breakwater classes from Table 5.12.

Table 5.14: Breakwater Labour and Equipment Costs of IJburg Case Study

Breakwater Alternative	Value	Units	Remarks
Rubble Mound	62	€/m	
Placed Block	2233	€/m	
Sheet Pile	69	€/m	
~~Caisson~~	>401	€/m	
~~Block Wall~~	>1147	€/m	
~~Floating~~	>165	€/m	
~~Timber Pile~~	>100	€/m	
~~Tire~~	>1037	€/m	
~~Reef Ball~~	-	€/m	
~~Gabion~~	>1188	€/m	
~~Screen~~	>744	€/m	
Geotube	50	€/m	
~~Synthetic~~	-	€/m	

The amounts of material, the size of structural elements and the required equipment are combined per chosen breakwater class. The direct costs are computed. The result is shown in Table 5.14. Indications and approximations of the material costs are implemented. Thus, these could vary per company and period of time.

STEP 14: EMVI DISCOUNT

In the tender phase of this project, BVP criteria were provided. These criteria are broad and ill-defined. EMVI criteria can be defined based on the BVP criteria, but it is unclear whether these criteria shall count and what the amount of discount/points will be. For this reason, an EMVI discount is not taken into account.

STEP 15: COST-BASED SELECTION

In the previous sections, the structural performance, material, labour and equipment costs are discussed. The structural performance is not cost-based, but is essential to determine whether the structure is functional or to expect additional costs to improve a structure. The direct costs and EMVI discount are shown in Table 5.15. The values are round up to the nearest hundreds in the last row. When breakwaters fulfil the requirement, the green shape is used. The red shape refers to breakwaters which fail the governing load.

Table 5.15: Cost-based Selection Scheme of IJburg Case Study

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER
I6	HYDRAULIC FULFILMENT							
I7	COSTS OF MATERIAL (€/m)	1068	1433	1280	>4579	>3384	>676	>1194
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	62	2233	69	>401	>1147	>165	>100
I9	EMVI DISCOUNT (€/m)	N/A	N/A	N/A	N/A	N/A	N/A	N/A
I10	TOTAL (€/m)	1300	3700	1400	>5000	>4600	>1000	>1300

CODE	ITEM	TIRE	REEF BALL	GABION	SCREEN	GEOTUBES	SYNTHETIC
I6	HYDRAULIC FULFILMENT						
I7	COSTS OF MATERIAL (€/m)	>58	N/A	>1180	>3024	1005	N/A
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	>1037	N/A	>1188	>744	50	N/A
I9	EMVI DISCOUNT (€/m)	N/A	N/A	N/A	N/A	N/A	N/A
I10	TOTAL (€/m)	>900	N/A	>2400	>3800	1100	N/A

The hydraulic fulfilment is initially met by the rubble mound, placed block, sheet pile and geotube breakwater. Excluding the expensive placed block breakwater, the breakwaters are in the same order of magnitude and therefore selected (Figure 5.16). These do not require improvements to be stable and fully operational. What is more, the placed block seems less attractive due to the relatively high price. This is in contrast with the floating, timber pile, tire breakwater, which are less expensive. One can reconsider these alternatives, when their strength is improved and when requirements are negotiable. The least feasible breakwaters are the caisson, blocks wall, gabion and screen breakwater. These are structurally failing and high in costs. As a result of a computed crest height and a classified crest height, which are more or less similar, cost ranges per breakwater alternative are not considered. In fact, if a structure is over- or under-dimensioned, the average costs between two classes can be assumed.

The choice of a breakwater alternative also depends on the experience in designing and construction. When a structure is not familiar to a contractor, the design and construction phase could consume both extra time and money. In particular, engineering companies could require consultation to perform the design, which provides additional costs. What is more, when issues occur during construction, the process of solving it could take time. Therefore, it is important to consider the strength of the contractor. This implies that the labour and equipment costs consist of the works and risk.

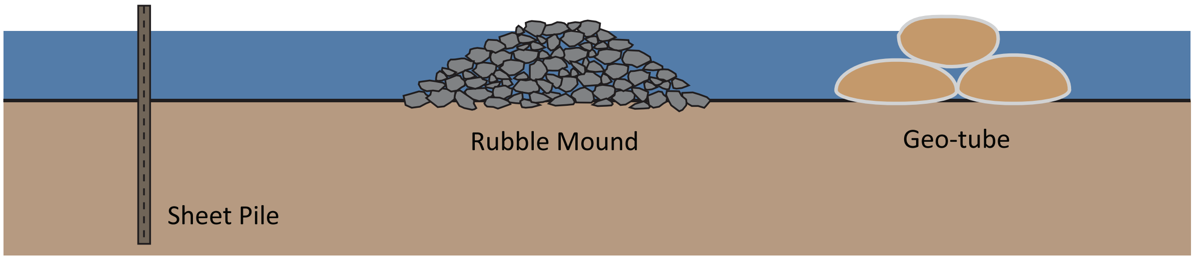


Table 5.16: Chosen Breakwater Alternatives after Cost-Based Selection of IJburg Case Study

STEP 16: DECISION

The selection tables are helpful tools for the pre-selection and cost-based selection. In the pre-selection, the sheet pile, caisson, block wall and screen breakwater are declared feasible alternatives (Table 5.7). These show no instant conflict with the (mandatory or negotiable) requirements. The type of contract focussed on the ability of the contractor to design, build and maintain. These items refer to the experience of a contractor, the complexity of the works, the sensitivity to damage and the simplicity of the repair works. What is more, legislation and the pre-limiting conditions (location, purpose and boundary conditions) show no direct complications.

Within the cost-based selection, optional breakwaters are rubble mound, placed block, sheet pile and geotube breakwater (Table 5.15). An interesting find is that the sheet pile breakwater is the only option, which is feasible in both selection phases and has a low price compared to the other breakwater alternatives. It is also lower in costs than the floating and tire breakwater. However, improving these structures to withstand ice loads would increase the costs, which could probably exceed the costs of the sheet pile breakwater. Therefore, the sheet pile breakwater is recommended for further design phases.

5.2. REFLECTION ON CASE STUDY 1

This section discusses the selected breakwater and governing aspects of the DSM.

5.2.1. SELECTED BREAKWATER

Contractor Hakkers selected the sheet pile breakwater as most effective breakwater at IJburg which is similar to the chosen breakwater in the case study. This is an unexpected outcome since the breakwater was initially selected on the requirements only. In other words, the type of contract, laws, permits, pre-limiting conditions, hydraulic fulfilment and cost estimate were not considered. Accordingly, there was a high probability that due to these criteria another breakwater would have been chosen. Moreover, the sheet pile breakwater shows the highest score in the pre-selection. Among the four pre-selected breakwaters the sheet pile breakwater also shows the most preferred in costs. Therefore, the extra selection criteria and cost estimate emphasize the choice for the most effective breakwater.

5.2.2. GOVERNING ASPECTS

Within each design step, certain aspects dominate the assessment. Therefore, insight in the most important considerations is provided in this section.

The method is fully designed to select a breakwater alternative. This implies that the selection phase, consideration of alternatives and breakwaters are mandatory items to determine the applicability of the method. In contrast, the framework of inland waterways in the Netherlands could be circumvented, but the maximum magnitude of boundary conditions should be considered for coastal waters. In this matter, abroad the Dutch legislation should be neglected.

The governing functional requirements were prescribing that the structure should: provide mooring opportunities, prevent damage to vessels and enable walk-ability. Other requirements were mostly fulfilled or should be assessed by experts. For example, the structure should: reduce wave action, be observable and not block the concerned ship channel, which is fulfilled by almost all breakwaters. More difficult considerations are about the life cycle costs, aesthetics and local spatial shapes. The life cycle costs consist of the project costs plus the periodic maintenance costs. The maintenance is not incorporated, because it is rather difficult to predict. In addition, for the aesthetics and local spatial shapes, architects should be consulted.

The DB&M contract is governing since it takes into account the strength of the contractor. When contractors are not experienced with particular breakwater alternatives, significant risks are found in the design, build and maintenance phase. As a result, engineering time will increase, issues during construction can not be solved easily and maintenance is insecure. Thus, costs are likely to increase. The consideration of the type of contract is less concrete and decisive compared to the requirements.

The legislation could play a role due to noise and water quality in the case study. What is more, the number of permits is not an issue, although there is a higher probability that delays in the works occur. Therefore, experts should be consulted to draw conclusion about the interaction between laws, permits and legislation with breakwater alternatives.

The pre-limiting conditions are in the case study dominated by large waves (wave height > 1 m), ice and weak subsoil. It can directly be decided that ice governs all other forces. Additionally, the weak subsoil shows conflicts with the founded breakwaters. An advantage is that more breakwaters can be neglected in the pre-selection. Despite of this, more certainty about the hydraulic performance of a breakwater is found during the phase of dimensioning.

The most important boundary conditions are the water depth (2.5 m), the wave height (1 m), the subsoil (Light Clay) and the ice thickness (0.2 m). These determine mainly the crest height, strength and stability of a breakwater. To be more specific, the water depth and wave height determine the crest height and the ice load determine the strength and stability. A second stability check is performed by analysing the subsoil.

The phase of dimensioning contains the hydraulic fulfilment. In this case study the ice load is the governing load of multiple breakwaters. The floating breakwater fails with ice but is dimensioned on wave transmission.

The cost estimate together with the mandatory requirements are considered to be the most important aspects of the method. The difference between the cost estimation of the materials, labour and equipment differ. Therefore, their contribution is equally important.

5.3. CASE STUDY 2: DALEMSCHE GEUL BREAKWATER

Along the Upper Merwede between two river groynes, a small basin is located, which goes by the name Dalemsche Geul. The basin is about 600 m in length and 200 m in width. The reallocation to a multifunctional basin is a recent fictitious idea which implies a temporary marina with recreational beach and nearby an extended habitat area (Figure 5.2). A breakwater is requested to protect these facilities from significant wave attack. In addition, the floating jetties of the yacht harbour and artificial beach will be exploited only in the season of sailing sport (April to August). To support the development of flora and fauna, the jetties and breakwater will be withdrawn in the winter.

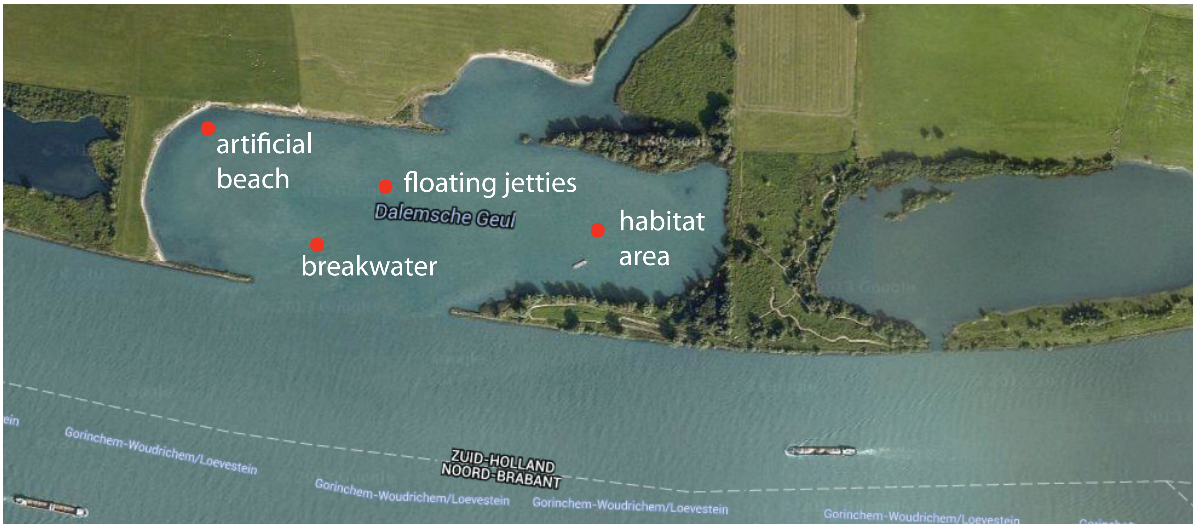


Figure 5.2: Topview of Location Dalemsche Geul Case Study

In the following sections, the project is briefly described and recommendations are formulated for the choice of an optional breakwater alternative.

5.3.1. PROJECT DESCRIPTION

The river and basin will be separated to support an ecological area. Owing to this, the area is not disturbed by the river flow. However, waves and ice should be considered. This is due to the gap between the river and the basin, which supports the exchange of water to enhance the water quality. Because a breakwater is not allowed in the river, it will be located in the basin. The breakwater is assumed to require 200 m of length with sufficient distance to the two current groynes, which are positioned parallel to the river.

The project location can be considered as a small lake area. This implies that a (short) fetch length should be considered, which is approximately similar to the width of the river. It is known that the water depth is relatively large compared to other inland water systems. Moreover, the waves in the basin are generated by pushed convoy vessels and high wind velocities. It is assumed that the subsoils in the surroundings consist of stiff soils and mainly sand. Owing to the fact that in the river polluted subsoils have been found, there is a high probability that the bed of the basin contains pollutant of contaminants, which can be exposed during excavation.

A Design & Construct contract is offered for a non-detached breakwater. The tender consist of an elaborated breakwater alternative, which is granted based on EMVI criteria. The four criteria define the structure as being robust (€65,000), ecologically (€150,000), noise limited (€110,000) and low-risk (€75,000).

5.3.2. DECISION SUPPORT METHOD IMPLEMENTATION

STEP 1: SYSTEM APPLICATION

The decision support method applicability is discussed Section 4.2. The following can be concluded in the validity range:

1.	Phase: Selection	Satisfied
2.	Subject: Consideration of alternatives	Satisfied
3.	Structure: Breakwaters	Satisfied
4.	Location: Inland waterways in the Netherlands	Satisfied
5.	Request: Design (and construct)	Satisfied
6.	Tender: EMVI-based (optional)	Satisfied

In contrast to case study 1 (Section 5.1), the EMVI tender approach is applied. Another difference is the D&B contract, which excludes the responsibility of maintenance of the contractor.

STEP 2: LIST OF REQUIREMENTS

The breakwater will be subjected to the functional requirements, operational requirements and stakeholder conditions. The following functional requirements are formulated. The structure:

Table 5.17: Functional Requirements of Dalemsche Geul Case Study

Code	Description
F1	reduces wave action to enhance shipping.
F3	reduces wave action to reduce erosion.
F5	should not support sedimentation in the ship channel.
F6	shall be observable for navigation at all times.
F10	is not blocking the ship channel.
F11	shall not enable mooring opportunities.
F15	shall not affect or interrupt the aquatic ecology.
F16	should be maintenance-free.
F17	should have a sufficiently long lifetime.
F18	should not diminish the water quality.
F19	is entirely removable after the expiration date.
F20	it should not hinder shipping, nearby traffic and local residents during its construction.
F21	should have the lowest investment costs.
F22	should have low life cycle costs.
F23	should support the requested aesthetics.
F24	should take limited space.
F25	should fit the local spatial shapes.
F26	should be able to cope with vessel collision.
F27	should be movable.

Additional requirements are F26 and F27. The requirement about vessel collision is formulated due to the absence of a guiding structure that takes into account vessel collision. The sensitivity to structural damage will be considered. The client also emphasize that the structure should be movable. Since these requirements are not part of the list, these should get special attention in the pre-selection (Step 8).

The operational requirements consist of an allowed wave height in the sheltered zone and a minimum lifetime. The structure:

Table 5.18: Operational Requirements of Dalemsche Geul Case Study

Code	Related Requirement	Description
O1	F1/F2/F3	reduces the significant wave height to 0.15 m.
O4	F16	has a lifetime of 20 years.

STEP 3: EMVI TENDER APPROACH

The tender phase is dominated by the EMVI criteria. These concern: robust, ecologically, noise limited and low-risk. The robustness of a structure refers to the strength and damage sensitivity. In contrast, the ecology aspect will discuss the disturbance to wild life and vegetation. Since a habitat area is close to the location of the breakwater, it should be considered. Related to ecology is noise which can be reduced by limited time of construction and construction methods. Within the classified EMVI criteria, the following items are considered:

1. System Quality
2. Ecological Impact
3. Noise
4. Risks

STEP 4: CONTRACT DEPENDENCY

The D&B contract compels the contractor to design and construct a breakwater. Maintenance is not taken into account. But the knowledge and experience in designing and constructing a breakwater alternative will be taken into account.

STEP 5: RELEVANT LEGISLATION

The Merwede is a typical Dutch water system, which is subjected to the Waterwet. While the area is already a natural site, initially the Natura2000, Vogel- en Habitatrichtlijn, Natuurbeschermingswet and Natuurcompensatie should be taken into account. The laws to be considered are enclosed in Table 5.19.

Table 5.19: Laws of Dalemsche Geul Case Study

Code	Type of Law	English Translation
L1	Waterwet (Ww)	Law of Water
L2	Flora- en Faunawet	Law of Flora and Fauna
L3	Wet Bodembescherming (Wbb)	Law of Underground Protection
L4	Natuurbeschermingswet	Law of Nature Protection
L5	Wet Milieubeheer (Wm)	Law of Nature Preservation
L6	Scheepvaartverkeerswet (SVW)	Law of Vessel Traffic
L7	Ontgrondingenwet	Law of Excavation
L8	Natuurcompensatie	Compensation of Nature
L9	Kaderrichtlijn Water (KRW)	European Water Directive

The required permits are enlisted in Table 5.20.

Table 5.20: Permits of Dalemsche Geul Case Study

Code	Type of Permit	English Translation
P1	Omgevingsvergunning	Permit of Surroundings
P2	Milieuvergunning	Environmental Permit
P3	Watervergunning (WBR and KEUR)	Water Permit
P4	Watermelding	Water Notification
P5	Ontgronding	Excavation
P6	Besluit Uniforme Saneringen (BUS)	Decision Uniform Soil Restoration
P7	Saneringsvergunning	Permits of Soil Restoration
P8	Bergingsverzoek Slibdepot	Storage Request Silt Deposit
P9	Kabels en Leidingen Informatie Centrum (KLIC)	Centre of Information about Cables and Pipes
P10	Meldpunt Opbrekingen Openbare Ruimte (MOOR)	Contact Point of Public Area
P11	Bouwvergunning	Permit of Construction

The considered regulations are shown in Table 5.21.

Table 5.21: Regulations of Dalemsche Geul Case Study

Code	Type of Regulation	English Translation
R1	Besluit Lozen Buiten Inrichtingen (BBI)	Decree Discharge Outside Interior
R2	Besluit Bodemkwaliteit (Bbk)	Decree Soil Quality
R3	Beheer- en Ontwikkelplan voor de Rijkswateren (BPRW)	Management and Development Plan of Waterways
R4	Activiteitenbesluit	Decree of Activities

STEP 6: PRE-LIMITING CONDITIONS

The breakwater shall be constructed parallel to the river in the basin area. Thus, flow velocities due to discharge are not considered. As a consequence, not a river groyne, but a lake breakwater is considered. The breakwater function is to protect the yachts along the jetties, the beach against scour and the habitat area. Taking into account the various boundary conditions, it is expected that:

Large waves can be neglected;
Large water depth is present;
High flow velocities can be neglected;
Ice is not considered;
Weak subsoil is absent;
Earthquakes can be neglected.

STEP 7: BREAKWATER ALTERNATIVES

The system provides an extensive set of breakwaters, which are the: rubble mound, placed block, sheet pile, caisson, block wall, floating, timber pile, tire, reef ball, gabion, screen, geotubes and synthetic breakwater.

STEP 8: PRE-SELECTION

The interaction of the requirements, contracts, legislation and pre-limiting conditions with the various breakwater alternatives are discussed. The pre-selection tends to apostatise non-feasible breakwaters. While other aspects provide recommendations about approving a breakwater alternative, the functional requirements are not negotiable. The findings are shown in Table 5.3.2. In the table the colour of the shapes indicate the conflict between a factor of influence and a breakwater alternative combination. The red shapes refer to 'Impossible or Not Preferable', the green shapes are 'Possible or Preferable' and the yellow shapes refer to 'Depends on Situation'. The blue shape implies many rules to meet and conflicts with certain permits.

Table 5.22: Pre-selection Scheme of Dalemsche Geul Case Study

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE
I1	REQUIREMENTS	Red	Red	Red	Red	Red	Yellow	Red
I2	CONTRACT	Green	Green	Green	Green	Green	Yellow	Green
I3	LEGISLATION	Yellow	Yellow	Blue	Blue	Blue	Green	Blue
I4	PRE-LIMITING CONDITIONS	Green	Green	Green	Green	Green	Yellow	Yellow
I5	FEASIBILITY	Red	Red	Red	Red	Red	Green	Red

CODE	ITEM	TIRE	REEF BALL	GABION	SCREEN	GEOTUBES	SYNTHETIC
I1	REQUIREMENTS	Green	Red	Red	Red	Red	Green
I2	CONTRACT	Yellow	Yellow	Yellow	Green	Yellow	Yellow
I3	LEGISLATION	Green	Blue	Blue	Yellow	Yellow	Green
I4	PRE-LIMITING CONDITIONS	Yellow	Red	Green	Green	Red	Yellow
I5	FEASIBILITY	Green	Red	Red	Red	Red	Green

The interaction of the breakwaters with the requirements and contracts is enclosed in Appendix M *Sub Flow Chart A: Requirement Verification and Contract Conflicts*. The movability requirement (F27) is governing for most alternatives. Hence, the rubble mound, placed block, sheet pile, caisson, block wall, gabion, reef ball, screen and geotube breakwater have to be discarded. The residuals are the floating, tire and synthetic breakwater, where the floating breakwater is the most sensitive to vessel collision (F26). It is presumed that the more flexible tire and synthetic breakwater will deflect without severe damage. The type of contract shows the lack of experience of the contractor. It is presumed that less known structures are the floating, tire, reef ball, gabion and geotube breakwater.

Providing the conflicts with laws, permits and legislation in Appendix N, it is expected that due to the polluted subsoil and the natural preserve multiple breakwaters are strongly recommended to be discarded.

Importantly, the permits on excavation and surroundings will probably not be granted to prevent exposure of pollutants of contaminants. Therefore, the excavation activities for the caisson, block wall, reef ball and gabion breakwater are causing conflicts. Driving piles for the sheet pile and timber pile breakwater are negatively interacting with the nearby habitat area. Therefore, floating breakwaters becoming attractive. Moreover, since the rubble mound, placed block, screen breakwater and geotube interact with the bed, the risk of delays due to conflicting permits is high.

The pre-limiting conditions are not providing any issues for most breakwaters. The interaction between these aspects and breakwaters are elaborated in Appendix O *Sub Flow Chart C: Pre-limiting Conditions*. The floating, tire and synthetic breakwater rely on circumstances without ice. The client prefers to remove the breakwater in the winter. Therefore, it is unclear whether it should be designed to withstand ice or not. It is assumed that the breakwater is only subjected to wave forces instead of ice loads. The reefbal and geotubes will be eliminated for the reason that large water depths are considered.

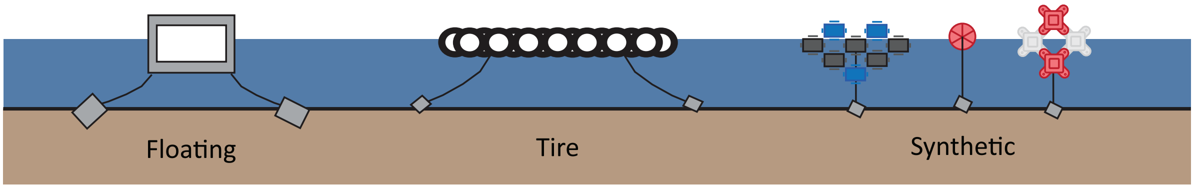


Table 5.23: Chosen Breakwater Alternatives after Pre-selection of Dalemsche Geul Case Study

The implementation of the pre-selection shows that the floating, tire and synthetic breakwater are yet the most promising (Figure 5.23). To observe the breakwater alternatives with the lowest costs, all breakwater alternatives are analysed in the subsequent design steps.

In the case study all breakwater alternatives are taken into account in both selection phases. In this manner also insight in non pre-selected breakwaters is obtained. Nevertheless, the method is designed to incorporate only the pre-selected breakwaters for the subsequent design steps.

STEP 9: QUANTIFIED BOUNDARY CONDITIONS

The boundary conditions consist of wind, vessel, flow velocity, wave, ice, subsoil and earthquake data. The assumed magnitudes are adapted to the classification of the boundary conditions, which are to be found in Table 5.24 and its references. These values are used for computations of the design considerations.

To obtain a safe structure, a return period of the design storm of 1/2000 years is assumed. This results in wind velocity of 27.4 m/s. Within the wind velocity classification, this becomes 30 m/s. According to Bureau Voorlichting Binnenvaart, the largest vessel at the Merwede is the pushed convoy vessel with 4 barges. This is similar to a type VIc vessel, which is approximately 193 m in length, 30 m in width and 4 m in draught. Within the vessel classification, these dimensions can be matched to the International Convoy Vessel (2). Moreover, Small Yachts are considered, because this is the type of vessel which will sail along the breakwater to the marina.

The local conditions imply a maximum water depth of 5.5 m. This large water depth exceeds the upper limit of 4 $m$in the classification. In the subsequent steps, this could cause additional calculations by hand and loss of time. Since the basin is separated from the main river, the flow velocity from discharge are assumed to be 0 m^3/s. Similarly, the vessels are sailing at a minimum distance of 50 m to the actual breakwater. Moreover, return flows are not likely to affect the location significantly.

To determine the wind-generated waves, the width of the river shall be presumed as fetch. This is approximately 300 m. The wind direction could differ from being perpendicular to the river. Therefore, a fetch length of 400 to 600 m is taken into account. This becomes 500 m in the classified fetch lengths. This fetch length, a water depth larger than 4 m and a design wind velocity of 30 m/s result in wave heights of 0.67 m. A classified wave height of 0.5 m is taken into account to prevent overestimation in case of a classified wave height of 1 m. It is expected that the dimensions of the classified breakwater can compensate the inequality.

Both convey vessels and yachts are likely to sail along the basin. These vessel will respectively develop waves of 0.22 m and 0.45 m. The waves generated by yachts are the most unfavourable and will lead to a classified wave height of 0.5 m. Another significant load is ice, which is not taken into account. First of all, since the breakwater is expected to be moved before the winter, ice will not be a design load. Secondly, the basin is relatively small and closed of from the behaviour of ice in the river.

Table 5.24: Boundary Conditions of Dalemsche Geul Case Study

Boundary Conditions	**Magnitude**	**Units**	**Reference**
Wind velocity	30	m/s	Appendix E.1
Type of Vessel (river)	International Convoy Vessel (2)	–	Appendix E.2
Type of Vessel (basin)	Small Yacht	–	Appendix E.2
Vessel Properties (river)	200 33 4.5	*Length* (m) *Beamwidth* (m) *Draught* (m)	Appendix E.2
Vessel Properties (basin)	15 5 2.0	*Length* (m) *Beamwidth* (m) *Draught* (m)	Appendix E.2
Water Depth	5.5	m	Appendix E.3
Flow Velocity from Discharge	0	m/s	Appendix E.4
Flow Velocity from Return Flow Velocities	0	m/s	Appendix E.4
Flow Velocity from Main Propeller	1.2	m/s	Appendix E.4
Flow Velocity from Bow Thruster	0.75	m/s	Appendix E.4
Type of System	Medium River	–	Appendix E.5
Fetch length	500	m	Appendix E.5
Wave Height by Wind	0.5 12.50 2.8	*Wave Height* (m) *Wave Length* (m) *Wave Period* (s)	Appendix E.5
Wave Height by Vessel	0.5 12.50 2.8	*Wave Height* (m) *Wave Length* (m) *Wave Period* (s)	Appendix E.5
Ice thickness	n/a	m	Appendix E.6
Subsoil	Heavy Clay	–	Appendix E.7
Modulus of Subgrade Reaction	0.090	N/mm^3	Appendix E.7
Earthquake	50	cm/s^2	Appendix E.8

The last boundary conditions are the subsoil and earthquakes. The area is characterized by heavy sand and clays. It is assumed that heavy clay is dominating the subsoil, which has a modulus of vertical subgrade reaction of 0.09 N/mm^3. Considering earthquakes, The Merwede lays within Zone C of the earthquake classes. With a probability of occurrence of 1/475 years, the structure should be designed to withstand horizontal accelerations of 50 cm/s^s.

STEP 10: DESIGN CONSIDERATIONS

The various boundary conditions are classified and can be used for subsequent computations to determine the impact of flow velocities, wave heights, wave forces, ice forces and subgrade reactions (Table 5.25). Moreover, aspects of wave behaviour at the structure is considered by taking into account wave run-up, wave overtopping, wave transmission and wave reflection (Table 5.26).

River groynes are not considered. Accordingly, no data is provided. The structure will be located in the sheltered zone, where flow velocity due to discharge is negligible. It is expected that only flow velocities

Table 5.25: Design Considerations of Dalemsche Geul Case Study

Design Consideration	Magnitude on Slope	Magnitude on Wall	Units	Reference
Flow Velocity from Discharge around River Groyne	n/a	n/a	m/s	Appendix F.1
Flow Velocity from Vessel around River Groyne	n/a	n/a	m/s	Appendix F.1
Distance Propulsion to Structure	8	8	m	Appendix F.1
Flow Velocity from Main Propeller on Structure	0.4	0.4	m/s	Appendix F.1
Flow Velocity from Bow Thruster on Structure	n/a	n/a	m	Appendix F.1
Water Level Rise from Wave	n/a	0.1	m	Appendix F.2
Wave Pressure (impact zone)	n/a	5.0	kN/m^2	Appendix F.2
Wave Pressure (at bed)	n/a	0	kN/m^2	Appendix F.2
Vessel Collision Force	n/a	n/a	kN	Appendix F.3
Ice Force	n/a	0	kN/m	Appendix F.4
Vertical Subgrade Reaction	0.27	0.27	N/mm^2	Appendix F.5
Wind Set-up	..	..	m	Appendix F.6

generated by vessel propellers are present. In addition, the convoy vessel is equipped with a large main propeller and bow thrusters. This is in contrast to the smaller yachts with small propellers and no bow thrusters. Despite the higher flow velocities from convey vessels, these will not sail close enough to provide flow velocity forces to the breakwater structure. Moreover, the assumed distance of the vessels to the structure will be approximately 50 m. Therefore, the flow velocities generated by yachts and sailing boats is more interesting to take into account. At a distance of 8 m and a water depth of 4 m, the flow velocities could increase to 0.4 m/s.

When wave impact is considered, it can be concluded that a wave height of 0.5 m will exert a pressure force to a vertical wall. It is expected that the water level increase is 0.07 m, the wave pressure at the impact zone is 4.9 N/mm^2 and the wave pressure at the bed is 0 N/mm^2. Discussing the subsoil, the subgrade reaction pressure is 0.27 N/mm^2 for heavy clay, which is for an allowed settlement of 50 mm. Lastly, the wind set-up is a function of 500 m of fetch length, a water depth bigger than 4 m and a design wind velocity of 30 m/s. This results in 0.014 m water level increase, which is about zero. Since it is of a far smaller magnitude than the water depth and wave height, this aspect is neglected.

Table 5.26: Wave-Structure Interaction of Dalemsche Geul Case Study

Design Consideration	Magnitude on Slope	Magnitude on Wall	Units	Reference
Wave Run-up	0.8	n/a	m	Appendix F.8
Wave Overtopping	n/a	n/a	m	Appendix F.9
Wave Transmission	0.3	0.3	m	Appendix F.11
Wave Reflection	n/a	n/a	m	Appendix F.12

The client did not provide any requirements about the allowed wave overtopping and wave reflection. Moreover, the breakwater should not be designed to give access to pedestrians or vehicle, according to the requirements. Therefore, only wave run-up and wave transmission are taken into account. The wave run-up computation of rubble mound with a slope angle of 34 deg and a wave height of 0.5 m results in 0.69 m requested freeboard, which is similar to the classified wave run-up of 0.8 m. In the phase of crest height and floating structure dimensions, the wave transmission should be considered. It is assumed that vessels are not obstructed by wave heights of 0.15 m. From the wave height of 0.5 m, the wave transmission coefficient becomes (0.15/0.5=) 0.3.

STEP 11: BREAKWATER DIMENSIONS

The material quantities and construction duration highly depend on the dimensions of the various breakwaters. A distinction is made between sloped, vertical and floating structures.

Sloped structure:

Crest height	= water depth + wind set-up + freeboard
	= 5.5 + 0 + 0.5 = 6.0 m
Freeboard	= wave run-up, wave transmission or wave overtopping

Vertical structure:

Crest height	= water depth + wind set-up + water level rise by waves + freeboard
	= 5.5 + 0 + 0.1 + 0.5 = 6.1 m
Freeboard	= wave transmission or wave overtopping

Floating structure:

dimensions ~ wave transmission = 0.3

According to the wave transmission coefficient of 0.3, the sloped rubble structure requires a freeboard of 0.5 m (when the crest width is 1 m). The vertical structure should have a freeboard of 0.5 m to obtain a wave transmission coefficient lower than 0.3.

For the breakwater several approaches are considered to determine the dimensions. Of influence are the wave height or wave length and wave transmission for floating structure. While sloped and wall structures should take into account horizontal forces of waves and ice. Frequently, the ice force is dominant, since it is significantly higher than the wave pressure. However, in this case ice will be out of scope. The load including the partial safety factors is:

Wave force = 3.4 kN/m

Also the bearing capacity of the subsoil should be verified by consideration of the vertical subgrade reaction. The magnitude of this pressure should be close to the exerted pressure on the subsoil or additional settlement will occur.

Subgrade reaction 270 kN/m^2

The classified boundary and design considerations are impacts on the breakwaters. The breakwaters are subsequently analysed paying strict attention to Appendix I. In this appendix the required dimensions and structural performance are considered. An overview of breakwater considerations and classes can be found in Appendix Q *Sub Flow Chart E: Load-Structure Interaction and Breakwater Classification*. Extensive tables with information about the breakwater performance are left out and referred to.

Rubble Mound Breakwater: 0.5 m of wave height leads to a nominal stone diameter of 241 mm, which corresponds with Rubble Mound 7. The critical flow velocity for this stone size is 1.07 m/s, which is not exceeded by flow velocities of 0.4 m/s from the main propeller of a Small Yacht. The crest height should be at least 6 m from the classification, which result is pressure onto the subsoil of 57 kN/m^2, which is below the allowed subgrade reaction.

Placed Block Breakwater: A design wave height of 0.5 m results in a block thickness of 0.15 m. This corresponds with Placed Block 1. Similar to the rubble mound, the heavy clay subsoil provides a stable base, but earthquakes of 50 cm/s^2 are a threat. Displacement of the blocks could lead to gaps and washout of filter material including core materials. The classified crest height has to be 6 m. The self weight provides a pressure on the subsoil of 75 kN/m^2, which is below the upper limit.

Sheet Pile Breakwater: The sheet pile breakwater will also have a crest height of 6 m. Due to the low wave force, both a SG-525 and AZ 14-700 can be applied.

Caisson Breakwater: The caisson finds a stable foundation with heavy clay but is sensitive to earthquakes of the considered magnitude. Caisson 3 provides a sufficient crest height of 6 m. This provides a bending moment and sliding resistance due to its selfweight, which can withstand the classified wave force. The allowed pressure on the subsoil is met with 58 kN/m^2.

Block Wall Breakwater: Heavy clay reduces the risks of deformation for the block wall. however, the earthquakes above 50 cm/s^2 could have a negative effect on the stability. To obtain a crest height of 6 m, Block Wall 3 is chosen. Since the wave force is limited, the smallest block suffices, which is Block 1. The created structure results in a vertical pressure on the subsoil of 170 kN/m^2, which lays in the order of a few centimetres of allowed settlements.

Floating Breakwater: For most water depths, the floating breakwater is applicable. To fulfil the requirement of a wave transmission of 0.3 and a wave height of 0.5 m, the most favourable and smallest floating breakwater is chosen. This is the Inter Boat Marina M3816 with a wave transmission of 28%.

Timber Pile Breakwater: To make a proper estimate of the wave transmission, the pile breakwater should get a crest height of 6 m, which results in Pile 3. The occurring tensile and compression stress in the outer fibre of the piles do not exceed the design stress. To obtain the required wave transmission, the piles should have a spacing of 0.15 m, which result in 27% wave transmission.

Tire Breakwater: The wave transmission of 30% can be achieved by choosing a 14 m wide tire breakwater, which is similar to Tire Breakwater 5.

Reef Ball Breakwater: The maximum water depth is 2 m for reef balls. A larger water depth will result in 100% wave and ice transmission.

Gabion Breakwater: Heavy clay is a proper foundation for gabion structures. To obtain a crest height of 6 m, Gabion 3 is chosen. The friction and bending moment of the upper cage suffices. Also the foundation pressure of 61 kN/m^2 is below the maximum.

Table 5.27: Breakwater Classes of Dalemsche Geul Case Study

Breakwater Alternative	**Classification**	**Performance**
Rubble Mound	Rubble Mound 7	
Placed Block	Placed Block 1	earthquake risk of deformation
Sheet Pile	SG-525 and AZ 14-700	
Caisson	Caisson 3	earthquake risk of deformation
Block Wall	Block Wall 3 and Block 1	earthquake risk of deformation
Floating	Inter Boat Marina M3816	
timber pile	Pile 3	
Tire	Tire Breakwater 5	
Reef Ball	-	
Gabion	Gabion 3	
Screen	Screen Breakwater 3 (TP219 and IPE 120)	
Geotube	~~Geotube 5~~	maximum height is exceed by water depth
Synthetic	WaveBrake 5	

Screen Breakwater: Classified Screen Breakwater 3 has a crest height of 6 m. The wave transmission requirement is met by taking 7.5 % porosity. A bending moment of 15 kNm/m is found for the tubular piles. This results in profile TP219. Moreover, the support beam, which is subjected to a bending moment of 11 kNm/m, becomes an IPE120.

Geotube Breakwater: The water depth exceeds the maximum crest height, which result in 100% wave transmission. This is not allowed, which will apostatize this alternative unless the crest height is increases. The performance of sliding and bending moment of the largest geotubes breakwater, Geotube 5, is approved. Moreover, the vertical pressure at the subsoil is approximately 115 kN/m^2, which is below the maximum stress.

Synthetic Breakwater: The WaveBrake 5 shows 15% wave transmission in case of wave heights of 0.4 m. It is expected that 0.5 m would result in a wave transmission between 20 and 40 %. Therefore, this breakwater alternative is declared feasible.

The optional breakwater alternatives obtained their dimensions and are studied on their structural performance. This leads to dissuasion of various breakwaters when their capacities are exceeded. The chosen breakwater classes and remarks on their performance are presented in Table 5.27.

STEP 12: MATERIAL COST ESTIMATE

The breakwaters obtain a material costs by paying strict attention to Appendix I and by following the chosen breakwater classes from Table 5.27. It is optional to extract the costs from Appendix R *Sub Flow Chart F: Material, Labour and Equipment Costs.*

Table 5.28: Breakwater Material Costs of Dalemsche Geul Case Study

Breakwater Alternative	**Value**	**Units**	**Remarks**
Rubble Mound	2560	€/m	
Placed Block	3047	€/m	(=685+78+20+1171)
Sheet Pile (SG-525)	715	€/m	
Sheet Pile (AZ14-700)	1554	€/m	
Caisson	7902	€/m	
Block Wall	1628	€/m	
Floating	637	€/m	
timber pile	1535	€/m	
Tire	58	€/m	
~~Reef Ball~~	-	€/m	
Gabion	2170	€/m	
Screen	4737	€/m	(=3896+830+11)
~~Geotube~~	>1347	€/m	
Synthetic	1292	€/m	

The type, the amount and the material costs are combined per chosen breakwater class. The computed costs per stretched meter is shown in Table 5.28. The values are indications and approximations. Thus, these could vary per company and period of time.

STEP 13: LABOUR AND EQUIPMENT COST ESTIMATE

The breakwaters obtain a labour and equipment costs by paying strict attention Appendix I and by following the chosen breakwater classes from Table 5.12. It is optional to extract the costs from Appendix R *Sub Flow Chart F: Material, Labour and Equipment Costs.*

The amounts of material, the size of structural elements and the required equipment are combined per chosen breakwater class. The direct labour and equipment costs are computed, excluding mobilisation and administration costs. The results are shown in Table 5.29. Indications and approximations of the material costs are implemented. Thus, these could vary per company and period of time.

STEP 14: EMVI DISCOUNT

In Step 3 the EMVI criteria were discussed. These consist of System Quality, Ecological Impact, Noise and Risks. The maximum discount per item is respectively €65,000, €150,000, €110,000) and €75,000.

The criteria will be scored from 1 to 5, which are respectively the lowest and highest values. In Appendix S *Sub Flow Chart G: EMVI Score and Selection Matrix* a quantification based on engineering judgement is presumed, which is applied to this case study. The predicted discounts per breakwater alternative are shown

Table 5.29: Breakwater Labour and Equipment Costs of Dalemsche Geul Case Study

Breakwater Alternative	**Value**	**Units**	**Remarks**
Rubble Mound	177	€/m	
Placed Block	3388	€/m	
Sheet Pile	103	€/m	
Caisson	584	€/m	
Block Wall	601	€/m	
Floating	165	€/m	
timber pile	129	€/m	
Tire	1037	€/m	
~~Reef Ball~~	-	€/m	
Gabion	2033	€/m	
Screen	1092	€/m	
~~Geotube~~	>59	€/m	
Synthetic	84	€/m	

in Table 5.30. To obtain the cost reduction per stretched meter, the total discount is divided by 200, which is the length of the breakwater.

Table 5.30: EMVI Score Matrix of Dalemsche Geul Case Study

	RUBBLE MOUND			PLACED BLOCK			SHEET PILE		
	Score	Perc.	Disc.	Score	Perc.	Disc.	Score	Perc.	Disc.
System Quality	4	0.8	52000	3	0.6	39000	3	0.6	39000
Ecological Impact	4	0.8	120000	3	0.6	90000	3	0.6	90000
Noise	3	0.6	66000	3	0.6	66000	1	0.2	22000
Risks	4	0.8	60000	3	0.6	45000	3	0.6	45000
		Total (€)	298000		Total (€)	240000		Total (€)	196000
		Total (€/m)	1490		Total (€/m)	1200		Total (€/m)	980

	CAISSON			BLOCK WALL			FLOATING		
	Score	Perc.	Disc.	Score	Perc.	Disc.	Score	Perc.	Disc.
System Quality	5	1	65000	2	0.4	26000	4	0.8	52000
Ecological Impact	2	0.4	60000	2	0.4	60000	4	0.8	120000
Noise	3	0.6	66000	3	0.6	66000	3	0.6	66000
Risks	5	1	75000	2	0.4	30000	4	0.8	60000
		Total (€)	266000		Total (€)	182000		Total (€)	298000
		Total (€/m)	1330		Total (€/m)	910		Total (€/m)	1490

	TIMBER PILE			TIRE			GABION		
	Score	Perc.	Disc.	Score	Perc.	Disc.	Score	Perc.	Disc.
System Quality	5	1	65000	5	1	65000	4	0.8	52000
Ecological Impact	2	0.4	60000	1	0.2	30000	4	0.8	120000
Noise	3	0.6	66000	3	0.6	66000	3	0.6	66000
Risks	5	1	75000	5	1	75000	4	0.8	60000
		Total (€)	266000		Total (€)	236000		Total (€)	298000
		Total (€/m)	1330		Total (€/m)	1180		Total (€/m)	1490

	SCREEN			GEOTUBE			SYNTHETIC		
	Score	Perc.	Disc.	Score	Perc.	Disc.	Score	Perc.	Disc.
System Quality	3	0.6	39000	2	0.4	26000	5	1	65000
Ecological Impact	2	0.4	60000	2	0.4	60000	4	0.8	120000
Noise	3	0.6	66000	3	0.6	66000	4	0.8	88000
Risks	3	0.6	45000	2	0.4	30000	5	1	75000
		Total (€)	210000		Total (€)	182000		Total (€)	348000
		Total (€/m)	1050		Total (€/m)	910		Total (€/m)	1740

The synthetic breakwater gained the highest score to an assumed system quality and risk. A high system quality comes from the fact that most plastics are long lasting and repair works are easily performed. The risk of damage or a failing structure is presumed low due to the flexible lightweight structure, which cooperates with the direction of a force. Also the rubble mound, caisson, placed block, timber pile and gabion have a relatively high score, which is due to a combination of EMVI criteria. Due to pile driving, the sheet pile and timber pile score low at the ecological impact and noise. These are a few conclusions, which can be disapproved by experts. A designer should choose the preferred score per alternative.

STEP 15: COST-BASED SELECTION

In the previous sections, the structural performance, material, labour and equipment costs are discussed. The structural performance is not cost-based, but is essential to determine the functionality of the structure or to expect additional cost to improve the structure. The direct costs and EMVI discount are shown in Table 5.3.2. When breakwaters fulfil the requirement, the green shape is used. The red shape refers to breakwaters which fail under the considered load.

Hydraulic fulfilment is in conflict due to the design earthquake and design water depth. Placed block, caisson and block wall are assumed to be not applicable in an environment with large horizontal accelerations due to earthquakes. Moreover, the reef ball and geotubes breakwater can not cope with the large water depth in the Dalemsche Geul.

Table 5.31: Cost-based Selection Scheme of Dalemsche Geul Case Study

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER
I6	HYDRAULIC FULFILMENT							
I7	COSTS OF MATERIAL (€/m)	2560	3047	715 / 1554	7902	1628	637	1535
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	177	3388	103	584	601	165	129
I9	EMVI DISCOUNT (€/m)	-1490	-1200	-980	-1330	-910	-1490	-1330
I10	TOTAL (€/m)	1300	5300	-200 / 700	7200	1300	-700	400

CODE	ITEM	TIRE	REEF BALL	GABION	SCREEN	GEOTUBES	SYNTHETIC
I6	HYDRAULIC FULFILMENT						
I7	COSTS OF MATERIAL (€/m)	58	N/A	2170	4737	>1347	1292
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	1037	N/A	2033	1092	>59	84
I9	EMVI DISCOUNT (€/m)	-1180	N/A	-1490	-1›050	-910	-1740
I10	TOTAL (€/m)	-100	N/A	2700	4800	>500	-400

When costs are considered, it can be observed that the floating breakwater, tire breakwater and synthetic breakwater and synthetic sheet pile are interesting (Figure 5.3). The negative values mean that the EMVI discount is larger than the summation of the material, labour and equipment costs. Moreover, when the total costs are calculated, one will find that the EMVI discount is a small percentage. So, in this matrix the negative values reveal the breakwater with the lowest direct costs only. Less preferable breakwaters cost-wise are the placed block, caisson, gabion and screen breakwater. Due to the large water depth, these structures increase in width and quantity of the material which results in high costs. The rubble mound is relatively cheap since the placement costs are low which is similar to the sheet piles.

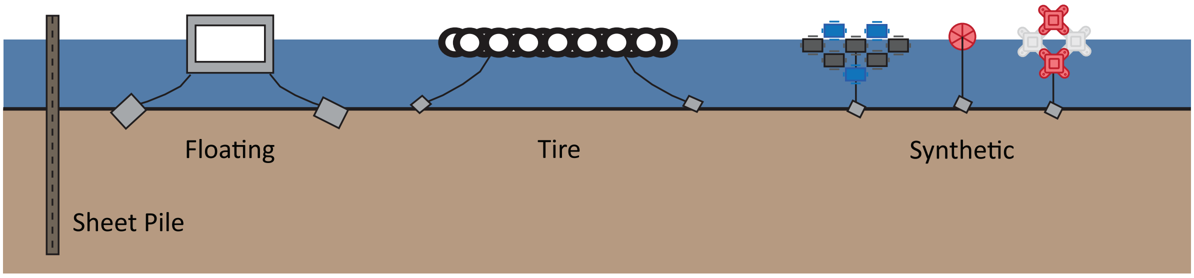


Figure 5.3: Chosen Breakwater Alternatives after Cost-Based Selection of Dalemsche Geul Case Study

The choice of a breakwater alternative also depends on the experience in designing and construction. When a structure is not familiar to a contractor, the design and construction phase could consume both extra time and money. In particular, engineering companies might need to be consulted to perform the design, which provides additional costs. What is more, when issues occur during construction, the process of solving it could take time. Therefore, it is important to consider the strength of the contractor. This implies that the labour and equipment costs consist of the works and risk.

STEP 16: DECISION

Helpful tools are the selection tables from the pre-selection and cost-based selection. In the pre-selection, the floating, tire and synthetic breakwater are declared feasible alternatives (Table 5.3.2). These fulfil the functional requirements and are recommended when the permits of excavation and surroundings are considered. It is expected that the polluted subsoil and nearby habitat area will obstruct obtaining the required permits. The D&C contract should refer to the experience of the contractor in designing and construction a type of breakwater. The three pre-selected breakwater alternatives are presumed to be less familiar to the average contractor.

In the cost-based selection, the pre-selected breakwaters show opportunities. Moreover, the synthetic sheet pile breakwater is also a potential breakwater alternative. Despite of this, this breakwater is not movable, and therefore out of scope. Since the last comparison between the floating, tire and synthetic breakwater can only be based on the costs, it is recommended to further design the floating breakwater (Table 5.3.2). When the material, labour and equipment costs are with certainty determined, this breakwater alternative has the lowest costs. However, the EMVI discount is higher for the synthetic breakwater. On the other hand, it should be said that there is a high uncertainty in the predicted discount.

5.4. REFLECTION ON CASE STUDY 2

This section discusses the selected breakwater and governing aspects of the DSM.

5.4.1. SELECTED BREAKWATER

The Dalembergsche Geul case study is characterised by different circumstances compared to the IJburg case study. The location is a sheltered area next to a river with a small fetch length. This resulted in a limited wave height, which is interesting for small floating structures. Subsequently, due to the request for a temporary structure in a habitat area with contaminated subsoil, it was expected that a floating type of breakwater would be selected. The governing variables which led to the choice are discussed in the following section.

5.4.2. GOVERNING ASPECTS

There are three essential requirements from the case study, namely the structure: is entirely removable, can cope with vessel collision and is movable. For all requirements, the floating, tire and synthetic breakwater are interesting.

The D&B contract requires the experience of a contractor in the design and construction phase. Since floating type of breakwaters are non-familiar structures to contractors, it depends on each contractor to declare them preferable.

Important legislation concerns excavation activities. The contaminated subsoil will cause problems for the water quality and habitat area. Therefore, breakwaters with excavation activities can cause permits (Omgevingsvergunning (Permit of Surroundings)) not to be granted.

The boundary conditions are not dominated by ice, but by a wave height of 0.5 m and a water depth of 5.5 m. Smaller waves also imply smaller horizontal loads. As a result, most breakwaters are found stable in the hydraulic fulfilment.

The cost estimate is interfered by the EMVI discount, which is large in comparison with the direct costs. The case study reveals that the discount can have a significant effect on the costs.

6
Conclusions

In this chapter conclusions are drawn according to the research questions and a discussion on the hypotheses is provided.

6.1. Research Questions

In this section the findings and conclusions are discussed based on the main research question and sub-questions.

What is an efficient method for the selection of the most effective breakwater alternative in Dutch inland waterways?

A decision support method (DSM) has been developed to consider breakwater alternatives in a factual, systemic, comprehensive and quick matter. The method takes into account the: requirements, contracts, legislations, boundary conditions, structural performances and cost estimates including EMVI discounts (Figure 6.1).

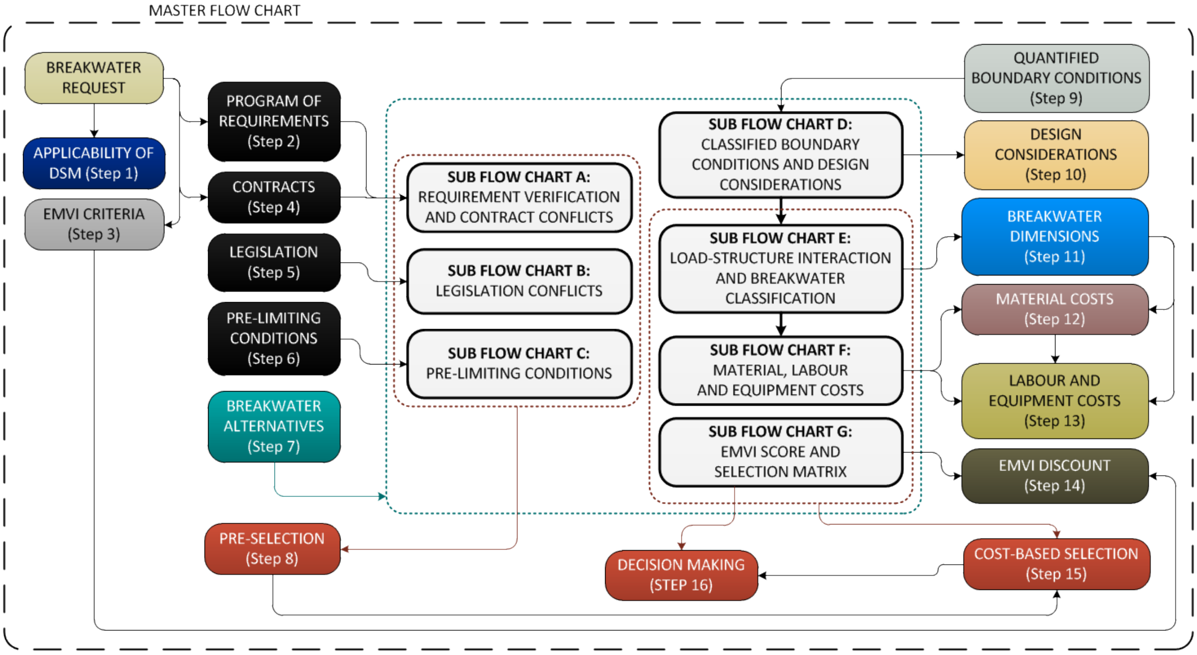


Figure 6.1: Overview of Decision Support Method

The DSM deviates from cost-based methods or trade-off matrices which only consider requirements and/or costs. So, it can be concluded that the DSM provides a more comprehensive assessment as it also includes EMVI criteria, contracts, permits, laws and regulations. Moreover, in the trade-off matrices the requirements have subjective weighing factors (e.g., 1, 2, 5) and scores (for instance, ++, 0, -). These are avoided in the method to be assured of a more factual decision. However, based on experiences of experts, subjectivity of engineering judgement remains. Therefore, the DSM supports a more well-justified selection for the most effective breakwater.

The method is divided into a pre-selection (Step 8) and a cost-based selection (Step 15). After the pre-selection a limited number of breakwater alternatives are dimensioned and estimated in costs. Accordingly, in Step 11, 12 and 13 this saves engineering time. What is more, selection tables are provided in the selection phases, which contain the interaction between a breakwater and selection criteria, and the cost estimates. Subsequently, a designer is able to rapidly oversee the conflicts and costs per breakwater alternative to achieve the required insight. This process guaranties an efficient method.

What are suitable breakwater alternatives when the requirements, criteria, type of contract and tender approach are provided?

The composition always changes of the pre-selected breakwater alternatives after the consideration of the requirements (functional, operational and technical), clients criteria and contracts (D&B and DB&M). In the pre-selection, the interaction between these aspects and the breakwater alternatives is instantaneously observable. This interaction can either apostatize or approve a breakwater alternative. Apart from this, the tender approach is focused on an EMVI and LP registration, because BVP has criteria with regard to the companies organisation and CO2 Performance is included as an EMVI criterion. Furthermore, EMVI prescribes criteria to obtain a fictitious discount. As a result, EMVI is considered in the cost estimates.

From two case studies the governing aspects are found in the functional requirements and the laws, permits and regulations. In case study 1, a group of dominant functional requirements are that the breakwater should allow mooring opportunities and should not damage vessels. Based on these, the screen, sheet pile, block wall and caisson breakwater are pre-selected. Case study 2 showed important requirements which are that the structure should be entirely removable and movable. Since the subsoil was polluted and a habitat was near, a conflict with the Omgevingsvergunning (Permit of Surroundings) is probable. Consequently, the floating, tire and synthetic breakwater were the most promising breakwater alternatives. The DB&M and D&B contracts in respectively case 1 and 2 affect the breakwater selection in lesser extent, because this depends on the strength of a contractor in the design, construction and maintenance. It is expected that this aspect will play an important role in practise.

How to consider the governing boundary and initial conditions in order to reject breakwater alternatives?

The pre-limiting conditions support insight into the governing boundary conditions (for example, high waves, high water depth and ice). These are assumed by experts based on engineering judgement which supports the elimination of breakwaters in the pre-selection. Subsequently, the boundary conditions are classified which means that variables are enclosed in classes. For example, the considered ice thickness are 0.1, 0.2, 0.3 and 0.4 m. These ranges of the various boundary conditions are limited due to the inland waterway conditions. Furthermore, the classified boundary conditions do not reject a breakwater, but are required for the dimensions. These dimensions provide cost estimates on which the breakwaters are finally rejected or preserved.

Due to various descriptions in the method, known and unknown boundary conditions are easily incorporated. The classifications make the method quick and simple since all the possible results are provided. Therefore, there is no need for calculation sheets which is an advantage compared to other methods without guidelines. Apart from this, two cases have been elaborated which provide insight in governing aspects. The dominant variables in case study 1 are the significant wave height (1 m), the ice thickness (0.2 m) and the subsoil (Light Clay). When this combination is considered, ice determines the strength, waves determine the dimensions and subsoil determines the stability of the structure. In contrast, case study 2 shows a different set of governing variables, namely: a large water depth (5.5 m), stiff subsoil

(Heavy Clay) and small significant wave height (0.5 m). Subsequently, the water depth plus waves determine the dimensions, and the waves are the only load considered. The force exerted by the waves is low compared to ice.

How are the breakwater alternatives and characteristic approaches adapted to the classified design conditions?

The boundary conditions and design considerations are classified and provide the lower and upper limits of the breakwater dimensions. Subsequently, dimension classes per breakwater alternative are provided to make a quick comparison. More important, the classifications enable computations with limited results which are easily read by designers. Owing to this, a significant amount of engineering time is saved which contributes to efficiency of the DSM.

The breakwater dimensions are roughly classified which devalue the accuracy of the cost estimates. For example the crest height of a block wall is 2, 4, 6, 8 and 10 m. On the other hand, the order of magnitude of the dimensions and the cost estimates are comparable between breakwater alternatives. Also the material, labour and equipment costs can be provided per classified breakwater. Extensive calculation sheets are once more circumvented. This shows the efficiency of the method.

How do the equipment and construction methods contribute in the selection?

The equipment and construction methods provide the labour and equipment costs. These costs are part of the cost-based selection. Since land-based equipment cannot be carried by small-scale breakwaters, only water-borne equipment is considered. As a result, the typical equipment and construction procedures are fixed per breakwater alternative. The DSM provides first insight in construction and placement costs.

The time of construction varies per breakwater alternative, which result in varying cost estimates. These costs include only the labour and equipment for placement of the breakwater. This means that the transport and mobilisation costs are not taken into consideration, because these are not generically difficult to predict per breakwater alternative. From the case studies it can be concluded that the labour and equipment costs and material costs contribute both significantly. Apart from this, the construction aspects are also taken into account in the design and build contracts. In this case, the contractor will verify the available equipment and his experience in construction of a certain breakwater alternative.

What is the most favourable breakwater alternative in terms of meeting requirements and costs?

This is the breakwater alternative which at least meets the functional requirements and is the lowest in the combination of the direct costs and EMVI discount. The DSM also considers the impact of the tender approaches, contracts and legislations. This well-justifies a certain breakwater being more favourable. Moreover, while other methods take only requirements and/or costs into account, the DSM considers both to ensure the most effective structure.

From the case studies it is concluded that a list of requirements and cost estimations play a significant role in the breakwater selection. For instance, in case study 1 the sheet pile breakwater was chosen for the reason that it was able to fulfil the functional requirements (the structure enables mooring opportunities, should not damage vessels and enables walk-ability) and it was the lowest in direct costs. Another example is case study 2 where the functional requirements (structure should be entirely removable, able to cope with vessel collision and movable) were fulfilled and the direct costs, including EMVI discount, resulted in the lowest price for the floating breakwater. This example showed that it is important to include polluted subsoil to guaranty no obstruction in construction.

A special feature is the EMVI tender approach to which the DSM can respond. The method is designed in such a way that EMVI criteria can be recognised and assessed in monetary terms. Despite, the BVP tender approach is also supported, because the applied knowledge in and professional application of the DSM can be in favour of BVP criteria.

6.2. DISCUSSION ON HYPOTHESES

In an earlier phase, the expectations of the research results have been formulated in three hypotheses. In this section these hypotheses are discussed.

It is expected that the DSM will result in a different breakwater alternative than the chosen IJburg breakwater, which consists of steel sheet piles.

The DSM resulted also in the steel sheet pile breakwater as the most effective breakwater alternative. In the pre-selection, the requirements about enabling mooring opportunities and no damage to vessels rejected eight out of thirteen breakwaters, which included the sheet pile breakwater. Yet, it is known that steel structures are relatively expensive due to the high material costs. This was the main reason to assume that the sheet pile breakwater would not be chosen. In other words, dumping rubble mound would be quicker and cheaper. However, due to the slender sheet piles, material quantities are limited compared to the mound or monolithic breakwaters. This resulted in lower direct costs for the sheet pile breakwater in the cost-based selection.

It is expected that the choice of a breakwater alternative will be different when a cost estimate is provided besides the requirements.

The breakwater at IJburg was initially based on the requirements only and resulted in the sheet pile breakwater. In the DSM this alternative was also selected in the pre-selection, apart from the caisson, block wall and screen breakwater. Subsequently, an estimation of the material, labour and equipment costs for all breakwater shows a shift in the selection. This is because the rubble mound, placed block and geotube breakwater provide the lowest direct costs in the cost-based selection. Thus, it can be concluded that the selection based on the functional requirements leads to a different selection than based on the costs.

It is expected that compared with previous performed considerations of breakwater alternatives, the DSM provides the same results in less time.

The method is designed to make quick analyses. This is achieved by prescribing and predicting the positive or negative interaction of the factors of influence to a breakwater alternative. Hence, an extensive number of influences are given to directly conclude preferable, non-preferable and dependent breakwaters. What is more, the design steps about engineering are supported by tables containing the results of relevant design formulas. Thus, the results have to be chosen which prevents doing research to obtain the correct design formulas and to circumvent large spreadsheets to be developed with yet unknown input. Moreover, the computations are to be overseen due to the limited classifications of the boundary conditions, design considerations and breakwater dimensions. Therefore, it can be expected that the developed method and guideline saves time. From experience, it can be said that the DSM provides a selection within days. In contrast, without the method and taking into account the same aspects the expected time for tendering is between two and three weeks. The additional time is mainly an result of the involvement of multiple disciplines. For instance, these are calculators, designers/engineers and quality, working conditions & environment.

7 RECOMMENDATIONS

This chapter discusses the recommendations for the implementation and improvement of the DSM.

An comprehensive method has been developed with multiple selection criteria. However, it is possible that aspects are currently lacking which can discard or involve certain breakwater alternatives. It is recommended to add these to the DSM.

Likewise, it is recommended to include as many breakwater alternatives to obtain a broader selection procedure. In fact, the current breakwaters can have alternatives themselves, which are developed by adjustments. These can be improvements as well to meet the requirements. For example, a horizontal concrete plate could enable pedestrians on a rubble mound and a separate mooring structure enables mooring facilities at a placed block breakwater.

For significant improvements of the breakwater dimensions and structural assessment, it is proposed to increase the number of classified variables and apply consistent spacing to prevent significant over- and under-estimation. Thus, instead of wave heights of 0.25, 0.3, 0.5 and 1 m, it is more useful to apply 0.2, 0.3, 0.4, ..., 1 m.

Apart from this, insignificant values of boundary conditions and design considerations can be left out. For example, wave heights of 0.1 m and flow velocities up to 0.2 m/s can be neglected as loads. This should be supported by a comparison study of the impact on the various breakwaters.

The engineering approach has the disadvantage that the load always exerts at the highest point of the breakwaters, which is the most unfavourable scenario. This is to achieve a stable structure for unknown water depths and position of the loads. It is advised to incorporate the exact position of the design water level, where the actual point of loading is.

The direct cost estimate may be extended by the costs of mobilisation, transport and manufacture. Yet, these are not taken into account, because these have many unknown variables (e.g. travel distance) and are time-dependent. However, this would be a supplement to the provided material, labour and equipment costs. Moreover, a risk margin and maintenance costs could vary per breakwater. The prediction per breakwater alternative requests additional research.

It is advised to consult experts to consider the EMVI criteria and relevant legislation. For example, ecologist and architects can be helpful to discuss the EMVI score, while juridical experts could provide a clear understanding of the laws, permits and regulations.

It is also recommended to consult experts in the field of hydraulic engineering to review the DSM. Besides this, testing the method with other case studies would provide more insight in the governing variables. With more cases, the essential variables can be recognized.

The DSM is not designed for river groynes. These have the function to guide a river and enable sailing of container vessels. Since these structures are found in rivers, the processes on flow development should be considered. This is in contrast with breakwaters which concern wave impact. Therefore, it is proposed to develop a separate document about the alternatives of river groynes and the selection phase.

The breakwaters could have more categorised dimensions which increases the accuracy of the cost estimations. For the reason that this will result in a significant amount of data in a written DSM, one could develop a decision support system (DSS). This is an interactive software tool, which enables designers to implement the method in an user-friendly environment. An expectation is that the DSM will be quicker implemented.

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UNITS

Unit	Property
Bft	$Beaufort$
J	$N\dot{m}$
kg	$kilogram$
m	$meter$
N	$kg\dot{m}/s^2$
Pa	N/m^2
s	$second$
W	J/s
€	$euro$

ABECEDARY

Symbol	Unit	Property
a	m	Vertical wave motion
a_{em}	–	Empirical coefficient in Young and Verhagen
A''	–	Vessel geometry coefficient
$A1$	–	Variable of formula in Young and Verhagen
b_{em}	–	Empirical coefficient in Young and Verhagen
B	m	Maximum beam width
$B2$	–	Variable of formula in Young and Verhagen
c	m/s	Wave velocity
	N/mm^2	Cohesion of soil (drained soil)
c_h	N/m^3	Coefficient of Hudson
C	$m^{1/2}/s$	Chézy coefficient of roughness
C_c	–	Compression index
C_D	–	Drag coefficient
C_s	–	Swelling index
c_v	–	Consolidation coefficient
C_k	$m^{1/2}/s$	Parameter of roughness
d	m	Water depth
d_i	m	Thickness of the ice layer
d_p	m	Propeller diameter
d_{50}	m	The medium diameter in which 50% of the mixture is finer
d_{90}	m	The medium diameter in which 90% of the mixture is finer
D	m	Draught of vessel
	m	Grain classification/size
	m	Distance between sea bed level en minimum water level
E	m^2	Total energy
	N/m^2	Modulus of elasticity
f_p	Hz	Peak wave frequency
F	N	Force of the ice layer
g	m/s^2	Acceleration of gravity
h	m	Original water level
h_c	m	Crest height
h_{gd}	m	Guaranteed depth
h_{net}	m	Keel clearance
H	m	Wave height (regular waves)
H_b	m	Maximum wave height at breaker line
F_{HB}	N	Breaking load of the ice layer
F_{HP}	N	Push load where the sheet ice moves through the ice rubble
F_{HR}	N	Push load where the ice blocks moves through the ice rubble up a slope
F_{HL}	N	Lift load where the ice rubble is lifted
F_{HT}	N	Turn load where the block at the top is turned
Fr	–	Froude number
G	N	Stone weight where the highest load occurs
h_{bb}	m	Distance from bed to centre of bow thruster
H_{m0}	m	Significant wave height, zero-th moment of the total energy spectrum
H_{rms}	m	Root-mean-square wave height
H_r	m	Wave height of the reflected wave
H_t	m	Wave height of the transmitted wave
H_i	m	Wave height of the incoming wave height
$H_{1/3}$	m	Significant wave height, mean of 1/3 of the highest waves
i	–	Water surface gradient
k	–	Permeability

k_d	–	Coefficient of the stability
K_d	–	Coefficient of diffraction
K_r	–	Coefficient of reflection
K_t	–	Coefficient of transmission
K_w	–	Saunders coefficient
l_c	–	Strength parameter of the ice layer
L	m	General notation for the wave length
L_e	m	Vessel length
L_{sea}	m	Wave length of locally generated waves
L_{swell}	m	Wave length of swell
L_0	m	Deep-water wave length
M	kNm	Bending moment
M_{Ed}	kNm	Design bending moment
M_E	kNm	Design rotation
M_{Rd}	kNm	bending moment resistance
M_R	kNm	Rotational resistance
n	–	Relative density/porosity
	–	Number of measurements
	–	Number of propellers
	–	Variable as function on the dimensionless depth
N	–	Number of measurements
	N	Normal force
P	m	Water level difference
	kW	Power of the engine
$P_{thruster}$	kW	Power of the bow thruster
Q	m^s/s	Discharge
q	$m/s^3/m$	Unit discharge
R	m	Hydraulic radius
R	–	Resistance parameter in probabilistic computations
R_c	m	Freeboard
$R_{u2\%}$	m	Wave run-up exceeded by 2% of all waves
S	–	Load parameter in probabilistic computations
S_{max}	m	Maximum sinkage
T	s	Wave period
	T	Tonnage (1000 kg)
T_p	s	Peak wave period, maximum wave period in an arbitrary energy spectrum
u	m	Average flow velocity
u_b	m/s	Flow velocity at the bed
u_{bb}	m/s	Flow velocities from bow thruster
u_{b-max}	m/s	Maximum flow velocity at the bed
u_c	m/s	Critical flow velocity
u_0	m/s	Flow velocity at the propeller/bow thruster
s_u	N/mm^2	Undrained shear strength
u_{10}	m/s	Wind speed at 10 m above water surface
v_s	m/s	Flow velocity of the water
v_v	m/s	Sailing speed of the vessel
W	m^3	Displacement volume
	m	Height of the structure, distance between sea bed level to crest level
x_{eff}	m	Effective length of fetch
x	m	Length of fetch
x_i	m	Distance to land
x^*	–	Dimensionless distance sailing to wave measurement location (–)
X	–	Dimensionless fetch
z	m	Water level depression
z_b	m	Keel clearance
Z_c	m	Distance between crest level and maximum water level

GREEK

Symbol	Unit	Property
α	rad	Angle of the sloped structure
α_{Fr}	$-$	variable depending on the Froude number
γ	kN/m^3	General notation for the density
γ_{br}	$-$	Breaker index
γ_b	$-$	Reduction factor for influence of the berm
γ_f	$-$	Reduction factor for armour layer roughness
γ_i	$degrees$	Angle of deviation
γ_β	$-$	Reduction factor for obliqueness of the waves
δ	m	Water depth
ϵ	$-$	Dimensionless total energy
$\triangle d$	m	Size of the elements used
η_w	m	Water level increase
η_{max}	m	Wave set-up
θ	rad	Directional spreading
ϕ	rad	Angle of internal friction (drained soil)
ρ	kg/m^3	Density
ρ_{air}	kg/m^3	Density of air
ρ_r	kg/m^3	Density of rock
ρ_{sat}	kg/m^3	Density of wet soil
ρ_{unsat}	kg/m^3	Density of dry soil
ρ_w	kg/m^3	Density of water
σ	N/mm^2	In situ stress
σ_d	N/mm^2	Design stress
σ_f	N/mm^2	Flexural strength of the ice layer
ξ	$-$	Parameter of the slope
ξ_p	$-$	Iribarren number

9

LIST OF ACRONYMS

Abbreviation	Definition
BIM	Building Information Model
BVP	Best Value Procurement
CD	Conceptual Design
D&B	Design & Build
DB&M	Design, Build & Maintenance
DBF&M	Design, Build, Finance & Maintenance
DBFM&O	Design, Build, Finance, Maintenance & Operate
D&C	Design & Construct
DD	Detailed Design
DSS	Decision Support System
DSM	Decision Support Method
E&C	Engineering & Construct
EMS	European Marco-seismic Scale
EMVI	Most Economical Registration
FoS	Factor of Safety
JONSWAP	Joint North Sea Wave Observation Project
KRW	European Water Directive
LLC	Life Cycle Costs
LP	Lowest Price
MCA	Multi-criteria Analysis
NAP	Normalised Amsterdams Level
NLL	Nominal Lower Limit
NUL	Nominal Upper Limit
OCR	Over Consolidation Ratio
PD	Preliminary Design
PPS	Public Private Cooperation
RAW	Consent Registration Works
SLS	Serviceability Limit State
SSK	Standard Method of Cost Management
ULS	Ultimate Limit State

10

LIST OF TERMS

Term	**Dutch Translation**
Best Value Procurement	Prestatie-inkoop
Coefficient of planishment	Uitvlakkingscoefficient
Concept Design	Conceptueel Ontwerp
Consent Registration Works	Regeling Aanbesteding Werken
Decision Support Method	Beslissingsondersteunende Methode
Decision Support System	Beslissingsondersteunend Systeem
Dilation waterways	Ontsluitingswateren
Directorate-General for Public Works and Water Management	Rijkswaterstaat
Most Economical Registration	Economische Meest Voordelige Inschrijving
European Water Directive	Kaderrichtlijn Water
Detailed Design	Definitief Ontwerp
Lowest Price	(Gunnen op) Laagste Prijs
Local water managers	Waterbeheerders
New Level of Amsterdam	Normaal Amsterdams Peil
Preliminary Design	Voorlopig Ontwerp
Public Private Cooperation	Publiek Private Samenwerking
Tender Guide	Inschrijvingsleidraad
Transitional waterways	Verbindingswateren

APPENDICES

A

Standard Requirements

The tender phase of a civil project is subjected to requirements. These are the functional, performance and technical requirements. Firstly, these will be outlined to the type of tenders. Secondly, lists of typical requirements will be shown and discussed.

A.1. Tender Dependency

The tender guideline will constrain the designer and builder by requirements. The principles will also affect the types of requirements provided by the client Table A.1. The principle BVP is provided with qualitative requirements excluding number, amounts and values. Whereas, the EMVI, CO2 Performance Ladder and LP approaching are provided with all types of requirements.

Table A.1: Relation between Requirements and Contract Approaches

Type of Law	**BVP**	**EMVI**	**CO2 Performance Ladder**	**LP**
Functional Requirements	x	x	x	x
Performance Requirements		x	x	x
Technical Requirements		x	x	x
Stakeholder Conditions	x	x	x	x

A.2. Typical Functional Requirements

The functional requirements are spatial and user requirements. The description consists of the future function of a structure. Systems Engineering translates the functional requirements into top-level functions.

One can observe that the requirements about ecology and water quality are to be found in the Kader Richtlijn Water (KRW) and the Waterwet. In case of ecology, fine sediments and sound pollution should be taken into account for birds, fish and shellfish.

Typical functional requirements defined by the client are provided in Table A.2. The structure:

Table A.2: Typical Functional Requirements

Code	Description
F1	reduces wave action to enhance shipping.
F2	reduces wave action to enhance safe recreation.
F3	reduces wave action to reduce erosion.
F4	reduces flow velocity to reduce erosion.
F5	should not support sedimentation in the ship channel.
F6	shall be observable for navigation at all times.
F7	shall not affect the view of the landscape.
F8	shall be adjusted to the environment.
F9	is able to be extended or to be shortened without removal of the expired structure.
F10	is not blocking the ship channel.
F11	shall enable mooring opportunities.
F12	shall not damage vessels.
F13	shall enhance the walk-ability.
F14	shall provide facilities to stay overnight.
F15	shall not affect or interrupt the aquatic ecology.
F16	should be maintenance-free.
F17	should have a sufficiently long lifetime.
F18	should not diminish the water quality.
F19	is entirely removable after the expiration date.
F20	it should not hinder shipping, nearby traffic and local residents during its construction.
F21	should have the lowest investment costs.
F22	should have low life cycle costs.
F23	should support the requested aesthetics.
F24	should take limited space.
F25	should fit the local spatial shapes.
F26	has partial reflection of waves.
F27	has no transmission of waves.

The typical functional requirement can be used. It is important to determine whether the individual requirement is negotiable or mandatory.

A.3. TYPICAL PERFORMANCE REQUIREMENT

In contrast to the functional requirements, the performance requirements are focused on certain properties, which have ambiguously measured or calculated tolerances. These requirements are defined as specifications to the functional requirements. Typical performance requirements defined by the client are provided in Table A.3. The structure:

Table A.3: Typical Operational Requirements

Code	Related Requirement	Description
O1	F1/F2/F3	reduces the significant wave height to 0.30 m.
O2	F1/F2/F3	reduces wind-generated waves to waves occurring at 4 Bft.
O3	F1/F2/F3	allows wave overtopping of 10 l/s/m.
O4	F16	has a lifetime of 50 years.

The typical performance requirement can be used. It is of importance to determine whether the individual requirement is negotiable or mandatory.

A.4. TYPICAL TECHNICAL REQUIREMENTS

Technical requirements originate from the functional and/or performance requirements. These describe parts of the structure in values, quantities or provide technical information. In most breakwater designs, designers are not mandated to take into account certain requirements, but in fact they determine these.

The technical requirements originate from the calculations and shall not be considered in the earlier stages.

A.5. TYPICAL STAKEHOLDER CONDITIONS

Next to the requirements originating from codes and guidelines, stakeholders and clients could also have several conditions, which are mandatory or negotiable. The mandatory requirements are considered as functional requirements. Typical stakeholder conditions are provided in Table A.4. The structure:

Table A.4: Typical Stakeholder Conditions

Code	**Description**
SC1	incorporates mooring or berthing opportunities of recreational sailing.
SC2	should enhance the safety of recreational sailing vessels.
SC3	provides lay-by facilities.
SC4	is fixed to one location or movable.

The typical stakeholder conditions can be used. It is of importance to determine whether the individual requirement is negotiable or mandatory.

B

TENDER APPROACHES

In the phase of registration or tendering, certain principles are of paramount importance. These provide the aspects which are assessed and graded, and will weight up against the total project costs. The most common approaches are found in Table B.1. In practice also combinations occur.

Table B.1: Approaches of Registration

Abbreviation	**Definition**
BVP	Best Value Procurement
EMVI	Economical Most Beneficial Registration
CO2 Performance Ladder	-
LP	Lowest Price

B.1. BVP

The highest value and the lowest price underlie the Best Value Procurement (Rijkswaterstaat, 2014a). The method was created by an American named Dean Kashiwagi working at the Performance Based Studies Research Group of the Arizona State University (PIANOo, 2014). Parties can profile themselves by showing there expertise during the registration phase. While the client gives global requirements (functional requirements) and explains what is preferred ('state of the art') about the organisation, the tenderer will show in which manner more project value can be generated. The client will foresee a situation where both parties have advantages. For example, the contractor will use his freedom plus expertise to distinguish himself from the other submitters and the client will have a proper design without verification. The result will be lower not-suspected costs, higher profit perspective and less loss of time for discussion. The BVP procedure is in sequence: preparation by the client, assessment of the tenders, argumentation by the engineers and execution of the design. The criteria in the assessment are the file about the risks, file about chances and the arguments of performance, which is focused on the organisation only and not on the resulting structure. The client will have a clear definition of the risk, performances and planning of the works, which is overcoming uncertainties in the construction phase. Possible risks reduction, taking of chances and performance providing value to the project should be mapped.

BVP is not considered.

B.2. EMVI

The Economical Most Beneficial Registration is balancing the quality and benefits of a product (Rijkswaterstaat, 2014b). Eventually, the benefits will make the difference between the multiple parties.

Table B.2: EMVI Criteria with Definition

EMVI Criterion	Definition
System Quality	The structure is not sensitive to damage and the damage is easy to repair.
Durability	The lifetime of the structure is long, the environmental aspect is considered and energy resources are properly handled.
Innovation	New concepts and techniques are applied for the structure.
Ecological Impact	Animals are not disturbed by the structure and water quality is not affected.
CO2 Ambition Level	CO2 emission in production of the structure and during construction are low.
Hindrance	Pedestrians and vehicles can safely pass and are not endangered by construction activities.
Noise	Annoying sounds due to construction are not present.
Risks	The structure is easy to construct and there is less uncertainty in the performance.
Life Cycle Cost	The cost during the lifetime of the structure is considered, reduced and/or low.

The client will provide a complete list of EMVI-criteria, where a discount (fictional amount of money) can be obtained. Common criteria are summed in Table B.2. In this way, the client will be ensured due to a fully specified product. The contents of each registration will be assessed qualitatively and quantitatively, which results in the allocation of a score in money. The result is a public-oriented, durable and risks-controlled product. More benefits within registration will lead to more discount. The highest bidder could have to lowest price, in case he is able to fulfil the conditions of the client as described in the EMVI criteria.

EMVI takes into account specified criteria from the client including the maximum monetary discount and a design with the lowest price.

B.3. CO2 PERFORMANCE

The CO2 Performance Ladder is a tool to manage and reduce carbon dioxide emission (Stichting Klimaatvriendelijk Aanbesteden & Ondernemen, 2014). The total emission is awarded with a fictional discount, as in case of the EMVI approach. The CO2 Performance Ladder is also regularly included in the EMVI criteria. An advantage for the contractor is that the costs of using energy is reduced and that savings are found in materials.

CO2 Performance Ladder takes into account the total CO2 emission of the construction and is part of the EMVI-criteria.

C
CONTRACTS

Many types and combinations of contracts are developed for civil engineering, which mean varying responsibilities, advantages and risks for the contractor and client. An outline is provided of the contracts: Design & Construct; Design, Build & Maintain; Design, Build, Finance & Maintain; Design, Build, Finance, Maintain & Operate; Engineering & Construct and Consent Registration Works.

C.1. D&B

A Design & Build (or Design & Construct) contract is an agreement between two parties. The accepter of the assignment is responsible for the design and construction based on functional requirements. The optimisation of these phases are resulting in an efficient process. A disadvantage is that the client is not able to fully influence the design, because functions are prescribed and not the lay out, while occurring problems are mainly for the contractor, unless it has been included in the tender guide and notified by the client. An advantage for the contractor is that the work is accomplished internally, which will affect the complexity, capacities, time and money in a positive manner. However, innovation is less obvious, since the risks are to be at the expense of the contractor. Well-known structures and construction methods are more likely to be used.

D&B should consider the design and construction as most important cost drivers. The aspects maintenance and reduction of risks should be given less attention, unless prescribed by the contracts as functional requirement.

C.2. DB&M

The Design, Build & Maintain contract extends the above-mentioned contract. After construction a period of inspection and maintenance starts. This is an extra responsibility for the contractor. An advantage is that the client is assured that the considered structure is well-designed and -constructed to reduce high maintenance costs. Also the Life Cycle Costs can be controlled in a large extend, balancing the investment and the costs of maintenance for multiple years.

DB&M should consider the design, construction and maintenance as most important cost drivers. The aspect reduction of risks should be given less attention, unless prescribed by the contracts as functional requirement.

C.3. DBF&M

Close to this type of contract is the Design, Build, Finance & Maintain contract. An additional feature is that the finance is performed by the contractor initially. However, this contract is rarely applied to breakwater projects. During the selection, the risks and liabilities are given to the party, which proved to be able to manage them. This will not have influence on the selection of a breakwater alternative and is as a consequence not incorporated in the method.

DBF&M is not considered in the assessment of the method.

C.4. DBFM&O

Also the Design, Build, Finance, Maintain & Operate contract is rarely found for breakwater requests. For the reason that the term operation is not relevant for these static structures, where no revenues are directly gained. Therefore, this type of contract is neglected.

DBFM&O is not considered in the assessment of the method.

C.5. E&C

In case of an Engineering & Construct contract, the client can influence the design in contrast to the D&C contract. In the E & C contract the object is already determined. The next phase is that the contractor will dimension the structure, develop a final design report and provide technical drawings. A study of alternatives is already performed, which makes this type of contract irrelevant in this report.

E&C is not considered in the assessment of the method.

C.6. RAW

The contractor is particularly familiar with RAW (Consent Registration Works) contracts. These contracts are about the construction phase only. The RAW-contract is subjected to UAV-2012 (Uniform Administrative Conditions) and standard RAW-2010, which consists of regulations and conditions about the responsibilities of the client and contractor. More important, the study of alternatives and final design phase is accomplished before the RAW tendering. As a result, also this contract will be neglected in the method description.

RAW is not considered in the assessment of the method.

D

LAWS, PERMITS AND REGULATIONS

Legislation should be met for civil projects. Discussed are relevant laws, permits and regulation for construction of breakwaters.

D.1. LAWS

Enclosed in the program of requirements are the laws, which are mainly dealing with the protection and improvement of aquatic ecosystems. Like the permits, these could exclude certain structures or construction methods. The most common laws concerning the body of soil and the water quality (Rijksoverheid, 2014) and (Rijkswaterstaat, 2014d) are provided in Table D.1.

Table D.1: Laws for Construction of Hydraulic Structures

Code	Type of Law	English Translation	Contents
L1	Waterwet (Ww)	Law of Water	Surface and ground water quality; return drainage and temporary storage.
L2	Flora- en Faunawet	Law of Flora and Fauna	Disturbance of vegetations and wildlife.
L3	Wet Bodembescherming (Wbb)	Law of Subsoil Protection	Pollution in the underground.
L4	Natuurbeschermingswet	Law of Nature Protection	Preservation of vegetations and wildlife areas.
L5	Wet Milieubeheer (Wm)	Law of Nature Preservation	Transport of dredging materials; dumping of materials; return drainage and temporary storage.
L6	Scheepvaartverkeerswet (SVW)	Law of Maritime Traffic	No hinder of the vessel traffic; no damage to hydraulic structure by vessels.
L7	Ontgrondingenwet	Law of Excavation	Permit is required for excavation.
L8	Natuurcompensatie	Compensation of Nature	Compensation of damage to nature.
L9	Kaderrichtlijn Water (KRW)	European Water Directive	Water quality and pollution.

The Vogel- en Habitatrichtlijn (translation: Birds and Habitat Regulation) is included in the Flora- en Faunawet and Natuurbeschermingswet. The Nature2000 areas in The Netherlands are subjected to this regulation.

An additional document is the Law of Tendering (Aanbestedingswet). It discusses the procedure of registration. In other words, the Law of Tendering concerns the works after the design phase is completed and is therefore not affecting the design choices.

The relevant laws should be listed and considered.

D.2. PERMITS

Various permits are required before the construction of water related structures. Table D.2 gives a list of the most common and relevant permits.

Table D.2: Permits for Hydraulic Structures

Code	Type of Permit	English Translation	Remarks
P1	Omgevingsvergunning	Permit of Surroundings	Required for construction works (in protected area).
P2	Milieuvergunning	Environmental Permit	Omgevingsvergunning originates from the Wet Milieubeheer.
P3	Watervergunning (WBR and KEUR)	Water Permit	Required for construction works in surface waters.
P4	Watermelding	Water Notification	Replaces the Watervergunning under certain conditions.
P5	Ontgronding	Excavation	Required for excavation works.
P6	Besluit Uniforme Saneringen (BUS)	Decision Uniform Soil Restoration	Required for excavation of polluted subsoils.
P7	Saneringsvergunning	Permits of Soil Restoration	Originates from Besluit Uniforme Saneringen.
P8	Bergingsverzoek Slibdepot	Storage Request Silt Deposit	Required for dumping excavated materials.
P9	Kabels en Leidingen Informatie Centrum (KLIC)	Centre of Information about Cables and Pipes	Excavation Notification to prevent damage to cables and pipes.
P10	Meldpunt Opbrekingen Openbare Ruimte (MOOR)	Contact Point	Notification of activities in public area.
P11	Bouwvergunning	Permit of Construction	Replaced by Omgevingsvergunning.
P12	Sloopvergunning	Demolish Permit	Replaced by Omgevingsvergunning.

Hakkers, 2015

These permits could restrict certain activities or types of structures. For example, in case of polluted soil, permits regarding excavation and soil quality could cause issues during the construction phase. Therefore, structures which expose the soil to the water and therefore causes polluted soil going into suspension should be avoided initially.

The relevant permits should be listed and considered.

D.3. REGULATIONS

Regulations are written by the government and municipalities to control public areas. The laws are supplemented by the regulations from administrative body. Table D.3 provides a number of relevant regulations for breakwaters.

Table D.3: Regulations for Construction of Hydraulic Structures

Code	Type of Regulation	English Translation	Contents
R1	Besluit Lozen Buiten Inrichtingen (BBI)	Decree Discharge Outside Interior	Originates from the Waterwet and Wet Milieubeheer. Required for dredging works.
R2	Besluit Bodemkwaliteit (Bbk)	Decree Soil Quality	Management of subsoil quality. Required for dredging works.
R3	Beheer- en Ontwikkelplan voor de Rijkswateren (BPRW)	Management and Development Plan of Waterways	Maintenance and management of hydraulic structure.
R4	Activiteitenbesluit	Decree of Activities	Companies should notify their activities. Originates from the Wet Milieubeheer.

Construction works should be according to the regulations. When the works are not according to the rules, construction activities can be delayed, interrupted or cancelled.

The relevant regulations should be listed and considered.

E

Boundary Conditions

This appendix consists of the following sections which are independently focused on the boundary conditions relevant for a breakwater design.

Appendix E.1	Wind
Appendix E.2	Vessel
Appendix E.3	Water Depth
Appendix E.4	Flow Velocity
Appendix E.5	Waves
Appendix E.6	Ice
Appendix E.7	Subsoil
Appendix E.8	Earthquakes

These sections consist of an extensive description of the boundary conditions. It includes professional expressions and typical units, which are accordingly classified to be implemented in subsequent sections.

E.1. WIND

Wind is a generator of waves and therefore usually an indirect load. It could be taken into account for floating breakwaters. Obviously, higher wind velocities result in larger wave heights and longer wave periods.

E.1.1. WIND ROSE

The wind classification includes information about the direction, the maximum occurring velocity and the number of readings. A common visualization is a wind-rose (Figure E.1).

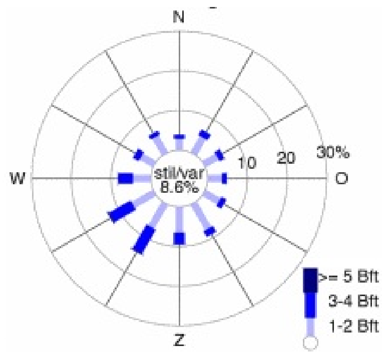


Figure E.1: Wind Rose
(KNMI. (2014). *Klimatologie: Windrozen van de Nederlandse hoofdstations*. Retrieved from http://www.knmi.nl/. Accessed on September 26, 2014.)

In the following table the Beaufort scale is explained. This type of scale is not applicable in many design formulas.

Table E.1: Classification of Wind Velocity by Beaufort

Beaufort Scale (Bft)	**Wind Velocity** (m/s)
0	0.3
1	0.3 - 1.5
2	1.6 - 3.3
3	3.4 - 5.4
4	5.5 - 7.9
5	8.0 - 10.7
6	10.8 - 13.8
7	13.9 - 17.1
8	17.2 - 20.7
9	20.8 - 24.4
10	24.5 - 28.4
11	28.5 - 32.6
11	28.5 - 32.6

(Wallbrink and Koek, 2009)

E.1.2. DESIGN WIND

The design wind velocity depends on the decision of the designer and/or the client. The client can provide a velocities for the wind during storm conditions and for the wind the structure should reduce the wave actions. The wind can be provided in multiple units, in which the most common is Beaufort and m/s (Table E.1).

Moreover, designers normally use the return period of certain wind conditions. Examples of return periods are 1/50, 1/100 or 1/200 years. Wind conditions up to 1/10000 years are provided in Table E.3. It is obvious that the smaller the probability of occurrence the higher the wind velocity. When the wind is known a first estimate can be made by dividing the national storm conditions in four groups (CUR, 2000). These are assumed to be respectively 15, 20, 25 (heavy storm) and 30 (very heavy storm) m/s.

Table E.2: Classification of Design Wind

Type of Wind Conditions	**Wind Velocity** (m/s)
Strong Wind	15
Very Strong Wind	20
Heavy Storm	25
Very Heavy Storm	30

The four wind groups suffice to obtain a rough estimate of the design wind velocity. An observed or provided value can be round up to obtain a higher level of safety.

E.1.3. WIND DATA

Local weather stations collect wind data throughout the years. In the Netherlands stations are homogeneously distributed over the country (Figure E.2). The wind conditions in the long term can be found here. Global and rough climate information including wind states is elaborated in Young and Holland (1996) and Hogben et al. (1986).

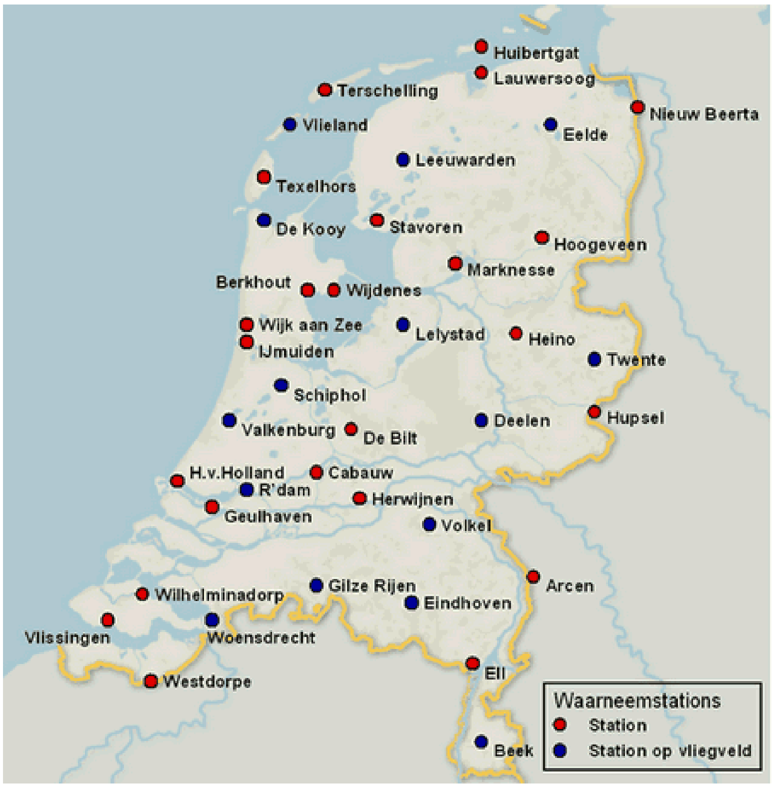
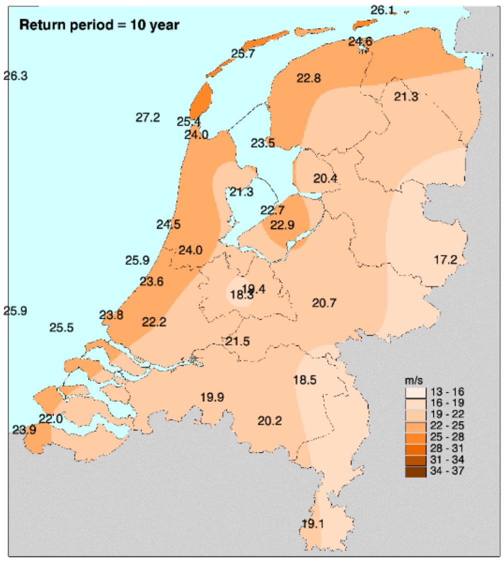
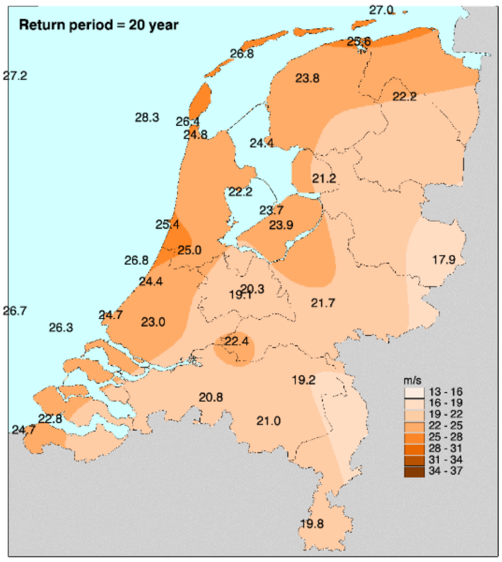
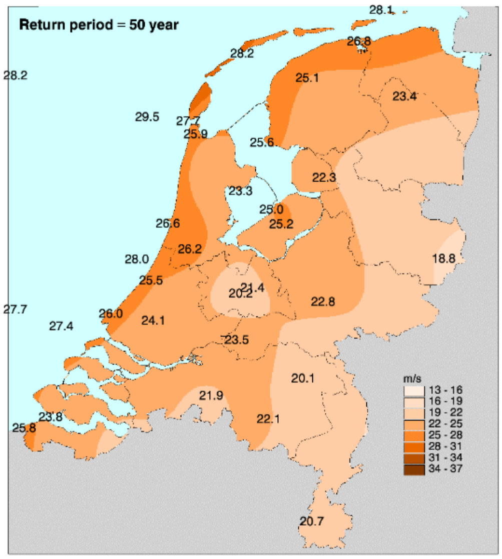


Figure E.2: Weather Observation Stations in the Netherlands
(KNMI. (2014). *Weer: Waarnemingsstations in Nederland*. Retrieved from http://www.knmi.nl/. Accessed on November 4, 2014.)

Figure E.3 shows wind velocities over The Netherlands with a return period.


Return period = 10 year
26.1
24.6
25.7
26.3
22.8
21.3
27.2
25.4
24.0
23.5
20.4
21.3
22.7
22.9
24.5
24.0
25.9
17.2
23.6
19.4
18.3
20.7
25.9
25.5
23.8
22.2
21.5
18.5
19.9
22.0
20.2
23.9
19.1
m/s
13 - 16
16 - 19
19 - 22
22 - 25
25 - 28
28 - 31
31 - 34
34 - 37


Return period = 20 year
27.0
25.6
26.8
27.2
23.8
22.2
28.3
26.4
24.8
24.4
21.2
22.2
23.7
23.9
25.4
25.0
26.8
17.9
24.4
20.3
19.1
21.7
26.7
26.3
24.7
23.0
22.4
19.2
20.8
22.8
24.7
21.0
19.8
m/s
13 - 16
16 - 19
19 - 22
22 - 25
25 - 28
28 - 31
31 - 34
34 - 37


Return period = 50 year
28.1
26.8
28.2
28.2
25.1
23.4
29.5
27.7
25.9
25.6
22.3
23.3
25.0
25.2
26.6
26.2
28.0
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25.5
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27.4
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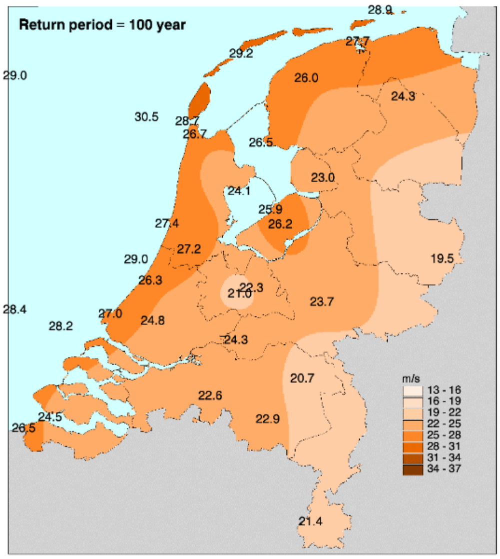
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27.7
29.2
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27.4
27.2
29.0
19.5
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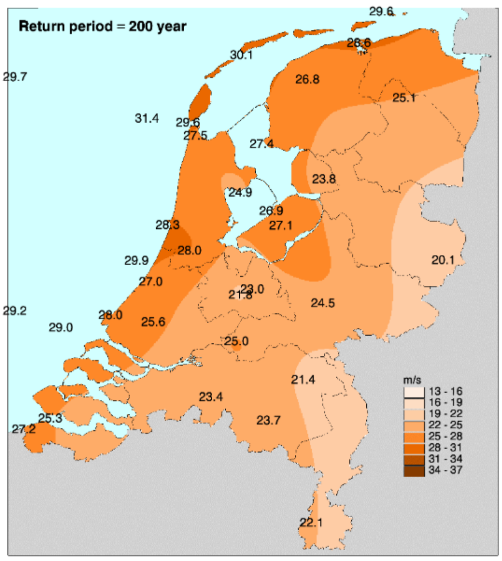
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30.1
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26.8
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24.9
26.9
27.1
28.3
28.0
29.9
20.1
27.0
23.0
21.8
24.5
29.2
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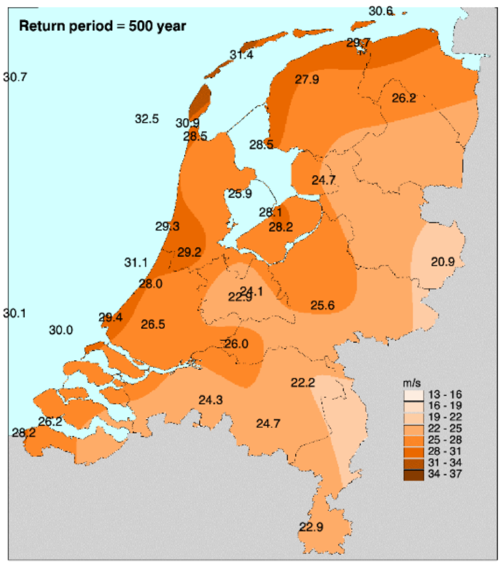
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31.4
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27.9
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32.5
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24.1
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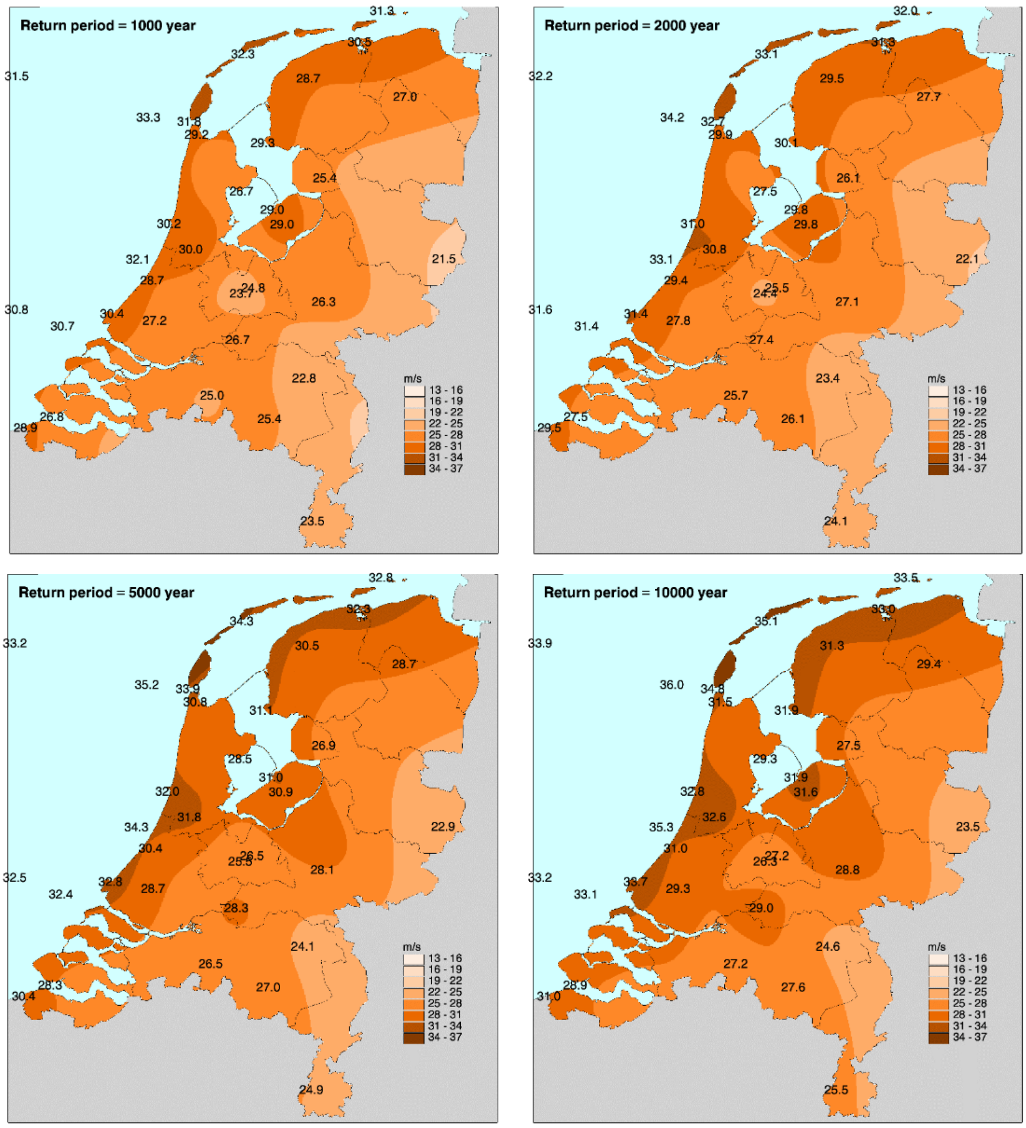


Table E.3: Return Period of Wind in The Netherlands
(Smits, 2001)

E.2. VESSEL

Various vessel are sailing along banks and ports protected by breakwater. The type and shape of vessels determine the magnitude of the vessel-generated: wave loading, flow velocity forces, (additional) mooring forces and accidental collision forces.

E.2.1. INLAND VESSELS

Besides the vessels mentioned in the international CEMT waterway classes Table E.4), a more limited set of vessels is found in inland waterways in the Netherlands (Table E.5 and Table E.6).

Type of inland waterways		**Classes of navigable waterways**	**Motor vessels and barges**					**Minimum height under bridges (m)**
			Type of vessel and general characteristics					
			Designation	**Length (m)**	**Beam (m)**	**Draught (m)**	**Tonnage (T)**	
Of regional importance	**To West of Elbe**	I	Barge	38.50	5.05	1.80-2.20	250-400	4.00
		II	Kampine-barge	50-55	6.60	2.50	400-650	4.00-5.00
		III	Gustav Koenigs	67-80	8.20	2.50	650-1000	4.00-5.00
	To East of Elbe	I	Grosse Finow	41	4.70	1.40	180	3.00
		II	BM-500	57	7.50-9.00	1.60	500-630	3.00
		III		67-70	8.20-9.00	1.60-2.00	470-700	4.00
Of international importance		IV	Johann Welker	80-85	9.5	2.50	1000-1500	5.25 or 7.00
		Va	Large Rhine Vessels	95-110	11.40	2.50-2.80	1500-3000	5.25
		Vb						7.00 or 9.10
		VIa						7.00 or 9.10
		VIb		140	15.00	3.90		7.00 or 9.10
Type of inland waterways		**Classes of navigable waterways**	**Pushed convoys**					**Minimum height under bridges (m)**
			Type of convoy: general characteristics					
			Designation	**Length (m)**	**Beam (m)**	**Draught (m)**	**Tonnage (T)**	
Of regional importance		I						3.00
		II						3.00
		III		118-132	8.20-9.00	1.60-2.00	1000-1200	4.00
Of international importance		IV		85	9.5	2.50-2.80	1250-1450	5.25 or 7.00
		Va		95-110	11.40	2.50-4.50	1600-3000	5.25
		Vb		172-185	11.40	2.50-4.50	3200-6000	7.00 or 9.10
		VIa		95-110	22.80	2.50-4.50	3200-6000	7.00 or 9.10
		VIb		185-195	22.80	2.50-4.50	6400-12000	7.00 or 9.10
		VIc	6 barges, long 6 barges, wide	270-280 193-200	22.80 33.00-34.20	2.50-4.50 2.50-4.50	9600-18000 9600-18000	9.10
		VII		285 195	33.00-34.20	2.50-4.50	14500-27000	9.10

Table E.4: European Classification of Waterways and Inland Waterway Vessels (European Conference of Ministers of Transport, 1992)

Klasse		
I	 Spits Lengte 38,5 meter - breedte 5,05 meter - diepgang 2,20 meter - laadvermogen 350 ton	14 x
II	 Kempenaar Lengte 55 meter - breedte 6,60 meter - diepgang 2,59 meter - laadvermogen 655 ton	22 x
III	 Dortmund-Eemskanaalschip (Dortmunder) Lengte 67 meter - breedte 8,20 meter - diepgang 2,50 meter - laadvermogen 1.000 ton	40 x
IV	 Rijn-Hernekanaalschip (Europaschip) Lengte 85 meter - breedte 9,50 meter - diepgang 2,50 meter - laadvermogen 1.350 ton	54 x
Va	 Groot Rijnschip Lengte 110 meter - breedte 11,40 meter - diepgang 3,00 meter - laadvermogen 2.750 ton	120 x
Vb	 Groot Rijnschip Lengte 135 meter - breedte 11,40 meter - diepgang 3,5 meter - laadvermogen 4.000 ton	160 x
VIa	 Tweebaksduwstel Lengte 172 meter - breedte 11,40 meter - diepgang 4 meter - laadvermogen 5.500 ton	220 x
VIb VIc	 Vier- of zesbaksduwstel Lengte 193 meter - breedte 22,80 / 34,20 meter - diepgang 4 meter - laadvermogen 11.000 / 16.500 ton	440 / 660 x
Va	 Standaard tanker Lengte 110 meter - breedte 11,40 meter - diepgang 3,50 meter - laadvermogen 3.000 ton	120 x

Klasse		
Vb	Grote tanker Lengte 135 meter - breedte 21,80 meter - diepgang 4,40 meter - laadvermogen 9.500 ton	380 x
Va	Autoschip Lengte 110 meter - breedte 11,40 meter - diepgang 2,00 meter - laadvermogen 530 auto's	60 x
III	Containerschip Kempenaarsklasse Lengte 63 meter - breedte 7 meter - diepgang 2,50 meter - laadvermogen 32 TEU	16 x
Va	Standaard containerschip Lengte 110 meter - breedte 11,40 meter - diepgang 3,00 meter - laadvermogen 200 TEU	100 x
Vb	Groot containerschip Lengte 135 meter - breedte 17 meter - diepgang 3,50 meter - laadvermogen 500 TEU	250 x
Va	Ro-roschip Lengte 110 meter - breedte 11,40 meter - diepgang 2,50 meter	72 x
VIb	Koppelverband (schip met duwbak) Lengte gemiddeld 185 meter - breedte 11,40 meter - diepgang 3,50 meter - laadvermogen 6.000 ton	240 x
VIb	Koppelverband (schip met schip) Lengte gemiddeld 185 meter - breedte 11,40 meter - diepgang 3,50 meter - laadvermogen 6.000 ton	240 x

Table E.5: Types of Inland Waterway Vessels in The Netherlands
(Bureau Voorlichting Binnenvaart (2014, November 4) *Scheepstypen*. Retrieved from http://www.bureauvoorlichtingbinnenvaart.nl)

Table E.6: Map of Inland Waterway Vessels in The Netherlands
(Bureau Voorlichting Binnenvaart (2014, December 9) *Vaarwegen*. Retrieved from http://www.bureauvoorlichtingbinnenvaart.nl)

E.2.2. RECREATIONAL VESSELS

The recreational boats are split up for transitional and dilation waterways (Rijkswaterstaat, 2005). The following table provides four vessels in length (L), beamwidth (W) and draught (D).

Table E.7: Recreational Vessels

Type of Vessel	**Length** (m)	**Beamwidth** (m)	**Draught** (m)
Engine Power and Sail (Transitional Waterways)	15	4.25	2.1
Engine Power and Sail (Dilation Waterways)	15	6.25	1.9
Charter (BVA)	35	7.00	1.4
Charter (BVB)	25	6.00	1.2

(Rijkswaterstaat, 2005)

The power of the engine (P) and the main propeller diameter (d) can be approximated by the following formulas.

$$P = 0.661L(2D + B) \tag{E.1}$$

$$d = 0.7^{2\cdot}D \tag{E.2}$$

E.2.3. VESSEL DATA

Depending on the location of the river, lake or canal, information can be obtained by the Directorate-General for Public Works and Water Management (Rijkswaterstaat). This division of the government provides waterway data for larger vessels. Local water managers can provide data for the areas with recreational boat activity. The managers will also be stakeholder when a project in their water district is concerned.

E.2.4. DESIGN VESSEL

CEMT inland waterway classes and the classification by the Dutch government provides accurate data about vessels to take into account in the preliminary design phase. In the study of alternatives it is recommended to use the following vessel dimensions for a first estimate.

Table E.8: Classification of Design Vessels

Type of Vessel	**Length** (m)	**Beamwidth** (m)	**Draught** (m)	**Engine Power** (kW)	**Propeller Diameter** (m)
Small Yacht	15	5	2.0	89	1.0
Sailing Boat	35	7	1.5	231	0.7
Barge	50	7	1.5	331	0.7
International Vessel	100	12	3.0	1190	1.5
Regional Convoy Vessel	150	9	2.0	1289	1.0
International Convoy Vessel (1)	150	22	3.5	2875	1.7
International Convoy Vessel (2)	200	33	4.5	5552	2.2

These classes are extracted from the CEMT waterway classes and determined by engineering judgement about vessels entering yacht harbours. The seven design vessels suffice to obtain a rough estimate of the design vessel.

The two recreational boats and five cargo vessels suffices to obtain a rough estimate of the design vessel. Observed or provided dimensions can be round up to obtain a higher level of safety.

E.3. WATER DEPTH

The water depth strongly effects the wave height, the wave velocity and the dimensions of a hydraulic structure. When the water depth increases the occurring wave heights will become larger. Similarly, the dimensions of the structure will increase as well. As a consequence, more material, labour and heavy equipment are required for construction.

The water depth is the difference between the water level and bed level (Figure E.3). The levels are often provided in NAP (New Level of Amsterdam).

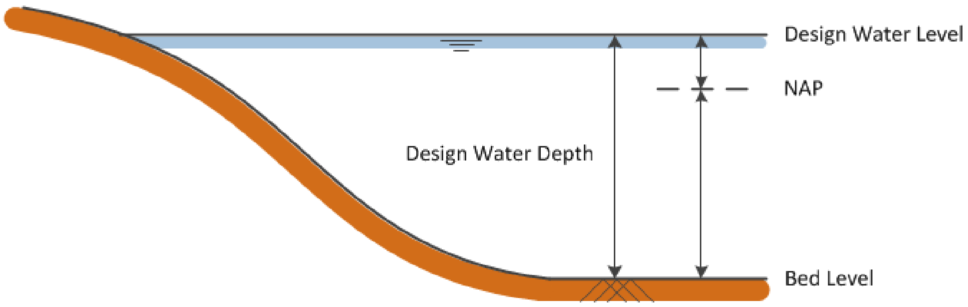


Figure E.3: Water and Bed Levels with respect to NAP

The bathymetrical data consist of hydrographic maps, which contain 2D and/or 3D depth information. Lake- and riverbed shapes are created by depth contour lines. In a cross-sectional view the depths are mostly shown relative to the waterline, where the water depth is zero. NAP (New Level of Amsterdam) is also taken as reference level.

E.3.1. INLAND WATERS

Inland waterways deal with limited water depths compared to seas and oceans. For instance, The Veluwemeer, Ketelmeer, Markermeer, Zwarte Meer, Gooimeer, Tjeukemeer, Lauwersmeer, Veerse-meer in The Netherlands have water depths between 1 m and 2 m (Nature2000, 2014). A larger water depth is found in the IJmeer, where the IJburg breakwater is constructed. A design water depth of 2.5 m (water level NAP 0.0 m and bed level NAP -2.5 m) was assumed (Haghgoo Daryasari, 013b). Besides, an extreme value is found in the IJsselmeer in The Netherlands in which water depths are up to 7 m (IJsselmeeralmanak, 2014). Average water levels are between NAP -0.4 to -0.2 m and bed level is at NAP -6 to -7 m. The deepest parts of this lake are not nearby harbour facilities, which could be protected by breakwaters. Parts of the lake eligible for breakwaters are assumed to be designed for the largest inland waterway vessels, which are found in the Nieuwe Waterweg in Zuid-Holland. An reasonable assumption is that the water depth varies between 1 and 4 m along the side of rivers and lakes.

E.3.2. CHANNEL DEPTH

The required channel depth is determined by the waterway classification of the CEMT international standard (PIANC, 1992). Regularly this is provided as an functional requirement. Inland waterway vessels of class I to VIIb have a draught of respectively 1.8 m to 4.5 m (Table E.4). Including the variables in the formula for channel depths Ligteringen and Velsink (2012) the maximum design water depth can be determined.

$$h_{gd} = D + s_{max} + a + h_{net} \tag{E.3}$$

In which:

h_{gd} Guaranteed depth (m)
D Draught design vessel (m)
S_{max} Maximum sinkage (m)
a Wave amplitude (m)
h_{net} Keel clearance (m)

E.3.3. WATER DEPTH DATA

Information about the exact water depths at the project location are normally provided by local water managers or available depth charts. Information can also be gained by executing survey. Common methods of measurement are single and multibeam echo sounding. Survey by echosounding is executed by the sending of a sound signal(s) or single pulse(s). The time between sending this signal and receiving reflected signal from the bed will determine the distance between the echosounders and bed (Schiereck et al., 2012). This measurement is only applicable to hard surfaces (for instance, rubble). For a small area of seabed a singlebeam echosounder, while for larger areas multibeam sounding is more preferable (Figure E.4).

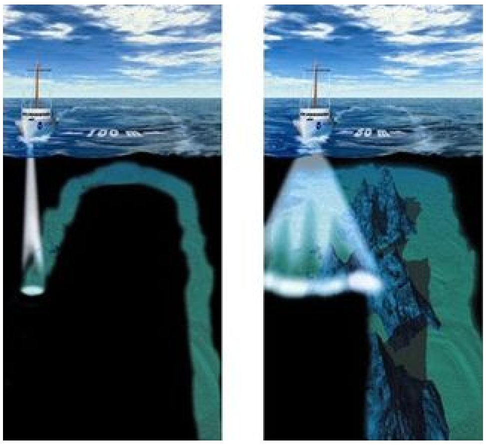


Figure E.4: Single Beam (left) and Multi Beam (right) Echosounding
(Ayres Associates. (2011). *Hydro Survey: Multibeam vs. Single Beam (Part 1)*. Retrieved from http://ayresriverblog.com/. Accessed on October 30, 2014.)

Multibeam echosounders send multiple single pulses per strip over an angle of ±60 to ±70 degrees with respect to the transducer. The echosounders become more accurate in shallow waters. It also depends on the quality of measuring equipment and survey operation (CUR, 2014). Two kinds of error can occur. The systematic errors are considered to be incorrect: tidal and chart data, draught and installation, and speed of the sound signal. Another contributor to the inaccuracy is the random error. This is defined as the difference between the real and measured value. The total survey precision of the singlebeam and multibeam is found in Table E.9.

Table E.9: Total Survey Precision of Echosounding by using dGNSS Positioning Systems

Quality Operational Water Level System	Bad (dGNSS)	Good (dGNSS)	Good (RTK dGNSS)
Horizontal Precision	3	1-2	0.5
Vertical Precision	0.4	<0.2	<0.1

Additionally, hydrographic departments have long-term bathymetry charts on scale. For detailed engineering these charts are not sufficient and additional hydrographic survey has to be performed. But for a first estimate, the charts deliver valuable information.

E.3.4. DESIGN WATER DEPTH

In the determination of the design water depths, the effects of wind setup and seasonal variations shall be included. The water depths along river and lakes fluctuate roughly between 1 and 4 m (CUR, 2000) for regions where hydraulic structures could be requested. The seasonal changes of precipitation will mainly affect the discharge of rivers and water level in lakes. Statics of the hydraulics of a certain water system should be gathered to determine 1/50, 1/100 or 1/500 years maximum water levels.

The determined water depth can be fitted into the water depth classes in Table J.1. It assumed that by using 0.5 m difference between the design water depths the over- or underestimation is limited compared to 1 m difference. In contrast, 0.25 m difference would result in an accurate estimation to be considered in this high level investigation of the alternatives.

Table E.10: Classification of Design Water Depth

Property	**Water Depth** (m)
Lowest	1
Mean Low	1.5
Low	2
Mean	2.5
High	3
Mean High	3.5
Highest	4

The defined groups of water depth suffice to obtain a rough estimate of the design water depth. An observed or provided value can be round up to obtain a higher level of safety.

E.4. FLOW VELOCITY

A less relevant parameter of most breakwater designs is the flow velocity. Compared to the impact of waves, these forces are negligible. Nevertheless, the motion of water is able to cause scour and erosion, which could result in the collapse of a hydraulic structure. Flow velocities are generated in open channels (e.g. canals and rivers), which can be approximated by formulas of Chézy and Manning.

E.4.1. FORMULA OF CHÉZY

For most open channels the governing discharges and flow velocities have to be determined. Also simplified formulas can be applied. For instance, Chézy (1775) provides an equation, which approaches the physics of flow velocities (Nortier and De Koning, 1996). The general formula for uniform turbulent flow reads:

$$u = C\sqrt{Ri} \tag{E.4}$$

The averages flow velocity, u (m), depends on the Chézy coefficient of roughness, C $(m^{1/2}/s)$, the hydraulic radius, R (m) and the water surface gradient, i $(-)$. The determination of the Chézy coefficient is the most challenging part.

E.4.2. FORMULA OF MANNING

The Irish engineer Manning (1980) proposed a similar formula. The Manning coefficient (n) indicates the roughness of the open channels side material. Although the Manning formula is straightforward and quick, the Chézy seems more applicable in unique circumstances, according to literature. The Manning equation is as follows:

$$u = \frac{R^{2/3} i^{1/2}}{n} \tag{E.5}$$

u (m) is the average flow velocity, R (m) is the hydraulic radius and i $(-)$ is the water surface gradient.

E.4.3. PROPELLER FLOW VELOCITIES

Flow velocities are also generated by vessels. These are a result of the spinning propellers causing high turbulence in the body of water. While yachts are equipped with a jet engine, more impact on the bed, bank and shoreline is found due to propellers of larger vessels. The outflow velocity of the propeller, u_0 (m/s), can be approximated by the following formula. The power of the engine, P (kW), the density of water, ρ_w (kg/m^3), and the diameter of the propeller, d (m), are the independent variables.

$$u_0 = 1.15\left(\frac{P}{\rho_w d^2}\right)^{1/3} \tag{E.6}$$

E.4.4. RETURN FLOW VELOCITIES

Also flow velocities along and under vessels are considered. Along a sailing vessel on both sides a water depression is observed (Figure E.5). Due to this local water level decrease, the flow velocities have to increase and the water displaces itself from the bow to the back of the vessel.

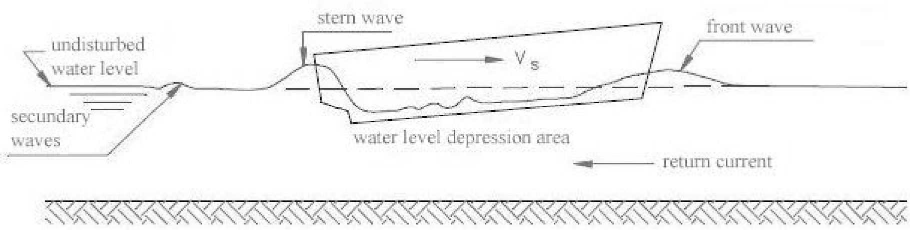


Figure E.5: Vessel-generated Waves, Currents and Water levels (1)
(Waterloopkundig Laboratorium, 1997)

The equation of Bernoulli provides a method to determine the increase of the flow velocities (kinetic energy) when the water level drops (potential energy). The formulas read:

Bernoulli:

$$d + \frac{{v_s}^2}{2g} = d - z + \frac{(v_s + u_r)^2}{2g} \tag{E.7}$$

Equation of continuity:

$$bdv_s = (bd - BD - bz)(v_s + u_r) = Q \tag{E.8}$$

In which:

- v_s Sailing velocity of the vessel (m/s)
- u_r Flow velocity of the return flow (m/s)
- d Original water depth (m)
- D Draught of the vessel (m)
- b Width of the waterway (m)
- B Width of vessel (m)
- z Water level depression (m)
- Q Discharge (m^s/s)

E.4.5. BOW THRUSTER FLOW VELOCITIES

PIANC/Marcom 180 (2015) provides a broad outline of all kinds of bow thrusters. It is said that inland waterways vessels have ducted propeller in the bow of 350 to 2000 kW. A relation is found between length, L_e (m), and draught, D (m) of the largest vessels (for example, international, regional convoy, international concoy vessel) and the bow thruster power, $P_{thruster}$ (kW).

$$P_{thruster,generalcargo} = 1.75 L_e D - 150 \tag{E.9}$$

$$P_{thruster,container} = 2.0 L_e D - 250 \tag{E.10}$$

Smaller vessels, like the small yacht, sailing boat and barge, could have bow thrusters, but impact on mooring is assumed to be relatively low and the frequency of occurring is low. Therefore, the thrusters of these vessels are not taken into account.

Generally, the flow velocities originating from the bow thruster, u_0 (m/s), can be determined by the engine power P (kW) and the propeller diameter of the bow thruster d_p (m).

$$d_p = 0.1636 P^{0.3656} \tag{E.11}$$

Similarly, Equation E.6 can be solved.

E.4.6. FLOW VELOCITY DATA

Most rivers and lake areas have well-known discharges, from which the flow velocities can be extracted. These data are obtained from previous hydraulic studies and by measurements carried out by researchers. It concerns averaged discharges, which are not representative for local appearing flow conditions. As a result, studies have to be carried out to approach the development of the flow close to a structure.

E.4.7. DESIGN FLOW VELOCITY

Within inland waterways average flow velocities fluctuate between 0.1 and 2 m/s (CUR, 2000). For example, during governing discharge of the Rhine (16.000 m^3/s) the average flow velocity is approximately 1 m/s. When rivers are deeper and steeper, flow velocities will increase considerably.

The designer should analyse the geometry of the open channel and seasonal discharges. A design discharge is based on probabilities of occurrence like 1/100, 1/500, 1/1250, often provided as functional requirement. These strongly depend on the behaviour of system, which is the catchment area and discharge channel. The following classification in Table E.11 is assumed for inland rivers and canals.

Table E.11: Classification of Design Flow Velocities from Discharge

Property	**Flow Velocity** (m/s)
Lowest	0.10
Mean Low	0.25
Low	0.50
High	1.00
Mean High	1.50
Highest	2.00

In contrast to the flow velocities as a result of discharge, the return flow along vessels measured in Dutch rivers and channel is between 0.1 and 1 m/s (CUR, 2000). The following classification of the return flow in Table E.12 is recommended.

Table E.12: Classification of Design Return Flow Velocities

Property	**Flow Velocity** (m/s)
Lowest	0.10
Mean Low	0.20
Low	0.30
High	0.50
Mean High	0.75
Highest	1.00

The third group of flow velocities is generated by the chosen design vessels (Section E.2.4). The flow velocity in front of the propeller are determined by the rules of thumb. The result is found in the Table E.13.

Table E.13: Classification of Design Flow Velocities from Main Propeller

Type of Vessel	**Flow Velocities** (m/s)
Small Yacht	0.5
Sailing Boat	0.9
Barge	1.0
International Vessel	0.9
Regional Convoy Vessel	1.3
International Convoy Vessel (1)	1.1
International Convoy Vessel (2)	1.2

Besides a main propeller, vessels could have thrusters at the bow. These are able to produce impacts by flow velocities on a wall or sloped structure and at the bed. A classification is developed for the four largest inland waterways.

Table E.14: Classification of Design Flow Velocities from Bow Thruster

Type of Vessel	**Type of Formula**	**Engine Power** (kW)	**Propeller Diameter** (m)	**Flow Velocity** (m/s)
Small Yacht	-	-	-	-
Sailing Boat	-	-	-	-
Barge	-	-	-	-
International Vessel	General Cargo	375	1.4	0.65
Regional Convoy Vessel	General Cargo	375	1.4	0.65
International Convoy Vessel (1)	Container	800	1.9	0.70
International Convoy Vessel (2)	Container	1550	2.4	0.75

The defined groups of flow velocities suffice to obtain a rough estimate of the design flow velocity. An observed or provided value can be round up to obtain a higher level of safety.

E.5. WAVES

Waves in inland waters are generated by wind and vessels. These waves are called *short waves* (wave period is shorter than 30 seconds) and the wave period is normally between the 5 and 10 seconds. The vertical accelerations in the water column cannot be neglected due to the quickly varying water level. Their character is irregular in wave height and wave period. On the other hand, there are *long waves* (wave period is longer than 30 seconds), which are mainly generated at sea (e.g. tidal waves and seiches). Due to the slow variation of the water level the vertical acceleration is approximately zero and the pressure distribution is hydrostatic (Battjes and Labeur, 2014). Regular waves do not occur in nature. As a consequence, wave height, wave period, direction and wave shape are fluctuating consistently.

E.5.1. WAVE RELATIONS

The short-crested waves are characterized by three parameters, the wave height H (m), the wave period T (s) and the wave length, L (m). These waves are also defined as propagating surface gravity waves. The wave height and wave length are not correlated. The following definition holds for the wave length (Nortier and De Koning, 1996):

$$L = cT \tag{E.12}$$

Inland waters have limited water depth. Accordingly, the wave velocity of a short wave, c (m/s), is given by the following equation:

$$c = \sqrt{\frac{gL}{2\pi}\tanh\frac{2\pi h}{L}} \tag{E.13}$$

The practical relation between the wave period and wave period in shallow waters (approximate 10 m) is:

$$T = (3.5...4)\sqrt{H} \tag{E.14}$$

A sea surface can be in- or outside the wind zone. Within a wind zone locally generated waves are found (wave steepness between 0.1 and 0.005). On the contrary, swell (wave steepness below 0.025) is the group of waves found outside the generation zone. The following definition holds:

$$wave - steepness = \frac{H}{L} \tag{E.15}$$

Due to the forcing or absence of wind the wave length becomes as follows.

$$L_{sea} = (10...20)\dot{}H \tag{E.16}$$

$$L_{swell} = 40H \tag{E.17}$$

Since all kinds of waves (i.e. long period, short period, high wave height, low wave height, etc.) are present in a wave field, one introduced a dominant wave. In literature this wave is called the significant wave, while in conversation *the* wave height is used. Study of Rayleigh on wave statistics resulted in the following significant wave heights (Table E.15).

Table E.15: Wave Height and Chance of Higher Wave Heights

Chance of higher wave height (%)	**Wave height** (m)	**Remarks**
0.1	$1.86H_s$	maximum wave height
1	$1.67H_s$	-
2	$1.40H_s$	-
10	$1.07H_s$	-
50	$0.59H_s$	mean wave height

(Nortier and De Koning, 1996)

E.5.2. WAVE DIVISION

The wave zones are characterized by the ratio of the mean water depth and wave length ratio. Deep, transition and shallow waters are considered, in which the behaviour of the larger waves is influenced by a closer bed. In Figure E.6 the definition is clarified.

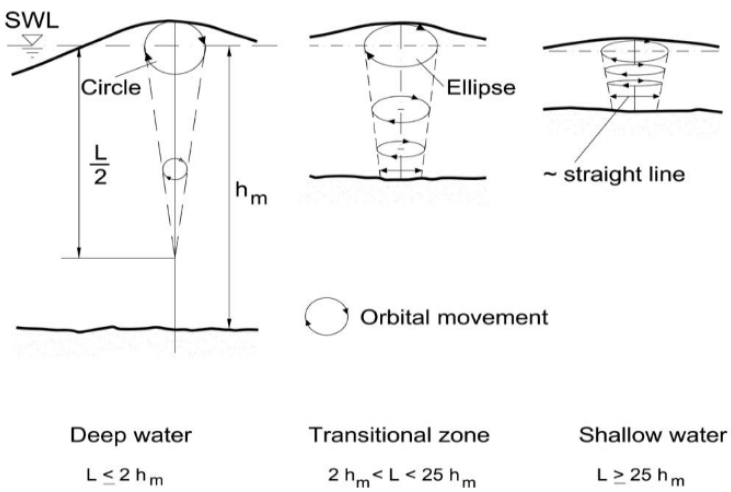


Figure E.6: Waves in Deep, Transition and Shallow Waters
(BAW, 2011)

E.5.3. WAVE MEASUREMENTS

Also wave measurements can be considered to determine these parameters. An example of the results of these measurements is found in Figure E.7.

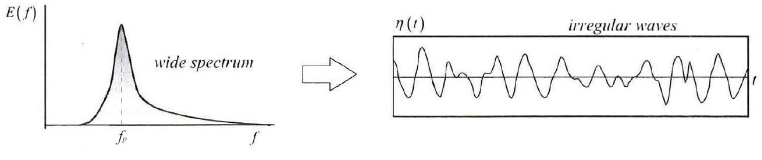


Figure E.7: Spectrum and Measurements of Irregular Waves
(Holthuijsen, 2007)

For simplicity, waves (in deep water) are characterized by two parameters, H_s (significant wave height, the average of the highest one-third measured waves) and T_p (peak wave period of a wave spectrum) (Holthuijsen, 2007). The significant wave height is determined by the following formula:

$$H_{1/3} = \frac{1}{N/3} \sum_{j=N}^{2/3N} H_j \tag{E.18}$$

$$T_p = \frac{1}{f_p} \tag{E.19}$$

In which:

$H_{1/3}$	Significant wave height (m)
N	Number of waves $(-)$
H_j	Wave height (m)
T_p	Peak wave period (s)
f_p	Peak wave frequency (Hz)

Several wave conditions are occurring over time. Therefore, short term and long term measurement could give significantly different results. To prevent over- and underestimation of the design wave, long term wave data and extreme events should be accounted for.

E.5.4. WIND-GENERATED WAVES

The formula of Young and Verhagen (1996) is developed to translate wind velocities to wave energy for a given fetch. It reads:

$$\epsilon = 3.64 \cdot 10^{-3} \{tanh(A1) tanh(\frac{B2}{tan(A1)})\}^n \tag{E.20}$$

In which:

$A1$	$= 0.292^{1/n} \delta^{1/n}$	Variable 1 (-)
$B2$	$= (4.396 \cdot 10^{-5})^{1/n} X^{1/n}$	Variable 2 (-)
ϵ	$= \frac{g^2 E}{{u_{10}}^4}$	Dimensionless total energy $(-)$
δ	$= \frac{gd}{{u_{10}}^2}$	Water depth $(-)$
X	$= \frac{gx}{{u_{10}}^2}$	Dimensionless fetch $(-)$
n	$= 1.74$	Parameter $(-)$
g	$= 9.81$	Gravitational acceleration (m/s^2)
E		Total energy (J)
u_{10}		Wind speed above 10 meters (m/s)
x		Length of fetch (m)
d		Water depth (m)

The formula replacing an older formula of Brettschneider and others (CERC, 1977). This earlier relationship becomes consistent with the JONSWAP (Joint North Sea Wave Observation Project) data (CERC, 1984).

The effective fetch is considered when the width of the lake is limited. The shape of the lake is taken into account regularly resulting in a lower fetch length (Rogala, 1997). This effective fetch length, x_i (m), is determined by taking into account several distances (Figure E.8).

$$x_{eff} = \frac{\Sigma x_i \cos \gamma_i}{\Sigma \cos gamma_i} \tag{E.21}$$

7 lines on both sides of chosen wind direction are drawn where the wind enters and leaves the water surface. The distance to land measured is x_i (m). The angle between these is 6 degrees. The deviation angle is γ_i $(degrees)$.

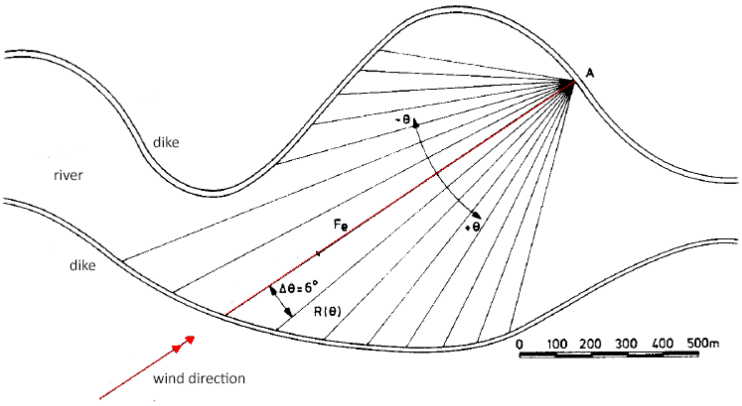


Figure E.8: Effective Fetch Distance-lines
Rijkswaterstaat (2007)

The dimensionless total energy can be rewritten as the total energy of the spectrum.

$$E = \frac{\epsilon\, u_{10}^4}{g^2} \tag{E.22}$$

The wave height, H_{m0} (m), can be calculated as follows:

$$H_{m0} = 4\sqrt{E} \tag{E.23}$$

E.5.5. VESSEL-GENERATED WAVES

In channels and lakes vessel-generated waves could dominate wind-generated waves. When vessels sail two types of waves can be distinguished, respectively primary and secondary waves. The primary waves originate from the bow and are the most visible as a result of the larger wave height, while the secondary waves from the stern are smaller in wave height (Sorensen, 1997) (Figure E.9).

The vessel-generated wave characteristics strongly depend on the size, shape and speed of the vessel. The celerity is similar to the vessel speed, but does not have the same direction. Due to the directional change the wave velocity in the propagation direction of the vessel is less. What is more, the water depth decrease to the shoreline affects the wave characteristics. These can be quantified by the Froud number (Fr).

$$c = v_v cos\theta \tag{E.24}$$

$$Fr = \frac{v_v}{\sqrt{gd}} \tag{E.25}$$

In which:

c	Wave celerity (m/s)
v_v	Vessel velocity (m/s)
θ	Directional spread (rad)
Fr	Froude number $(-)$
g	Gravitational acceleration (m/s^2)
d	Water depth (m)

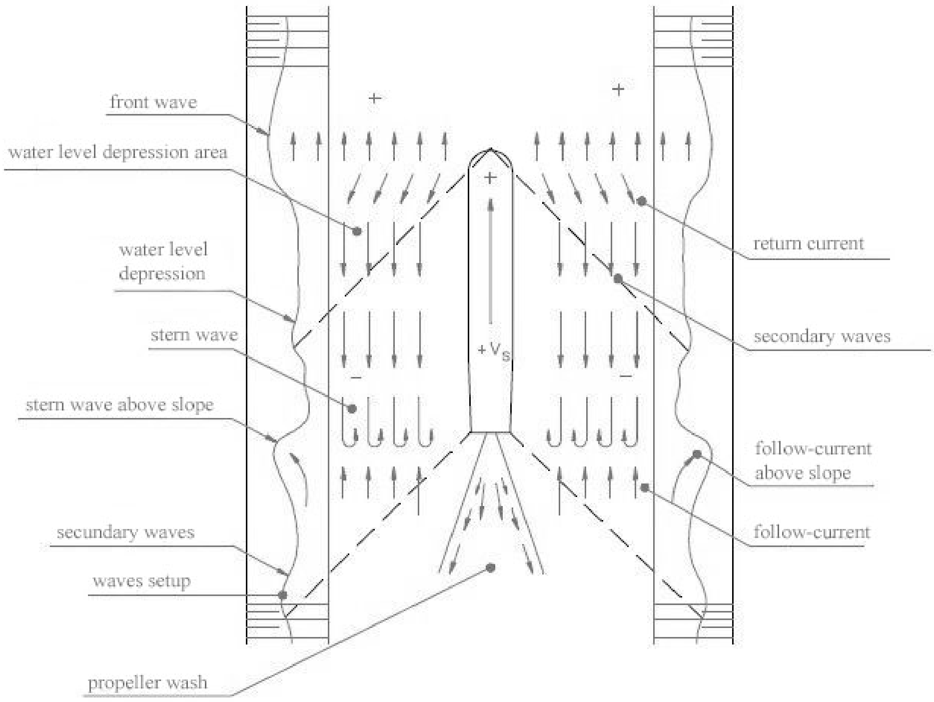


Figure E.9: Vessel-generated Waves, Currents and Water levels (2)
(Waterloopkundig Laboratorium, 1997)

Generally, at a Froude number of 0.7 waves start to interact with the bottom. Shallow water conditions are considered. Between the Froude numbers 0.7 and 1 the wave height increases significantly. Consequently, the wave will break when the number exceeds 1. The vessel, with higher speed pushes the waves into breaking with a resulting lower wave height. A Froude number equal to 1 achieves the peak wave height.

The angle of the diverging waves is calculated by the following empirical formula (Fr<1) (Sorensen and Weggel, 1984):

$$\theta = 35.27(1 - e^{12(Fr-1)}) \tag{E.26}$$

When the celerity and water depth are known the following formula can be solved to wave length (L). The wave length is directly correlated to the wave period and wave celerity. This is calculated as follows:

$$c^2 = \frac{gL}{2\pi} \tanh \frac{2d\pi}{L} \tag{E.27}$$

$$T = L/C \tag{E.28}$$

In which:

L Wave length (m)
T Wave period (s)

A lot of research is done in the determination of the wave vessel-generated wave height. Sorensen (1997) discussed eight of these prediction models with different limitations and point of discussion. Measured vessel waves and laboratory tests were conducted to obtain the influence of the: dimensions of the vessels, shape of the vessels, sailing speed, width of the channel, geometry of the channel and water depth. There is more uncertainty in the wave height prediction due to these factors and the ignorance of certain factors. The majority of the discussed formulas do not account for wave height decay from sailing line to shore and the hull geometry. Others do not consider the correct relation sailing speed and maximum wave height. Valid equations to estimate the wave height are: Gates and Herbich (1977) for large vessels in deep water (Equation E.29 for Fr<0.7); PIANC (1987) and Verheij & Bogaerts (1989) for inland ship in deep water (Equation E.30 for Fr<0.7); Sorensen and Weggel (1984 and 1986) for vessels in deep and shallow water (Equation E.32 for 0.2<Fr< 0.8). The mean wave heights are respectively:

$$H_{m,GH} = \frac{K_w B}{L_e} \frac{{v_v}^2}{2g} \tag{E.29}$$

$$H_{m,VB} = A'' d \left(\frac{S}{d}\right)^{-0.33} Fr^4 \tag{E.30}$$

$$A'' = \frac{KD}{L_e} \tag{E.31}$$

$$H_{m,SW} = \alpha_{Fr} (x^*)^n W^{0.33} \tag{E.32}$$

In which:

K_w	Saunders coefficient $(-)$
B	Maximum beam with (m)
L_e	Vessel length (m)
A''	Vessel geometry coefficient $(-)$
K	Bow geometry $(-)$
D	Draught (m)
d	Water depth (m)
W	Displacement volume (m^3)
α_{Fr}	Variable depending on the Froude number $(-)$
n	Variable as function on the dimensionless depth $(-)$
x^*	Dimensionless distance sailing to wave measurement location $(-)$

The size of a vessel is strongly related to the wave height to investigate the stability and possibility of vessel-structure damage. For wave conditions at the berths Table E.16 can be used.

Table E.16: Maximum Wave Height for Vessels at Berth

Type of Vessel	**Maximum Wave Height** (m)
Pleasure Craft	0.15 - 0.25
Fishing Vessels	0.40
Dredges and dredge barges	0.80 - 1.00
General Cargo (<30,000 DWT)	1.00 - 1.25
Dry Bulk Cargo (<30,000 DWT)	1.00 - 1.25
Dry Bulk Cargo (up to <100,000 DWT)	1.50
Oil Tankers	1.00 - 1.25
Oil Tankers	1.50 - 2.50
Oil Tankers	2.50 - 3.00
Passenger Vessels	0.70

(Schiereck et al., 2012)

E.5.6. WAVE BREAKING CRITERIA

Breaking waves in shallow water are depth-induced. On the contrary, deep water waves break due to a limit of the wave steepness. An outdated rule of thumb of engineers for the conditions under which waves break in shallow water is for regular waves:

$$H_{br} > 0.75d \tag{E.33}$$

For irregular waves, the following wave height of the breaking wave is:

$$H_{br} > 0.4d \tag{E.34}$$

After breaking the wave height changes which is in contrast with the wave period. The significant wave height becomes:

$$H_s \approx \frac{1}{2}d \tag{E.35}$$

This equation is conservative and does not include the wave steepness ($s = H/L$). Research found that the depth, bottom slope and the wave steepness are strongly related. A higher water depth delays the wave height of breaking. In other words, a wave 'feels the bottom' later in case of larger depth.

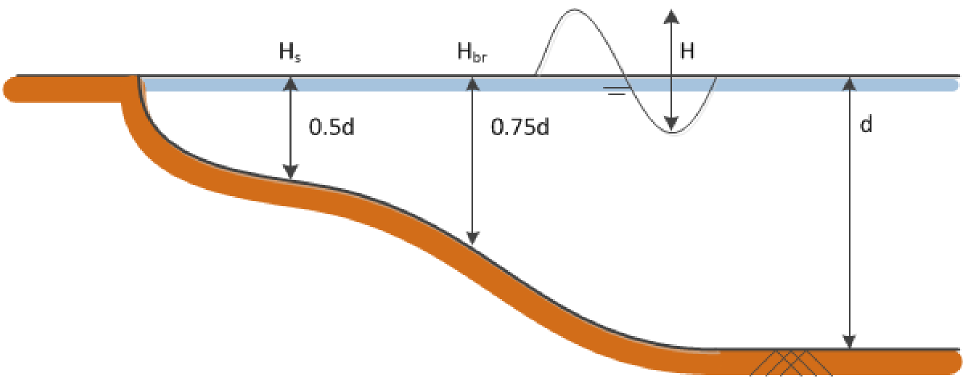


Figure E.10: Bathymetry and Behaviour of the Waves

Additionally, mild slopes will cause spilling waves, while steep slopes provide the waves to plunge. The plunging waves are accompanied by high impact. Surging wave do not occur in lake unless the water is deep and the slopes are steep.

The criteria of the wave breaking caused by steepness and water depth (CIRIA, CUR, CETMEF, 2007) are respectively,

$$\frac{H}{L} \leq [\frac{H}{L}]_{max} = 0.14 \tanh 2\pi \frac{H}{L} \tag{E.36}$$

$$\frac{H}{L} \leq \gamma_{br} = [\frac{H}{h}]_{max} = \frac{H_b}{h_b} \tag{E.37}$$

E.5.7. WAVE DATA

Designing hydraulic structures long term wave information is required. The design process is regularly short in such a way that the designing company collects wave information itself. Easy access and reliable information of waves, all over the world, can be found in meteorological institutes or wave data banks. In this matter data for inland waterways is seldomly available.

E.5.8. DESIGN WAVE

The design wave is determined by the classes of the design wind and vessels. To obtain a set of wind-generated wave properties typical fetches within The Netherlands has been defined. The size of these lakes vary between 500-1000 m (Veluwemeer) and 30000 m (IJsselmeer). What is more, rivers (for instance, River Rhine and Meuse) and other small waters have widths between approximately 50 and 500 m. This results in the classification of the fetch length in Table E.17.

Table E.17: Classification of the Fetch Length

Type of System	**Fetch Length (m)**
Small River	50
Medium River	100
Large River	500
Small Lake	1000
Medium Lake	5000
Large Lake	10000
Largest Lake	30000

Consequently, for water depths of 1 to 4 m and fetch lengths between 50 and 30000 m the wave height wind-generated waves are determined by Equations E.20, E.22 and E.23. The results are shown in Table E.18, E.19, E.20 and E.21.

Table E.18: Dimensionless Wave Energy and Wave Height of Wind-Generated Waves u_{10} = 15

∈ (-)		**d (m)**	1	1.5	2	2.5	3	3.5	4
		delta (-)	0.0436	0.0654	0.0872	0.109	0.1308	0.1526	0.1744
x (m)	**X (-)**	**B2 / A1**	0.0814	0.1028	0.1213	0.1379	0.1531	0.1673	0.1807
50	2.18	0.005	3.5E-07	3.4E-07	3.4E-07	3.4E-07	3.4E-07	3.4E-07	3.4E-07
100	4.36	0.007	6.9E-07	6.9E-07	6.8E-07	6.8E-07	6.8E-07	6.7E-07	6.7E-07
500	21.80	0.018	3.4E-06	3.4E-06	3.4E-06	3.4E-06	3.4E-06	3.4E-06	3.3E-06
1000	43.60	0.027	6.5E-06	6.6E-06	6.7E-06	6.7E-06	6.7E-06	6.7E-06	6.6E-06
5000	218.00	0.069	2.4E-05	2.7E-05	2.9E-05	3.0E-05	3.0E-05	3.1E-05	3.1E-05
10000	436.00	0.103	3.5E-05	4.3E-05	4.8E-05	5.1E-05	5.4E-05	5.5E-05	5.7E-05
30000	1308.00	0.194	4.5E-05	6.4E-05	7.9E-05	9.3E-05	1.0E-04	1.1E-04	1.2E-04
H_{m0} (m)		**x (m) / d (m)**	1	1.5	2	2.5	3	3.5	4
		50	0.05	0.05	0.05	0.05	0.05	0.05	0.05
		100	0.08	0.08	0.08	0.08	0.08	0.08	0.08
		500	0.17	0.17	0.17	0.17	0.17	0.17	0.17
		1000	0.23	0.24	0.24	0.24	0.24	0.24	0.24
		5000	0.45	0.48	0.49	0.50	0.51	0.51	0.51
		10000	0.54	0.60	0.64	0.66	0.67	0.68	0.69
		30000	0.61	0.73	0.82	0.88	0.93	0.97	1.00

Table E.19: Dimensionless Wave Energy and Wave Height of Wind-Generated Waves u_{10} = 20

∈ (-)		**d (m)**	1	1.5	2	2.5	3	3.5	4
		delta (-)	0.0245	0.0368	0.0491	0.0613	0.0736	0.0858	0.0981
x (m)	**X (-)**	**B2 / A1**	0.0585	0.0739	0.0871	0.0991	0.1100	0.1202	0.1298
50	1.23	0.004	2.0E-07	1.9E-07	1.9E-07	1.9E-07	1.9E-07	1.9E-07	1.9E-07
100	2.45	0.005	3.9E-07	3.9E-07	3.9E-07	3.9E-07	3.9E-07	3.9E-07	3.8E-07
500	12.26	0.013	1.9E-06	1.9E-06	1.9E-06	1.9E-06	1.9E-06	1.9E-06	1.9E-06
1000	24.53	0.020	3.7E-06	3.7E-06	3.8E-06	3.8E-06	3.8E-06	3.8E-06	3.8E-06
5000	122.63	0.050	1.4E-05	1.5E-05	1.6E-05	1.7E-05	1.7E-05	1.8E-05	1.8E-05
10000	245.25	0.074	2.0E-05	2.4E-05	2.7E-05	2.9E-05	3.1E-05	3.2E-05	3.2E-05
30000	735.75	0.139	2.5E-05	3.6E-05	4.5E-05	5.2E-05	5.9E-05	6.4E-05	6.8E-05
H_{m0} **(m)**		**x (m) / d (m)**	1	1.5	2	2.5	3	3.5	4
		50	0.07	0.07	0.07	0.07	0.07	0.07	0.07
		100	0.10	0.10	0.10	0.10	0.10	0.10	0.10
		500	0.22	0.23	0.23	0.23	0.23	0.23	0.23
		1000	0.31	0.32	0.32	0.32	0.32	0.32	0.32
		5000	0.60	0.64	0.66	0.67	0.68	0.68	0.69
		10000	0.72	0.80	0.85	0.88	0.90	0.92	0.93
		30000	0.82	0.98	1.09	1.18	1.25	1.30	1.35

Table E.20: Dimensionless Wave Energy and Wave Height of Wind-Generated Waves u_{10} = 25

∈ (-)		**d (m)**	1	1.5	2	2.5	3	3.5	4
		delta (-)	0.0157	0.0235	0.0314	0.0392	0.0471	0.0549	0.0628
x (m)	**X (-)**	**B2 / A1**	0.0453	0.0572	0.0674	0.0767	0.0851	0.0930	0.1004
50	2.18	0.005	3.5E-07	3.5E-07	3.5E-07	3.5E-07	3.5E-07	3.4E-07	3.4E-07
100	4.36	0.007	6.9E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07
500	21.8	0.018	3.2E-06	3.3E-06	3.3E-06	3.4E-06	3.4E-06	3.4E-06	3.4E-06
1000	43.6	0.027	5.7E-06	6.1E-06	6.3E-06	6.4E-06	6.5E-06	6.6E-06	6.6E-06
5000	218	0.069	1.4E-05	1.8E-05	2.1E-05	2.3E-05	2.5E-05	2.6E-05	2.7E-05
10000	436	0.103	1.6E-05	2.3E-05	2.8E-05	3.3E-05	3.6E-05	4.0E-05	4.2E-05
30000	1308	0.194	1.7E-05	2.5E-05	3.3E-05	4.1E-05	4.8E-05	5.5E-05	6.2E-05
H_m0 **(m)**		**x (m) / d (m)**	1	1.5	2	2.5	3	3.5	4
		50	0.15	0.15	0.15	0.15	0.15	0.15	0.15
		100	0.21	0.21	0.21	0.21	0.21	0.21	0.21
		500	0.45	0.46	0.46	0.47	0.47	0.47	0.47
		1000	0.61	0.63	0.64	0.65	0.65	0.65	0.66
		5000	0.96	1.09	1.17	1.23	1.27	1.30	1.32
		10000	1.02	1.21	1.35	1.46	1.54	1.60	1.66
		30000	1.04	1.27	1.46	1.62	1.77	1.89	2.00

Table E.21: Dimensionless Wave Energy and Wave Height of Wind-Generated Waves $u_{10} = 30$

∊ (-)		**d (m)**	1	1.5	2	2.5	3	3.5	4
		delta (-)	0.0109	0.0164	0.0218	0.0273	0.0327	0.03815	0.0436
x (m)	**X (-)**	**B2 / A1**	0.0367	0.0463	0.0547	0.0622	0.0690	0.0754	0.0814
50	2.18	0.005	3.4E-07	3.5E-07	3.5E-07	3.5E-07	3.5E-07	3.5E-07	3.5E-07
100	4.36	0.007	6.8E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07	6.9E-07
500	21.8	0.018	3.0E-06	3.2E-06	3.3E-06	3.3E-06	3.3E-06	3.3E-06	3.4E-06
1000	43.6	0.027	5.2E-06	5.8E-06	6.1E-06	6.2E-06	6.4E-06	6.4E-06	6.5E-06
5000	218	0.069	1.1E-05	1.5E-05	1.8E-05	2.0E-05	2.2E-05	2.3E-05	2.4E-05
10000	436	0.103	1.1E-05	1.7E-05	2.1E-05	2.5E-05	2.9E-05	3.2E-05	3.5E-05
30000	1308	0.194	1.2E-05	1.7E-05	2.3E-05	2.9E-05	3.4E-05	4.0E-05	4.5E-05
H_m0 **(m)**		**x (m) / d (m)**	1	1.5	2	2.5	3	3.5	4
		50	0.22	0.22	0.22	0.22	0.22	0.22	0.22
		100	0.30	0.30	0.30	0.30	0.30	0.30	0.30
		500	0.64	0.65	0.66	0.67	0.67	0.67	0.67
		1000	0.84	0.88	0.90	0.92	0.92	0.93	0.94
		5000	1.20	1.40	1.54	1.63	1.71	1.76	1.80
		10000	1.24	1.50	1.70	1.85	1.98	2.08	2.17
		30000	1.25	1.53	1.76	1.97	2.15	2.31	2.46

In which:

$g \quad = 9.81\ m/s^2$
$n \quad = 1.74\ (-)$

The results vary between 0.1 and 2.5 m of wave height, in which the wave-steepness is assumed to be 4%. A classification of the significant wave heights by wind is provided in Table E.22.

Table E.22: Classification of Wind-generated Wave Heights (H_{m0})

Property	**Wave Height** (m)	**Wave Length** (m)	**Wave Period** (s)
Lowest	0.25	6.25	2.0
Mean Low	0.50	12.50	2.8
Low	1.00	25.00	4.0
Mean	1.50	37.50	4.9
High	1.75	43.75	5.3
Mean High High	2.00	50.00	5.7
Highest	2.50	62.50	6.3

Verheij & Bogaerts (1989) has established an appropriate formula for the wave height of vessel-generated waves in inland waters (Equation E.30). In Table E.23, E.24, E.25 and E.26 the vessel types are subjected to the classified water depth.

Table E.23: Wave Height of Vessel-Generated Waves S = 10 m

Type of Vessel	**A" (-)**	**V (m/s)**	**Fr (-) / d (m)**	1	1.5	2	2.5	3	3.5	4
Small Yacht	1	5.5	0.6	0.11	0.17	0.22	0.28	0.34	0.39	0.45
Sailing Boat	1	5.5	0.6	0.11	0.17	0.22	0.28	0.34	0.39	0.45
Barge	0.5	7.5	0.8	0.06	0.08	0.11	0.14	0.17	0.20	0.22
International Vessel	0.5	7.5	0.8	0.06	0.08	0.11	0.14	0.17	0.20	0.22
Regional Convoy Vessel	0.5	7.5	0.8	0.06	0.08	0.11	0.14	0.17	0.20	0.22
International Convoy Vessel (1)	0.5	7.5	0.8	0.06	0.08	0.11	0.14	0.17	0.20	0.22
International Convoy Vessel (2)	0.5	7.5	0.8	0.06	0.08	0.11	0.14	0.17	0.20	0.22

Table E.24: Wave Height of Vessel-Generated Waves S = 25 m

Type of Vessel	**A" (-)**	**V (m/s)**	**Fr (-) / d (m)**	1	1.5	2	2.5	3	3.5	4
Small Yacht	1	5.5	0.6	0.08	0.12	0.17	0.21	0.25	0.29	0.33
Sailing Boat	1	5.5	0.6	0.08	0.12	0.17	0.21	0.25	0.29	0.33
Barge	0.5	7.5	0.8	0.04	0.06	0.08	0.10	0.12	0.15	0.17
International Vessel	0.5	7.5	0.8	0.04	0.06	0.08	0.10	0.12	0.15	0.17
Regional Convoy Vessel	0.5	7.5	0.8	0.04	0.06	0.08	0.10	0.12	0.15	0.17
International Convoy Vessel (1)	0.5	7.5	0.8	0.04	0.06	0.08	0.10	0.12	0.15	0.17
International Convoy Vessel (2)	0.5	7.5	0.8	0.04	0.06	0.08	0.10	0.12	0.15	0.17

Table E.25: Wave Height of Vessel-Generated Waves S = 50 m

Type of Vessel	**A" (-)**	**V (m/s)**	**Fr (-) / d (m)**	1	1.5	2	2.5	3	3.5	4
Small Yacht	1	5.5	0.6	0.07	0.10	0.13	0.17	0.20	0.23	0.26
Sailing Boat	1	5.5	0.6	0.07	0.10	0.13	0.17	0.20	0.23	0.26
Barge	0.5	7.5	0.8	0.03	0.05	0.07	0.08	0.10	0.12	0.13
International Vessel	0.5	7.5	0.8	0.03	0.05	0.07	0.08	0.10	0.12	0.13
Regional Convoy Vessel	0.5	7.5	0.8	0.03	0.05	0.07	0.08	0.10	0.12	0.13
International Convoy Vessel (1)	0.5	7.5	0.8	0.03	0.05	0.07	0.08	0.10	0.12	0.13
International Convoy Vessel (2)	0.5	7.5	0.8	0.03	0.05	0.07	0.08	0.10	0.12	0.13

Table E.26: Wave Height of Vessel-Generated Waves S = 100 m

Type of Vessel	**A" (-)**	**V (m/s)**	**Fr (-) / d (m)**	1	1.5	2	2.5	3	3.5	4
Small Yacht	1	5.5	0.6	0.05	0.08	0.11	0.13	0.16	0.18	0.21
Sailing Boat	1	5.5	0.6	0.05	0.08	0.11	0.13	0.16	0.18	0.21
Barge	0.5	7.5	0.8	0.03	0.04	0.05	0.07	0.08	0.09	0.11
International Vessel	0.5	7.5	0.8	0.03	0.04	0.05	0.07	0.08	0.09	0.11
Regional Convoy Vessel	0.5	7.5	0.8	0.03	0.04	0.05	0.07	0.08	0.09	0.11
International Convoy Vessel (1)	0.5	7.5	0.8	0.03	0.04	0.05	0.07	0.08	0.09	0.11
International Convoy Vessel (2)	0.5	7.5	0.8	0.03	0.04	0.05	0.07	0.08	0.09	0.11

Remarkably, the maximum wave height generated by wind and vessel have a significant difference of 2 m. But since lower wind velocities have been found, the vessel-generated waves could still dominate. The variation in wave height as a result of sailing vessels is between 0.1 and 0.5 m. Also for these waves the wave-steepness is assumed 4%. This result in the following classification.

Table E.27: Classification of Vessel-generated Wave Heights ($H_{m,VB}$)

Property	**Wave Height** (m)	**Wave Length** (m)	**Wave Period** (s)
Lowest	0.10	2.50	1.3
Low	0.20	5.00	1.8
Mean	0.30	7.50	2.2
High	0.50	12.50	2.8

In which:

Tugs, patrol and inland motor boats	$A'' = 1$
Europian barges	$A'' = 1$

In Table E.28 the depth-induced breaking wave criterion is computed by Equation E.34. Breaking and no breaking is shown as respectively 1 and 0. One can observe that the highest wind-generated waves break in the more shallow waters. Accordingly, wave heights of 1 m or higher should be analysed and if necessary the accompanied significant wave height should be lowered.

Table E.28: Breaking Wave Criterium (0 = no breaking wave, 1 = breaking wave)

$d(m)$ / $H_0(m)$	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00	2.50
1.0	0	0	0	0	1	1	1	1	1	1
1.5	0	0	0	0	0	1	1	1	1	1
2.0	0	0	0	0	0	1	1	1	1	1
2.5	0	0	0	0	0	0	1	1	1	1
3.0	0	0	0	0	0	0	1	1	1	1
3.5	0	0	0	0	0	0	1	1	1	1
4.0	0	0	0	0	0	0	0	1	1	1

After the breaking of waves the significant wave height is changed. In Table E.29 the adjusted wave heights are shown. These are computed according to Equation E.35 One can observe that wave heights of 2.5 m are not present due to the maximum classified water depth of 4 m. Therefore, a maximum wave height of 2 m is considered.

Table E.29: Significant Wave Height (after Breaking)

$d(m)$ / $H_0(m)$	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00	2.50
1.0	0.10	0.20	0.25	0.30	0.50	0.50	0.50	0.50	0.50	0.50
1.5	0.10	0.20	0.25	0.30	0.50	0.75	0.75	0.75	0.75	0.75
2.0	0.10	0.20	0.25	0.30	0.50	1.00	1.00	1.00	1.00	1.00
2.5	0.10	0.20	0.25	0.30	0.50	1.00	1.25	1.25	1.25	1.25
3.0	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.50	1.50	1.50
3.5	0.10	0.20	0.25	0.30	0.50	1.00	1.75	1.75	1.75	1.75
4.0	0.10	0.20	0.25	0.30	0.50	1.00	1.50	2.00	2.00	2.00

The final classification of the design wave height becomes:

Table E.30: Classification of All Wave Heights (H_{m0})

Property	**Wave Height** (m)	**Wave Length** (m)	**Wave Period** (s)
Lowest	0.10	2.50	1.3
Low Low	0.20	5.00	1.8
Low	0.25	6.25	2.0
Mean Low	0.30	7.50	2.2
Mean	0.50	12.50	2.8
Mean High	1.00	25.00	4.0
High	1.50	37.50	4.9
High High	1.75	43.75	5.3
Highest	2.00	50.00	5.7

The defined groups of wave heights suffice to obtain a rough estimate of the design wave height. An observed or provided value can be round up to obtain a higher level of safety.

E.6. ICE

Ice is found in lakes and canals where flow velocities are low (close to 0 m/s) and temperatures are below zero degrees. Also the wind, cloudiness, water depth and location of the water is a factor of influence (KNMI, 2013). Long periods of these circumstances will thicken the layer of ice, which is increasing its mass, strength and stiffness.

E.6.1. ICE BEHAVIOUR

There are four relevant phenomena of ice which are: thermal expansion, ice accumulation, collision and ice attachment. Ice accumulation and collision are the most relevant for breakwater water designs, while expansion and attachment of ice are mainly considered for single or multiple piled structures, like oil platforms.

Various periods of cold weather conditions have been present the past decades in The Netherlands. However, due to climate change the winters are less cold and significant ice formation is not regularly seen. Due to all the changes there is still a chance that during a cold weather period, thick ice layers develop and be a threat for hydraulic structures.

E.6.2. ICE DATA

Ice data can be found in national institutes of meteorology. These gather the amount of cold days and the correlations to the ice thickness. Also a long term estimate of ice behaviour is to be expected in the nearby future.

E.6.3. DESIGN ICE THICKNESS

In the Dutch lakes and rivers, the ice thickness fluctuated severely. For example, in 1963 the maximum thickness of ice measured was approximately 0.40 m. Sub-sequentially, in 1996 and 1997 it was respectively 0.25 and 0.32 m. After the year 2000, researchers found a decrease in the development of ice during the winter period. It became more or less stable. The ice thickness has been approximately 0.10 m since. Therefore, the following classification is suggested.

Table E.31: Classification of Design Ice Thickness

Property	**Ice Thickness** (m)
Lowest	0.1
Low	0.2
Mean	0.3
High	0.4

The defined groups of ice thickness suffice to obtain a rough estimate of the design ice thickness. An observed or provided value can be round up to obtain a higher level of safety.

E.7. SUBSOIL

To assess the bearing capacity and the macro stability of breakwaters, geotechnical data is required. The settlements as well as erosion and scour have to be estimated with regard to the required crest level.

E.7.1. GEOTECHNICAL PARAMETERS

An impression of the basic geotechnical parameters is given in Table E.32. Each location has its unique soil profile. These profiles can be obtained by CPT- and SPT-penetration tests or boring holes for laboratory analysis.

Table E.32: Geotechnical Parameters

Name	**Symbol**
Grain classification/size	D
Piezometric pressure	p
Permeability	k
Dry/wet density	ρ_{unsat}, ρ_{sat}
Relative density/porosity	n
Drained shear strength	c,ϕ
Undrained shear strength	s_u
Compressibility	C_c, C_s
Consolidation coefficient	c_v
Moduli of elasticity	G, E
In sity stress	σ
Stress history	OCR
Stress/strain curve	G,E

(CIRIA, CUR, CETMEF, 2007)

E.7.2. GEOLOGICAL MAP

Over the years soil data is obtained over The Netherlands. These give an approximation of the soil profile for Dutch project locations, which is found in Figure E.11. 10 soil types are to be found in the figure. The soils are derived from a soil sample 0.8 m below ground level. The accuracy of the measurements is 0.10 to 0.25 m. Peat, sand, clay and loam subsoils are dominant in more than 50% of the sample. The grounds of peat in the Eastern part of the Netherlands changed by 47%. Between 2001 and 2004 investigation showed that the soil type was adjusted. What is more, due to oxidation of peat the ground levels decrease. In the drawing these ground are counted among mineral grounds.

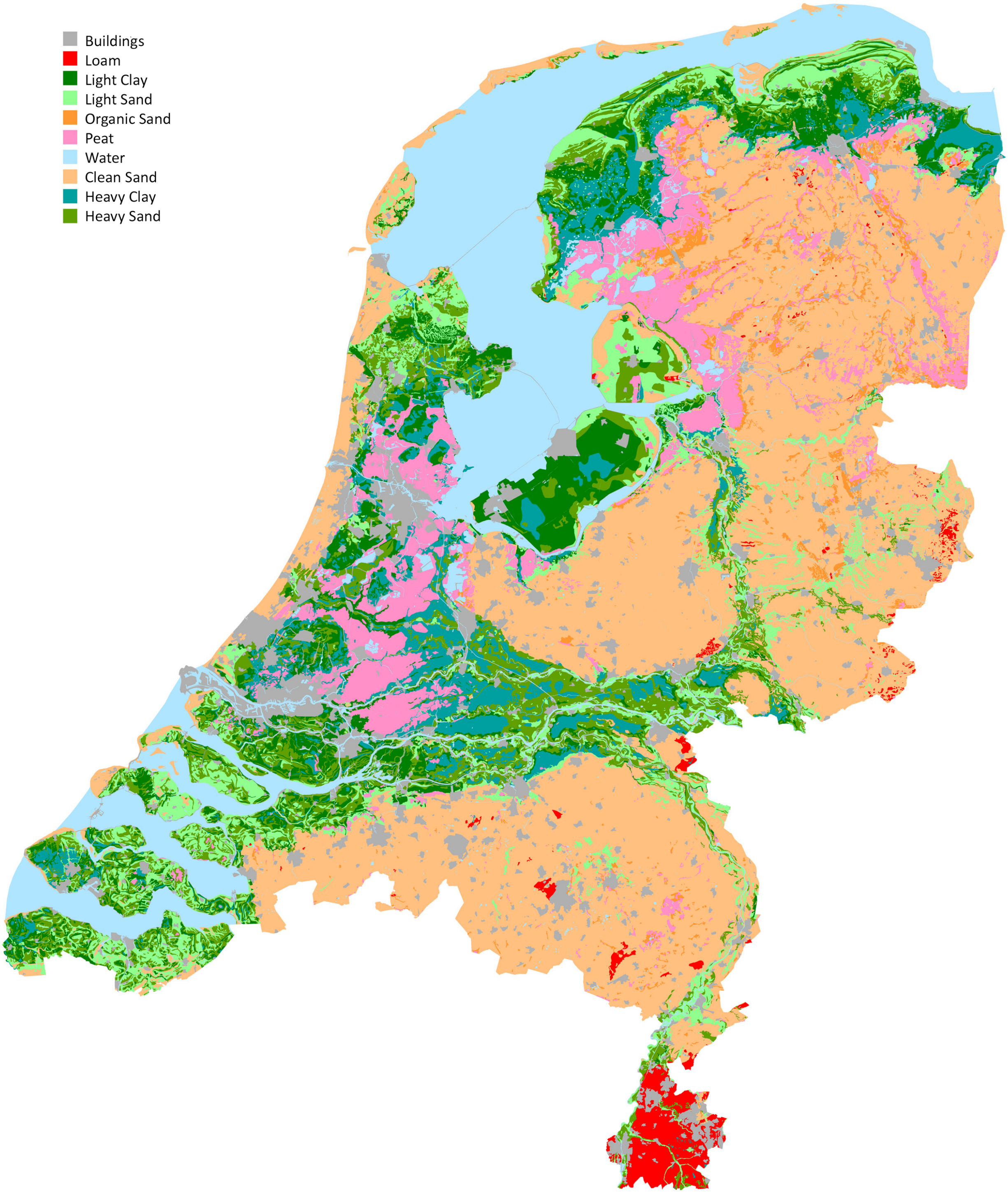


Figure E.11: Soil map of The Netherlands
(Wagingen UR Alterra. (2014). *Grondsoortenkaart 2006 - Simplified Soil Map of the Netherlands*. Retrieved from http://www.wageningenur.nl/. Accessed on September 30, 2014.)

Soil Type					Soil Characteristics									
Name	**Division**	**Coherence**	**γ (kN/m)**	**γ_{sat} (kN/m)**	**q_c (MPa)**	**C'_p**	**C'_a**	**$C_c/(1+e_0)$ (-)**	**C_a (-)**	**$C_{sw}/(1+e_0)$ (-)**	**E_{100} (MPa)**	**φ (°)**	**c' (kPa)**	**c_u (kPa)**
Gravel	Weak silty	Loose	17	19	15	500	∞	0.0046	0	0.0015	45	32.5	-	-
		Poor	18	20	25	1000	∞	0.0023	0	0.0008	75	35.0	-	-
		Compacted	19 20	21 22	30	1200 1400	∞	0.0019 0.0016	0	0.0006 0.0005	90 105	37.5 40.0	-	-
	Strong silty	Loose	18	20	10	400	∞	0.0058	0	0.0019	30	30.0	-	-
		Poor	19	21	15	600	∞	0.0038	0	0.0013	45	32.5	-	-
		Compacted	20 21	22 22.5	25	1000 1500	∞	0.0023 0.0015	0	0.0008 0.0005	75 110	35.0 40.0	-	-
Sand	Clean	Loose	17	19	5	200	∞	0.0115	0	0.0038	15	30.0	-	-
		Poor	18	20	15	600	∞	0.0038	0	0.0013	45	32.5	-	-
		Compacted	19 20	21 22	25	1000 1500	∞	0.0023 0.0015	0	0.0008 0.0005	75 110	35.0 40.0	-	-
	Weak silty clay	-	18 19	20 21	12	450 650	∞	0.0051 0.0035	0	0.0017 0.0012	35 50	27.0 32.5	-	-
	Strong silty clay	-	18 19	20 21	8	200 400	∞	0.0115 0.0058	0	0.0038 0.0019	15 30	25.0 30.0		
Loam	Weak sandy	Soft	19	19	1	25	650	0.0920	0.0037	0.0307	2	27.5 30.0	0	50
		Poor	20	20	2	45	1300	0.0511	0.0020	0.0170	3	27.5 32.5	1	100
		Compacted	21 23	21 22	3	70 100	1900 2500	0.0329 0.0230	0.0013 0.0009	0.0110 0.0077	5 7	27.5 35.0	2.5 3.8	200 300
	Strong sandy	-	19 20	19 20	2	45 70	1300 2000	0.0511 0.0329	0.0020 0.0013	0.0170 0.0110	3 5	27.5 35.0	0 1	50 100
Clay	Clean	Soft	14	14	0.5	7	80	0.3286	0.0131	0.1095	1	17.5	0	25
		Poor	17	17	1.0	15	160	0.1533	0.0061	0.0511	2	17.5	5	50
		Compacted	19 20	19 20	2.0	25 30	320 500	0.0920 0.0767	0.0037 0.0031	0.0307 0.0256	4 10	17.5 25.0	13 15	100 200
	Weak sandy	Soft	15	15	0.7	10	110	0.2300	0.0092	0.0767	1.5	22.5	0	40
		Poor	18	18	1.5	20	240	0.1150	0.0046	0.0383	3	22.5	5	80
		Compacted	20 21	20 21	2.5	30 50	400 600	0.0767 0.0460	0.0031 0.0018	0.0256 0.0153	5 10	22.5 27.5	13 15	120 170
	Strong Sandy	-	18 20	18 20	1.0	25 140	320 1680	0.0920 0.0164	0.0037 0.0007	0.0307 0.0055	2 5	27.5 32.5	0 1	0 10
	Organic	Soft	13	13	0.2	7.5	30	0.3067	0.0153	0.1022	0.5	15.0	0 1	10
		Poor	15 16	15 16	0.5	10 15	40 60	0.2300 0.1533	0.0115 0.0077	0.0767 0.0511	1.0 2.0	15.0	0 1	25 30
Peat	Not preloaded	Soft	10 12	10 12	0.1	5 7.5	20 30	0.4600 0.3067	0.0230 0.0153	0.1533 0.1022	0.2 0.5	15.0	1 2.5	10 20
	Poor preloaded	Poor	12 13	12 13	0.2	7.5 10	30 40	0.3067 0.2300	0.0153 0.0115	0.1022 0.0767	0.5 1.0	15.0	2.5 5	20 30
Coefficient of Variation			0.05	0.05	-	0.25	0.25	0.25	0.25	0.25	0.25	0.10	0.20	0.20

Table E.33: Soil Types and Characteristics
(NEN, 2012)

E.7.3. SUBGRADE REACTION

An approximation of the soil-structure interaction can be achieved by the model of subgrade reaction, also known by the name "Winkler Springs Model". In this model the subsoil is simplified as a system of distributed springs (LUSAS, 2013).

Table E.34: Modulus of Subgrade Reaction (Terzaghi, 1955)

Type of Soil	**Characteristics**	**Modulus of Vertical Subgrade Reaction** (N/mm^3)
Cohesion Soils	Stiff	0.023
	Very stiff	0.045
	Hard	0.090
Cohesionless Soils	Loose	0.080
	Medium	0.250
	Dense	1.000

Typical values for the modulus of vertical subgrade reaction for a square plate 1ft x 1ft (k_{s1}), are shown Table E.34.

E.7.4. GEO DATA

Other ways to obtain geological data is by scientific libraries, which collect geotechnical-survey included in researches. Also national geographical services and universities have data.

E.7.5. DESIGN SOIL

Eight soil types are found in Figure E.11. Relating this types to the Eurocode leads to the following list of soil characteristics.

Table E.35: Classification of Design Soil

Name	**Eurocode Class**	**Modulus of Vertical Subgrade Reaction** (N/mm^3)
Loam	Loam: Weak sandy compacted	0.023
Light Clay	Clay: weak sandy compacted	0.045
Light Sand	Sand: weak silty clay	0.080
Organic Sand	Clay: organic soft	0.023
Peat	Peat: not preloaded	0.000
Clean Sand	Sand: clean compacted	0.025
Heavy Clay	Clay: strong sandy	0.090
Heavy Sand	Gravel: weak silty compacted	1.000

The defined groups of subsoil suffice to obtain a rough estimate of the design soil characteristics.

E.8. EARTHQUAKES

Earthquakes are to be found all over the world with varying causes and magnitudes, which could cause the collapse of hydraulic structures. This section discusses the incorporation of earthquake impact on breakwaters.

E.8.1. SEISMIC ACTIVITY

The kinds of earthquake to be recognized are tectonic, volcanic, collapse, explosion earthquakes (Liang, 2011). The tectonic and collapse earthquake are relevant for The Netherlands, which is due to the absence of vulcanos. Furthermore, explosions of chemical plants, nuclear devices or instruments of war is rare and more applicable in abroad. On the other hand, tectonic activity is scientifically proved to be present. The cause is the sliding of tectonic plates over each other, affecting large areas with intense vibrations. More regional earthquakes originate from the collapse of mines, caverns and gas/oil fields (due to the extraction of the fluids).

The overall seismic activity causes short lasting accelerations in both vertical as horizontal direction. A seismograph can determine the magnitudes of the vibration at certain distances to the epicenter. These significant vibrations will affect the stability of many structure attached to the ground. Particularly, buildings in Japan are designed with sufficient earthquake resistant.

E.8.2. SEISMIC SCALES

The intensity of an earthquake can be measured at various scales. In The Netherlands the scale of Richter is broadly used, while in Europe the European Marco-seismic Scale (EMS) is often applied. A third scale is the internationally accepted Mercalli scale.

E.8.3. EARTHQUAKE SCALE UNITY

Table E.36: Scale of Richter, Mercalli and EMS

Scale of Richter	**Scale of Mercalli**	**Acceleration (cm/s^2)**	**EMS Specification**
<2	I-II	<1.7	Not felt by most people.
3	III	1.7-13.7	Felt indoors by some people.
4	IV-V	13.7-90.3	Felt by most people; dishes rattle, some break.
5	VI-VII	90.3-333.5	Felt by all; many windows and some masonry cracks or falls.
6	VII-IX	333.5-1216.4	People frightened; most chimneys fall; major damage to poorly built structures.
7	X	>1216.4	People panic; most masonry structures and bridhes destroyed.
8	XII		Nearly total damage to masonry structures; major damage to bridges, dams; rails bend.
9	>XII		Nearly total destruction; people see ground surface move in waves; objects thrown into air.

(Crondall Weather. (2005). *Earthquakes and Earthquake Tracker for the UK area.* Retrieved from http://www.crondallweather.co.uk/. Accessed on November 17, 2014.)

E.8.4. EUROPEAN MACRO-SEISMIC SCALE

Table E.37: European Macro-seismic Scale (EMS)

EMS	Specification	Observations
1	Not felt	Not felt, even under the most favourable circumstances.
2	Scarcely felt	Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.
3	Weak	The vibration is weak and is felt indoors by a few people. People at rest feel a swaying or light trembling.
4	Largely observed	The earthquake is felt indoors by many people, outdoors by very few. A few people are awakened. The level of vibration is not frightening. Windows, doors and dishes rattle. Hanging objects swing. The earthquake is felt indoors by most, outdoors by few. Many sleeping people awake. A few run outdoors. Buildings tremble throughout. Hanging objects swing considerably. China and glasses clatter together. The vibration is strong. Top heavy objects topple over. Doors and windows swing open or shut.
5	Strong	The earthquake is felt indoors by most, outdoors by few. Many sleeping people awake. A few run outdoors. Buildings tremble throughout. Hanging objects swing considerably. China and glasses clatter together. The vibration is strong. Top heavy objects topple over. Doors and windows swing open or shut.
6	Slightly damaging	Felt by most indoors and by many outdoors. Many people in buildings are frightened and run outdoors. Small objects fall. Slight damage to many ordinary buildings e.g. fine cracks in plaster and small pieces of plaster fall.
7	Damaging	Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many ordinary buildings suffer moderate damage: small cracks in walls; partial collapse of chimneys. Furniture may be overturned. Many ordinary buildings suffer damage: chimneys fall; large cracks appear in walls and a few buildings may partially collapse.
8	Heavily damaging	Furniture may be overturned. Many ordinary buildings suffer damage: chimneys fall; large cracks appear in walls and a few buildings may partially collapse.
9	Destructive	Monuments and columns fall or are twisted. Many ordinary buildings partially collapse and a few collapse completely.
10	Very destructive	Many ordinary buildings collapse.
11	Devastating	Most ordinary buildings collapse.
12	Completely devastating	Practically all structures above and below ground are heavily damaged or destroyed.

(Grunthal, 1998)

E.8.5. EARTHQUAKE MAP OF THE NETHERLANDS

An earthquake map of The Netherlands is also found Figure E.12 and Figure E.13. It provides the EMS classes with a return period of ones in the 475 years. Also four areas are indicated with known accelerations.

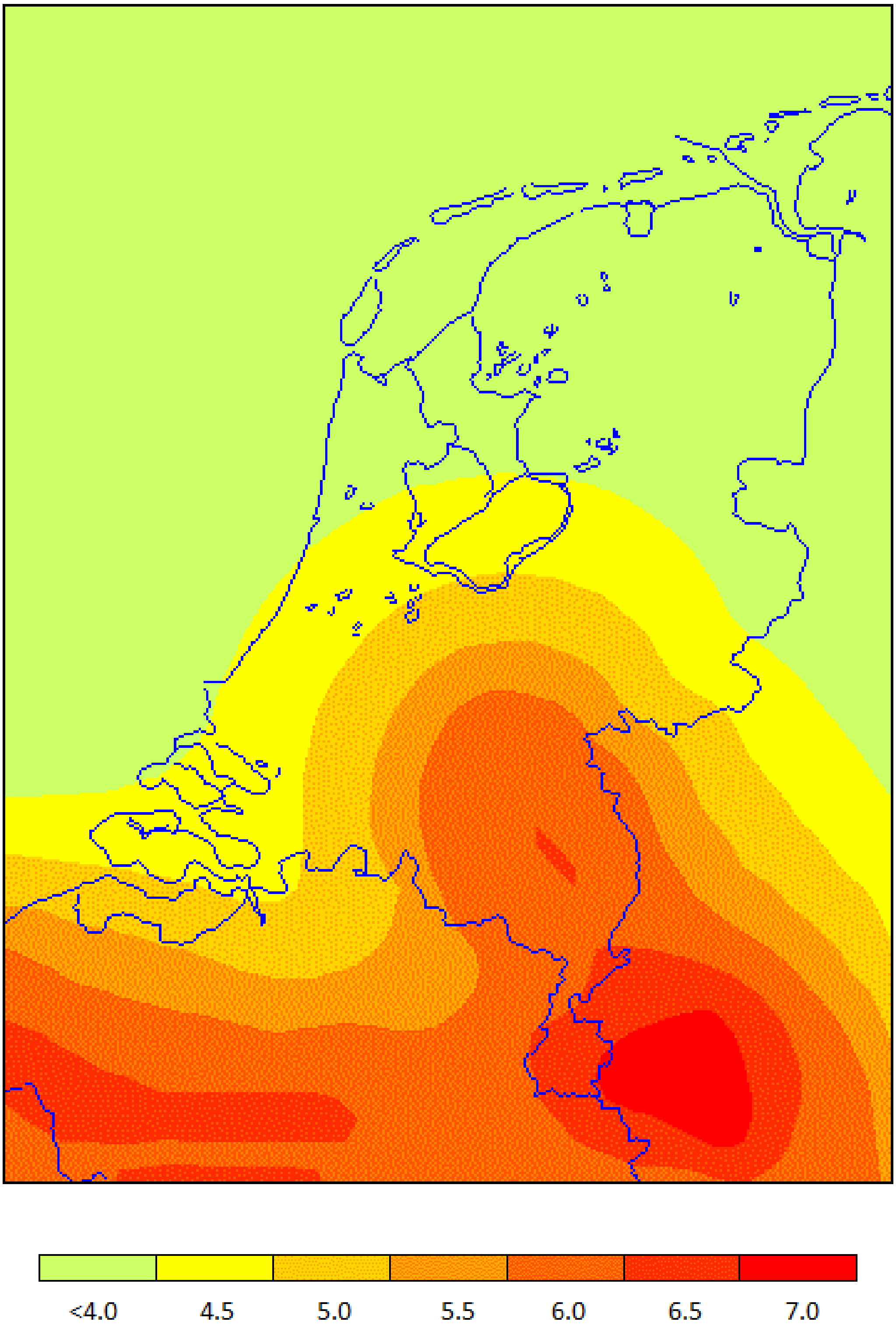


Figure E.12: Earthquake Map of The Netherlands: EMS Classification, Probability of Occurrence 1/475 (Crook, 1996)

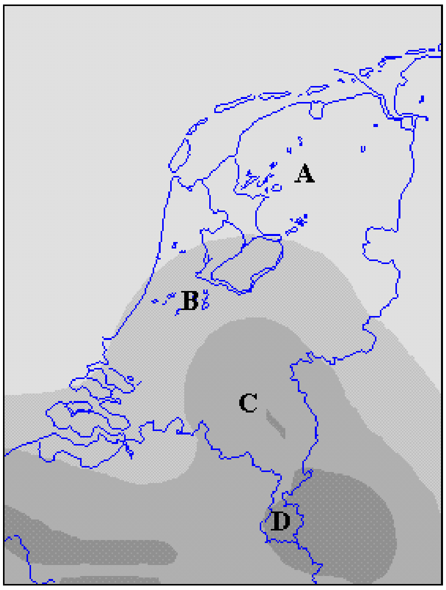


Figure E.13: Earthquake Map of The Netherlands: Zones Division, Probability of Occurrence 1/475 (Crook, 1996)

The seismotectonic zones of The Netherlands are divided into the zones A to D, which have a horizontal acceleration of respectively 10, 22, 50 en 100 cm/sec^2.

Not incorporated on the map are the regular seismic activity in the southern part of The Netherlands, at the Belgium and Germany border, and the northern part, where gas fields are situated in Groningen (Brouwer et al., 2010). Particularly for the extraction of gas, the settlements of the ground are expected to continue for decades. In Groningen one should consider a peak velocity of 20 to 60 mm/s and a magnitude fluctuating from 0.8 to 3.5 on the scale of Richter, which is assumed to be EMS scale 3 and lower. Thus, most of these earthquakes are not felt by the local population, but result in small cracks and damage to houses.

E.8.6. EARTHQUAKE DATA

Design codes and institutes of meteorology can recommend accelerations due to earthquake for certain parts of The Netherlands.

E.8.7. DESIGN EARTHQUAKE

According to the map in Figure E.12 and E.13 the following classification can be applied. The probability of occurrence of the provided values is 1/475 years.

Table E.38: Classification of Design Earthquake

Zone	**Horizontal Acceleration** (cm/s^2)
A	*10*
B	*22*
C	*50*
D	*100*

(CIRIA, CUR, CETMEF, 2007)

The defined groups of earthquakes suffice to obtain a rough estimate of the design earthquake accelerations.

F

DESIGN CONSIDERATIONS

This appendix consists of the following sections which are focused on the design considerations relevant for a breakwater design.

These sections consist of an extensive description of the design considerations. It includes professional expressions and typical units which are accordingly classified to be implemented in subsequent sections.

F.1. LOADS BY FLOW VELOCITY

Flow velocities occur perpendicular or parallel to breakwaters or river groynes. While rivers and channels would cause parallel flows around the head, propellers of vessels would frequently provide impact perpendicular to breakwaters and mooring structures.

F.1.1. CRITERION ENTERING VESSEL

The water depth is an important aspect, which determines whether a vessel can enter a lake or river. It assumes that the water depth (d (m)) should be larger than the draught (D (m)) including 0.5 m keel clearance. The requirement becomes:

$$d > D + 0.5 \tag{F.1}$$

The classified water depth and vessel draughts are input to Equation F.1. The results are shown in Table F.1.

Table F.1: Criterion Vessel Draught and Water Depth (0 = vessel is able to sail, 1 = wave vessel is unable to sail)

Type of Vessel	**D (m) / d (m)**	1	1.5	2	2.5	3	3.5	4.5
Small Yacht	2	0	0	0	1	1	1	1
Sailing Boat	1.5	0	0	1	1	1	1	1
Barge	1.5	0	0	1	1	1	1	1
International Vessel	3	0	0	0	1	1	1	1
Regional Convoy Vessel	2	0	0	0	1	1	1	1
International Convoy Vessel (1)	3.5	0	0	0	1	1	1	1
International Convoy Vessel (2)	4.5	0	0	0	1	1	1	1

With this information, a designer can determine what kind of vessel can be considered at a certain location.

F.1.2. FLOW VELOCITY AROUND RIVER GROYNES

The cross-sectional profile of the flow velocity in a water column is decreasing over depth. Also along the breakwater velocities decrease to zero.

On and in a bed protection the flow force exerts lift, shear and drag forces. These are determined by the average flow velocity. In case of river groynes, piers and harbour dams (Nortier and De Koning, 1996) the flow lines are pressed together resulting in flow velocities 1.5 times higher at head of the structure then the flow velocity in the main stream.

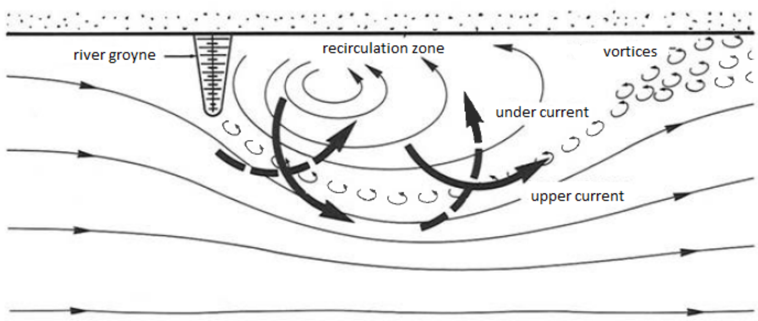


Figure F.1: Flow Behaviour around a River Groyne
(Nortier and De Koning, 1996)

F.1.3. PROPELLER VELOCITIES ON A SLOPE

Flow velocities are also generated by vessels. The highest velocities are a result of the spinning propellers. Two scenario's for ships could create flow velocities near breakwaters. The first scenario is when vessels depart from there mooring position. Bow thrusters and propellers will disturb the water by forcing it backwards resulting in high flow velocities and turbulence (Figure F.2). For a slope and vertical wall the following equations are provided.

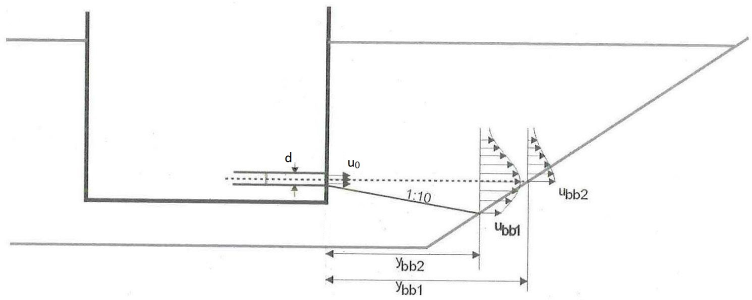


Figure F.2: Flow Velocities from Bow Thruster on a Slope
(Schiereck and Verhagen, 2001)

The equations used to determine the magnitude of this type of load depends on the flow velocities on the bed.

$$u_{b-max} = 0.3 u_0 \frac{d_p}{z_b} \sqrt{n} \tag{F.2}$$

$$u_b = u_{b-max} - 0.5 v_v \tag{F.3}$$

The following parameters are found in the above mentioned formulas:

u_{b-max}	Maximum flow velocity at the bed (m/s)
u_0	Flow velocity of the propeller (m/s)
d_p	Diameter of the propeller (m)
z_b	Keel clearance (m)
n	Number of propellers (-)
u_b	Flow velocity at the bed (m/s)
v_s	Sailing speed of the vessel (m/s)

Flow velocity on a slope by a bow thruster can be calculated by the following two formulas. $u_b b1$ is the flow velocity horizontally to the slope and $u_b b2$ is the flow velocity with an angle of 1:10 downward from the propeller axes.

$$u_{bb1} = 6.3 \frac{u_0 d}{y_{bb1}} \tag{F.4}$$

$$u_{bb2} = 3.15 \frac{u_0 d}{y_{bb2}} \tag{F.5}$$

With the requirement:

$$max(u_{bb1} and u_{bb2}) < u_b$$

Flow velocity on a vertical wall by a bow thruster:

$$u_{bb3} = 1.03\frac{u_0 d}{h_{bb}} < 6.3\frac{u_0 d}{y_{bb3}} < u_b \tag{F.6}$$

In which:

u_{bb1}, u_{bb2} and u_{bb3}	Flow velocities from bow thruster (m/s)
y_{bb1}, y_{bb2} and y_{bb3}	Distances ship to slope/wall (m)
h_{bb}	Distance bottom and centre of bow thruster (m)

F.1.4. FLOW VELOCITIES ON STRUCTURE BY MAIN PROPELLER

Implementation of Equation F.4 for main propellers and the requirement of Equation F.1 result in the Table F.2. It contains flow velocities with a distance x (m) to a vertical wall and slope for several water depths.

d (m)	1							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.0	0.0	0.0	0.0	0.0
Sailing Boat	0.9	1.5	0.7	0.0	0.0	0.0	0.0	0.0
Barge	1.0	1.5	0.7	0.0	0.0	0.0	0.0	0.0
International Vessel	0.9	3	1.5	0.0	0.0	0.0	0.0	0.0
Regional Convoy Vessel	1.3	2	1.0	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

d (m)	1.5							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.0	0.0	0.0	0.0	0.0
Sailing Boat	0.9	1.5	0.7	0.0	0.0	0.0	0.0	0.0
Barge	1.0	1.5	0.7	0.0	0.0	0.0	0.0	0.0
International Vessel	0.9	3	1.5	0.0	0.0	0.0	0.0	0.0
Regional Convoy Vessel	1.3	2	1.0	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

d (m)	2							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.0	0.0	0.0	0.0	0.0
Sailing Boat	0.9	1.5	0.7	1.0	0.7	0.5	0.4	0.3
Barge	1.0	1.5	0.7	1.1	0.8	0.6	0.5	0.4
International Vessel	0.9	3	1.5	0.0	0.0	0.0	0.0	0.0
Regional Convoy Vessel	1.3	2	1.0	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

d (m)	2.5							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.8	0.5	0.4	0.3	0.3
Sailing Boat	0.9	1.5	0.7	1.0	0.7	0.5	0.4	0.3
Barge	1.0	1.5	0.7	1.1	0.8	0.6	0.5	0.4
International Vessel	0.9	3	1.5	0.0	0.0	0.0	0.0	0.0
Regional Convoy Vessel	1.3	2	1.0	2.0	1.3	1.0	0.8	0.7
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

d (m)	3							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.8	0.5	0.4	0.3	0.3
Sailing Boat	0.9	1.5	0.7	1.0	0.7	0.5	0.4	0.3
Barge	1.0	1.5	0.7	1.1	0.8	0.6	0.5	0.4
International Vessel	0.9	3	1.5	0.0	0.0	0.0	0.0	0.0
Regional Convoy Vessel	1.3	2	1.0	2.0	1.3	1.0	0.8	0.7
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

d (m)	3.5							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.8	0.5	0.4	0.3	0.3
Sailing Boat	0.9	1.5	0.7	1.0	0.7	0.5	0.4	0.3
Barge	1.0	1.5	0.7	1.1	0.8	0.6	0.5	0.4
International Vessel	0.9	3	1.5	2.2	1.5	1.1	0.9	0.7
Regional Convoy Vessel	1.3	2	1.0	2.0	1.3	1.0	0.8	0.7
International Convoy Vessel (1)	1.1	3.5	1.7	0.0	0.0	0.0	0.0	0.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

Table F.2: Flow Velocities by Main Propeller on Slope/Vertical Wall

d (m)	4							
Type of Vessel	u_0 **(m/s)**	**D (m) / x (m)**	**d (m)**	4	6	8	10	12
Small Yacht	0.5	2	1.0	0.8	0.5	0.4	0.3	0.3
Sailing Boat	0.9	1.5	0.7	1.0	0.7	0.5	0.4	0.3
Barge	1.0	1.5	0.7	1.1	0.8	0.6	0.5	0.4
International Vessel	0.9	3	1.5	2.2	1.5	1.1	0.9	0.7
Regional Convoy Vessel	1.3	2	1.0	2.0	1.3	1.0	0.8	0.7
International Convoy Vessel (1)	1.1	3.5	1.7	3.1	2.1	1.5	1.2	1.0
International Convoy Vessel (2)	1.2	4.5	2.2	0.0	0.0	0.0	0.0	0.0

F.1.5. FLOW VELOCITIES ON STRUCTURE BY BOW THRUSTER

It should be noticed that Small Yacht, Sailing Boats and Barges are regularly built without bow thruster, although nowadays more of these vessels are delivered with thruster. But the size of these bow thrusters is assumed to be too small to produce significant flow velocities. What is more, the International Convoy Vessel is neglected due to the large draught.

Implementation of Equation F.4 for bow thrusters and the requirement of Equation F.1 result in the Table F.3. It contains flow velocities with a distance x (m) to a vertical wall and slope for several water depths.

d (m)	1							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	0	0	0	0	0
Regional Convoy Vessel	0.65	2	1.4	0	0	0	0	0
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

d (m)	1.5							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	0	0	0	0	0
Regional Convoy Vessel	0.65	2	1.4	0	0	0	0	0
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

d (m)	2							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	0	0	0	0	0
Regional Convoy Vessel	0.65	2	1.4	0	0	0	0	0
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

d (m)	2.5							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	0	0	0	0	0
Regional Convoy Vessel	0.65	2	1.4	5.9	2.9	2.0	1.5	1.2
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

d (m)	3							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	0	0	0	0	0
Regional Convoy Vessel	0.65	2	1.4	5.9	2.9	2.0	1.5	1.2
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

d (m)	3.5							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	5.9	2.9	2.0	1.5	1.2
Regional Convoy Vessel	0.65	2	1.4	5.9	2.9	2.0	1.5	1.2
International Convoy Vessel (1)	0.70	3.5	1.9	0	0	0	0	0
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

Table F.3: Flow Velocities by Bow Thruster on Slope/Vertical Wall

d (m)	4							
Type of Vessel	u_0 **(m/s)**	**D (m)**	**d (m) / x (m)**	1	2	3	4	5
Small Yacht	0	2	0	0	0	0	0	0
Sailing Boat	0	1.5	0	0	0	0	0	0
Barge	0	1.5	0	0	0	0	0	0
International Vessel	0.65	3	1.4	5.9	2.9	2.0	1.5	1.2
Regional Convoy Vessel	0.65	2	1.4	5.9	2.9	2.0	1.5	1.2
International Convoy Vessel (1)	0.70	3.5	1.9	8.3	4.2	2.8	2.1	1.7
International Convoy Vessel (2)	0.74	4.5	2.4	0	0	0	0	0

F.1.6. DESIGN LOADS BY FLOW VELOCITY

It is concluded that flow velocities around the head of a river groyne increases. These can be determined from the flow velocities of open channels, which are previously classified in Table E.11. Multiplying these values by factor 1.5 results in Table F.4.

Table F.4: Classification of Design Flow Velocities by Discharge around River Groyne

Property	**Flow Velocity** (m/s)
Lowest	0.15
Mean Low	0.38
Low	0.75
High	1.50
Mean High	2.25
Highest	3.00

The velocities as a consequence of vessels and return flows is shown in Table E.12. When vessels pass close to a river groyne or breakwater, the flow velocities could increase to the values in Table F.5.

Table F.5: Classification of Design Return Flow Velocities by Vessels around River Groyne

Property	**Flow Velocity** (m/s)
Lowest	0.15
Mean Low	0.30
Low	0.45
High	0.75
Mean High	1.10
Highest	1.50

The flow velocities on a wall are mainly caused by the main propeller and bow thruster. For the classification of the propeller load, it is assumed that the main propeller is providing an impact from a larger distance than a bow thruster (Table F.6).

Table F.6: Classification of Distance Propulsion to Structure

Property	**Distance to slope/wall** (m)
Main Propeller	4, 6, 8, 10 and 12
bow Thruster	1, 2, 3, 4 and 5

The requirement for vessels is that the draught plus keel clearance should be equal to or smaller than the water depth. If this is not the case, the value is set zero, because vessels could not enter these waters. One can see that the 'International Convoy Vessel (2)' is unable to enter shallow water areas with a water depth between 1 and 4 m.

In the several tables above the criterion and flow velocities generated by the main propeller and bow thruster are determined. The classification of the main propeller and bow thruster loads on a structure can be found in respectively Table F.7 and Table F.8.

Table F.7: Classification of Design Flow Velocities from Main Propeller on Structure

Property	**Flow Velocities** (m/s)
Lowest	0.4
Mean Low	0.8
Low	1.0
High	1.5
Mean High	2.0
Highest	3.0

Table F.8: Classification of Design Flow Velocities from Bow Thruster on Structure

Property	**Flow Velocities** (m/s)
Lowest	1.0
Mean Low	2.0
Low	3.0
Mean	4.0
High	6.0
Mean High	7.0
Highest	8.0

The defined groups of flow velocities suffice to obtain a rough estimate of the design flow velocity on a structure. An observed or provided value can be round up to obtain a higher level of safety.

F.2. LOADS BY WAVES

The loading of structures by wave impact is regularly the most governing scenario to design a structure against failure. In this section the wave breaking criteria, wave impact on a structure and the wave pressure is discussed.

F.2.1. WAVE IMPACT ON A STRUCTURE

The wave impact on a structure depends mainly on the angle of the slope, the slope roughness and the water depth. When a wave enters shallow water, the wave height increases resulting in a higher force on a gentle slope to a vertical wall. Another situation is when a wave is entering deep water, which does not effect the wave height. But due to a steep slope, a compulsive force occurs. Thus, visualized in the following figures.

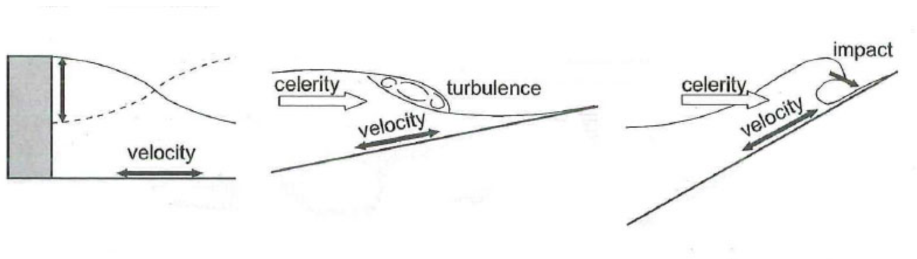


Figure F.3: Behaviour of Waves on Slope Types
(Schiereck and Verhagen, 2001)

F.2.2. WAVE BREAKING ON SLOPE

It is assumed that a foreshore in inland water is absent and that an horizontal bed is expected. Therefore, only the sloped structure could influence present waves to spill, plunge, collapse or surge. The slopes of rubble mound structures are normally between 1:1 to 1:10 (up to 1:12). Considering waves with limited wave height and wave length, the geometry of a sloped structure will determine behaviour of waves (Figure F.4).

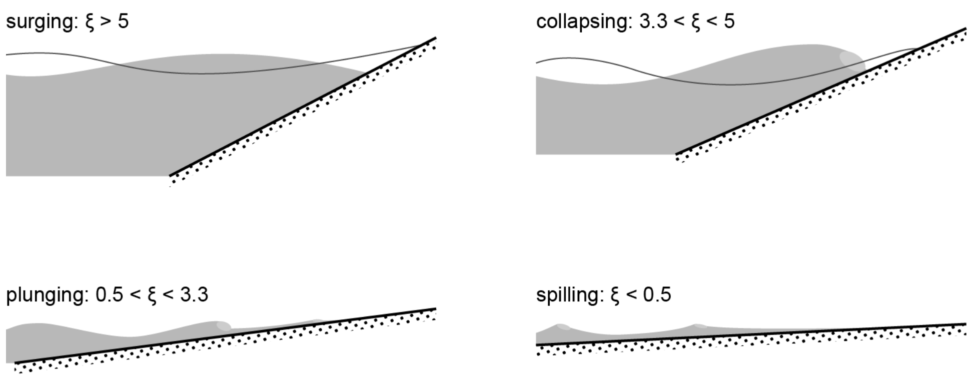


Figure F.4: Types of Wave Breaking
(Bosboom and Stive, 2013)

The Iribarren number (ξ) determines the magnitude of wave impact. In the following equation the ratio between the structures slope α ($degree$) and the wave-steepness. The wave-steepness is defined as the local wave height divided by deep water length. When a slope is steeper, the wave impact will increase.

$$\xi = \frac{tan\alpha}{\sqrt{H_s/L_0}} \tag{F.7}$$

F.2.3. WAVE PRESSURE ON A WALL

Waves are exerting pressure in case of a vertical wall structure. These pressure can be calculated by Sainflou for preliminary design phase calculations (Sainflou, 1928). Sainflou takes into account waves modelled by the Stokes' second order wave theory and overestimated the actual occuring wave pressures. Sainflou's theory holds:

$$h_0 = \frac{1}{2} k {H_i}^2 coth(kd) \tag{F.8}$$

The related pressure are:

$$p_1 = \rho g H_i \tag{F.9}$$

$$p_0 = \frac{\rho g H_i}{coshkd} \tag{F.10}$$

In which:

h_0 Height increase of the middle level (m)
H_i Wave amplitude of the incoming wave (m)
H Wave height of incoming wave (m)
k Wave number of the incoming wave (m^{-1})
p_1 Maximum wave pressure within the wave impact zone (N/m^2)
p_0 Minimum wave pressure at bed level (N/m^2)

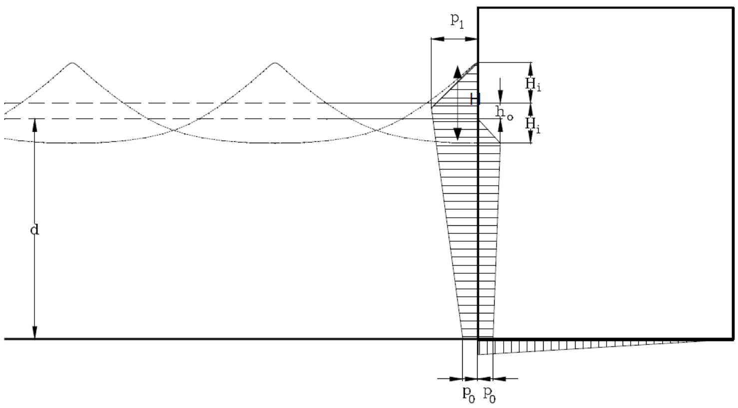


Figure F.5: Wave Pressure Distribution by Sainflou
(Sainflou, 1928)

In contrast, the formulas of Rundgren and Goda are used for final design calculations (Vrijling et al., 2011). Rundgren included higher order wave theory and adjusted the formula of Sainflou, while Goda provide a formula based on broken and breaking waves.

Breaking waves are causing a shock in a wall structures. Three models are to be distinguished to give rough estimates of the dynamic loading, namely Minikin, CERC 1984 and Goda-Takahashi. These short lasting pulses require special attention and shall not be taken into account in the method.

F.2.4. WAVE PRESSURE DETERMINATION

Tables F.9, F.10 and F.11 show the results of the computations about the wave pressures and water level increase by waves.

Table F.9: Water Level Increase, h_0 (m)

H (m)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
d (m) / k (rad/m)	2.51	1.26	1.01	0.84	0.50	0.34	0.25	0.17	0.17
1.0	0.01	0.03	0.04	0.06	0.14	0.13	0.13	0.13	0.13
1.5	0.01	0.03	0.03	0.04	0.10	0.20	0.20	0.19	0.19
2.0	0.01	0.03	0.03	0.04	0.08	0.29	0.27	0.26	0.26
2.5	0.01	0.03	0.03	0.04	0.07	0.24	0.35	0.33	0.33
3.0	0.01	0.03	0.03	0.04	0.07	0.22	0.44	0.41	0.41
3.5	0.01	0.03	0.03	0.04	0.07	0.20	0.54	0.49	0.49
4.0	0.01	0.03	0.03	0.04	0.07	0.19	0.37	0.57	0.57

Table F.10: Maximum Wave Pressure within the Wave Impact Zone, p_1 (kN/m^2)

H (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
p_1 (kN/m^2)	1.0	2.0	2.5	2.9	4.9	9.8	14.7	17.2	19.6

Table F.11: Minimum Wave Pressure at Bed Level, p_0 (kN/m^2)

H (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
k (rad/m)	2.51	1.26	1.01	0.84	0.50	0.34	0.25	0.17	0.17
d (m) / p_1 (kN/m^2)	1.0	2.0	2.5	2.9	4.9	9.8	14.7	17.2	19.6
1.0	0.2	1.0	1.6	2.1	4.3	9.3	14.3	16.9	19.3
1.5	0.0	0.6	1.0	1.5	3.8	8.7	13.7	16.6	19.0
2.0	0.0	0.3	0.6	1.1	3.2	8.0	13.0	16.2	18.6
2.5	0.0	0.2	0.4	0.7	2.6	7.2	12.2	15.8	18.0
3.0	0.0	0.1	0.2	0.5	2.1	6.3	11.3	15.2	17.4
3.5	0.0	0.0	0.1	0.3	1.6	5.5	10.4	14.6	16.7
4.0	0.0	0.0	0.1	0.2	1.3	4.8	9.5	13.9	15.9

F.2.5. DESIGN LOADS BY WAVES

Structures with a slope can be affected by plunging, collapsing and surging waves. Assuming that the wave height is relatively small, the wave behaviour is directly influenced by the slope angle of the structure. Accordingly, it will determine the breaker type. A classification for slopes between 7 and 45 degrees is provided in Table F.12. In inland waters the wave steepness (H_s/L_0) is assumed to be approximately 4%.

Table F.12: Classification of Breaker Type on Slope

Type of Slope	**Slope Angle** (deg)	**Breaker Index** (ksi)	**Breaker Type**
1:1	45.0	5.0	collapsing/surging
1:1.5	33.7	3.3	plunging/collapsing
1:2	26.6	2.5	plunging
1:3	18.4	1.7	plunging
1:4	14.0	1.3	plunging
1:5	11.3	1.0	plunging
1:6	9.5	0.8	plunging
1:7	8.1	0.7	plunging
1:8	7.1	0.6	plunging

The horizontal load on a vertical wall breakwater is mainly caused by wind- en vessel-generated waves. Moreover, larger vessel and larger water depth will cause higher waves, and therefore higher forces to be retained by the structure. Mind that the combination of h_0, p_0 and p_1 is not fixed, but are valued to the minimum, maximum and values in between in Table F.13. The variables are computed by the formulas of Sainflou.

Table F.13: Classification of Wave Pressure on Wall

Property	h_0 (m)	p_1 (kN/m^2)	p_0 (kN/m^2)
	0.1	1.0	0.5
	0.2	2.0	1.0
	0.4	3.0	3.0
	0.5	5.0	5.0
		10.0	10.0
		15.0	15.0
		20.0	20.0

In case of breakwaters, on both sides is a body of water. Therefore, the resultant force is composed by the water level increase of waves and the wave force determined.

The defined groups of wave consequences suffice to obtain a rough estimate of the horizontal design impact on a sloped and vertical structure. An observed or provided value can be round up to obtain a higher level of safety.

F.3. VESSEL COLLISION

For waterfront structures, the impact of vessel collision should be incorporated in the design. This accidental load is exceeding the load during berthing of a vessel. The berthing load depends on the dimensions of the vessels, the velocity of berthing, the type of fenders used, the deformation of the hull of the vessel and the characteristics of the structure (Committee for Waterfront Structures, 2012).

F.3.1. COMPUTATION OF VESSEL COLLISION FORCE

To determine the force exerted on a stiff structure, the following formulas can be applied:

$$E = \frac{1}{2} m {v_v}^2 \tag{F.11}$$

$$F_{max,m} = 3.3\sqrt{E} + 5.6 \tag{F.12}$$

In which:

E	Kinetic energy of the vessel (kNm)
m	Mass of the vessel (kg)
v_v	Sailing velocity of the vessel (m/s)
$F_{max,m}$	Maximum collision force of the vessel (kN)

Rijkswaterstaat (2013) provides not only the equation, but also suggest minimal force of 500 kN (perpendicular-to-structure component) and 250 kN (parallel-to-structure component).

F.3.2. DESIGN IMPACT OF VESSEL COLLISION

Enclosing the chosen design vessels, the forces to take into account are shown in Table F.14.

Table F.14: Classification of Design Vessel Collision

Type of Vessel	**Tonnage** (T)	**Sailing Velocity** (m/s)	**Collision Force** (kN)
Small Yacht	-	-	- (will be 500 kN)
Sailing Boat	-	-	- (will be 500 kN)
Barge	400	4.5	226 (will be 500 kN)
International Vessel	1500	5.3	508
Regional Convoy Vessel	1200	5.5	472 (will be 500 kN)
International Convoy Vessel (1)	3000	5.5	742
International Convoy Vessel (2)	6000	5.5	1048

According to the guidelines, the force has its impact over 0.5 m of structure length.

The vessel collision forces can be used to design prevention structures in front of the breakwaters, which are considered in this document. Consequently, vessel collision shall not be incorporated to determine the breakwater dimensions.

F.4. LOADS BY ICE

The magnitude of loads due to the motion of ice is less predictable. Literature is scarcely providing guidelines with regard to ice on hydraulic structures. The result is that the probability of under- or overestimation is high.

F.4.1. ICE FORCES

Ice exerts forces on a structures by means of four mechanisms, namely: thermal expansion, ice accumulation, ice collision and ice attachment. In case of breakwaters and river groynes, the collision and accumulation of ice is considered. The layer of ice is then pushed against hydraulic structures due to shear forces from the wind on top or from flow velocity under the ice. The result is the pile-up or ride-up of ice (Figure F.6). These forces have to be carried by the structure.

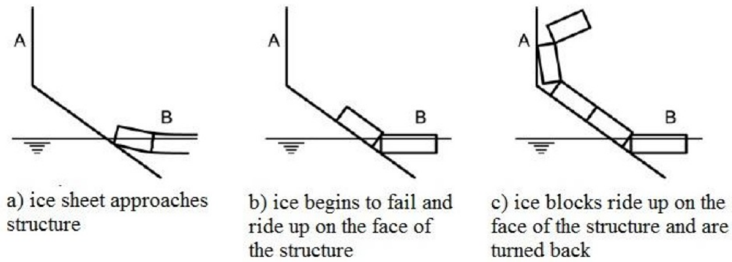


Figure F.6: Ice Behaviour on a Slope
(International Organization for Standardization [ISO], 2010)

The International Organization for Standardization [ISO] (2010) provides equations to determine the horizontal and vertical forces acting on a breakwater. The scheme of forces is shown in Figure F.7.

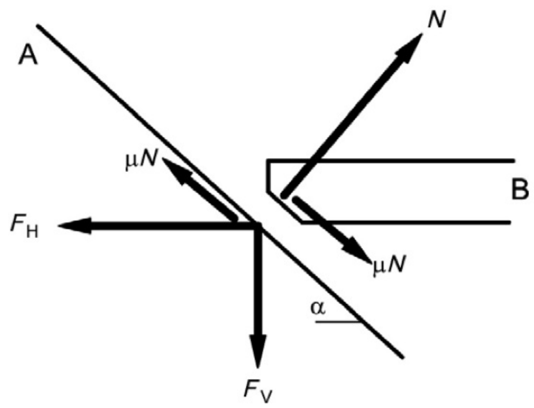


Figure F.7: Components of the Ice Force on a Slope
(International Organization for Standardization [ISO], 2010)

Obviously, vertical structures have no vertical component due to the absence of inclination. Assuming both horizontal and vertical forces the following equations can be applied.

$$F_H = \frac{F_{HB} + F_{HP} + F_{HR} + F_{HL} + F_{HT}}{1 - \frac{F_{HB}}{\sigma_f l_c d_i}} \tag{F.13}$$

$$F_V = F_H / \xi \tag{F.14}$$

In which:

F_H	Horizontal force of the ice layer (N)
F_V	Vertical force of the ice layer (N)
F_{HB}	Breaking load of the ice layer (N)
F_{HP}	Push load where the sheet of ice moves through the ice rubble (N)
F_{HR}	Push load where the ice blocks move through the ice rubble up a slope (kN)
F_{HL}	Lift load where the ice rubble is lifted (N)
F_{HT}	Turn load where the block at the top is turned (N)
σ_f	Flexural strength of the ice layer (N/mm^2)
l_c	Strength parameter of the ice layer $(-)$
d_i	Thickness of the ice layer (m)
ξ	Parameter of the slope $-$

It gives a global understanding of the forces resulting in ice motion. But there are still a lot of uncertainties. For example, the ice-ice friction and cohesion of the rubble of ice are unclear. On the contrary, the mentioned equations and the formulas to calculate the variables provide loads to check the global stability of structures.

F.4.2. ICE RULES

Guidelines or regulations to determine the ice loads are rare or do not discuss the topic extensively. For example, the Eurocode does not provide guidelines and the NEN refers to German literature.

The relevant ice load is ice collision. The Rijkswaterstaat (2013), CUR (2012) and Committee for Waterfront Structures (2012) propose for floating ice the following horizontal forces. Additionally, when ice accumulation is taken into account, a vertical force component should also be considered.

- ice thickness 0.2 m: 150 kN/m
- ice thickness 0.5 m: 400 kN/m

For short structures the force is modelled as a line load, in which the length of the ice and ice thickness is relevant. The following formula holds:

$$q_i = p_h \frac{d_i}{1} 0.33 \tag{F.15}$$

The formula for the ice force per unit of length (q_i) contains: a parameter p_h (horizontal pressure of 2500 kN/m^2 for ice in fresh water), a ratio of the actual to 1 m ice thickness, and a coefficient of planishment (0.33).

F.4.3. DESIGN LOADS BY ICE

From Table E.31 and the implementation of Equation F.15, the following ice forces can be obtained. These values are marginally higher than the forces mentioned in the previous section. This is for the reason that the order of magnitude is more important than exact values, which is a result of all the uncertainties in this load type.

Table F.15: Classification of Design Ice Force

Property	**Ice Thickness** (m)	**Ice Force** (kN/m)
Lowest	0.1	83
Low	0.2	165
Mean	0.3	248
High	0.4	330

It can be concluded that the behavior of ice on a structure is clear, but that the concerned load to be considered is unsettled. Exclusively the force by ice movement and collision is rather determined. Moreover, the effect of ice accumulation and attachment is still unknown. But the Directorate-General for Public Works and Water Management designed more than 600 hydraulic structures with the above mentioned formula Hooijschuur (2014). These structures are designed to a design ice load resulting from 0.20 m ice thickness, which suffice thus far.

The defined groups of ice suffice to obtain a rough estimate of the design ice forces. An observed or provided value can be round up to obtain a higher level of safety.

F.5. SUBGRADE REACTION

To discuss the stability of a structure on soil, the subgrade reaction is analysed. A relation is found between the applied load and the vertical/horizontal displacement.

F.5.1. SUBGRADE REACTION DETERMINATION

The following general expression of the modulus of subgrade reaction, k, is written as:

$$k = q/\delta \tag{F.16}$$

One can also distinguish the pressure (q) and the deflection (δ). The typical values of k_{s1} are provided in Table E.34 and should not be used for designs. To obtain an applicable subgrade reaction constant, one should consider the geometry of the structure. Therefore, the width of the structure, B (m), is included.

$$k_s = k_{s1}\frac{0.305}{B} \tag{F.17}$$

The specific formula to determine to allowable vertical force exerted on the subsoil becomes:

$$f_{sub} = k_{s1}\delta \tag{F.18}$$

F.5.2. DESIGN SUBGRADE REACTION

Assuming a general width for all breakwaters of 5 m, the correction factor becomes 0.06. This result in a lower k-value. By setting an allowable displacement of the structure of 50 mm, the bearing pressure of the soil is computed. Table F.16 provides the design values.

Table F.16: Design Modulus of Vertical Subgrade Reaction

Subsoil	k_{s1} (N/mm^3)	k_s (N/mm^3)	f_{sub} (N/mm^2)
Loam	0.023	0.001	0.07
Light Clay	0.045	0.003	0.14
Light Sand	0.080	0.005	0.24
Organic Sand	0.023	0.001	0.07
Peat	0.000	0.000	0.00
Clean Sand	0.025	0.002	0.08
Heavy Clay	0.090	0.005	0.27
Heavy Sand	1.000	0.060	3.00

The defined groups suffice to obtain a rough estimate of the modulus of vertical subgrade reaction. An observed or provided value can be round down to obtain a higher level of safety.

F.6. WATER LEVEL BY WIND

Besides waves, the wind is also a contributor of an increase of the water level. In this section, the principal and classification of wind set-up is discussed.

F.6.1. WIND SET-UP

Wind on a water surface is able to lift and lower the ends. The shear stress exerted by wind on the water surface counteract with the water gradient.

The gradient of the water surface can be determined by a formula developed for closed water systems and constant water depths. Application within inland waterways is found in lakes. The analytical solution (CIRIA, CUR, CETMEF, 2007) is as follows:

Equation of continuity:

$$\eta_w = \frac{1}{2}\frac{\rho_{air}}{\rho_w} C_D \frac{{u_{10}}^2}{gd} x \tag{F.19}$$

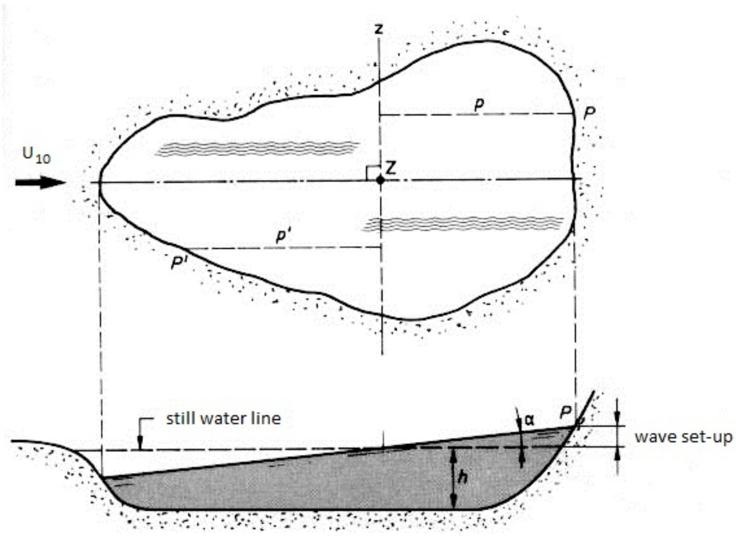


Figure F.8: Wind Set-up
(Nortier and De Koning, 1996)

The water level increase, η_w (m), is the difference between the still water level and the wind set-up. The equation depends on densities of air and water, respectively ρ_{air} (1.21 kg/m^3) and ρ_w (1000-1035 kg/m^3). The C_D coefficient ($0.8*10^{-3}$ to $3.0*10^{-3}$) is the air to water drag coefficient. This coefficient depends on the wind speed. The higher the wind speed, the more drag will occur. Other required parameters are the wind speed at 10 m height (u_{10}), the water depth (d) and the fetch length (x) considered.

F.6.2. WIND SET-UP DETERMINATION

The wind set-up for the classified water depth and wind velocities is determined by Equation F.19.

x (m)	50							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.0006	0.0004	0.0003	0.0002	0.0002	0.0002	0.0001
20	0.0012	0.0015	0.0010	0.0007	0.0006	0.0005	0.0004	0.0004
25	0.0016	0.0031	0.0021	0.0015	0.0012	0.0010	0.0009	0.0008
30	0.0020	0.0056	0.0037	0.0028	0.0022	0.0019	0.0016	0.0014

x (m)	100							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.001	0.001	0.001	0.000	0.000	0.000	0.000
20	0.0012	0.004	0.003	0.002	0.002	0.001	0.001	0.001
25	0.0016	0.006	0.004	0.003	0.002	0.002	0.002	0.002
30	0.0020	0.011	0.007	0.006	0.004	0.004	0.003	0.003

x (m)	500							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.006	0.004	0.003	0.002	0.002	0.002	0.001
20	0.0012	0.015	0.010	0.007	0.006	0.005	0.004	0.004
25	0.0016	0.031	0.021	0.015	0.012	0.010	0.009	0.008
30	0.0020	0.056	0.037	0.028	0.022	0.019	0.016	0.014

x (m)	1000							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.011	0.007	0.006	0.004	0.004	0.003	0.003
20	0.0012	0.030	0.020	0.015	0.012	0.010	0.008	0.007
25	0.0016	0.062	0.041	0.031	0.025	0.021	0.018	0.015
30	0.0020	0.111	0.074	0.056	0.044	0.037	0.032	0.028

x (m)	5000							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.056	0.037	0.028	0.022	0.019	0.016	0.014
20	0.0012	0.148	0.099	0.074	0.059	0.049	0.042	0.037
25	0.0016	0.308	0.206	0.154	0.123	0.103	0.088	0.077
30	0.0020	0.555	0.370	0.278	0.222	0.185	0.159	0.139

x (m)	10000							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.111	0.074	0.056	0.044	0.037	0.032	0.028
20	0.0012	0.296	0.197	0.148	0.118	0.099	0.085	0.074
25	0.0016	0.617	0.411	0.308	0.247	0.206	0.176	0.154
30	0.0020	1.110	0.740	0.555	0.444	0.370	0.317	0.278

Table F.17: Wind Set-up

x (m)	30000							
u_{10} **(m/s)**	C_D **/ d (m)**	1	1.5	2	2.5	3	3.5	4
15	0.0008	0.33	0.22	0.17	0.13	0.11	0.10	0.08
20	0.0012	0.89	0.59	0.44	0.36	0.30	0.25	0.22
25	0.0016	1.85	1.23	0.93	0.74	0.62	0.53	0.46
30	0.0020	3.33	2.22	1.67	1.33	1.11	0.95	0.83

F.6.3. DESIGN WATER LEVEL SET-UP BY WIND

It can be concluded that up to 1 km of fetch the wind can not affect the water level close to a structure, since approximation of 1 mm to 10 cm are computed. In contrast, after 5, 10 and 30 km the wind set-up fluctuates from 3 mm to 3.3 m. Table F.18 provides the classification of the wind set-up for further use.

Table F.18: Classification of Wind Set-up

Property	**Wind Set-up** (m)
Lowest	0.2
Mean Low	0.4
Low	0.6
Mean	0.8
High	1
Mean High	2
Highest	3

The defined groups of wind suffice to obtain a rough estimate of the design water level increase by wind. An observed or provided value can be round up to obtain a higher level of safety.

F.7. WATER LEVEL BY WAVES

Besides the local still water level and the wind set-up, also waves contribute to the water level. This phenomenon is called wave-set-up and is discussed in this section.

F.7.1. WAVE SET-UP

The increase of the water level is caused by depth-induced wave breaking. The reduction of the wave height is translated into the wave set-up.

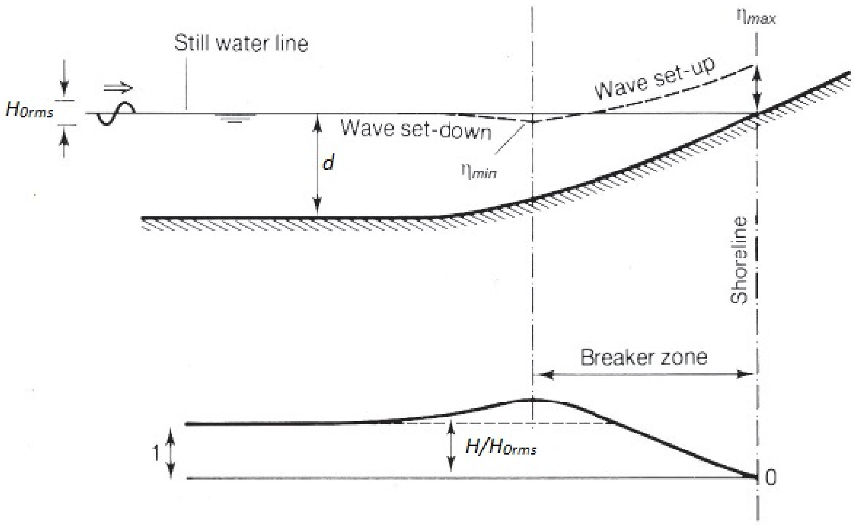


Figure F.9: Wave Set-up
(CIRIA, CUR, CETMEF, 2007)

Battjes (1974) derived an equation for the wave set-up, η_{max} (m) of regular waves, normal to the contour lines, using the breaker index, γ_{br} $(-)$, maximum wave height at breaker line, H_b (m).

$$\eta_{max} = 0.3\gamma_{br} H_b \tag{F.20}$$

When research evolved in this area of expertise Hanslow and Nielsen (1992) conducted an experiment in the field and numerical model which resulted in an equation for irregular waves.

$$\eta = 0.38 H_{0rms} \tag{F.21}$$

$$\eta = 0.0488\sqrt{H_{0rms} L_0} \tag{F.22}$$

The root-mean-square deep-water wave height, $H_{0_{rms}}$ (m), and the deep-water wave length, L_0 (m), are applied.

F.7.2. DESIGN WATER LEVEL SET-UP BY WAVES

The wave set-up for the design wave heights, resulting from Equation F.21 varies between 0.04 m (with H_0 = 0.1 m) and 0.95 m (with H_0 = 2.5m). This is only relevant to obtain water pressures on a slope due to an increased water depth, which is regularly used to design closed filters during significant wave actions. A more interesting aspect is the consequences of waves.

The wave set-up is part of the determination of the wave run-up and wave overtopping. The height of a breakwater is dominated by two factors only. Therefore, it should be avoided that this factor is taken into account.

F.8. WAVE RUN-UP

Waves arriving at a slope could increase the maximum of the crest height. The phenomenon is defined as wave run-up and affects the dimensions of a breakwater.

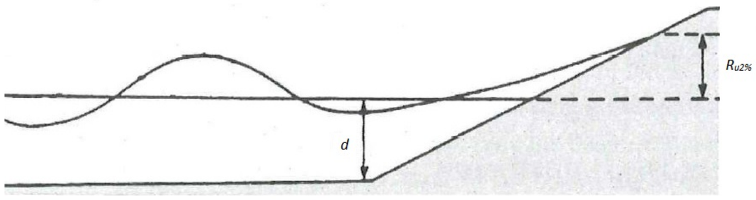


Figure F.10: Wave Run-up
(Schiereck and Verhagen, 2001)

Various researchers are interested in the determination of the run-up Van der Meer and Stam (1992). The first formula, originating from Delft and well-known in this area, was developed by Wassing (1957) for regular waves and smooth slopes.

$$R_{u2\%} = 8H_s \tan\alpha \tag{F.23}$$

Battjes (1974) came up with a run-up equation widely used in The Netherlands for outer slopes (sea) dikes covered with asphalt or stones under irregular waves. For smooth and rock slopes and ξ_{op} between 0.5 and 2, the following formulas can be used, respectively:

$$R_{u2\%} = 1.5H_s\xi_p \tag{F.24}$$

$$R_{u2\%} = 0.83H_s\xi_p \tag{F.25}$$

For steeper rock slopes with collapsing and surging waves ($\xi_p > 2$), a reduction factor of 0.5 to 0.6 can be used.

In which:

$R_{u2\%}$	Wave run-up exceeded by 2% of all waves (m)
H_s	Significant wave height (m)
α	Angle of the structure (rad or deg)
ξ_p	Iribarren number ($-$)

The formula of Wassing was improved by adding more influence factors, which could be used in further design stages. For a smooth impermeable slope the expression holds:

$$\frac{R_{u2\%}}{H_{m0}} = A\gamma_b\gamma_f\gamma_\beta\xi_p \tag{F.26}$$

In which:

H_{m0}	Wave height from total energy in the wave-spectrum (m)
γ_b	Factor berm ($-$)
γ_f	Factor roughness and permeability ($-$)
γ_β	Factor approach angle of the waves ($-$)

For more information and the determination of the parameters the Pullen et al. (2007) can be used.

F.8.1. WAVE RUN-UP DETERMINATION

The wave run-up results (Table F.19) are obtained by implementing Equation F.25.

Table F.19: Wave Run-up

α *(deg)*	45.0	ξ_p *(-)*	5.0	*reduction factor (-)*	0.5				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.21	0.42	0.52	0.62	1.04	2.08	3.11	3.63	4.15

α *(deg)*	33.7	ξ_p *(-)*	3.3	*reduction factor (-)*	0.5				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.14	0.28	0.35	0.42	0.69	1.38	2.08	2.42	2.77

α *(deg)*	26.6	ξ_p *(-)*	2.5	*reduction factor (-)*	0.5				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.10	0.21	0.26	0.31	0.52	1.04	1.56	1.82	2.08

α *(deg)*	18.4	ξ_p *(-)*	1.7	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.14	0.28	0.35	0.42	0.69	1.38	2.08	2.42	2.77

α *(deg)*	14.0	ξ_p *(-)*	1.3	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.10	0.21	0.26	0.31	0.52	1.04	1.56	1.82	2.08

α *(deg)*	11.3	ξ_p *(-)*	1.0	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.08	0.17	0.21	0.25	0.42	0.83	1.25	1.45	1.66

α *(deg)*	9.5	ξ_p *(-)*	0.8	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.07	0.14	0.17	0.21	0.35	0.69	1.04	1.21	1.38

α *(deg)*	8.1	ξ_p *(-)*	0.7	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.06	0.12	0.15	0.18	0.30	0.59	0.89	1.04	1.19

α *(deg)*	7.1	ξ_p *(-)*	0.6	*reduction factor (-)*	1				
H_0 *(m)*	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
$R_{2\%}$ *(m)*	0.05	0.10	0.13	0.16	0.26	0.52	0.78	0.91	1.04

F.8.2. DESIGN WAVE RUN-UP

The run-up of waves is determined for sloped structures. Equation F.25 provides an approach for wave run-up on a rock slope. Implementation of Table F.19 leads to the classification in Table F.20.

Table F.20: Classification of Wave Run-up

Property	*Wave Run-up (m)*
Lowest	0.2
Mean Low	0.4
Low	0.8
Mean	1
High	2
Mean High	3.5
Highest	5.5

The defined groups suffice to obtain a rough estimate of the wave run-up on a sloped structure. An observed or provided value can be round up to obtain a higher level of safety.

F.9. WAVE OVERTOPPING

Besides the determination of the structure height by the wave run-up one also has to consider the amount of water going over the structure. This is defined as wave overtopping in the unit of specific discharge q ($m^3/s/m$ or $l/s/m$). To have less energy of waves behind a structure it is important to reduce the discharge on top of the structure sufficiently. In Europe the lower and upper boundaries are 0.1 and 10 l/s/m which are frequently applied (Pullen et al., 2007).

F.9.1. WAVE OVERTOPPING REQUIREMENTS

For various circumstances, the allowable wave overtopping varies. It depends on the type of breakwater, the functions and the acceptable damage. Table F.21 and Table F.22 provide recommended values for requirement O3.

Table F.21: Allowable Wave Overtopping (1)

Purpose	**Overtopping Discharge ($l/s/m$)**
Dike with no quality slope	<0.1
Dike with normal slope	<1.0
Dike with high quality slope	<10
Safe passage of cars	<0.0001
Safe passage of pedestrians	<0.005
No Damage to buildings	<0.01
Acceptable damages to buildings	<0.02

(Schiereck and Verhagen, 2001)

Table F.22: Allowable Wave Overtopping (2)

Purpose	**Overtopping Discharge ($l/s/m$)**
Trained pedestrians	1-10
Aware pedestrians	0.1
Low speed driving vehicles	10-50
Moderate/high speed driving vehicles	0.01-0.05
Damage/sinking of larger yachts	50
Sinking of small boats, damage of larger yachts	10
Building structure elements	1
Damage of equipment	0.4

Pullen et al. (2007)

F.9.2. WAVE OVERTOPPING DETERMINATION

In this context Schiereck and Verhagen (2001) provide two equations for sloped structures and vertical walls. The overtopping depends on the roughness of the armour layer, the angle of the slope and the incoming wave.

The general formula for the quantity of water during overtopping is (Pullen et al., 2007):

$$q = a_{em} e^{(-b_{em} \frac{R_C}{\gamma})} \tag{F.27}$$

In case of steep slopes the following equation is applicable:

$$\frac{q}{\sqrt{g H_{m0}{}^3}} = 0.2 e^{-2.3 \frac{R_C}{H_{m0} \gamma_f \gamma_\beta}} \tag{F.28}$$

Considering vertical walls the following equation can be applied:

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = a_{em}e^{-b\frac{R_c}{H_{m0}}} \tag{F.29}$$

for $0.1 < R_c/H_{m0} < 3.5$ and in which:

$$R_c = \frac{h_c}{H_s}\frac{1}{\xi} \tag{F.30}$$

In which:

q	Unit discharge $(m^3/s/m)$
R_c	Dimensionless freeboard (-)
h_c	Crest height of the structure (m)
a_{em}, b_{em}	Empirical parameters $(-)$

For more information and the determination of the parameters the Pullen et al. (2007) can be used.

F.9.3. WAVE OVERTOPPING ON SLOPED STRUCTURES

The following parameters are provided

surging waves
$a_{em} = 0.2$ (-) (used in computation for collapsing and surging waves)
$b_{em} = 2.6$ (-) (used in computation for collapsing and surging waves)

plunging waves
$a_{em} = 0.067$ (-) (used in computation for plunging waves)
$b_{em} = 4.75$ (-) (used in computation for plunging waves)

reduction factor
$\gamma_{gras} = 0.95$
$\gamma_{rubble1} = 0.70$ (single layer)
$\gamma_{rubble2} = 0.55$ (double layer) (used in computation)

Inserting these in Equations F.30 and F.27 result in the following tables.

Table F.23: Wave Overtopping on Sloped Structure (freeboard = 0.1 m)

α (deg)	45.0	ξ (-)	5.0	*surging*	*R (m)*	0.5			
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	1.00	0.50	0.40	0.33	0.20	0.10	0.07	0.06	0.05
Q (l/s/m)	0.00	0.02	0.03	0.04	0.08	0.12	0.15	0.15	0.16

α (deg)	33.7	ξ (-)	3.3	collapsing					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	1.50	0.75	0.60	0.50	0.30	0.15	0.10	0.09	0.08
Q (l/s/m)	0.00	0.01	0.01	0.02	0.05	0.10	0.12	0.13	0.14

α (deg)	26.6	ξ (-)	2.5	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	2.00	1.00	0.80	0.67	0.40	0.20	0.13	0.11	0.10
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.02	0.03

α (deg)	18.4	ξ (-)	1.7	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.02	0.02

α (deg)	14.0	ξ (-)	1.3	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	4.00	0.08	0.10	0.12	0.20	0.40	0.60	0.70	0.80
Q (l/s/m)	0.00	0.03	0.03	0.02	0.01	0.00	0.00	0.00	0.00

α (deg)	11.3	ξ (-)	1.0	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	5.00	2.50	2.00	1.67	1.00	0.50	0.33	0.29	0.25
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01

α (deg)	9.5	*ξ (-)*	0.8	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01

α (deg)	8.1	ξ (-)	0.7	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	7.00	3.50	2.80	2.33	1.40	0.70	0.47	0.40	0.35
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	ξ (-)	0.6	plunging					
Hs (M)	0.100	0.200	0.250	0.300	0.500	1.000	1.500	1.750	2.000
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.24: Wave Overtopping on Sloped Structure (freeboard = 1.0 m)

α (deg)	45.0	ξ (-)	5.0	*surging*	h_c *(m)*	1.0			
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	2.00	1.00	0.80	0.67	0.40	0.20	0.13	0.11	0.10
Q (l/s/m)	0.00	0.00	0.00	0.01	0.03	0.08	0.11	0.12	0.12

α (deg)	33.7	ξ (-)	3.3	collapsing					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
Q (l/s/m)	0.00	0.00	0.00	0.00	0.01	0.05	0.08	0.09	0.10

α (deg)	26.6	ξ (-)	2.5	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01

α (deg)	18.4	ξ (-)	1.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01

α (deg)	14.0	ξ (-)	1.3	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	11.3	ξ (-)	1.0	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	10.00	5.00	4.00	3.33	2.00	1.00	0.67	0.57	0.50
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	9.5	ξ (-)	0.8	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	8.1	ξ (-)	0.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	14.00	7.00	5.60	4.67	2.80	1.40	0.93	0.80	0.70
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	ξ (-)	0.6	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	16.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.25: Wave Overtopping on Sloped Structure (freeboard = 1.5 m)

α (deg)	45.0	ξ (-)	5.0	*surging*	h_c *(m)*	1.5			
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
Q (l/s/m)	0.00	0.00	0.00	0.00	0.01	0.05	0.08	0.09	0.10

α (deg)	33.7	ξ (-)	3.3	collapsing					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.50	2.25	1.80	1.50	0.90	0.45	0.30	0.26	0.23
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.02	0.05	0.06	0.07

α (deg)	26.6	ξ (-)	2.5	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01

α (deg)	18.4	ξ (-)	1.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	9.00	4.50	3.60	3.00	1.80	0.90	0.60	0.51	0.45
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	14.0	ξ (-)	1.3	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	11.3	ξ (-)	1.0	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	15.00	7.50	6.00	5.00	3.00	1.50	1.00	0.86	0.75
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	9.5	ξ (-)	0.8	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	18.00	9.00	7.20	6.00	3.60	1.80	1.20	1.03	0.90
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	8.1	ξ (-)	0.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	21.00	10.50	8.40	7.00	4.20	2.10	1.40	1.20	1.05
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	ξ (-)	0.6	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	24.00	12.00	9.60	8.00	4.80	2.40	1.60	1.37	1.20
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.26: Wave Overtopping on Sloped Structure (freeboard = 2.0 m)

α (deg)	45.0	ξ (-)	5.0	*surging*	h_c *(m)*	2.0			
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.03	0.06	0.07	0.08

α (deg)	33.7	ξ (-)	3.3	collapsing					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.01	0.03	0.04	0.05

α (deg)	26.6	ξ (-)	2.5	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	18.4	ξ (-)	1.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	14.0	ξ (-)	1.3	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	16.00	8.00	6.40	5.33	3.20	1.60	1.07	0.91	0.80
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	11.3	ξ (-)	1.0	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	20.00	10.00	8.00	6.67	4.00	2.00	1.33	1.14	1.00
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	9.5	ξ (-)	0.8	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	24.00	12.00	9.60	8.00	4.80	2.40	1.60	1.37	1.20
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	8.1	ξ (-)	0.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	28.00	14.00	11.20	9.33	5.60	2.80	1.87	1.60	1.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	ξ (-)	0.6	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	32.00	16.00	12.80	10.67	6.40	3.20	2.13	1.83	1.60
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.27: Wave Overtopping on Sloped Structure (freeboard = 2.5 m)

α (deg)	45.0	ξ (-)	5.0	*surging*	h_c **(m)**	2.5			
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	5.00	2.50	2.00	1.67	1.00	0.50	0.33	0.29	0.25
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.02	0.04	0.05	0.06

α (deg)	33.7	ξ (-)	3.3	collapsing					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	7.50	3.75	3.00	2.50	1.50	0.75	0.50	0.43	0.38
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.01	0.02	0.03	0.03

α (deg)	26.6	ξ (-)	2.5	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	10.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	18.4	ξ (-)	1.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	15.00	7.50	6.00	5.00	3.00	1.50	1.00	0.86	0.75
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	14.0	ξ (-)	1.3	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	20.00	10.00	8.00	6.67	4.00	2.00	1.33	1.14	1.00
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	11.3	ξ (-)	1.0	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	25.00	12.50	10.00	8.33	5.00	2.50	1.67	1.43	1.25
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	9.5	ξ (-)	0.8	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	30.00	15.00	12.00	10.00	6.00	3.00	2.00	1.71	1.50
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	8.1	ξ (-)	0.7	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	35.00	17.50	14.00	11.67	7.00	3.50	2.33	2.00	1.75
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	ξ (-)	0.6	plunging					
Hs (M)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	40.00	20.00	16.00	13.33	8.00	4.00	2.67	2.29	2.00
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

F.9.4. WAVE OVERTOPPING ON WALL STRUCTURES

Inserting these in Equations F.30 and F.29 result in the following tables.

h_c (m)	0.1	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	0.20	0.10	0.08	0.07	0.04	0.02	0.01	0.01	0.01
Q (l/s/m)	0.00	0.02	0.03	0.06	0.18	0.59	1.12	1.43	1.75

h_c (m)	0.5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	1.00	0.50	0.40	0.33	0.20	0.10	0.07	0.06	0.05
Q (l/s/m)	0.00	0.00	0.00	0.01	0.08	0.48	1.03	1.33	1.66

h_c (m)	1.0	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	2.00	1.00	0.80	0.67	0.40	0.20	0.13	0.11	0.10
Q (l/s/m)	0.00	0.00	0.00	0.00	0.03	0.37	0.91	1.22	1.56

h_c (m)	1.5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
Q (l/s/m)	0.00	0.00	0.00	0.00	0.01	0.29	0.81	1.12	1.46

h_c (m)	2.0	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.22	0.72	1.03	1.37

h_c (m)	2.5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	5.00	2.50	2.00	1.67	1.00	0.50	0.33	0.29	0.25
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.17	0.65	0.95	1.28

h_c (m)	3	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.13	0.58	0.87	1.20

Table F.28: Wave Overtopping on Wall (freeboard = 0.1 to 5.0 m)

h_c **(m)**	3.5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	7.00	3.50	2.80	2.33	1.40	0.70	0.47	0.40	0.35
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.10	0.51	0.80	1.12

h_c **(m)**	4	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.08	0.46	0.74	1.05

h_c **(m)**	4.5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	9.00	4.50	3.60	3.00	1.80	0.90	0.60	0.51	0.45
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.06	0.41	0.68	0.99

h_c **(m)**	5	*surging*							
Hs (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	10.00	5.00	4.00	3.33	2.00	1.00	0.67	0.57	0.50
Q (l/s/m)	0.00	0.00	0.00	0.00	0.00	0.05	0.36	0.62	0.92

F.9.5. DESIGN WAVE OVERTOPPING

In Tables F.23 to F.28 the discharge over structures is computed for both sloped and wall breakwaters. This has been done by using Equation F.27 and F.29. The tables show higher values of overtopping for walls, which is a result of no run-up, approximately no wave energy loss and intensive splashes. The total overtopping varies between 0.00 and 0.95 l/s/m. Therefore, the following classification is chosen in Table F.29.

Table F.29: Classification of Wave Overtopping

Property	**Wave Overtopping** ($l/s/m$)
Lowest	0.01
Mean Low	0.02
Low	0.1
Mean	0.4
High	0.6
Mean High	0.8
Highest	1.0

The defined groups suffice to obtain a rough estimate of the wave overtopping on a sloped structure and walls. An observed or provided value can be round up to obtain a higher level of safety.

F.10. PILING-UP

The phenomena of piling-up is rarely described in literature. Manuals on breakwater designs also do not take into account accumulation of water behind a breakwater due to wave overtopping (immersed breakwater) or energy dissipation of waves (submerged breakwater) resulting in a water level difference (Diskin et al., 1970).

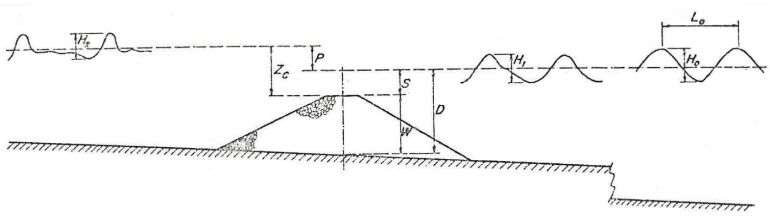


Figure F.11: Wave Piling-up
(Diskin et al., 1970)

The higher water levels in the sheltered area causes an outflow of water equal to the inflow by overtopping. The flow creates a pressure force on the inside breakwater and a lift force on the outside. In combination with collapsing wave, a dangerous situation reveals itself. Also piping could occur when the core consists of fine material. The general formula reads:

$$P = W + Z_c - D = Z_c - S \tag{F.31}$$

In which:

P	Water level difference (m)
W	Height of the structure, distance between sea bed level to crest level (m)
Z_c	Distance between crest level and maximum water level (m)
D	Distance between sea bed level en minimum water level (m)

Correspondingly, in only two situations piling-up occurs; a breakwater enclosing an area and a breakwater parallel to the bank of sufficient length.

F.10.1. DESIGN PILING-UP

The accumulation behind a breakwater or dike is mainly a result of an impermeable structure with a significant amount of wave overtopping. Another factor of influence is the length of a breakwater. Namely, the longer the structure, the less able water is to flow back to the side and front of a structure.

Since there is dealt with smaller breakwaters in inland waterways, which are loaded by small short-crested wind waves, insignificant wave overtopping can be assumed. What is more, the concerned structures will be relatively short for the small ports and bank section than in coastal zones. In other words, the water level on both sides shall be approximately the same and unaffected, which results in the absence of the phenomena piling-up.

Piling-up is not considered.

F.11. WAVE TRANSMISSION

When hydraulic structures are permeable waves can transmit their energy besides over the structure also through it. In this matter the high waves are more likely to go over the structure and the waves with longer wave periods to go through it when the armour and core layer are sufficiently permeable.

The general formula of reflection reads (Schiereck et al., 2012):

$$K_t = \frac{H_t}{H_i} \tag{F.32}$$

In which:

K_t	Transmission coefficient $(-)$
H_t	Wave height of the transmitted wave (m)
H_i	Wave height of the incoming/initial wave height (m)

When K_t is 1 there is full transmission. This value implies that there is no structure or a very low crest. But when K_t is 0, no wave energy is transmitted. In other words, there is a structure higher than the highest level of a wave and the core is impermeable and semi-permeable.

Initially, the formula of Deamen was developed applicable to low crested armour stone breakwaters. Years later, it was adjusted by d'Angremond et al. (1996) by making the freeboard dimensionless.

For $B/H_s < 8$:

$$K_t = -0.4\frac{R_c}{H_i} + 0.64\left(\frac{B}{H_i}\right)^{-0.31}(1 - e^{-0.5\xi}) \tag{F.33}$$

For wider crests the following equation was presented by DELOS.

For $B/H_s > 12$:

$$K_t = -0.35\frac{R_c}{H_i} + 0.51\left(\frac{B}{H_i}\right)^{-0.65}(1 - e^{-0.41\xi}) \tag{F.34}$$

The wave transmission for vertical structure is determined by Goda (2000). The formula reads:

for $0 < R_c/H_{m0} < 1.25$

$$K_t = 0.45 - 0.3\frac{R_c}{H_{m0}} \tag{F.35}$$

F.11.1. WAVE TRANSMISSION ON SLOPED STRUCTURE

The wave transmission on sloped structures is determined by Equation F.33.

Table F.30: Wave Transmission on Sloped Structure (freeboard = 0.5 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	0.5	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	1.00	0.50	0.40	0.33	0.20	0.10	0.07	0.06	0.05
K_t (-)	0.00	0.00	0.00	0.00	0.31	0.55	0.65	0.69	0.72

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	1.50	0.75	0.60	0.50	0.30	0.15	0.10	0.09	0.08
K_t (-)	0.00	0.00	0.00	0.00	0.18	0.46	0.56	0.60	0.63

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	2.00	1.00	0.80	0.67	0.40	0.20	0.13	0.11	0.10
K_t (-)	0.00	0.00	0.00	0.00	0.05	0.38	0.48	0.52	0.55

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.24	0.36	0.39	0.42

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.14	0.27	0.30	0.33

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	5.00	2.50	2.00	1.67	1.00	0.50	0.33	0.29	0.25
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.05	0.20	0.23	0.26

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.14	0.18	0.21

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	7.00	3.50	2.80	2.33	1.40	0.70	0.47	0.40	0.35
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.14	0.17

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.10	0.13

Table F.31: Wave Transmission on Sloped Structure (freeboard = 1.0 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	1	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	2.00	1.00	0.80	0.67	0.40	0.20	0.13	0.11	0.10
K_t (-)	0.00	0.00	0.00	0.00	0.15	0.51	0.63	0.67	0.71

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.40	0.54	0.58	0.61

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.30	0.45	0.49	0.53

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.12	0.30	0.35	0.39

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.25	0.29

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	10.00	5.00	4.00	3.33	2.00	1.00	0.67	0.57	0.50
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.17	0.21

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.10	0.15

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	14.00	7.00	5.60	4.67	2.80	1.40	0.93	0.80	0.70
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.10

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	16.00	8.00	6.40	5.33	3.20	1.60	1.07	0.91	0.80
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05

Table F.32: Wave Transmission on Sloped Structure (freeboard = 1.5 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	1.5	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	3.00	1.50	1.20	1.00	0.60	0.30	0.20	0.17	0.15
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.47	0.61	0.66	0.70

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	4.50	2.25	1.80	1.50	0.90	0.45	0.30	0.26	0.23
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.34	0.51	0.56	0.60

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.22	0.41	0.46	0.51

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	9.00	4.50	3.60	3.00	1.80	0.90	0.60	0.51	0.45
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.31	0.36

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.12	0.20	0.25

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	15.00	7.50	6.00	5.00	3.00	1.50	1.00	0.86	0.75
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.10	0.16

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	18.00	9.00	7.20	6.00	3.60	1.80	1.20	1.03	0.90
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.09

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	21.00	10.50	8.40	7.00	4.20	2.10	1.40	1.20	1.05
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	24.00	12.00	9.60	8.00	4.80	2.40	1.60	1.37	1.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.33: Wave Transmission on Sloped Structure (freeboard = 2.0 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	2	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.43	0.60	0.65	0.69

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.28	0.48	0.54	0.58

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.14	0.38	0.44	0.49

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.27	0.33

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	16.00	8.00	6.40	5.33	3.20	1.60	1.07	0.91	0.80
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.14	0.21

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	20.00	10.00	8.00	6.67	4.00	2.00	1.33	1.14	1.00
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.11

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	24.00	12.00	9.60	8.00	4.80	2.40	1.60	1.37	1.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	28.00	14.00	11.20	9.33	5.60	2.80	1.87	1.60	1.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	32.00	16.00	12.80	10.67	6.40	3.20	2.13	1.83	1.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.34: Wave Transmission on Sloped Structure (freeboard = 2.0 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	2	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	4.00	2.00	1.60	1.33	0.80	0.40	0.27	0.23	0.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.43	0.60	0.65	0.69

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	6.00	3.00	2.40	2.00	1.20	0.60	0.40	0.34	0.30
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.28	0.48	0.54	0.58

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	8.00	4.00	3.20	2.67	1.60	0.80	0.53	0.46	0.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.14	0.38	0.44	0.49

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	12.00	6.00	4.80	4.00	2.40	1.20	0.80	0.69	0.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.27	0.33

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	16.00	8.00	6.40	5.33	3.20	1.60	1.07	0.91	0.80
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.14	0.21

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	20.00	10.00	8.00	6.67	4.00	2.00	1.33	1.14	1.00
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.11

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	24.00	12.00	9.60	8.00	4.80	2.40	1.60	1.37	1.20
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	28.00	14.00	11.20	9.33	5.60	2.80	1.87	1.60	1.40
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	32.00	16.00	12.80	10.67	6.40	3.20	2.13	1.83	1.60
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.35: Wave Transmission on Sloped Structure (freeboard = 2.5 m, crest width = 1 m)

α (deg)	45.0	**ξ (-)**	5.0	*surging*	**R (m)**	2.5	**B (m)**	1	
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
B/H_s (-)	10.0	5.0	4.0	3.3	2.0	1.0	0.7	0.6	0.5
R (-)	5.00	2.50	2.00	1.67	1.00	0.50	0.33	0.29	0.25
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.39	0.58	0.63	0.68

α (deg)	33.7	**ξ (-)**	3.3	collapsing					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	7.50	3.75	3.00	2.50	1.50	0.75	0.50	0.43	0.38
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.22	0.46	0.52	0.57

α (deg)	26.6	**ξ (-)**	2.5	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	10.00	5.00	4.00	3.33	2.00	1.00	0.67	0.57	0.50
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.06	0.34	0.41	0.47

α (deg)	18.4	**ξ (-)**	1.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	15.00	7.50	6.00	5.00	3.00	1.50	1.00	0.86	0.75
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.14	0.23	0.30

α (deg)	14.0	**ξ (-)**	1.3	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	20.00	10.00	8.00	6.67	4.00	2.00	1.33	1.14	1.00
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.17

α (deg)	11.3	**ξ (-)**	1.0	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	25.00	12.50	10.00	8.33	5.00	2.50	1.67	1.43	1.25
Kt (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06

α (deg)	9.5	**ξ (-)**	0.8	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	30.00	15.00	12.00	10.00	6.00	3.00	2.00	1.71	1.50
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	8.1	**ξ (-)**	0.7	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	35.00	17.50	14.00	11.67	7.00	3.50	2.33	2.00	1.75
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

α (deg)	7.1	**ξ (-)**	0.6	plunging					
H_s (m)	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
R (-)	40.00	20.00	16.00	13.33	8.00	4.00	2.67	2.29	2.00
K_t (-)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table F.36: Wave Transmission on Vertical Wall (freeboard = 0.1 to 2)

R (m)	0.1								
H_s **(m)**	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
K_t **(-)**	0.15	0.30	0.33	0.35	0.39	0.42	0.43	0.43	0.44

R (m)	0.5								
H_s **(m)**	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
K_t **(-)**	0	0	0	0	0.15	0.30	0.35	0.36	0.38

R (m)	1								
H_s **(m)**	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
K_t **(-)**	0	0	0	0	0	0.15	0.25	0.28	0.30

R (m)	1.5								
H_s **(m)**	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
K_t **(-)**	0	0	0	0	0	0	0.15	0.19	0.23

R (m)	2								
H_s **(m)**	0.10	0.20	0.25	0.30	0.50	1.00	1.50	1.75	2.00
K_t **(-)**	0	0	0	0	0	0	0	0.11	0.15

F.11.2. DESIGN WAVE TRANSMISSION

The classification is mainly focused on rubble structure with varying freeboard and crest width. The typical freeboard is discussed in the previous section and hold 0.1 to 2.5 m. The crest width is assumed to be between 1 and 4 m. For reason of simplicity the maximum and minimum values are combined to compute the transmission coefficient. The results can be found in Tables F.30 to F.36 with the chosen classification in Table F.37.

Table F.37: Classification of Wave Transmission

Property	**Wave Transmission** (−)
Lowest	0
Low	0.2
Mean	0.4
High	0.6
Highest	0.8

The defined groups suffice to obtain a rough estimate of the wave transmission on a sloped structure. An observed or provided value can be round up to obtain a higher level of safety.

F.12. WAVE REFLECTION

Incoming waves on breakwaters and revetments could be partially reflected. This depends on the total wave energy dissipating within the structure. The energy losses are achieved by gaps and pores, which are dominated by friction when water particles enter and exit. Therefore, energy dissipation can be neglected in case of vertical solid walls. When reflected waves are in phase with the incoming wave in canals, navigation could be hindered due to an increasing wave crest and a decreased wave trough.

The general formula of reflection reads:

$$K_r = \frac{H_r}{H_i} \tag{F.36}$$

In which:

K_r Reflection coefficient $(-)$
H_r Wave height of the reflected wave (m)
H_i Wave height of the incoming/initial wave height (m)

A more advanced equation is developed for both smooth and rough slopes. The rough slopes are divided into rock (permeable/impermeable), Tetrapods, Core-Loc, Xbloc, Accropode, Antifer, Cubes (one and two layers).

$$K_r = \tanh(a\xi_{m-1,0}^{b}) \tag{F.37}$$

$$a = 0.167\left[1 - e^{-3.2\gamma_f}\right] \tag{F.38}$$

$$b = 1.49\left(\gamma_f - 0.38\right)^2 + 0.86 \tag{F.39}$$

For more information and the values of the parameters the Zanuttigh and Van der Meer (2007) and Zanuttigh and van der Meer (2008) can be used.

F.12.1. WAVE REFLECTION DETERMINATION

Implementation of Equation F.37 for several slope angles and surfaces leads to the following formula.

Table F.38: Wave Reflection of Rock and Smooth Slopes

ξ (-)	K_r **(rock, permeable)**	K_r **(rock, impermeable)**	K_r **(smooth slopes)**
5.0	0.45	0.53	0.92
3.3	0.33	0.39	0.71
2.5	0.26	0.31	0.53
1.7	0.18	0.22	0.32
1.3	0.14	0.17	0.22
1.0	0.12	0.14	0.16
0.8	0.10	0.12	0.12
0.7	0.09	0.10	0.10
0.6	0.08	0.09	0.08

The parameters for the three materials assumed, are:

rock, permeable
a = 0.12
b = 0.87
γ = 0.4

rock, impermeable
a = 0.14
b = 0.9
γ = 0.55

smooth slopes
a = 0.16
b = 1.43
$\gamma = 1$

F.12.2. DESIGN WAVE REFLECTION

The design wave reflection has been classified taken into account rock and smooth sloped structured (Table F.38). Similarly to the wave transmission, walls structures are assumed to have a reflection coefficient of 0, unless there is dealt with semi-permeable structures. In Table F.39 the classification of reflection coefficients is shown.

Table F.39: Classification of Wave Reflection

Property	**Wave Reflection** (−)
Lowest	0.0
Mean Low	0.1
Low	0.3
Mean	0.5
High	0.7
Mean High	0.9
Highest	1.0

The defined groups suffice to obtain a rough estimate of the wave reflection on a sloped structure. An observed or provided value can be round up to obtain a higher level of safety.

G

BREAKWATER TYPES

Many types of breakwaters are to be found all over the world. The stability strongly depends on the wave height (H) and size of the elements used ($\triangle$d). Table G.1 gives the relation between the type of structure and that ratio.

Table G.1: Characteristic Values of H/$\triangle$d

Type of Structure	**H/$\triangle$d**
Sandy Beach	>500
Gravel Beach	20-500
Rocky Slope	6-20
Berm Breakwater	3-6
Rubble Mound	1-4
Caisson	<1

(Schiereck et al., 2012)

Between the unstable and continuous changing morphology of the beaches, and the statically stable breakwaters withstanding all wave forces, is the berm breakwater. This breakwater has a stability under particular wave conditions and is able to reshape within limits. The rubble mound is therefore flexible and can cope with uneven settlement of the foundation. When failure occurs it takes effect gradually. Four categories shall be distinguished, respectively **mound**, **monolithic**, **composite** and **special breakwaters**.

G.1. MOUND TYPES

The first category of breakwaters consists of large loose elements (e.g. quarry stones, concrete blocks), which are dumped or accurately placed into a hill-shaped structures. A few parts can be distinguished. A core is required on which one or more filter layers (e.g., fine sand en gravel) are constructed. Within these layers geotextile could be used. To prevent erosion of the filters an armour layer, consisted of stones or concrete blocks, is used. A toe-structure is required to prevent the armour layer of sliding and collapsing. Additional features are a berm and a crown wall to reduce the amount of wave overtopping.

Sloping mound breakwaters are relatively simple to construct and require less maintenance. During construction the accuracy of positioning is less important compared to a row of caissons or sheet pile walls. As a result constructions costs could be less. Another reduction factor on the costs for mound breakwaters is maintenance. In case of erosion blocks will settle themselves into a stable position and when the height of the structure is dropping additional blocks can be placed.

G.2. MONOLITHIC TYPES

The second category is the monolithic breakwater. These structures of a single mass are, for example, caissons, block walls and masonry structures.

The mound and monolithic structures differ in soil-interaction and failure mechanisms. Monolithic structures cannot handle uneven settlements, but are able to cope with high dynamic forcing when the foundation is sufficiently solid. What is more, large water depths where mound structures require large amounts of material, a vertical wall breakwater could be less expensive.

G.3. COMPOSITE TYPES

The third category is the composite (multiple materials) breakwater. In some cases a low-crested breakwater consisting of loose material with a monolithic breakwater on top is preferred. Weak subsoils do not provide a stable foundation for caisson. Moreover, liquefaction can be a threat in case of weaker subsoils. A low-crested berm breakwater, which is pre-loading the (weak) subsoil, is optional to create an appropriate foundation for caissons. Since loose rock is cheaper than concrete, a combination of both materials could also make the overall design less expensive.

G.4. SPECIAL TYPES

The fourth category is the special type of breakwater. These unconventional breakwaters are in exceptional circumstances applicable, mentioning small short-crested waves in deep water. Otherwise they are not feasible and/or economical, because they will require large dimensions to damp long-period waves.

H

ORIENTATION OPTIONS

Breakwater are positioned in numerous ways, which can influence the type or construction equipment. In this appendix, a distinction is made between **Detached Breakwater**, **Non-detached Breakwater**, **T- and L-head Breakwater**, **Immersed Breakwater** and **Submerged Breakwater**.

H.1. DETACHED BREAKWATER

Detached breakwaters do not have a connection to land and are positioned parallel to the coast or bank. When the governing wind direction is directed to a harbour area, beach or habitat area, often these structure are designed in front of them. In practise one can observe accumulation of particles behind the structure. The waves transport these particles, where it deposits when waves of similar magnitude damp each other (Bosboom and Stive, 2013). As a results in coastal areas Tombolos are developed. Only water-based equipment can be used to constructed these breakwaters.

Figure H.1: Detached Breakwater
(Hakusan Tedorigawa Geopark. (2014). *Mikawa and Hakusan coastal zone.* Retrieved from http://hakusan-geo.main.jp/. Accessed on November 11, 2014.)

H.2. NON-DETACHED BREAKWATER

Non-detached breakwaters are attached to land, which enables construction by means of land-based equipment only. Also here it depends on the design wind direction. The wind has to be perpendicular to the harbours entrance and parallel to areas of nature.

Figure H.2: Detached Breakwater
(Beeldbank RWS. (2014). *Kustfoto 1982 luchtfoto Den Helder Donkere duinen golfbrekers ID418929*. Retrieved from https://beeldbank.rws.nl/. Accessed on November 11, 2014).

H.3. T- AND L-HEAD BREAKWATER

The detached and non-detached breakwater can also be combined. Depending on the long term wave direction, a non-detached breakwater with T- or L- head can be established. The result is that the shedding zone is increased in case of a groyne head positioned perpendicular to the wave direction.

Figure H.3: L-head Breakwater
(Gelocation. (2010). *Arica - La (ex) Isla Alacran desde El Morro*. Retrieved from https://geolocation.ws/. Accessed on Februari 9, 2014).

H.4. IMMERSED BREAKWATER

Immersed breakwaters are not visible. The crest could be right at the water surface or just below it. The design wave height and the allowed transmission determine the level of the crest. Dump trucks cannot be used when there is a very low crest. Cranes on land or water should do the work.

Figure H.4: Immersed Breakwater
(Reef Innovations. (2013). *Breakwater Construction*. Retrieved from http://reefinnovations.com/. Accessed on November 10, 2014.)

H.5. SUBMERGED BREAKWATER

Submerged breakwaters are still visible. The most of a rubble mound or caisson is found underwater, but still a crest is to be seen. The crest level mainly depends on the design wave height. An advantage of submerged breakwater is that dump trucks can drive over it if the drive way is well-constructed to carry the weight of the trucks with armour material.

I

OPTIONAL BREAKWATER ALTERNATIVES

This appendix consists of the following sections which focus on the analysis of various breakwater alternatives.

Appendix I.1	Rubble Mound Breakwater
Appendix I.2	Placed Block Breakwater
Appendix I.3	Sheet Pile Breakwater
Appendix I.4	Caisson Breakwater
Appendix I.5	Block Wall Breakwater
Appendix I.6	Floating Breakwater
Appendix I.7	Timber Pile Breakwater
Appendix I.8	Tire Breakwater
Appendix I.9	Reef Ball Breakwater
Appendix I.10	Gabion Breakwater
Appendix I.11	Screen Breakwater
Appendix I.12	Geotube Breakwater
Appendix I.13	Synthetic Breakwater

These sections provide an extensive description per breakwater alternative. The governing characteristics, the impact of boundary conditions and the EMVI-score are mentioned. Also the classified breakwater dimensions, the structural approach and the various cost estimates are discussed.

I.1. RUBBLE MOUND BREAKWATER

I.1.1. GENERAL

The rubble mound breakwater consists of core and armour material (Figure I.1). The core contains finer coarse, which is protected by a coarser quarry stone. A more advanced rubble mound structure is the traditional multilayer breakwater. It consists of an armour layer, a first under-layer, a core, a toe, a filter and a crest. A special type of rubble mound is the berm breakwater. In front of this breakwater a terrace is constructed below the water level.

Figure I.1: Rubble Mound Breakwater
(JohnsonMatel. (2011). *World-Wide-Matel.* Retrieved from http://johnsonmatel.com/. Accessed on November 6, 2014.)

I.1.2. SCHEME

The rubble mound structure in shallow inland waters is modelled as a heap of stones without berm (Figure I.2). The steepest slope and the exclusion of a berm will lead to less material and therefore lower material costs. Moreover, the core of the structure is not consisting of sand. Only for large-scale coastal breakwaters a sand core would have significant influence on the overhead cost.

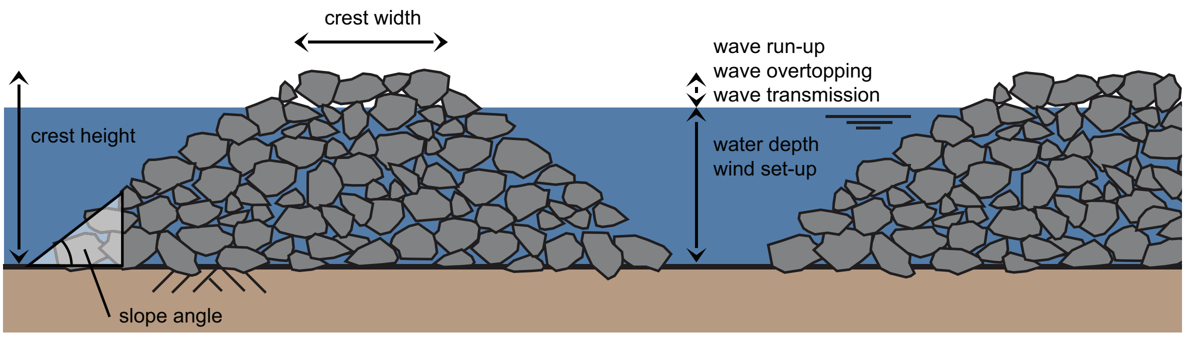


Figure I.2: Cross-sectional View (left) and Side View (right) of Rubble Mound Breakwater

I.1.3. CHARACTERISTICS

Complying to the laws and allocation of the permits, the following can be noticed:

- Water quality; suspended sediment due to rock dumping could lead to temporary decrease and release of polluted soil.
- Nature; temporary disturbance of wildlife (e.g. birds and fish) and damage to vegetation could be relevant. Even so, the rocks also provide shelter for animals and opportunities for vegetation to settle.
- Excavation; when stronger subsoil lays deeper.

Considering the functional requirements, the following can be said:

- Rock is a product of nature and fits perfectly in both urban as nature area landscapes.

- Rubble mound is not able to provide mooring facilities.
- Vessels could damage in case of interaction with vessels.
- The structure will reshape in time and is not maintenance free.
- Repair-works are carried out in a simple manner.

Regarding the boundary conditions, the following is assumed:

- Waves (governing); should always be considered.
- Flow velocities (governing); local discharge and vessels close-by.
- Subsoil (neglected); rubble mound are flexible structures, which settle evenly.
- Earthquakes (neglected); vertical and horizontal earthquakes in The Netherlands are limited and would allow the rubble mound to obtain a more stable position. A steep slope of the structure would be an disadvantage at this point and larger earthquakes could result in a lower crest height.
- Ice (neglected); it is assumed that the structures can withstand the ice loads due to its slope.

Design considerations to take into account are:

- Wind set-up.
- Wave run-up; for a slope.
- Wave overtopping; for a slope.
- Wave transmission; for a slope with (permeable) rocks.
- Wave reflection; for a slope with rocks.

I.1.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

There are no boundary conditions, which could instantaneously repel this breakwater alternative. But the subsoil could provide significant settlements and deformation to which it is advised to reject.

For the subsoil conditions:

Table I.1: Classification of Design Soil for Rubble Mound

Name	*Deformation*
Loam	Medium
Light Clay	Medium
Light Sand	Medium
Organic Sand	Medium
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

I.1.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.2: EMVI Criteria Score of Rubble Mound

Criterion	*Score*
System quality	4
Innovation	1
Ecological impact	3
CO2 ambition level	5
Durability	5
Hindrance	3
Noise	3
Risks	5

I.1.6. CLASSIFICATION

The stone classes considered, can be found in Table I.3.

Table I.3: Stone Classes

Class	D_{n50} (mm)
Rubble Mound 1	85
Rubble Mound 2	113
Rubble Mound 3	122
Rubble Mound 4	135
Rubble Mound 5	170
Rubble Mound 6	199
Rubble Mound 7	241
Rubble Mound 8	363
Rubble Mound 9	376
Rubble Mound 10	417
Rubble Mound 11	646
Rubble Mound 12	924

I.1.7. STRUCTURAL APPROACH

Using the simplified formula of Van der Meer, which is known as the Hudson formula in various scientific literature, the medium weight of armour stone or concrete armour units (W_{50} (N)) can be determined.

$$\frac{H}{\Delta D_{n50}}^{3} = K_D \cot\alpha \tag{I.1}$$

The Rock Manual CIRIA, CUR, CETMEF (2007) explains that the design wave height H is in fact $H_{1/10}$, which is equal to $1.27H_s$. Previously, the maximum significant wave height by wind in inland waterways is expected to be 2.5 m. The design wave height becomes $(1.27 \cdot 2.5 =)$ 3.2 m. Furthermore, Δ = 1.65 (-), g = 9.81 m/s^2, K_D = 3.5 (assuming non-breaking waves on the foreshore), $\cot\alpha$ = 1.5. The result is D_{n50} = 1.1 m, which is the same as a mean weight between 2090 and 4745 kg (Section K.1).

Loose non-cohesive grains could be mobilized under flow velocities. Examples are rock and gravel. Research found many empirical relations. Near the bed uniform flow becomes turbulent. A critical flow velocity, u_c m/s, can be defined, in which the load and strength in horizontal en vertical direction is out of balance. Izbash and Shields performed further research in the stability of stones. In 1936 Izbash published the following relation.

$$u_c = 1.2\sqrt{2\Delta g d} \tag{I.2}$$

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{I.3}$$

The variables are the relative density, Δ $(-)$, the acceleration of gravity, g (m/s^2), and the grain diameter, d (m). The bulk density of rubble is assumed to be 1590 kg/m^3.

I.1.8. DIMENSIONS

The stone size is a function of the wave height or flow velocity. Applying Equation I.1 and I.2 results in Table I.4.

Table I.4: Stone Size of Rubble Mound Breakwater

$Class$	D_{n50} (mm)	H_s (m)	u_c (m/s)
Rubble Mound 1	85	0.19	0.64
Rubble Mound 2	113	0.26	0.73
Rubble Mound 3	122	0.28	0.76
Rubble Mound 4	135	0.30	0.80
Rubble Mound 5	170	0.38	0.90
Rubble Mound 6	199	0.45	0.97
Rubble Mound 7	241	0.54	1.07
Rubble Mound 8	363	0.82	1.31
Rubble Mound 9	376	0.85	1.34
Rubble Mound 10	417	0.94	1.41
Rubble Mound 11	646	1.46	1.75
Rubble Mound 12	924	2.09	2.10

To determine the surface area per meter of length, the crest height is most important. The crest height is determined by taking into account the water level rise, wave run-up and wave overtopping. 1 m of crest width is assumed for the low volume structures and the slope is taken 1:1.5 (approximately 34 degrees). The relevant formulas are:

Crest height = water depth + wind set-up + wave run-up
Surface area/m = (crest height * crest width) + (0.5 * crest height * (crest height / sin(α)))

The crest height should be verified with:

Wave transmission and wave overtopping

The result can be fitted into the classified crest heights which are provided in Table I.5. The direct relation between the crest height (h_c) and the cross-sectional area of the rubble mound (A_{RM}) are given in Table I.5.

Table I.5: Dimensions of Rubble Mound Breakwater

h_c (m)	A_{RM} (m^2)
2	8
3	17
4	28
5	43
6	60
7	81
8	104
9	131
10	160

To determine the subgrade reaction, the mass of the stones taken are into account. The computed self-weight shall be divided by the surface area of the foundation (A_f) to obtain the pressure applied to the subsoil (Table I.16). One should draw conclusions by comparing the computed and allowed pressure.

$$f_{sub} = (F_G/A_f) - f_b \tag{I.4}$$

In which:

f_{sub}	Allowable pressure on subsoil (kN/m^2)
F_G	Force from selfweight (kN)
f_b	Buoyancy pressure (kN/m^2)
A_f	Surface area of structure on foundation (m^2)

Table I.6: Subgrade Reaction of Placed Block Breakwater

h_c (m)	F_G (kN)	A_f (m^2)	q_G (kN/m^2)
2	125	7	18
3	257	10	26
4	437	13	34
5	663	16	41
6	936	19	49
7	1256	22	57
8	1622	25	65
9	2036	28	73
10	2496	31	81

I.1.9. MATERIAL COSTS

In Table I.7, the unity costs for the stone classes are shown.

Table I.7: Stone Classes and Cost

Class	D_{n50} (mm)	*Cost of Rocks* (€/ton)	*Cost of Rocks* (€/m^3)
Rubble Mound 1	85	20	32
Rubble Mound 2	113	20	32
Rubble Mound 3	122	20	32
Rubble Mound 4	135	20	32
Rubble Mound 5	170	20	32
Rubble Mound 6	199	20	32
Rubble Mound 7	241	20	32
Rubble Mound 8	363	21	33
Rubble Mound 9	376	21	33
Rubble Mound 10	417	22	35
Rubble Mound 11	646	24	38
Rubble Mound 12	924	27	43

The general formula to computed the material costs is written as:

COST $(hour/m)$ = QUANTITY $(m^2/m$ or $m/m)$ / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions result in the material cost which are shown in Table I.8.

Table I.8: Material Cost of Rubble Mound Breakwater

h_c (m)	*Cost RM 1-7 (€/m)*	*Cost RM 8-9 (€/m)*	*Cost RM 10 (€/m)*	*Cost RM 11 (€/m)*	*Cost RM 12 (€/m)*
2	254	267	280	305	343
3	525	551	577	630	708
4	890	935	979	1068	1202
5	1352	1419	1487	1622	1825
6	1908	2003	2099	2290	2576
7	2560	2688	2816	3072	3456
8	3307	3473	3638	3969	4465
9	4150	4357	4565	4980	5602
10	5088	5342	5597	6106	6869

I.1.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of rock by hydraulic excavator	125	$m^3/hour$
Transport vessel with hydraulic excavator	250	€/$hour$
Pontoon to carry structure elements	25	€/$hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^2/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.9 can be constructed. In the last column, the labour and equipment costs are found.

Table I.9: Labour and Equipment Costs of Rubble Mound Breakwater

h_c (m)	*Placement of rock by hydraulic excavator (hour/m)*	*Cost of transport vessel with hydraulic excavator (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
2	0.06	16	2	18
3	0.13	33	3	36
4	0.22	56	6	62
5	0.34	85	9	94
6	0.48	120	12	132
7	0.64	161	16	177
8	0.83	208	21	229
9	1.04	261	26	287
10	1.28	320	32	352

I.1.11. CODES AND GUIDELINES

Further design rules for these structures and stability of the rocks can be found in CIRIA, CUR, CETMEF (2007) and PIANC/Marcom 40 (2003).

I.2. PLACED BLOCK BREAKWATER

I.2.1. GENERAL

A more structured breakwater compared to the randomly placed rubble mound and concrete blocks is the placed block breakwater (Figure I.3). These are frequently used as bank protection along rivers and lakes. In contrast, these are not applicable for extreme wave conditions. The placed blocks have various shapes. Regularly applied blocks are: Haringman, Hillblock, Basalton, Elastocoast, Interlock, Armorflex, Hydroblock, Asphalt, amongst others.

Figure I.3: Armorflex Breakwater
(External Works. (2014). *Armorflex concrete block revetment system.* Retrieved from http://www.externalworksindex.co.uk/. Accessed on December 4, 2014.)

I.2.2. SCHEME

Multiple materials are required for construction. The core consist of sand and has a large surface area. Between the sand and the blocks a permeable layer is design. Most often a filter layer of course grains and a geotextile are placed. To prevent the slope of blocks from sliding down, a simple toe of loose rock is constructed (Figure I.4).

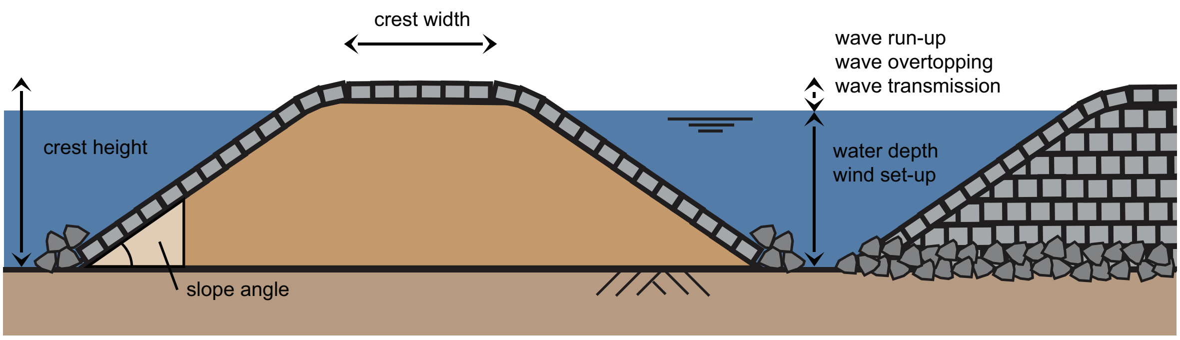


Figure I.4: Cross-sectional View (left) and Side View (right) of Placed Block Breakwater

I.2.3. CHARACTERISTICS

Complying to the laws and allocation of the permits, the following can be noticed:

- The dumping of sand and accurately placement of the blocks will not lead to the release of pollutants of contaminations.
- The water quality could be temporary diminished by very fine sand (dumped or dredged).
- Disturbance of wildlife (e.g. birds and fish) and damage to vegetation could.

Considering the functional requirements, the following can be said:

- The concrete block fits the urban area well in contrast to natural environments.
- Mooring facilities are not provided.
- In case of vessel collision, the structure will obtain a damaged part.
- The structure is not maintenance free and should be checked regularly.
- Repair-works are rather difficult, since these blocks are structurally placed and act as mattress. Thus one block can cause displacement of multiple blocks.

Regarding the boundary conditions, the following is assumed:

- Waves (governing); should always be considered.
- Flow velocities (neglected); for local discharge and vessels close-by.
- Subsoil (governing); placed block are not flexible structures.
- Earthquakes (governing); vertical and horizontal earthquakes in The Netherlands are limited, but could displace blocks. This results in an unstable structures, which vulnerable for wave attack and flow velocities.
- Ice (neglected); it is assumed that the structures can withstand the ice loads due to its slope.

Design considerations to take into account are:

- Wind set-up.
- Wave run-up; for a slope.
- Wave overtopping; for a slope.
- Wave transmission; for a slope.
- Wave reflection; for a slope.

I.2.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. Due to the type of subsoil and the earthquake acceleration this alternative can be repelled.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

In case of weak subsoil and intensive earthquakes, the placed block breakwater is assumed to be not capable to cope with the load.

For the subsoil conditions:

Table I.10: Classification of Design Soil for Placed Block

Name	*Deformation*
Loam	High
Light Clay	High
Light Sand	High
Organic Sand	High
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

For the earthquake acceleration:

Table I.11: Classification of Design Earthquake for Placed Block

Zone	*Horizontal Acceleration* (cm/s^2)	*Risk of Collapse*
A	*10*	Low
B	*22*	Low
C	*50*	High
D	*100*	High

I.2.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.12: EMVI Criteria Score of Placed Block

Criterion	*Score*
System quality	5
Innovation	3
Ecological impact	2
CO2 ambition level	1
Durability	4
Hindrance	3
Noise	3
Risks	2

I.2.6. CLASSIFICATION

The placed block classes (Holcim Betonproducten, 2009) considered are enclosed in Table I.13. d_b (m) is the variable for the block height.

Table I.13: Placed Block Classes

Class	$d_b(m)$
Placed Block 1	0.15
Placed Block 2	0.20
Placed Block 3	0.25
Placed Block 4	0.30
Placed Block 5	0.35
Placed Block 6	0.40
Placed Block 7	0.45
Placed Block 8	0.50

I.2.7. STRUCTURAL APPROACH

The stability of placed blocks depends on: the pressure difference between outer and inner side of the block, the weight of a block and the friction between the blocks. The most governing force is found under the block at the point where the maximum wave retreat it found. The permeability of the structure decreases the magnitude of the force. This is obtained by using geotextile or coarser grain directly below the block.

The Pilarzcyk formula (Bezuijen et al., 1990) can be applied to make a first guess for both rock and block revetments. The formula sounds:

$$\frac{H_s}{\Delta_m d} = \psi\Phi\frac{\cos\alpha}{{\epsilon_p}^b} \tag{I.5}$$

In which H_s (m) is significant wave height, which depends on the block thickness , d (m). Assumed is that:

Relative density, Δ_m $(-)$	= 1.5
System stability, ψ $(-)$	= 2 (Basalton, Haringman) or 2.5 (Armorflex)
Stability factor of incipient motion, Φ $(-)$	= 3 (first approximation within Acceptable tolerances)
Angle of the slope, α (rad)	= 18.4 (1:3)
Breaking index, ϵ $(-)$	= 1.7
Parameter, b $(-)$	= 1

The subgrade reaction pressure is analysed by determining the vertical pressure exerted by the structure onto the subsoil.

$$f_{sub} = (F_G/A_f) - f_b \tag{I.6}$$

In which:

f_{sub}	Allowable pressure on subsoil (kN/m^2)
F_G	Force from selfweight (kN)
f_b	Buoyancy pressure (kN/m^2)
A_f	Surface area of structure on foundation (m^2)

I.2.8. DIMENSIONS

The block thickness is a function of the wave height. Applying Equation I.5 results in Table I.14.

Table I.14: Placed Block Classes and Significant Wave Height

Class	d_b (m)	H_s (m)
Placed Block 1	0.15	0.75
Placed Block 2	0.20	1.00
Placed Block 3	0.25	1.26
Placed Block 4	0.30	1.51
Placed Block 5	0.35	1.76
Placed Block 6	0.40	2.01
Placed Block 7	0.45	2.26
Placed Block 8	0.50	2.51

To determine the surface area per meter of length the crest height is most important. By taking into account the water level rise, wave run-up and wave overtopping. 1m of crest width is assumed to be sufficient for the low volume structures and the slope is taken 1:3. The amount of core sand can be computed. The cost for the placed blocks, filter layer and geotextile is found by the two slopes plus crest. The relevant formulas are as follows:

Crest height = water depth + wind set-up + wave run-up
Surface area/m = (crest height * crest width) + (0.5 * crest height * (crest height / tan(α)))
Perimeter length/m = crest width + 2 * (crest height / cos(α)))

The crest height should be verified with:

Wave transmission and wave overtopping

The result can be fitted into the crest heights provided in Table I.15.

Table I.15: Dimensions of Placed Block Breakwater

h_c (m)	A_{sand} (m^2/m)	$L_{slopesandcrest}$ (m)	A_{filter} (m^2/m)
2	14	14	1
3	30	20	2
4	52	26	3
5	80	33	3
6	114	39	4
7	154	45	5
8	200	52	5
9	252	58	6
10	311	64	6

To determine the subgrade reaction, the mass of the core of sand and placed block cover (assuming 0.35 m block height) taken into account. The computed selfweight shall be divided by the surface area of the foundation (A_f) to obtain the pressure applied to the subsoil. One should draw conclusions by comparing the computed and allowed pressure.

Table I.16: Subgrade Reaction of Placed Block Breakwater

h_c (m)	F_G (kN)	A_f (m^2)	q_G (kN/m^2)
2	300	13	23
3	636	19	33
4	1095	25	44
5	1679	31	54
6	2386	37	64
7	3217	43	75
8	4172	49	85
9	5250	55	95
10	6453	61	106

I.2.9. MATERIAL COSTS

In Table I.17 the core material per barge, geotextile and filter layer (thickness 0.15 m) are determined for a given crest height. In Table I.18 the cost of the placed blocks (Basalton) are computed as function of the length of the outer perimeter and the block thickness. Note, that the dimensions and cost of the toe structure are neglected. These should be incorporated in preliminary design.

To calculate the material costs, the following unity costs are applied.

Core sand	6 (per barge)	$€/m^3$ (used)
	10 (per truck)	$€/m^3$
Geotextile	0.5	$€/m^2$
Filter layer	32	$€/m^2/m$
Basalton/Haringman/Armorflex	200	$€/m^3$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^2/m$ or $m/m)$ / PRICE OF MATERIAL ($€/m^3$ or $€/m^2$)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.17 and Table I.18.

Table I.17: Material Costs of Sand, Geotextile and Filter Layer of Placed Block Breakwater

h_c (m)	*Cost of Sand (€/m)*	*Cost of Filter Layer (€/m)*	*Cost of Geotextile (€/m)*
2	84	27	7
3	180	40	10
4	313	53	13
5	481	65	16
6	685	78	20
7	926	91	23
8	1202	103	26
9	1515	116	29
10	1864	129	32

I.2.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters,

Table I.18: Material Costs of Basalton/Haringman/Armorflex of Placed Block Breakwater

	Cost of Placed Blocks (€/m)	d_b (m)							
h_c (m)	*Length of Slopes with Crest Width (m)*	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
2	14	410	547	684	820	957	1094	1231	1367
3	20	600	800	1000	1201	1401	1601	1801	2001
4	26	790	1054	1317	1581	1844	2108	2371	2634
5	33	980	1307	1634	1961	2288	2614	2941	3268
6	39	1171	1561	1951	2341	2731	3121	3512	3902
7	45	1361	1814	2268	2721	3175	3628	4082	4535
8	52	1551	2068	2584	3101	3618	4135	4652	5169
9	58	1741	2321	2901	3482	4062	4642	5222	5803
10	64	1931	2574	3218	3862	4505	5149	5793	6436

the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of sand by hydraulic excavator	125	$m^3/hour$
Placement of rock by hydraulic excavator	125	$m^3/hour$
Placement of placed block by hydraulic excavator	15	$m^2/hour$
Transport vessel with hydraulic excavator	250	$€/hour$
Pontoon to carry structure elements	25	$€/hour$
Placement of geotextile	12	$€/m^2$
Slope finishing	50	$€/m^2$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 ($€/m$) = EQUIPMENT ($€/hour$) · HOURS OF WORKS($hour/m$)
- COST2 ($€/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.26 can be constructed. In the last column, the labour and equipment costs are found.

Table I.19: Labour and Equipment Costs of Placed Block Breakwater

h_c (m)	*Placement of sand by hydraulic excavator (hours/m)*	*Placement of rock by hydraulic excavator (hours/m)*	*Placement of placed block by hydraulic excavator (hours/m)*	*Transport vessel with hydraulic excavator (€/m)*	*Pontoon to carry structure elements (€/m)*	*Placement of geotextile (euro/m)*	*Slope finishing (€/m)*	*Total Cost (€/m)*
2	0.1	0.008	0.91	258	26	164	684	1131
3	0.2	0.008	1.33	396	40	240	1000	1676
4	0.4	0.008	1.76	545	55	316	1317	2233
5	0.6	0.008	2.18	707	71	392	1634	2804
6	0.9	0.008	2.60	881	88	468	1951	3388
7	1.2	0.008	3.02	1066	107	544	2268	3985
8	1.6	0.008	3.45	1264	126	620	2584	4595
9	2.0	0.008	3.87	1474	147	696	2901	5219
10	2.5	0.008	4.29	1696	170	772	3218	5856

I.2.11. CODES AND GUIDELINES

Further design rules for these structures and stability placed blocks are to be found in Pullen et al. (2007).

I.3. SHEET PILE BREAKWATER

I.3.1. GENERAL

A sheet pile breakwater is like a quay wall excluding the backfill. This type of breakwater can be constructed in case of limited space (Takahashi, 2002). The breakwater is regularly applied in small harbours and consist of steel or synthetic sheet piles. Also continuous concrete piles are possible.

The concrete wall, which goes by the name diaphragm wall, is not considered since this type of structure is applied for large water depths. Overall, the construction of diaphragm walls will be more time consuming and expensive than that of steel sheet piles.

For reasons of functionality or aesthetics a capping beam can be included. Also gaps in the piles can be applied which result in the regulation of water quality behind the breakwater. Since 2000 also plastic sheet piles are offered by various companies. These are durable and ecologically sound structures with a long lifetime. Therefore, these shall be taken into account.

Figure I.5: Sheet Pile Breakwater

I.3.2. SCHEME

The vertical wall breakwater consists of various sheet piles with interlocking at the edges. The sheets are drilled or hammered one by one into the subsoil. When the sheet pile row is completed a steel or concrete capping beam can be constructed on top (Figure I.6).

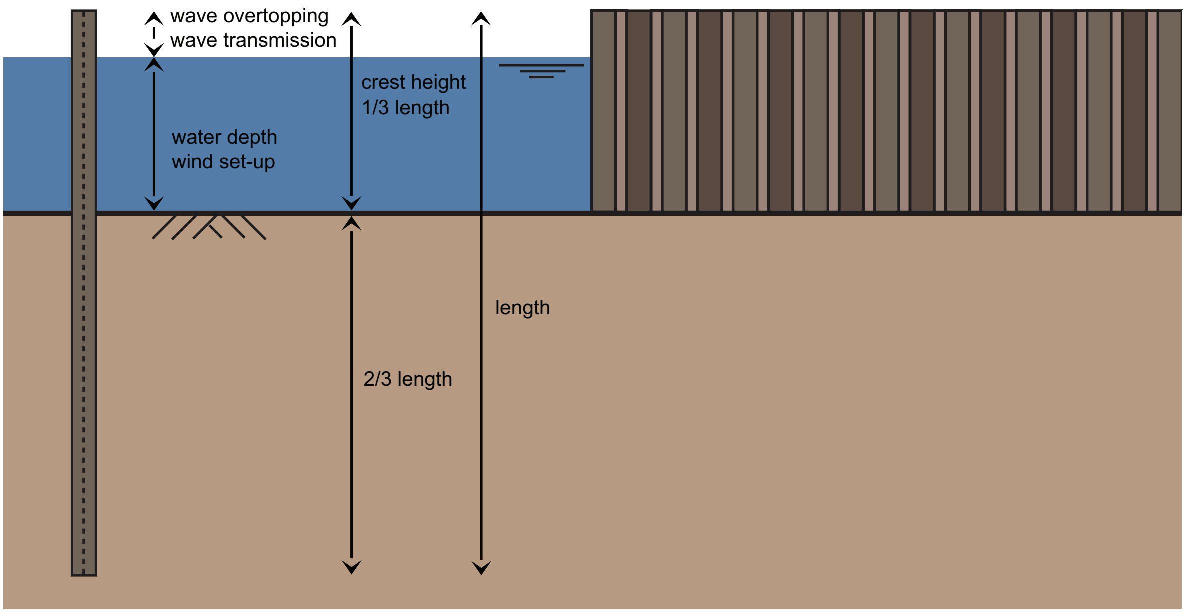


Figure I.6: Cross-sectional View (left) and Side View (right) of Sheet Pile Breakwater

I.3.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.3.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. Due to the earthquake this alternative can be repelled.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

Horizontal and vertical accelerations due to earthquakes could will be a threat for sheet piles. The impact can cause deformations of both the body of soil and piles. However, up to 100 cm/s^2 the deformation is assumed to be insignificant. In case of flow velocities, a designer should include bed protection around the sheet pile wall.

I.3.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.20: EMVI Criteria Score of Sheet Pile Breakwater

Criterion	*Score*
System quality	3
Innovation	4
Ecological impact	3
CO2 ambition level	3
Durability	2
Hindrance	3
Noise	1
Risks	3

I.3.6. CLASSIFICATION

The sheet pile classes considered are shown in Table I.22. These originate from Figure K.1 and K.2.

Table I.21: Sheet Pile Classes

Class	*Sheet Pile Type*
Sheet Pile 1	AZ 14-700
Sheet Pile 2	AZ 20-700
Sheet Pile 3	AZ 28-700
Sheet Pile 4	AZ 36-700N
Sheet Pile 5	AZ 44-700N
Sheet Pile 6	AZ 52-700
Sheet Pile 7	SG-525
Sheet Pile 8	SG-850
Sheet Pile 9	SG-950

The sheet pile types found in the table are made of the steel (AZ) and synthetic (SG). For the steel sheets a maximum design steel stress of $355N/mm^2$ is considered.

I.3.7. DIMENSIONS

To determine the overall dimensions, the crest height is most important. This can be computed according to the following formula:

Crest height = water depth + wind set-up

The crest height should be verified with:

Wave transmission and wave overtopping

The result can be fitted into the crest heights of 2, 4, 6, 8 and 10 m.

I.3.8. STRUCTURAL APPROACH

The structural scheme considered is shown in the Figure I.7. Due to wave level differences the force can be located at different heights. For this reason the governing point of the force is chosen to be at the top-level.

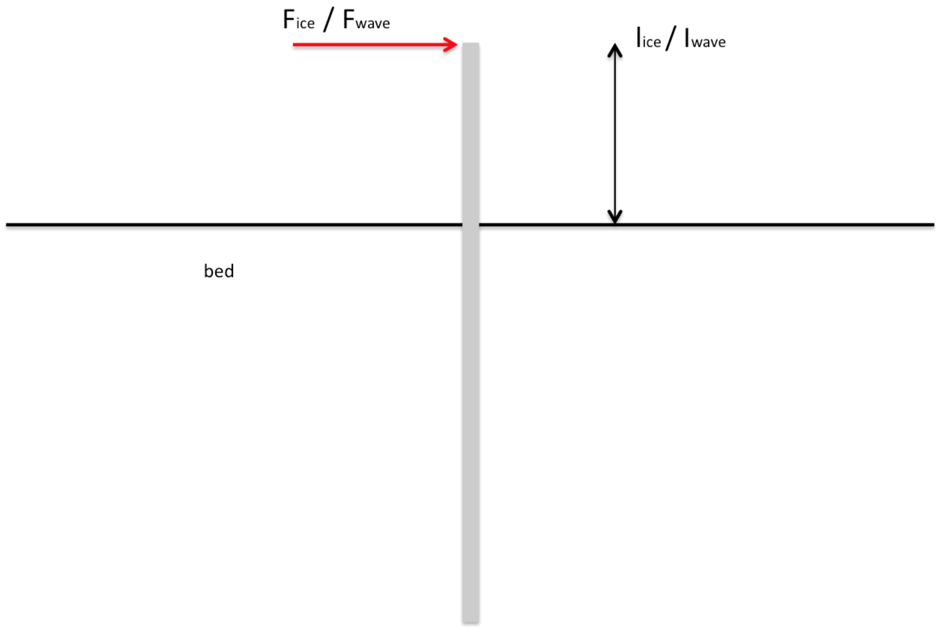


Figure I.7: Structural Model Sheet Pile Breakwater

For the structural performance of the sheet pile breakwater, the bending moments are investigated. It is assumed that the bearing capacity, shear force resistance, normal force equilibrium and sliding plane are within safety limits.

$$M_{Ed} < M_{Rd} \tag{I.7}$$

Assuming a fixed support the bending moment can be written as:

$$M_{Ed} = Fl \tag{I.8}$$

$$M_{Rd} = W\sigma \tag{I.9}$$

In which:

M_{Rd} Bending moment resistance (kNm)
M_{Ed} Design bending moment (kNm)

Consequently, the various design bending moments are determined which are enclosed in Table I.22 and I.23). The wave and ice loads are considered only.

Table I.22: Steel (AZ) and Synthetic (SG) Sheet Pile Characteristics

Class	*Sheet Pile Type*	W (cm^3/m)	M_{Rd} (kNm/m)
Sheet Pile 1	AZ 14-700	1205	428
Sheet Pile 2	AZ 20-700	1945	690
Sheet Pile 3	AZ 28-700	2725	967
Sheet Pile 4	AZ 36-700N	3590	1274
Sheet Pile 5	AZ 44-700N	4405	1564
Sheet Pile 6	AZ 52-700	5155	1830
Sheet Pile 7	SG-525	946	42
Sheet Pile 8	SG-850	2000	88
Sheet Pile 9	SG-950	3054	135

The steel class S355 (yield stress is 355 N/mm^2) is adopted to determine the internal bending moment. The modulus of the synthetic material (or yield strength) for vinyl and ultra composite is respectively 22 and 69 N/mm^2. Compared to the allowed steel stress, these values are significantly lower, which implies that the bending moment resistance will be less to, which can been observed.

I.3.9. MATERIAL COSTS

For the chosen sheet piles classes the following surface areas are considered.

Table I.23: Structural Performance of Sheet Pile Breakwater

h_c (m)	F (kN/m)	M_{Ed} (kNm/m)	*Applicable Sheet Pile*
2	3.4	7	SG-525 and AZ 14-700
	10.1	20	SG-525 and AZ 14-700
	20.3	41	SG-525 and AZ 14-700
	33.8	68	SG-850 and AZ 14-700
	83.0	166	AZ 14-700
	165.0	330	AZ 14-700
	248.0	496	AZ 20-700
	330.0	660	AZ 20-700
4	3.4	14	SG-525 and AZ 14-700
	10.1	41	SG-525 and AZ 14-700
	20.3	81	SG-850 and AZ 14-700
	33.8	135	SG-950 and AZ 14-700
	83.0	332	AZ 14-700
	165.0	660	AZ 20-700
	248.0	992	AZ 28-700
	330.0	1320	AZ 44-700N
6	3.4	20	SG-525 and AZ 14-700
	10.1	61	SG-850 and AZ 14-700
	20.3	122	SG-950 and AZ 14-700
	33.8	203	AZ 14-700
	83.0	498	AZ 20-700
	165.0	990	AZ 36-700N
	248.0	1488	AZ 44-700N
	330.0	1980	NO SHEET PILE
8	3.4	27	SG-525 and AZ 14-700
	10.1	81	SG-850 and AZ 14-700
	20.3	162	AZ 14-700
	33.8	270	AZ 14-700
	83.0	664	AZ 20-700
	165.0	1320	AZ 44-700N
	248.0	1984	NO SHEET PILE
	330.0	2640	NO SHEET PILE
10	3.4	34	SG-525 and AZ 14-700
	10.1	101	SG-850 and AZ 14-700
	20.3	203	AZ 14-700
	33.8	338	AZ 14-700
	83.0	830	AZ 28-700
	165.0	1650	AZ 52-700N
	248.0	2480	NO SHEET PILE
	330.0	3300	NO SHEET PILE

Table I.24: Steel (AZ) and Synthetic (SG) Sheet Pile Surface Areas

Sheet Pile Type	A_s (m^2/m)
AZ 14-700	0.012
AZ 20-700	0.015
AZ 28-700	0.020
AZ 36-700N	0.022
AZ 44-700N	0.027
AZ 52-700	0.032
SG-525	0.013
SG-850	0.023
SG-950	0.032

It is assumed that the cost of steel is 900 €/ton. Taking into account a steel density of 7800 kg/m^3, the cost becomes 7020 €/m^3. Including the unity costs of the breakwater materials, the material costs can be computed by the amounts of the materials. These cost exclusively refer to the price of the material. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.25.

Table I.25: Material Costs of Sheet Pile Breakwater

h_c (m)	$L_{sheetpile}$ (m)	*Sheet Pile Type*	*Cost of Sheet Piles (€/m)*
2	6	AZ 14-700	518
		AZ 20-700	640
		AZ 28-700	842
		AZ 36-700N	910
		AZ 44-700N	1150
		AZ 52-700	1335
		SG-525	238
		SG-850	416
		SG-950	568
4	12	AZ 14-700	1036
		AZ 20-700	1280
		AZ 28-700	1685
		AZ 36-700N	1820
		AZ 44-700N	2300
		AZ 52-700	2670
		SG-525	476
		SG-850	832
		SG-950	1136
6	18	AZ 14-700	1554
		AZ 20-700	1921
		AZ 28-700	2527
		AZ 36-700N	2729
		AZ 44-700N	3450
		AZ 52-700	4006
		SG-525	715
		SG-850	1247
		SG-950	1704
8	24	AZ 14-700	2072
		AZ 20-700	2561
		AZ 28-700	3370
		AZ 36-700N	3639
		AZ 44-700N	4600
		AZ 52-700	5341
		SG-525	953
		SG-850	1663
		SG-950	2272
10	30	AZ 14-700	2590
		AZ 20-700	3201
		AZ 28-700	4212
		AZ 36-700N	4549
		AZ 44-700N	5749
		AZ 52-700	6676
		SG-525	1191
		SG-850	2079
		SG-950	2840

I.3.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of sheet piles	35	$m^2/hour$
Transport vessel with hydraulic	250	$€/hour$
Pontoon to carry structure elements	25	$€/hour$
Vibration block	100	$€/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total costs of labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 ($€/m$) = EQUIPMENT ($€/hour$) · HOURS OF WORKS($hour/m$)
- COST2 ($€/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.26 can be constructed. In the last column, the labour and equipment costs are found.

Table I.26: Labour and Equipment Costs of Sheet Pile Breakwater

h_c (m)	L_{embed} (m)	*Placement of sheet piles (hours/m)*	*Transport vessel with hydraulic (€/m)*	*Pontoon to carry structure elements (€/m)*	*Vibration block (€/m)*	*Total Cost (€/m)*
2	4	0.1	28.6	2.9	2.9	34
4	8	0.2	57.1	5.7	5.7	69
6	12	0.3	85.7	8.6	8.6	103
8	16	0.5	114.3	11.4	11.4	137
10	20	0.6	142.9	14.3	14.3	171

I.3.11. CODES AND GUIDELINES

Design rules for vertical wall breakwaters are to be found in PIANC/Marcom 28 (2003).

I.4. CAISSON BREAKWATER

I.4.1. GENERAL

Caisson breakwaters consist of large concrete boxes. These are prefabricated with reinforced concrete. Since the inside of a caisson is initially filled with air, the empty space can be filled with loose rock or sand to enhance the stability after emerging. These concrete structures are relatively expensive.

Figure I.8: Caisson Breakwater
(Tracesofwar.com. (2014). *Phoenix Caissons Mulberry B Tracy-sur-Mer*. Retrieved from http://www.tracesofwar.com/. Accessed on November 7, 2014.)

I.4.2. SCHEME

A cross-sectional view is provided in Figure I.9. A stable foundation consists of rubble mound on which the caisson is placed. The permeable layer prevents liquefaction which can occur in sand, clays and other less porous materials.

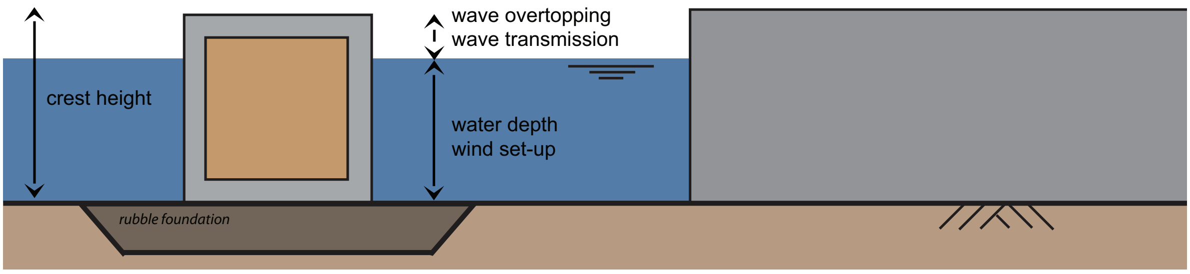


Figure I.9: Cross-sectional View (left) and Side View (right) of Caisson Breakwater

I.4.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.4.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. Due to the type of subsoil and the earthquake acceleration this alternative can be repelled.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

For the subsoil conditions:

Table I.27: Classification of Design Soil for Caisson Breakwater

Name	*Deformation*
Loam	High
Light Clay	High
Light Sand	High
Organic Sand	High
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

For the earthquake acceleration:

Table I.28: Classification of Design Earthquake for Caisson Breakwater

Zone	*Horizontal Acceleration (cm/s^2)*	*Risk of Collapse*
A	*10*	Low
B	*22*	Low
C	*50*	High
D	*100*	High

I.4.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.29: EMVI Criteria Score of Caisson

Criterion	*Score*
System quality	5
Innovation	4
Ecological impact	1
CO2 ambition level	2
Durability	2
Hindrance	3
Noise	3
Risks	4

I.4.6. CLASSIFICATION

The caisson classes considered are shown in Table I.30.

Table I.30: Caisson Classes

Class	*Height (m)*	*Width (m)*	*Thickness (m)*
Caisson 1	2	2	0.2
Caisson 2	4	4	0.3
Caisson 3	6	6	0.4
Caisson 4	8	8	0.5
Caisson 5	10	10	0.5

I.4.7. DIMENSIONS

To determine the overall dimensions, the crest height is most important. This can be computed according to the following formula:

Crest height = water depth + wind set-up + wave run-up

The crest height should be verified with:

Wave transmission and wave overtopping

The properties are provided in Table I.31.

I.4.8. STRUCTURAL APPROACH

The horizontal and rotational stability of the caissons are assessed. It is assumed that the bending moment resistance, shear force and normal force are conform the structural norms. The structural scheme considered is shown in the Figure I.10.

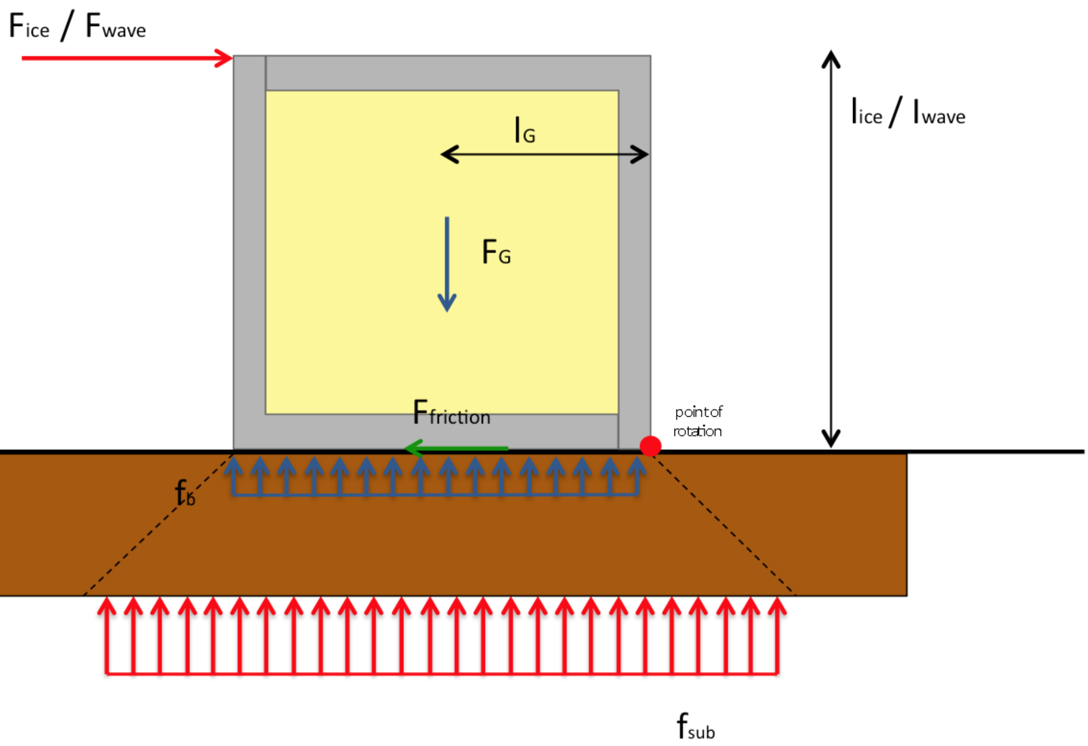


Figure I.10: Structural Model Caisson Breakwater

The requirement for the equilibrium of the horizontal forces is:

$$F_{fric} > F_H \tag{I.10}$$

$$F_{fric} = F_G * \delta \tag{I.11}$$

The friction factor between concrete and rubble is assumed to be 0.5.

The requirement for the equilibrium of the rotations is:

$$M_E < M_R \tag{I.12}$$

This can be written as:

$$F_H l_H < F_G l_G \tag{I.13}$$

Where:

F_H $= F_{ice}$ or F_{wave}
l_H $= l_{ice}$ or l_{wave}

In which:

F_H Horizontal load (kN)
l_H Arm of loading (m)
M_R Rotational resistance (kNm)
M_E Design rotation (kNm)
F_{fric} Friction force (kN)

Table I.31: Properties of Caisson Breakwater

Class	A_c (m^2)	G_c (kg/m)	A_s $(m2)$	G_s (kg/m)	G_{tot} (kg/m)	F_G (kN)	F_B (kN)	$F_{G,res}$ (kN)
Caisson 1	2.4	6000	2.6	5376	11376	114	39	75
Caisson 2	5.8	14500	11.6	24276	38776	388	157	231
Caisson 3	10.0	25000	27.0	56784	81784	818	353	465
Caisson 4	15.0	37500	49.0	102900	140400	1404	628	776
Caisson 5	19.0	47500	81.0	170100	217600	2176	981	1195

In which:

A_c Surface area of concrete
G_c Mass of concrete
A_s Surface area of sand
G_s Mass of sand
G_{tot} Total mass of concrete and sand
F_G Self-weight
F_B Buoyancy force
$F_{G,res}$ Resulting force

Table I.32: Structural Performance of Caisson Breakwater

h_c (m)	F_H (kN/m)	M_E (kNm/m)	M_R (kNm/m)	*Applicable (rotation)*	F_{fric} (kN)	*Applicable (friction)*
2	3.4	7	75	YES	57	YES
	10.1	20		YES		YES
	20.3	41		YES		YES
	33.8	68		YES		YES
	83.0	166		NO		NO
	165.0	330		NO		NO
	248.0	496		NO		NO
	330.0	660		NO		NO
4	3.4	14	462	YES	194	YES
	10.1	41		YES		YES
	20.3	81		YES		YES
	33.8	135		YES		YES
	83.0	332		YES		YES
	165.0	660		NO		YES
	248.0	992		NO		NO
	330.0	1320		NO		NO
6	3.4	20	1394	YES	409	YES
	10.1	61		YES		YES
	20.3	122		YES		YES
	33.8	203		YES		YES
	83.0	498		YES		YES
	165.0	990		YES		YES
	248.0	1488		NO		YES
	330.0	1980		NO		YES
8	3.4	27	3105	YES	702	YES
	10.1	81		YES		YES
	20.3	162		YES		YES
	33.8	270		YES		YES
	83.0	664		YES		YES
	165.0	1320		YES		YES
	248.0	1984		YES		YES
	330.0	2640		YES		YES
10	3.4	34	5975	YES	1088	YES
	10.1	101		YES		YES
	20.3	203		YES		YES
	33.8	338		YES		YES
	83.0	830		YES		YES
	165.0	1650		YES		YES
	248.0	2480		YES		YES
	330.0	3300		YES		YES

To determine the subgrade reaction, the mass of the core of sand and concrete walls are taken into account. The computed self-weight shall be divided by the surface area of the foundation (A_f) to obtain the pressure applied to the subsoil. One should draw conclusions by comparing the computed and allowed pressure.

$$f_{sub} = (F_G/A_f) - f_b \tag{I.14}$$

In which:

f_{sub} Pressure on subsoil (kN/m^2)
F_G Force from self-weight (kN)
f_b Buoyancy pressure (kN/m^2)
A_f Surface area of structure on foundation (m^2)

Table I.33 provides the pressure on the foundation for the caisson classes.

Table I.33: Subgrade Reaction of Caisson Breakwater

Class	f_{sub} (kN/m^2)
Caisson 1	19
Caisson 2	38
Caisson 3	58
Caisson 4	78
Caisson 5	100

I.4.9. MATERIAL COSTS

The following construction items are found:

Core sand	6 (per barge)	€$/m^3$
	10 (per truck)	€$/m^3$
Reinforced concrete with formwork	750	€$/m^3$
Rubble	32	€$/m^3$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST ($hour/m$) = QUANTITY (m^3/m or m/m) / PRICE OF MATERIAL (€$/m^3$ or €$/m^2$)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.34.

Table I.34: Material Costs of Caisson Breakwater

Class	*Cost of Reinforced Concrete (euro/m)*	*Cost of Sand (euro/m)*	*Cost of Rubble (euro/m)*	*Total Cost (euro/m)*
Caisson 1	1800	15	80	1895
Caisson 2	4350	69	160	4579
Caisson 3	7500	162	240	7902
Caisson 4	11250	294	320	11864
Caisson 5	14250	486	400	15136

The rubble mound foundation is provided with general dimensions. The length is chosen to be twice the width of the structure and the thickness is assumed to suffice with 1 m. The foundation depends on the subsoil. For example, a weaker layer requires more excavation for a stable foundation.

I.4.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Excavation of subsoil	40	$m^3/hour$
Placement of foundation	40	$m^3/hour$
Equalizing foundation	35	€$/m2$
Transport vessel with crawler crane	225	€$/hour$
Pontoon to carry structure elements	25	€$/hour$
Placement of caisson	2	$units/hour$
Connection of caisson	2	$units/hour$
Connecting structure segments	500	€$/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.35 can be constructed. In the last column, the total labour and equipment costs are found. The caisson length is assumed to be three times the width.

Table I.35: Labour and Equipment Costs of Caisson Breakwater

Class	*Excavation of subsoil (hours/m)*	*Placement of foundation (hours/m)*	*Equalizing foundation (€/m)*	*Transport vessel with crawler crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Connecting structure segments (€/m)*	*Total Cost (€/m)*
Caisson 1	0.1	0.1	140	45	5	42	232
Caisson 2	0.2	0.2	280	90	10	21	401
Caisson 3	0.3	0.3	420	135	15	14	584
Caisson 4	0.4	0.4	560	180	20	10	770
Caisson 5	0.5	0.5	700	225	25	8	958

I.4.11. CODES AND GUIDELINES

Design rules for caisson breakwaters are to be found in PIANC/Marcom 28 (2003) and British Standards Institution [BS] (1991).

I.5. Block Wall Breakwater

I.5.1. General

The block wall has a lot in common with caissons. Unreinforced concrete blocks are accumulated to the desired crest height. Also many quay walls in the Middle East are block walls due to the hard soils which cannot be penetrated by sheet piles or excavators. The block wall is attractive in the long-term.

Figure I.11: Block Wall Quay Wall
(New Port Project. (2012). *Building the New Port*. Retrieved from http://www.npp.com.qa/. Accessed on November 8, 2014.)

I.5.2. Scheme

The block wall requires a foundation of rubble. On top multiple blocks are constructed. The scheme is shown in Figure I.12.

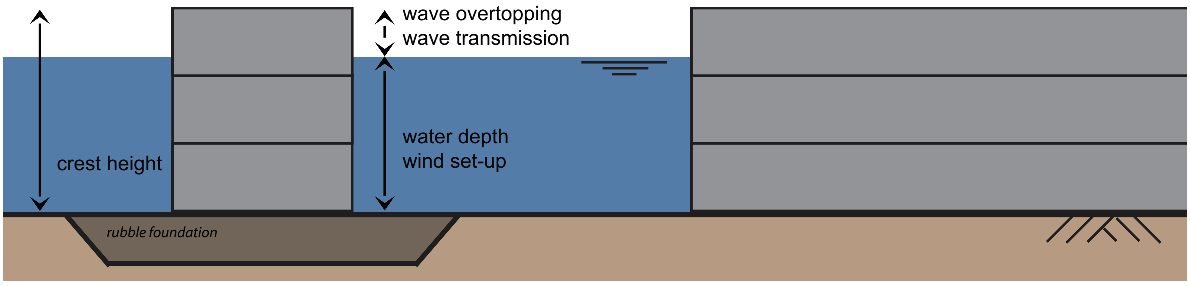


Figure I.12: Cross-sectional View (left) and Side View (right) of Block Wall Breakwater

I.5.3. Characteristics

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.5.4. Boundary Conditions

The general boundary conditions to be considered are enlisted below. Due to the type of subsoil and the earthquake acceleration this alternative can be repelled.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

For the subsoil conditions:

Table I.36: Classification of Design Soil for Block Wall

Name	*Deformation*
Loam	High
Light Clay	High
Light Sand	High
Organic Sand	High
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

For the earthquake acceleration:

Table I.37: Classification of Design Earthquake for Caisson

Zone	*Horizontal Acceleration* (cm/s^2)	*Risk of Collapse*
A	*10*	Low
B	*22*	Low
C	*50*	High
D	*100*	High

I.5.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.38: EMVI Criteria Score of Block Wall

Criterion	*Score*
System quality	2
Innovation	4
Ecological impact	1
CO2 ambition level	2
Durability	2
Hindrance	3
Noise	3
Risks	2

I.5.6. CLASSIFICATION

The block wall consists of multiple blocks. The assumed classification is shown in Table I.39 and I.40.

Table I.39: Block Classes

Type	*Height (m)*	*Width (m)*
Block 1	2	2
Block 2	2	4
Block 3	2	6

Table I.40: Block Wall Classes

Class	h_c (m)	*Applied Blocks*	*Number of Blocks*
Block Wall 1	2	Block 1	1
		Block 2	1
		Block 3	1
Block Wall 2	4	Block 1	2
		Block 2	2
		Block 3	2
Block Wall 3	6	Block 1	3
		Block 2	3
		Block 3	3
Block Wall 4	8	Block 1	4
		Block 2	4
		Block 3	4
Block Wall 5	10	Block 1	5
		Block 2	5
		Block 3	5

I.5.7. DIMENSIONS

To determine the overall dimensions, the crest height is most important. This can be computed according to the following formula:

Crest height = water depth + wind set-up + run-up

The crest height should be verified with:

Wave transmission and wave overtopping

The result can be fitted into the crest heights provided in Table I.40.

I.5.8. STRUCTURAL APPROACH

The block wall will be checked on the horizontal and rotational stability. It is assumed that the internal bending moments, shear forces and normal forces are conform the structural norms. The structural scheme considered is shown in the following Figure I.13.

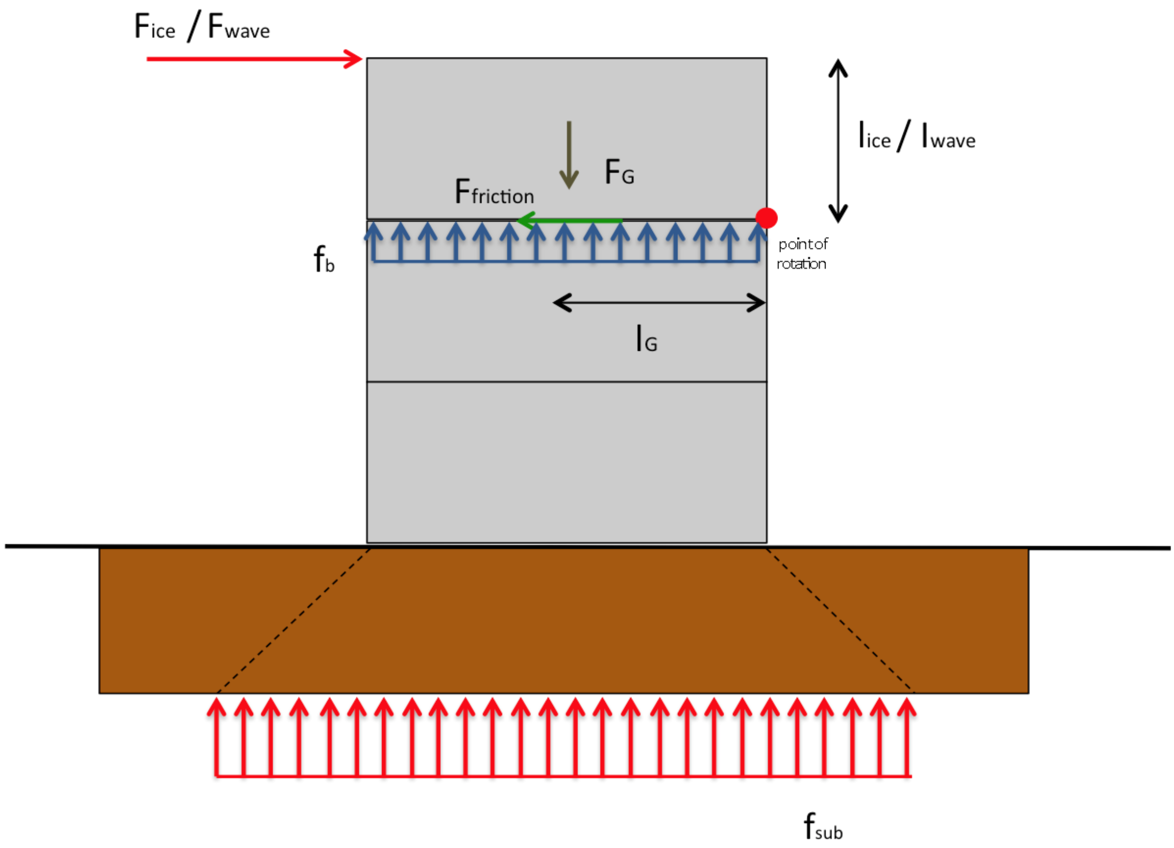


Figure I.13: Structural Model Block Wall Breakwater

The requirement for the equilibrium of the horizontal forces is:

$$F_{fric} > F_H \tag{I.15}$$

$$F_{fric} = F_G * \delta \tag{I.16}$$

The friction factor between concrete and rubble is 0.5.

The requirement for the equilibrium of the rotations is:

$$M_E < M_R \tag{I.17}$$

This can be written as:

$$F_H l_H < F_G l_G \tag{I.18}$$

Where:

F_H $= F_{ice}$ or F_{wave}
l_H $= l_{ice}$ or l_{wave}

In which:

F_H	Horizontal load (kN)
l_H	Arm of loading (m)
M_R	Rotational resistance (kNm)
M_E	Design rotation (kNm)
F_{fric}	Friction force (kN)

Including a friction between concrete and concrete of 0.8, the following verification Table I.42 of the horizontal force equilibrium and rotational stability is obtained.

Table I.41: Properties of Blocks

Class	A_c (m^2)	G_c (kg/m)	F_G (kN/m)	F_B (kN)	$F_{G,res}$ (kN)
Block 1	4	9600	96	39	57
Block 2	8	19200	192	78	114
Block 3	12	28800	288	118	170

In which:

A_c	Surface area of concrete
G_c	Mass of concrete
F_G	Self-weight
F_B	Buoyancy force
$F_{G,res}$	Resulting force

Table I.42: Structural Performance of Block Wall Breakwater

Class	*Block Height (m)*	M_R (kNm/m)	F_H (kN/m)	M_E (kNm/m)	*Applicable (rotation)*	F_{fric} (kN)	*Applicable (friction)*
Block 1	2	57	3.4	7	YES	77	YES
			10.1	20	YES		YES
			20.3	41	YES		YES
			33.8	68	NO		YES
			83.0	166	NO		NO
			165.0	330	NO		NO
			248.0	496	NO		NO
			330.0	660	NO		NO
Block 2	2	227	3.4	7	YES	154	YES
			10.1	20	YES		YES
			20.3	41	YES		YES
			33.8	68	YES		YES
			83.0	166	YES		YES
			165.0	330	NO		NO
			248.0	496	NO		NO
			330.0	660	NO		NO
Block 2	2	511	3.4	7	YES	230	YES
			10.1	20	YES		YES
			20.3	41	YES		YES
			33.8	68	YES		YES
			83.0	166	YES		YES
			165.0	330	YES		YES
			248.0	496	YES		NO
			330.0	660	NO		NO

To determine the subgrade reaction, the mass of the core of sand and concrete walls are taken into account. The computed selfweight shall be divided by the surface area of the foundation (A_f) to obtain the pressure applied to the subsoil. One should draw conclusions by comparing the computed and allowed pressure.

The subgrade reaction pressure is analysed by determining the vertical pressure exerted by the structure onto the subsoil.

$$f_{sub} = (F_G / A_f) - f_b \tag{I.19}$$

In which:

f_{sub}	Pressure on subsoil (kN/m^2)
F_G	Force from selfweight (kN)
f_b	Buoyancy pressure (kN/m^2)
A_f	Surface area of structure on foundation (m^2)

Table I.43 provides the pressure on the foundation for the block wall classes.

Table I.43: Subgrade Reaction of Block Wall Breakwater

Class	*Block Type*	*Number of Blocks*	f_{sub} (kN/m^2)
Block Wall 1	Block 1	1	57
	Block 2	1	114
	Block 3	1	170
Block Wall 2	Block 1	2	114
	Block 2	2	227
	Block 3	2	341
Block Wall 3	Block 1	3	170
	Block 2	3	341
	Block 3	3	511
Block Wall 4	Block 1	4	227
	Block 2	4	454
	Block 3	4	681
Block Wall 5	Block 1	5	284
	Block 2	5	568
	Block 3	5	851

I.5.9. MATERIAL COSTS

The following construction items are found:

Concrete with formwork	125	€$/m^3$
Rubble	32	€$/m^3$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL (€$/m^3$ or €$/m^2$)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.44.

The rubble mound foundation is provided with general dimensions. The length is chosen to be twice the width of the block and the thickness is assumed to suffice with 1 m. The foundation is highly depended on the subsoil. For example, a weaker layer requires more excavation for a stable foundation.

Table I.44: Material Costs of Block Wall Breakwater

Class	*Applied Blocks*	*Cost of Concrete (€/m)*	*Cost of Rubble (€/m)*	*Total Cost (€/m)*
Block Wall 1	Block 1	500	128	628
	Block 2	1000	256	1256
	Block 3	1500	384	1884
Block Wall 2	Block 1	1000	128	1128
	Block 2	2000	256	2256
	Block 3	3000	384	3384
Block Wall 3	Block 1	1500	128	1628
	Block 2	3000	256	3256
	Block 3	4500	384	4884
Block Wall 4	Block 1	2000	128	2128
	Block 2	4000	256	4256
	Block 3	6000	384	6384
Block Wall 5	Block 1	2500	128	2628
	Block 2	5000	256	5256
	Block 3	7500	384	7884

I.5.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Excavation of subsoil	40	$m^3/hour$
Placement of foundation	40	$m^3/hour$
Equalizing foundation	35	€$/m$
Transport vessel with crawler crane	225	€$/hour$
Pontoon to carry structure elements	25	€$/hour$
Placement of blocks	6	$units/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.45 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.45: Labour and Equipment Costs of Block Wall Breakwater

Class	*Applied Blocks*	*Excavation of subsoil (hours/m)*	*Placement of foundation (hours/m)*	*Equalizing foundation (€/m)*	*Placement of blocks (hours/m)*	*Transport vessel with crawler crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
Block Wall 1	Block 1	0.1	0.1	140	0.04	54	6	200
	Block 2	0.2	0.2	280	0.02	95	11	385
	Block 3	0.3	0.3	420	0.01	138	15	573
Block Wall 2	Block 1	0.2	0.2	280	0.08	109	12	401
	Block 2	0.4	0.4	560	0.04	189	21	770
	Block 3	0.6	0.6	840	0.03	276	31	1147
Block Wall 3	Block 1	0.3	0.3	420	0.13	163	18	601
	Block 2	0.6	0.6	840	0.06	284	32	1156
	Block 3	0.9	0.9	1260	0.04	414	46	1720
Block Wall 4	Block 1	0.4	0.4	560	0.04	189	21	770
	Block 2	0.8	0.8	1120	0.08	379	42	1541
	Block 3	1.2	1.2	1680	0.06	553	61	2294
Block Wall 5	Block 1	0.5	0.5	700	0.04	234	26	960
	Block 2	1.0	1.0	1400	0.10	473	53	1926
	Block 3	1.5	1.5	2100	0.07	691	77	2867

I.5.11. CODES AND GUIDELINES

Design rules of block wall breakwaters are to be found in PIANC/Marcom 28 (2003) and British Standards Institution [BS] (1991).

I.6. FLOATING BREAKWATER

I.6.1. GENERAL

Floating breakwaters have shown their functionality in damping short waves. Long waves are difficult to damp out, since it requires a lot of energy to dissipate these waves. For inland waters these structures could damp up to 90% of the wave energy. Floating breakwaters consist of reinforced concrete boxes, which are water tight or filled with material that exclude air inside.

Figure I.14: Floating Breakwater
(Maritime Journal. (2005). *Breakwater Beats the Weather at Holy Loch*. Retrieved from http://www.maritimejournal.com/. Accessed on November 10, 2014.)

I.6.2. SCHEME

The floating breakwater consists of a concrete box structure, which is filled with air or foam. To fix the breakwater to its location, piles or steel cables with concrete anchor are applied. The scheme is shown in Figure I.15.

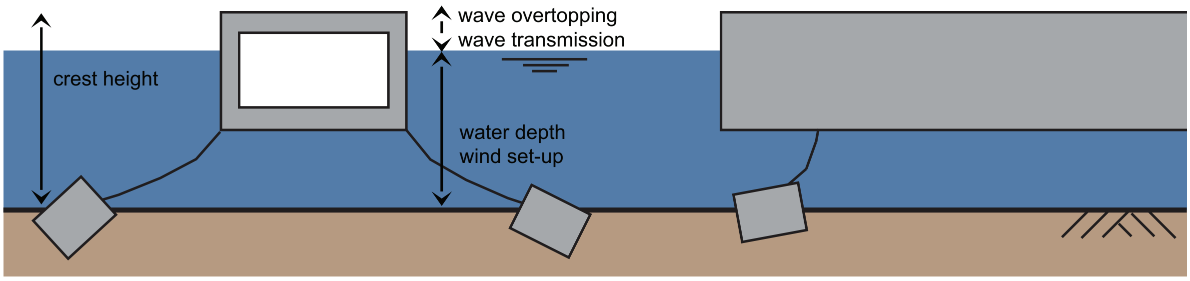


Figure I.15: Cross-sectional View (left) and Side View (right) of Floating Breakwater

I.6.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.6.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. Due to the water depth and the ice forces this alternative can be repelled.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

For the water depth:

Table I.46: Classification of Water Depth for Floating Breakwater

Water Depth (m)	*Application*
1	Able
1.5	Able
2	Able
2.5	Able
3	Able
3.5	Able
4	Able

It is assumed that ice forces cannot be carried by the cables and anchors designed for waves.

I.6.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.47: EMVI Criteria Score of Floating Breakwater

Criterion	*Score*
System quality	2
Innovation	4
Ecological impact	1
CO2 ambition level	2
Durability	2
Hindrance	3
Noise	3
Risks	2

I.6.6. CLASSIFICATION

The considered floating breakwater classes are shown in Table I.48 and I.49.

Table I.48: Floating Breakwater Classes (Inter Boat Marina, 2015)

Class	*Length (m)*	*Width (m)*	*Height (m)*	*Mass (ton)*	*Mass (Freeboard (m)*
Inter Boat Marina M2716	16.1	2.6	1.0	17.4	0.53
Inter Boat Marina M3316	16.1	3.2	1.0	20.8	0.55
Inter Boat Marina M3816	16.1	3.7	1.2	31.5	0.63
Inter Boat Marina M4316K	16.1	4.2	1.8	33.6	0.60

Table I.49: Floating Breakwater Classes

Class	*Length (m)*	*Width (m)*	*Height (m)*	*Mass (ton)*	*Freeboard (m)*
Macagno Floating Breakwater A	20	6	2.6	124.3	1.5
Macagno Floating Breakwater B	20	8	3.4	170.1	2.4
Macagno Floating Breakwater C	20	10	4.3	215.8	5.4

In practice, per 16.1 or 20 m of length four anchors are applied. These anchors consist of a steel chain with concrete block or Seaflex construction. Only the steel chain alternative is considered, since it is generally applied.

I.6.7. STRUCTURAL APPROACH

The floating breakwater types mentioned in the previous section are designed by an engineering company. The corresponding wave attenuation from laboratory tests is visualized in Figure I.16.

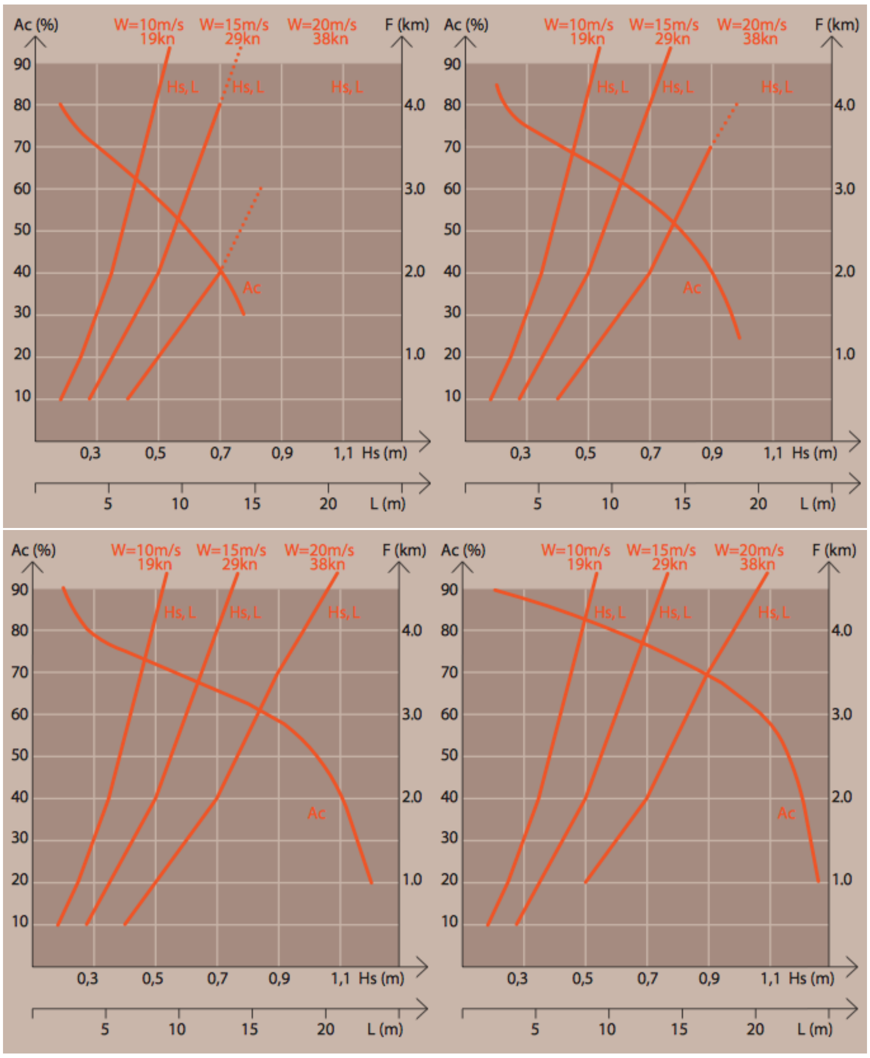


Figure I.16: Inter Boat Marinas floating Breakwater Performance
(M2700 and M3300 are on top, 3800 and M4300K are at bottom)

In the figure the following variables can be recognised:

F Effective fetch length (m)
W Wind velocity (m)
L Wave length (m)
H_s Significant wave height (m)
A_c Wave attenuation capacity (%)
H_t Transmitted wave height (m)
A_t Wave transmission capacity (%)

To achieve a functional and effective structure, the static and dynamic stability have been investigated. Also the equilibrium of vertical forces and bending moment are part of the static stability. In case of the dynamic stability, the natural frequencies are considered. These should not be close to the frequency of the waves.

Other dimensions of floating breakwaters are considered with Macagno's formula. This analytical method is to predict the transmission coefficient depending on the wave number (k), water depth (d), breakwater draught (D) and breakwater width (B) (Ruol et al., 2013).

$$K_t = 1/\sqrt{1+\left(kB\frac{\sinh kd}{2\cosh kd - kD}\right)^2} \tag{I.20}$$

The performance and applicability depends on the wave height and the type of floating breakwater. The larger breakwater can absorb more wave energy. A designer should compare the design wave height with the maximum allowable transmitted wave height.

I.6.8. DIMENSIONS

The width is important to determine the overall dimensions. One should consider the following aspect:

Wave transmission

The interaction of the classified wave heights and the floating breakwaters are shown in Table I.50.

It can be observed that the performances in Table I.50 andI.51 are approximately similar, while the dimensions vary significantly. For example, the Inter Boat Marine M4316 transmits 40% and the Macagno Floating breakwater 90% of a wave height of 1 m. This implies that properties of one is incorrect. A designer should take into account the risk of choosing a floating breakwater which has an insufficient performance. What is more, the classified floating breakwaters show an affect on a maximum wave height of 1 m. It is advised to consider other breakwaters as well, which can attenuate waves up to 2 m of wave height.

Table I.50: Performance of Inter Boat Marina Floating Breakwater

Class	H_s (m)	A_c (%)	A_t (%)	H_t (m)
Inter Boat Marina M2716	0.1	100	0	0.0
	0.2	76	24	0.0
	0.25	72	28	0.1
	0.3	70	30	0.1
	0.5	57	43	0.2
	0.75	36	64	0.5
	1	-	-	-
Inter Boat Marina M3316	0.1	100	0	0.0
	0.2	85	15	0.0
	0.25	77	23	0.1
	0.3	75	25	0.1
	0.5	67	33	0.2
	0.75	55	45	0.3
	1	25	75	0.8
Inter Boat Marina M3816	0.1	100	0	0.0
	0.2	90	10	0.0
	0.25	84	16	0.0
	0.3	78	22	0.1
	0.5	72	28	0.1
	0.75	65	35	0.3
	1	53	47	0.5
Inter Boat Marina M4316K	0.1	100	0	0.0
	0.2	90	10	0.0
	0.25	89	11	0.0
	0.3	88	12	0.0
	0.5	82	18	0.1
	0.75	75	25	0.2
	1	64	36	0.4

Table I.51: Performance of Macagno Floating Breakwater

Class	$D\ (m)$	$H_s\ (m)$	$L\ (m)$	$k\ (rad/m)$	$d\ (m)$	K_t (-)
Macagno Floating Breakwater A	1.0	0.50	12.50	0.50	2	0.5
		0.75	18.75	0.34		0.8
		1.00	25.00	0.25		0.9
		1.50	37.50	0.17		1.0
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	3	0.4
		0.75	18.75	0.34		0.7
		1.00	25.00	0.25		0.9
		1.50	37.50	0.17		1.0
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	4	0.4
		0.75	18.75	0.34		0.7
		1.00	25.00	0.25		0.8
		1.50	37.50	0.17		1.0
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
Macagno Floating Breakwater B	1.1	0.50	12.50	0.50	2	0.4
		0.75	18.75	0.34		0.7
		1.00	25.00	0.25		0.9
		1.50	37.50	0.17		1.0
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	3	0.3
		0.75	18.75	0.34		0.6
		1.00	25.00	0.25		0.8
		1.50	37.50	0.17		0.9
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	4	0.3
		0.75	18.75	0.34		0.5
		1.00	25.00	0.25		0.7
		1.50	37.50	0.17		0.9
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
Macagno Floating Breakwater C	1.1	0.50	12.50	0.50	2	0.3
		0.75	18.75	0.34		0.7
		1.00	25.00	0.25		0.8
		1.50	37.50	0.17		1.0
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	3	0.3
		0.75	18.75	0.34		0.5
		1.00	25.00	0.25		0.7
		1.50	37.50	0.17		0.9
		1.75	43.75	0.14		1.0
		2.00	50.00	0.13		1.0
		0.50	12.50	0.50	4	0.2
		0.75	18.75	0.34		0.5
		1.00	25.00	0.25		0.7
		1.50	37.50	0.17		0.9
		1.75	43.75	0.14		0.9
		2.00	50.00	0.13		1.0

I.6.9. MATERIAL COSTS

The following construction items are found:

Reinforced concrete with formwork	750	€$/m^3$
Anchor	200	€$/unit$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST ($hour/m$) = QUANTITY (m^3/m or m/m) / PRICE OF MATERIAL (€$/m^3$ or €$/m^2$)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.52.

Table I.52: Properties of Floating Breakwater

Class	*Mass (ton/m)*	*Reinforced concrete (m^2/m)*
Inter Boat Marina M2716	1.1	0.4
Inter Boat Marina M3316	1.3	0.5
Inter Boat Marina M3816	2.0	0.8
Inter Boat Marina M4316K	2.1	0.8
Macagno Floating Breakwater A	4.8	1.9
Macagno Floating Breakwater B	6.5	2.6
Macagno Floating Breakwater C	8.2	3.3

Table I.53: Material Costs of Floating Breakwater

Class	*Cost of Reinforced Concrete (€/m)*	*Cost of Anchors (€/m)*	*Total Cost (€/m)*
Inter Boat Marina M2716	324	50	374
Inter Boat Marina M3316	388	50	437
Inter Boat Marina M3816	587	50	637
Inter Boat Marina M4316K	626	50	676
Macagno Floating Breakwater A	1435	40	1475
Macagno Floating Breakwater B	1949	40	1989
Macagno Floating Breakwater C	2463	40	2503

I.6.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of structure	3	$units/hour$
Transport vessel with crawler crane	225	€$/hour$
Pontoon to carry structure elements	25	€$/hour$
Placement of anchors	600	€$/unit$
Connecting structure segments	3	$units/hour$
Connecting structure segments	500	€$/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.54 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.54: Labour and Equipment Costs of Floating Breakwater

Class	*Placement of structure (hours/m)*	*Transport vessel with crawler crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Connecting structure segments (€/m)*	Placement of anchors (€/m)	*Total Cost (€/m)*
Inter Boat Marina M2716	0.02	4.7	0.5	10.4	149.1	165
Inter Boat Marina M3316	0.02	4.7	0.5	10.4	149.1	165
Inter Boat Marina M3816	0.02	4.7	0.5	10.4	149.1	165
Inter Boat Marina M4316K	0.02	4.7	0.5	10.4	149.1	165
Macagno Floating Breakwater A	0.02	3.8	0.4	8.3	120.0	133
Macagno Floating Breakwater B	0.02	3.8	0.4	8.3	120.0	133
Macagno Floating Breakwater C	0.02	3.8	0.4	8.3	120.0	133

I.6.11. CODES AND GUIDELINES

Design rules of floating breakwaters are to be found in PIANC/Marcom 36 (1994).

I.7. TIMBER PILE BREAKWATER

I.7.1. GENERAL

Timber piles are mainly used in coastal areas to affect flow velocities and waves. The rows of piles are perpendicular positioned to the foreshore and located from the dry-zone of the beach to the breaker zone. Their primary function is to reduce erosion of beaches. However, in inland waters piled structures could deliver a contribution to the reduction of the wave height in a sheltered zone. An advantage is the low cost of the material and placement, but the disadvantage is that the lifetime is limited.

Figure I.17: Timber Pile Breakwater
(Zeelandnet. (2014). *Golfbrekers Strand Domburg*. Retrieved from http://www.zeelandnet.nl/. Accessed on November 8, 2014.)

I.7.2. SCHEME

The shape of the pile in a cross-sectional view is shown in Figure I.18. The configuration of the piles is presented with an empty space between two subsequent piles. This spacing affects the amount of wave transmission.

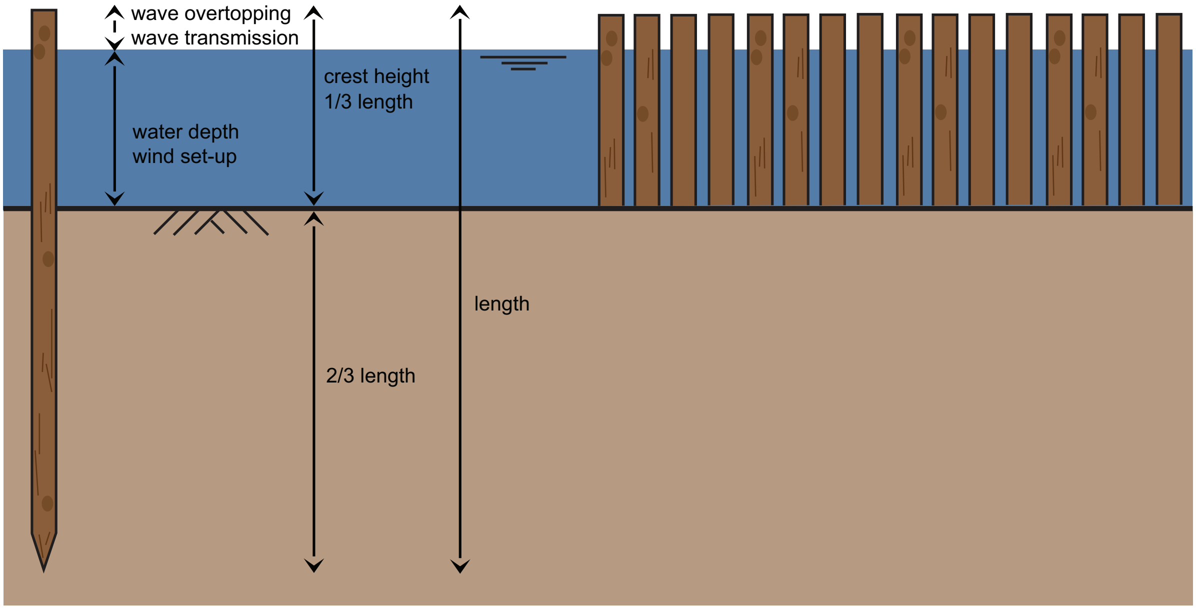


Figure I.18: Cross-sectional View (left) and Side View (right) of Timber Pile Breakwater

I.7.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.7.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. The ice forces can recommend to repel this alternative.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

The individual piles are not able to cope with ice. On the contrary, earthquakes are providing no problems. When displacements occur, the stability of a pile will not be affected. Only the porosity between the piles could be affected.

I.7.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.55: EMVI Criteria Score of Timber Pile Breakwater

Criterion	*Score*
System quality	5
Innovation	3
Ecological impact	1
CO2 ambition level	2
Durability	3
Hindrance	3
Noise	3
Risks	5

I.7.6. CLASSIFICATION

The assumed classified timber piles are shown in Table I.39.

Table I.56: Pile Breakwater Classes

Class	*Length (m)*	*Crest height (m)*
Timber Pile 1	6	2
Timber Pile 2	12	4
Timber Pile 3	18	6
Timber Pile 4	20	8

The piles are 0.2 m in diameter and can have a maximum length of 20 m. The specific type of wood is Basralocus or Azobé. The quality is assumed to be D40 which implies a design stress of 40 N/mm^2.

I.7.7. DIMENSIONS

To determine the overall dimensions, the crest height is most important. This can be computed according to the following formula:

crest height = water depth + wind set-up

The crest height should be verified with:

wave transmission and wave overtopping

I.7.8. STRUCTURAL APPROACH

The structural scheme considered is shown in the Figure I.19.

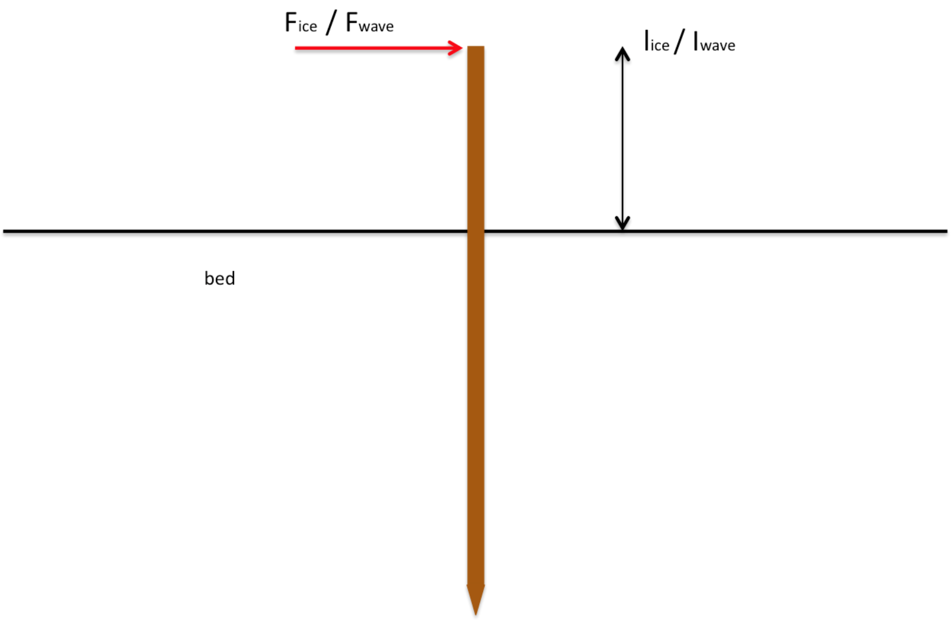


Figure I.19: Structural Model of Timber Pile Breakwater

For the stability of the pile breakwater, the bending moments are investigated. It is assumed that the bearing capacity, shear force resistance, normal force equilibrium and sliding plane are within safety limits.

$$M_{Ed} < M_{Rd} \tag{I.21}$$

$$M_{Ed} = Fl < M_{Rd} \tag{I.22}$$

$$f_{ext} < f_d \tag{I.23}$$

$$f_{ext} = M_{Ed}/W \tag{I.24}$$

$$W = \pi * D^3/32 \tag{I.25}$$

$$f_d = f_k/1.3 \tag{I.26}$$

M_{Rd} Bending moment resistance (kNm)
M_{Ed} Design bending moment (kNm)
f_{ext} Design tensile stress (N/mm^2)
f_d Maximum tensile stress (N/mm^2)
f_k Characteristic tensile stress (N/mm^2)
W Section modulus (mm^4)

The Basralocus is assumed to have a strength class D40, which refers to a maximum tensile strength in the outer fibre of 40 N/mm^2. A safety factor of 1.3 is found to determine the design stress.

The performance of the timber pile breakwater for the classified crest heights and the several loads are included in Table I.57.

Table I.57: Structural Performance of Timber Pile Breakwater

h_c (m)	F_H (kN/m)	M_{Ed} (kNm/m)	W (m^3/m)	f_{Ed} (N/mm^2)	f_{Rd} (N/mm^2)	*Applicable*
2	3.4	6.8	0.0039	1.7	31	YES
	10.1	20.3		5.2		YES
	20.3	40.5		10.3		YES
	33.8	67.5		17.2		YES
	83.0	166.0		42.3		NO
	165.0	330.0		84.0		NO
	248.0	496.0		126.3		NO
	330.0	660.0		168.1		NO
4	3.4	13.5		3.4		YES
	10.1	40.5		10.3		YES
	20.3	81.0		20.6		YES
	33.8	135.0		34.4		NO
	83.0	332.0		84.5		NO
	165.0	660.0		168.1		NO
	248.0	992.0		252.6		NO
	330.0	1320.0		336.1		NO
6	3.4	20.3		5.2		YES
	10.1	60.8		15.5		YES
	20.3	121.5		30.9		NO
	33.8	202.5		51.6		NO
	83.0	498.0		126.8		NO
	165.0	990.0		252.1		NO
	248.0	1488.0		378.9		NO
	330.0	1980.0		504.2		NO
8	3.4	27.0		6.9		YES
	10.1	81.0		20.6		YES
	20.3	162.0		41.3		NO
	33.8	270.0		68.8		NO
	83.0	664.0		169.1		NO
	165.0	1320.0		336.1		NO
	248.0	1984.0		505.2		NO
	330.0	2640.0		672.3		NO
10	3.4	33.8		8.6		YES
	10.1	101.3		25.8		YES
	20.3	202.5		51.6		NO
	33.8	337.5		85.9		NO
	83.0	830.0		211.4		NO
	165.0	1650.0		420.2		NO
	248.0	2480.0		631.5		NO
	330.0	3300.0		840.3		NO

The wave transmission is an important feature. Wiegel (1961) found a relation between the spacing of the piles and the transmitted wave.

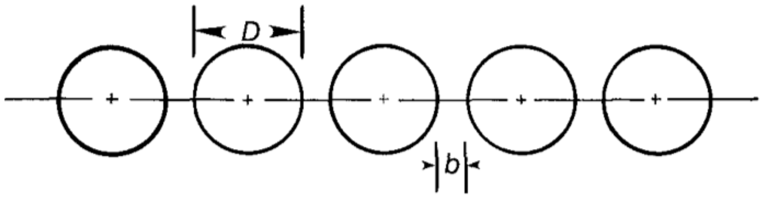


Figure I.20: Spacing of Timber Pile Breakwater

Including the definition in Figure I.20, the equation to determine the wave transmission reads:

$$K_t = H_t / H_i = \frac{b}{D+b} \tag{I.27}$$

In which:

D Diameter of the piles (m)
b Spacing (m)

Experiments found that the predicted wave height was 25 percent higher. This leads to the following formula:

$$H_t = 1.25 \frac{b}{D+b} H_i \tag{I.28}$$

To determine the wave transmission, the spacings in Table I.59 is chosen.

Table I.58: Spacing of Timber Pile Breakwater

b (m)
0.02
0.04
0.10
0.15

The classified spacings and wave heights provide the transmitted wave heights in Table I.59.

Table I.59: Transmitted Wave Height

H_s (m) / b (m)	0.02	0.04	0.10	0.15
0.10	0.01	0.02	0.04	0.05
0.20	0.02	0.04	0.08	0.11
0.25	0.03	0.05	0.10	0.13
0.30	0.03	0.06	0.13	0.16
0.50	0.06	0.10	0.21	0.27
0.75	0.09	0.16	0.31	0.40
1.00	0.11	0.21	0.42	0.54
1.50	0.17	0.31	0.63	0.80
1.75	0.20	0.36	0.73	0.94
2.00	0.23	0.42	0.83	1.00

I.7.9. MATERIAL COSTS

The following construction items are found:

Basralocus	950	€/m^3

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts of the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY (m^3/m or m/m) / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.60.

Table I.60: Material Costs of Timber Pile Breakwater

Class	*L (m)*	*s (m)*	*D (m)*	*V ($m^3/pile$)*	*Cost of Piles (€/m)*
Timber Pile 1	6	0.02	0.2	0.19	814
		0.04			746
		0.10			597
		0.15			512
Timber Pile 2	12	0.02		0.38	1628
		0.04			1492
		0.10			1194
		0.15			1023
Timber Pile 3	18	0.02		0.57	2442
		0.04			2238
		0.10			1791
		0.15			1535
Timber Pile 4	20	0.02		0.63	2713
		0.04			2487
		0.10			1990
		0.15			1705
Timber Pile 5	20	0.02		0.63	2713
		0.04			2487
		0.10			1990
		0.15			1705

I.7.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of piles	100	$m/hour$
Transport vessel with hydraulic	250	€/$hour$
Vibration block	100	€/$hour$
Pontoon to carry structure elements	25	€/$hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€/m) = EQUIPMENT (€/$hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€/m) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.61 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.61: Labour and Equipment Costs of Timber Pile Breakwater

Class	L_{embed} *(m)*	*s (m)*	*Placement of piles (hours/m)*	*Vibration block (€/m)*	*Transport vessel with hydraulic (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
Timber Pile 1	4	0.02	0.18	18.2	45.5	4.5	68
		0.04	0.17	16.7	41.7	4.2	63
		0.10	0.13	13.3	33.3	3.3	50
		0.15	0.11	11.4	28.6	2.9	43
Timber Pile 2	8	0.02	0.36	36.4	90.9	9.1	136
		0.04	0.33	33.3	83.3	8.3	125
		0.10	0.27	26.7	66.7	6.7	100
		0.15	0.23	22.9	57.1	5.7	86
Timber Pile 3	12	0.02	0.55	54.5	136.4	13.6	205
		0.04	0.50	50.0	125.0	12.5	188
		0.10	0.40	40.0	100.0	10.0	150
		0.15	0.34	34.3	85.7	8.6	129
Timber Pile 4	12	0.02	0.55	54.5	136.4	13.6	205
		0.04	0.50	50.0	125.0	12.5	188
		0.10	0.40	40.0	100.0	10.0	150
		0.15	0.34	34.3	85.7	8.6	129
Timber Pile 5	10	0.02	0.45	45.5	113.6	11.4	170
		0.04	0.42	41.7	104.2	10.4	156
		0.10	0.33	33.3	83.3	8.3	125
		0.15	0.29	28.6	71.4	7.1	107

I.7.11. CODES AND GUIDELINES

No codes and guidelines are available to the timber pile breakwater. Instead experience from practice is used.

I.8. TIRE BREAKWATER

I.8.1. GENERAL

A low-cost solution is a breakwater consisting of tires (Figure I.21). These tires from cars are normally used and run-down, but sufficiently strong to trap air and to create a chain of tires. The tire breakwater is mainly found in front of small fish farms and inland marinas (McGregor and Miller, 1978).

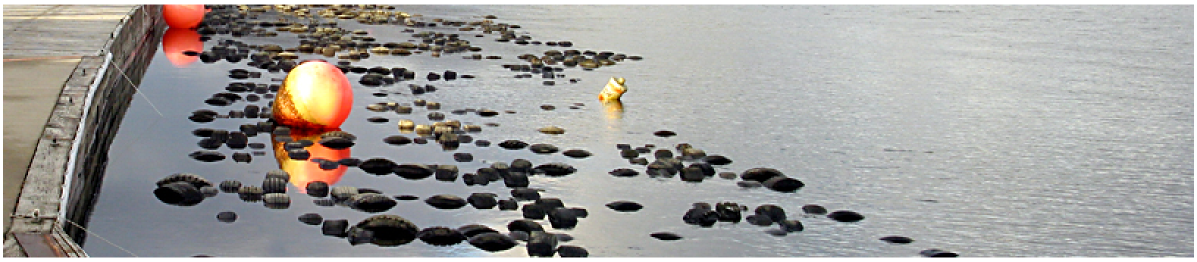


Figure I.21: (Floating) Tire Breakwater
(Westport Marina. (2008). *Floating tire breakwater along front dock for winter.* Retrieved from http://www.westportmarina.com/. Accessed on November 10, 2014.)

I.8.2. SCHEME

In Figure I.22 the cross-sectional view shows that the tires are alternately positioned. From the sides the tires can be closed to trap air inside. It is also said that the air in the upper part of the tire is sufficient for floating.

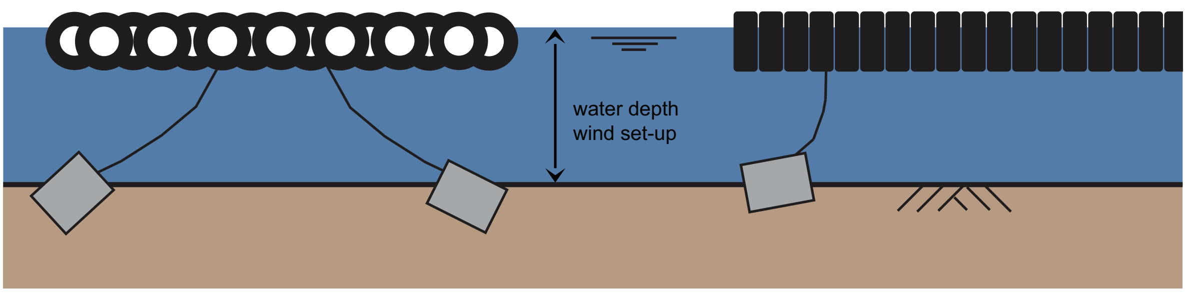


Figure I.22: Cross-sectional View (left) and Side View (right) of Tire Breakwater

I.8.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.8.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. This alternative is repelled in case of ice forces. These forces could either merely displace or damage the structure.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

I.8.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.62: EMVI Criteria Score of Tire Breakwater

Criterion	*Score*
System quality	5
Innovation	5
Ecological impact	3
CO2 ambition level	1
Durability	4
Hindrance	3
Noise	3
Risks	5

I.8.6. CLASSIFICATION

Multiple types of tires are applicable. Generally, car tires are used with the characteristics in Table I.63.

Table I.63: Tire Breakwater Classes

Class	*Width (m)*
Tire Breakwater 1	3
Tire Breakwater 2	6
Tire Breakwater 3	9
Tire Breakwater 4	12
Tire Breakwater 5	15

The car tires are 15 inch in inner diameter, 18 inch in outer diameter and 0.2 m in width.

I.8.7. STRUCTURAL APPROACH

The tire breakwater dimensions depend on the wave properties only. More specific, the width of the tire mat is a function of the wave length Harms (1980). Figure I.23 shows the transmission coefficient (C_t) compared to the ratio wave length (L) and breakwater width (B).

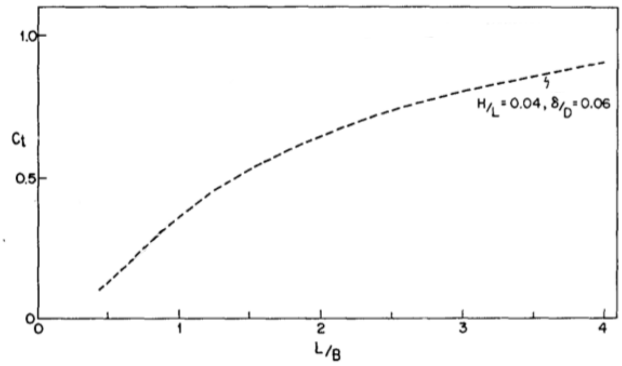


Figure I.23: Tire Breakwater Wave Transmission
(Harms, 1980)

The design wave heights are input for the computation of the breakwater width in Table I.65. The width range is between 1 m and 125 m. But a more realistic upper limit for the width is assumed to be 15 m. Thus, tire breakwaters wider than 15 m are out of scope.

I.8.8. DIMENSIONS

As can been seen in the previous section, the only dimension considered is the width. The crest height does not play parts. Thus, design aspect like wind set-up, wave run-up and wave overtopping are not considered.

I.8.9. MATERIAL COSTS

The following construction items are found:

Tires	0.2	€/$unit$
Anchor	200	€/$unit$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST ($hour/m$) = QUANTITY (m^3/m or m/m) / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.64. The tether/anchors are assumed to be positioned at 3 m center to center over the length of the breakwater.

Table I.64: cost of Cross-section of Tire Breakwater

Class	*B (m)*	*Number of tires over width*	*Number of tires per meter length*	*Number of tires*	*Cost of Tires (€/m)*	*Cost of Anchors (€/m)*	Total Cost (€/m)
Tire Breakwater 1	3	7	5	33	7	25	32
Tire Breakwater 2	6	13	5	66	13	25	38
Tire Breakwater 3	9	20	5	98	20	25	45
Tire Breakwater 4	12	26	5	131	26	25	51
Tire Breakwater 5	15	33	5	164	33	25	58

Table I.65: Width Determination of Tire Breakwater

H_0 (m)	L_0 (m)	K_t (-)	L/B (-)	B (m)	Applicable
0.1	2.5	0.1	0.4	6	YES
		0.2	0.7	4	YES
		0.3	0.9	3	YES
		0.4	1.2	2	YES
		0.5	1.8	1	YES
0.2	5	0.1	0.4	13	NO
		0.2	0.7	8	YES
		0.3	0.9	6	YES
		0.4	1.2	4	YES
		0.5	1.8	3	YES
0.25	6.25	0.1	0.4	16	NO
		0.2	0.7	10	YES
		0.3	0.9	7	YES
		0.4	1.2	5	YES
		0.5	1.8	3	YES
0.3	7.5	0.1	0.4	19	NO
		0.2	0.7	12	YES
		0.3	0.9	8	YES
		0.4	1.2	7	YES
		0.5	1.8	4	YES
0.5	12.5	0.1	0.4	31	NO
		0.2	0.7	19	NO
		0.3	0.9	14	YES
		0.4	1.2	11	YES
		0.5	1.8	7	YES
0.75	18.75	0.1	0.4	47	NO
		0.2	0.7	29	NO
		0.3	0.9	21	NO
		0.4	1.2	16	NO
		0.5	1.8	10	YES
1	25	0.1	0.4	63	NO
		0.2	0.7	38	NO
		0.3	0.9	28	NO
		0.4	1.2	22	NO
		0.5	1.8	14	YES
1.5	37.5	0.1	0.4	94	NO
		0.2	0.7	58	NO
		0.3	0.9	42	NO
		0.4	1.2	33	NO
		0.5	1.8	21	NO
1.75	43.75	0.1	0.4	109	NO
		0.2	0.7	67	NO
		0.3	0.9	49	NO
		0.4	1.2	38	NO
		0.5	1.8	24	NO
2	50	0.1	0.4	125	NO
		0.2	0.7	77	NO
		0.3	0.9	56	NO
		0.4	1.2	43	NO
		0.5	1.8	28	NO

I.8.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Connecting structure segments	5	$€/tire$
Connecting the tires	100	$€/hour$
Connecting the tires	20	$units/hour$
Transport vessel with hydraulic crane	250	$€/hour$
Placement of structure	3	$units/hour$
Placement of anchors	600	$€/unit$
Connecting structure segments	500	$€/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 ($€/m$) = EQUIPMENT ($€/hour$) · HOURS OF WORKS($hour/m$)
- COST2 ($€/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.66 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.66: Labour and Equipment Costs of Tire Breakwater

Class	*Connecting tire (hours /m)*	*Connecting tire (€/m)*	*Transport vessel with hydraulic crane (€/m)*	*Placement of anchors (€/m)*	*Connecting structure segments (€/m)*	*Total Cost (€/m)*
Tire Breakwater 1	1.6	164.0	5.6	200	11.1	381
Tire Breakwater 2	3.3	328.1	5.6	200	11.1	545
Tire Breakwater 3	4.9	492.1	5.6	200	11.1	709
Tire Breakwater 4	6.6	656.2	5.6	200	11.1	873
Tire Breakwater 5	8.2	820.2	5.6	200	11.1	1037

I.8.11. CODES AND GUIDELINES

No codes and guidelines are available to the tire breakwater. Instead experience from practice is used.

I.9. REEF BALL BREAKWATER

I.9.1. GENERAL

A more ecologically sound structure is the semi-permeable reef ball breakwater (Figure I.24). This concrete structure has gaps at all sides and provides shelter area for fish and living area for plants.

Figure I.24: Reef Ball Breakwater
(Reef Innovations. (2013). *Breakwater Construction.* Retrieved from http://reefinnovations.com/. Accessed on November 10, 2014.)

I.9.2. SCHEME

Generally, the reef ball breakwater is submerged, because the height of the structure is limited. The breakwater is typically used in shallow waters with water depths around a meter. For reasons of stability the foundation consists of rubble (Figure I.25).

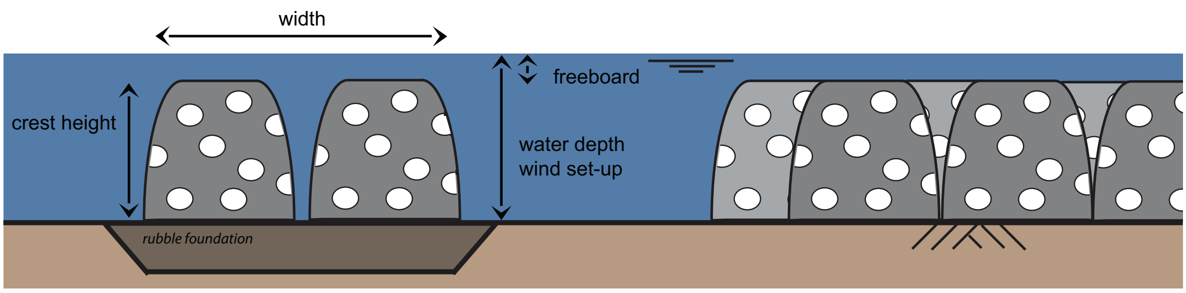


Figure I.25: Cross-sectional View (left) and Side View (right) of Reef Ball Breakwater

I.9.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.9.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below. The ice forces can recommend to repel this alternative.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

It is expected that the reef ball units will displace when a force is exerted on the structure from ice.

I.9.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.67: EMVI Criteria Score of Reef Ball Breakwater

Criterion	*Score*
System quality	3
Innovation	4
Ecological impact	2
CO2 ambition level	4
Durability	2
Hindrance	3
Noise	3
Risks	3

I.9.6. CLASSIFICATION

The reef ball classes considered are enclosed in Table I.68.

Table I.68: Reef Ball Classes

Style	*Width (m)*	*Height (m)*	*Weight (kg)*	*Concrete Volume (m^3)*
Bay Ball	0.91	0.61	255	0.08
Pallet Ball	1.22	0.88	841	0.25
Ultra Ball	1.68	1.31	1818	0.76

I.9.7. STRUCTURAL APPROACH

The structural performance of the reef ball breakwater is found in the horizontal, vertical and rotational stability. Since the breakwater is submerged, wave impact is limited to orbital motion, which refer to drag and lift forces to be considered. It is assumed that the structure is stable and that the dimensions only depend on the wave transmission.

I.9.8. DIMENSIONS

To determine the overall dimensions, the crest height is most important. It should be compared with the water depth which can be computed as follows:

Water depth = still water depth + wind set-up

The crest height should be verified with:

Wave transmission

The wave transmission is an important feature of the reef ball breakwater. d'Angremond et al. (1996) found a relation between the free board, initial wave and the transmitted wave. The equation reads:

$$K_t = a\frac{R_c}{H_i} + b * (1 - e^{-0.5\xi_{op}}) \tag{I.29}$$

The range of K_t is from 0.075 to 0.8, and the parameters in the formula are:

$a = 0.4$
$b = 0.64(B/H_i)^{-0.31}$

When the free board, R_c (m), is positive, spilling waves are assumed. Consequently, ξ_{op} becomes 0.5 (or lower). In case of a negative free board, a steep structure can be assumed, which results in a ξ_{op} of 5 (or higher). The wave transmission for one and two rows of reef ball are computed and shown in Table I.69.

Table I.69: Wave Transmission of Reef Ball Breakwater

					1 Row		*2 Row*	
h_c (m)	$Diameter$ (m)	d (m)	ξ (-)	H_s (m)	K_t (-)	H_t (m)	K_t (0)	H_t (m)
0.61	0.91	1	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.70	0.18	0.69	0.17
				0.3	0.60	0.18	0.59	0.18
				0.5	0.41	0.21	0.39	0.20
		1.5	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.80	0.20	0.80	0.20
				0.3	0.80	0.24	0.80	0.24
				0.5	0.80	0.40	0.79	0.40
				0.75	0.59	0.44	0.56	0.42
		2	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.80	0.20	0.80	0.20
				0.3	0.80	0.24	0.80	0.24
				0.5	0.80	0.40	0.80	0.40
				0.75	0.80	0.60	0.80	0.60
				1	0.68	0.68	0.65	0.65
0.88	1.22	1	0.5	0.1	0.53	0.05	0.80	0.08
				0.2	0.31	0.06	0.80	0.16
				0.25	0.26	0.07	0.68	0.17
				0.3	0.24	0.07	0.58	0.17
				0.5	0.19	0.09	0.38	0.19
		1.5	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.80	0.20	0.80	0.20
				0.3	0.80	0.24	0.80	0.24
				0.5	0.59	0.29	0.78	0.39
				0.75	0.43	0.32	0.56	0.42
		2	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.80	0.20	0.80	0.20
				0.3	0.80	0.24	0.80	0.24
				0.5	0.80	0.40	0.80	0.40
				0.75	0.70	0.52	0.80	0.60
				1	0.56	0.56	0.65	0.65
1.31	1.68	1	5	0.1	0.08	0.01	0.80	0.08
				0.2	0.08	0.02	0.80	0.16
				0.25	0.08	0.02	0.80	0.20
				0.3	0.08	0.02	0.78	0.24
				0.5	0.13	0.07	0.62	0.31
		1.5	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.44	0.09	0.80	0.16
				0.25	0.37	0.09	0.80	0.20
				0.3	0.32	0.10	0.80	0.24
				0.5	0.23	0.12	0.78	0.39
				0.75	0.19	0.15	0.55	0.41
		2	0.5	0.1	0.80	0.08	0.80	0.08
				0.2	0.80	0.16	0.80	0.16
				0.25	0.80	0.20	0.80	0.20
				0.3	0.80	0.24	0.80	0.24
				0.5	0.63	0.32	0.80	0.40
				0.75	0.46	0.35	0.80	0.60
				1	0.38	0.38	0.64	0.64

I.9.9. MATERIAL COSTS

The following construction items are found:

Concrete with formwork	125	€/m^3
Rubble	32	€/m^3

These result in the following costs for the classed reef balls:

Table I.70: Unit Cost of Reef Ball Breakwater

Class	*Cost of Concrete (€/unit)*
Bay Ball	10
Pallet Ball	31
Ultra Ball	95

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST ($hour/m$) = QUANTITY (m^3/m or m/m) / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.71 and I.72. Moreover, the rubble mound foundation is provided with general dimensions. The length is chosen to be twice the width of the reef balls and the thickness is assumed to suffice with 1 m. The foundation is highly depended on the subsoil.

Table I.71: Material Costs of Reef Ball Breakwater (1 Row)

Class	*Cost of Rubble (€/m)*	*Cost of Concrete (€/m)*	*Total Cost (€/m)*
Bay Ball	58	11	47
Pallet Ball	78	26	74
Ultra Ball	107	57	124

Table I.72: Material Costs of Reef Ball Breakwater (2 Row)

Class	*Cost of Rubble €/m)*	*Cost of Concrete (€/m)*	*Total Cost (€/m)*
Bay Ball	116	22	95
Pallet Ball	155	51	149
Ultra Ball	214	113	247

I.9.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Excavation of subsoil	40	$m^3/hour$
Placement of foundation	40	$m^3/hour$
Equalizing foundation	35	€/m^2
Placement of reef balls	6	$units/hour$
Transport vessel with hydraulic crane	250	€/$hour$
Pontoon to carry structure elements	25	€/$hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€/m) = EQUIPMENT (€/$hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€/m) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.73 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.73: Labour and Equipment Costs of Reef Ball Breakwater

Class	*Number of Rows*	*Excavation of subsoil (hours/m)*	*Placement of foundation (hours/m)*	*Equalizing foundation (€/m)*	*Placement of reef balls (hours/m)*	*Transport vessel with hydraulic crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
Bay Ball	1 row	0.05	0.05	63.7	0.18	68.5	6.9	139
Pallet Ball		0.06	0.06	85.4	0.14	64.7	6.5	157
Ultra Ball		0.08	0.08	117.6	0.10	66.8	6.7	191
Bay Ball	2 row	0.09	0.09	127.4	0.18	91.3	9.1	228
Pallet Ball		0.12	0.12	170.8	0.14	95.2	9.5	275
Ultra Ball		0.17	0.17	235.2	0.10	108.8	10.9	355

I.9.11. CODES AND GUIDELINES

No codes and guidelines are available to the reef ball breakwater. Instead experience from practice is used.

I.10. GABION BREAKWATER

I.10.1. GENERAL

Gabions consist of baskets with stones and are generally seen along rivers. These are mainly applied as bank and bed protection to prevent erosion. In some cases small-scale weirs and breakwaters can be designed with gabions, as in Figure I.26.

To overcome the need of large stones and to come with an alternative on steel cages, the company Kyowa came up with the idea of nets of synthetic materials filled with stones. This is an innovative solution. However, these are not considered in the method.

Figure I.26: Gabion Breakwater
(EcoCoast Contracting LLC. (2014). *Products Catalog*. Retrieved from http://ecocoast.com/. Accessed on November 12, 2014.)

I.10.2. SCHEME

A typical cross-section of a gabion breakwater is shown in Figure I.27. A rubble mound layer is provided to obtain a stable foundation to counteract extensive settlement. Subsequently, the gabions are placed on top of the foundation.

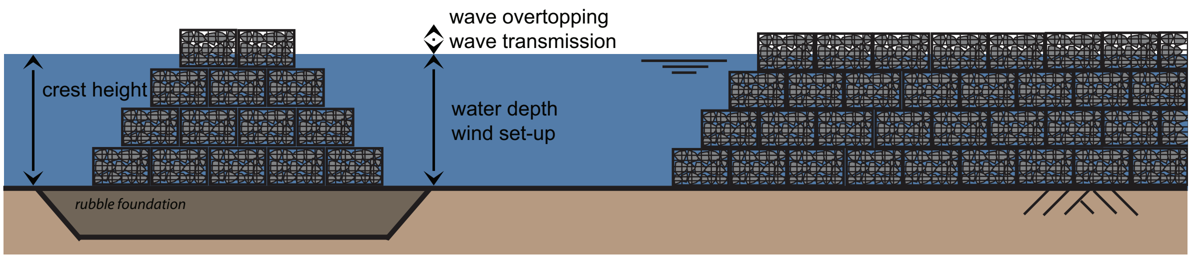


Figure I.27: Cross-sectional View (left) and Side View (right) of Gabion Breakwater

I.10.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.10.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

There are no boundary conditions, which could instantaneously repel this breakwater alternative. It is expected that the structure can withstand ice. The subsoil could provide significant settlements and deformation to which the gabions are not sensitive.

For the subsoil conditions:

Table I.74: Classification of Design Soil for Gabion Breakwater

Name	*Deformation*
Loam	Medium
Light Clay	Medium
Light Sand	Medium
Organic Sand	Medium
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

I.10.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.75: EMVI Criteria Score of Gabion Breakwater

Criterion	*Score*
System quality	4
Innovation	5
Ecological impact	1
CO2 ambition level	4
Durability	4
Hindrance	3
Noise	3
Risks	4

I.10.6. CLASSIFICATION

The gabion classes are considered are in Table I.76. The configuration is the number of boxing at the lowest layer to the highest.

The baskets are assumed to be 1 m in height, in width and in depth. The wire diameter fluctuates between 2 and 3.4 mm.

Table I.76: Gabion Classes

Class	*Crest Height (m)*	*Gabion Configuration (−)*
Gabion 1	2	3-2
Gabion 2	4	5-4-3-2
Gabion 3	6	7-6-5-4-3-2
Gabion 4	8	9-8-7-6-5-4-3-2
Gabion 5	10	11-10-9-8-7-6-5-4-3-2

I.10.7. DIMENSIONS

To determine the overall dimensions, the crest height is important. This can be computed according to the following formula:

Crest height = water depth + wind set-up + run-up

The crest height should be verified with:

Wave transmission and wave overtopping

I.10.8. STRUCTURAL APPROACH

The gabion breakwater is gravity-based structure due to the stability obtained from its self weight. For the stability checks the bearing force, sliding force and overturning moment should be taken into account Global Synthetics (2015) and Enviromesh (2015). The structural model to consider is shown in Figure I.28.

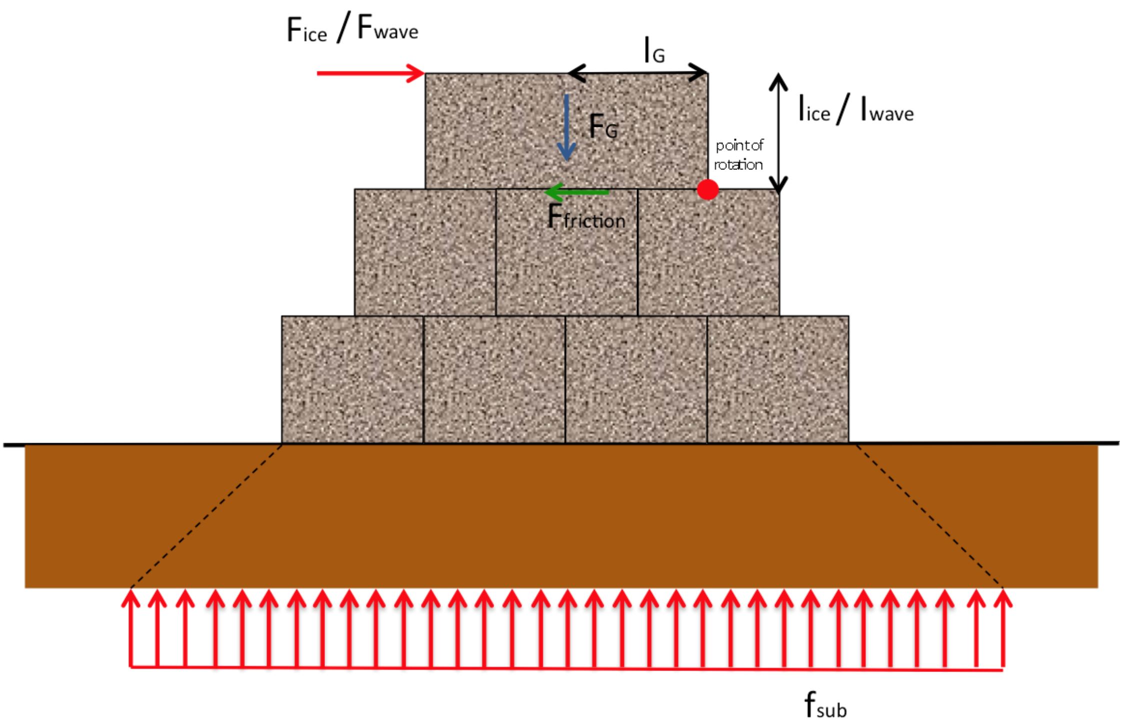


Figure I.28: Structural Model Gabion Breakwater

The requirement for the equilibrium of the horizontal forces, which is preventing sliding of the structure, is:

$$F_{fric} > F_H \tag{I.30}$$

$$F_{fric} = F_G * \delta \tag{I.31}$$

The friction factor between rubble and rubble is assumed to be 0.5.

The requirement for the equilibrium of the rotations is:

$$M_E < M_R \tag{I.32}$$

This can be written as:

$$F_H l_H < F_G l_G \tag{I.33}$$

Where:

F_H $= F_{ice}$ or F_{wave}
l_H $= l_{ice}$ or l_{wave}

In which:

F_H Horizontal load (kN)
l_H Arm of loading (m)
M_R Rotational resistance (kNm)
M_E Design rotation (kNm)
F_{fric} Friction force (kN)

Assuming that the double upper gabions are connected, the following overturning check can be performed:

Table I.77: Structural Performance of Gabion Breakwater

F_H (kN/m)	l_H (m)	M_E (kNm/m)	F_G (kN/m)	l_G (m)	M_R (kNm/m)	*Applicable (rotation)*	F_{fric} (kN)	*Applicable (friction)*
3.4	1	3	32	1	32	YES	16	YES
10.1	1	10	32	1	32	YES	16	YES
20.3	1	20	32	1	32	YES	16	NO
33.8	1	34	32	1	32	NO	16	NO
83.0	1	83	32	1	32	NO	16	NO
165.0	1	165	32	1	32	NO	16	NO
248.0	1	248	32	1	32	NO	16	NO
330.0	1	330	32	1	32	NO	16	NO

The subgrade reaction pressure is analysed by determining the vertical pressure exerted to the subsoil.

$$f_{sub} = (F_G / A_f) \tag{I.34}$$

In which:

f_{sub} Pressure on subsoil (kN/m^2)
F_G Force from selfweight (kN)
A_f Surface area of structure on foundation (m^2)

The results of the vertical pressure on the subsoil for the various breakwater classes is shown in Table I.78.

Table I.78: Soil Pressure of Gabion Breakwater

Class	A_f (m^2)	f_{sub} (kN/m^2)
Gabion 1	3	27
Gabion 2	5	45
Gabion 3	7	61
Gabion 4	9	78
Gabion 5	11	94

I.10.9. MATERIAL COSTS

To calculate the total cost of the materials, the following unity costs are used.

Stones	32	€$/m^3$
Gabion basket	50	€$/unit$
Rubble foundation	32	€$/m^3$

The stone classes for the gabions is normally 90/180 mm or 90/250. The baskets with stones from a barge result in the following cost.

Including the unity costs of the breakwater materials, the total costs can be computed by the amounts the materials required. These costs exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL (€$/m^3$ or €$/m^2$)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.79.

Table I.79: Material Costs of Gabion Breakwater

Class	*Number of boxes*	*Cost of Gabions (€/m)*	*Cost of Rubble (€/m)*	*Total Cost (€/m)*
Gabion 1	5	350	120	470
Gabion 2	14	980	200	1180
Gabion 3	27	1890	280	2170
Gabion 4	44	3080	360	3440
Gabion 5	65	4550	440	4990

The rubble mound foundation is provided with general dimensions. The length is chosen to be twice the width of the structure and the thickness is assumed to suffice with 1 m. The foundation is highly depended on the subsoil. For example, a weaker layer requires more excavation for a stable foundation.

I.10.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Transport vessel with hydraulic crane	250	€$/hour$
Pontoon to carry structure elements	25	€$/hour$
Excavation of subsoil	40	$m^3/hour$
Placement of foundation	40	$m^3/hour$
Equalizing foundation	35	€$/m$
Placement and fill cages with divers	225	€$/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.80 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.80: Labour and Equipment Costs of Gabion Breakwater

Class	*Excavation of subsoil (hours/m)*	*Placement of foundation (hours/m)*	*Equalizing foundation (€/m)*	l|*Placement of gabions (hours/m)*	*Placement and filling of gabions (€/m)*	*Transport vessel with hydraulic crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
Gabion 1	0.15	0.15	210	0.50	112.5	200	20.0	543
Gabion 2	0.25	0.25	350	1.40	315.0	475	47.5	1188
Gabion 3	0.35	0.35	490	2.70	607.5	850	85.0	2033
Gabion 4	0.45	0.45	630	4.40	990.0	1325	132.5	3078
Gabion 5	0.55	0.55	770	6.50	1462.5	1900	190.0	4323

I.10.11. CODES AND GUIDELINES

No codes and guidelines are available to the reef ball breakwater. Instead experience from practice is used.

I.11. SCREEN BREAKWATER

I.11.1. GENERAL

This breakwater alternative has much in common with the timber pile breakwater. It consists of tubular piles which hold a screen of wooden or steel plates (Figure I.29). The plates are positioned with a certain distance. This enables controlled transmission of waves. Screen breakwaters are practical and economical in regions with large tidal amplitudes and soft subsoil.

Figure I.29: Screen Breakwater
(TMS. (2014). *Breakwaters & Wave Screens*. Retrieved from http://www.tmsmaritime.co.uk/. Accessed on February 16, 2016.)

I.11.2. SCHEME

A typical cross-section of a single screen breakwater is shown in Figure I.30). In practice, double screens can be applied to obtain lower wave transmission. What is more, vertical or horizontal slots have the same results for the wave transmission and reflection.

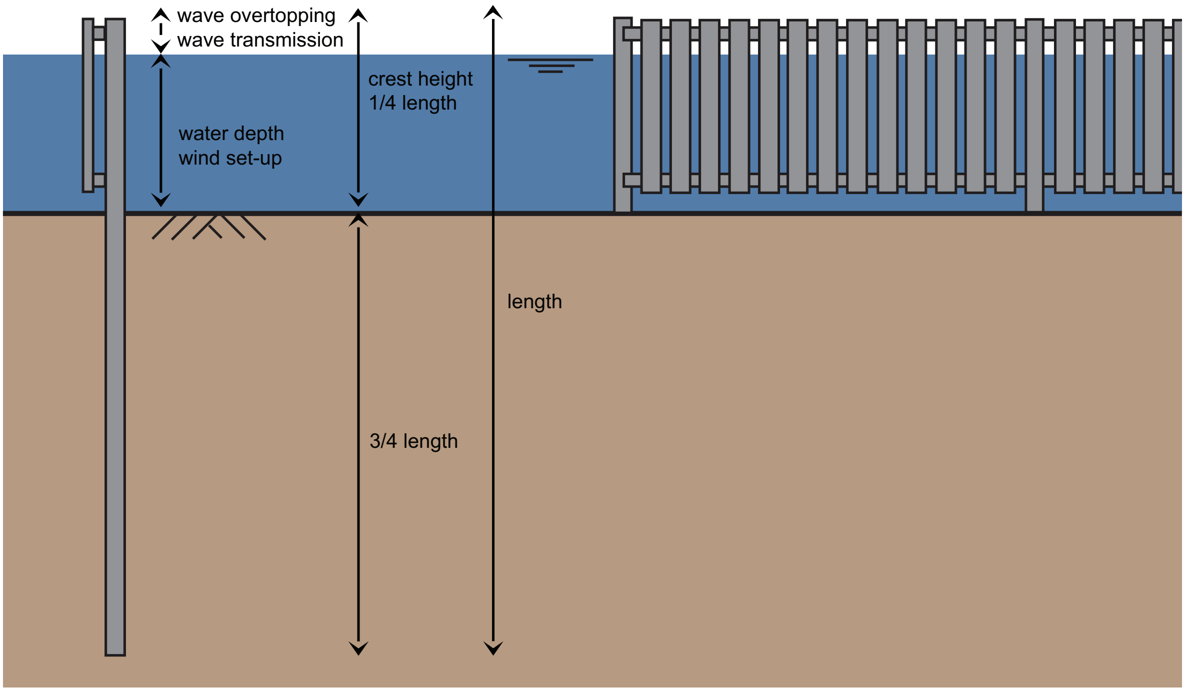


Figure I.30: Cross-sectional View (left) and Side View (right) of Screen Breakwater

I.11.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.11.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

There are no boundary conditions, which could instantaneously repel this breakwater alternative.

I.11.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.81: EMVI Criteria Score of Screen Breakwater

Criterion	*Score*
System quality	3
Innovation	4
Ecological impact	3
CO2 ambition level	2
Durability	3
Hindrance	3
Noise	3
Risks	3

I.11.6. CLASSIFICATION

The screen breakwater classes are enclosed in Table I.82.

Table I.82: Screen breakwater Classes

Class	*Length (m)*	*Crest Height (m)*	*Plate Length (m)*
Screen Breakwater 1	6	2	1.5
Screen Breakwater 2	12	4	3.5
Screen Breakwater 3	18	6	5.5
Screen Breakwater 4	24	8	7.5
Screen Breakwater 5	30	10	9.5

The gap between the bed and the lower side of the screen is about 0.5 m. Moreover, in the laboratory test a 1 m gap did not show any influence on the performance.

I.11.7. DIMENSIONS

To determine the overall dimensions, the crest height is important. This can be computed according to the following formula:

Crest height = water depth + wind set-up + run-up

The crest height should be verified with:

Wave transmission and wave overtopping

The wave transmission and reflection as a function of the porosity is presented in Figure I.31.

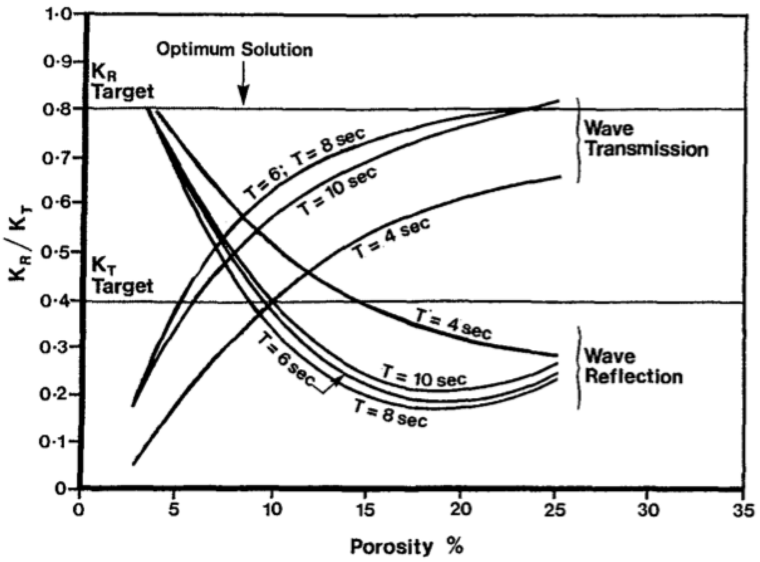


Figure I.31: Reflection coefficient (K_R) and transmission coefficient (K_T) for vertical screen breakwater (Gardner et al., 1986)

5%, 7.5% and 10% are porosities which are assumed to obtain sufficient wave reduction, according to the results by Gardner et al. (1986). Also the distance between screens can vary, but the exact influence on the wave transmission and reflection is unclear. Therefore, the breakwater is modelled as a continues screen. Subsequently, the typical wave transmission coefficient are determined in Table I.83.

Table I.83: Classified Porosity and Reflection Coefficient (K_r) and Transmission Coefficient (K_t) of Screen Breakwater

Porosity (%)	*T (s)*	K_t *(-)*	K_r *(-)*
5	4	0.15	0.75
	6	0.40	0.70
7.5	4	0.30	0.60
	6	0.50	0.50
10	4	0.40	0.50
	6	0.60	0.40

In Table I.84 the wave transmission for the classified wave heights is shown. It should be mentioned that the wave transmission coefficient for a given porosity strongly depends on the wave period.

Table I.84: Classified Transmission and Wave Heights of Screen Breakwater

H_s (m)	L_0 (m)	T (s)	Porosity (%)	K_t (-)	H_t (m)
0.1	2.5	1.3	5	0.15	0.02
			7.5	0.30	0.03
			10	0.40	0.04
0.2	5	1.8	5	0.15	0.03
			7.5	0.30	0.06
			10	0.40	0.08
0.25	6.25	2.0	5	0.15	0.04
			7.5	0.30	0.08
			10	0.40	0.10
0.3	7.5	2.2	5	0.15	0.05
			7.5	0.30	0.09
			10	0.40	0.12
0.5	12.50	2.8	5	0.15	0.08
			7.5	0.30	0.15
			10	0.40	0.20
0.75	18.75	3.5	5	0.15	0.11
			7.5	0.30	0.23
			10	0.40	0.30
1	25.00	4.0	5	0.15	0.15
			7.5	0.30	0.30
			10	0.40	0.40
1.5	37.50	4.9	5	0.15	0.23
			7.5	0.30	0.45
			10	0.40	0.60
1.75	43.75	5.3	5	0.4	0.70
			7.5	0.50	0.88
			10	0.60	1.05
2	50	5.7	5	0.4	0.80
			7.5	0.50	1.00
			10	0.60	1.20

I.11.8. STRUCTURAL APPROACH

The structural scheme considered is shown in the Figure I.32.

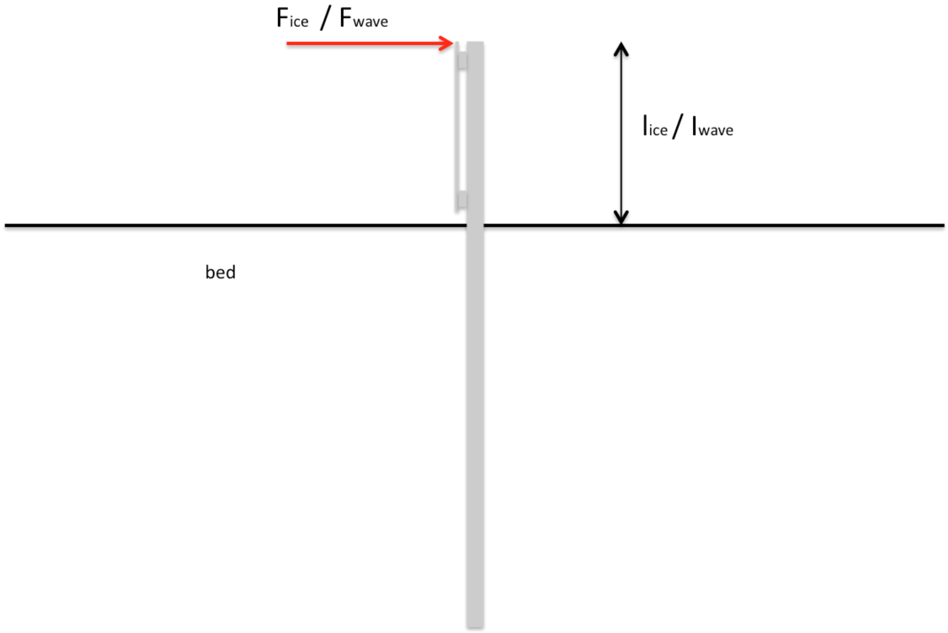


Figure I.32: Structural Model Screen Breakwater

For the stability of the screen breakwater, the bending moments are investigated. It is assumed that the bearing capacity, shear force resistance, normal force equilibrium and sliding plane are within safety limits.

$$M_{Ed} < M_{Rd} \tag{I.35}$$

In which:

M_{Ed} Internal bending moment (kNm)
M_{Rd} External bending moment (kNm)

The plates, tubular piles, support beams can be studied. The steel plates are assumed to have a width of 0.2 m and a fluctuating thickness of 10 to 25 mm. For several crest height, the thickness of the plates and porosity is provided in Table I.85.

The tubular piles are chosen to be at a distance of 5 m, center to center, in thickness 10 mm and tensile strength of 355 N/mm^2. The properties of the piles originate from Figure K.3. For the design bending moment in the tubular piles at the level of the bed the following formula is used:

$$M_{Ed} = F_H l_H \tag{I.36}$$

The bending moment resistance of multiple piles are shown in Table I.86.

Table I.85: Plate Properties of Screen Breakwater

h_c (m)	t_{plate} (mm)	Porosity (%)
2	10	5
		7.5
		10
4	15	5
		7.5
		10
6	20	5
		7.5
		10
8	25	5
		7.5
		10
10	30	5
		7.5
		10

Table I.86: Tubular Pile Properties of Screen Breakwater

Tubular Pile Type	D_{tp} (mm)	A_s (m^2)	W_el (cm^3/m)	M_{Rd} (kNm/m)
TP219	219.1	0.007	328	141
TP245	244.5	0.007	415	178
TP273	273.0	0.008	524	225
TP324	323.9	0.010	751	323
TP356	355.6	0.011	912	392
TP406	406.4	0.013	1205	518
TP457	457.0	0.014	1536	660
TP508	508.0	0.016	1910	821

The design bending moments due to wave and ice loads are enclosed in Table I.87.

To the tubular piles, the screens are connected, which are consisting of IPE beams and plates. IPE profiles are shown Figure K.4. The formula to compute the bending moment in the two I-beams is as follows:

$$M_{Ed} = \frac{1}{8} q l^2 \tag{I.37}$$

The considered properties and design bending moments are shown in respectively Table I.88 and I.89. The bending moment resistance is determined by an allowable steel stress of 355 N/mm^2.

Table I.87: Tubular Pile Performance of Screen Breakwater

F_H (kN/m)	F_H $(kN/4m)$	h_c (m)	M_{Ed} (kNm/m)	*Tubular Pile Type*
3.4	8	2	17	TP219
		4	34	TP219
		6	51	TP219
		8	68	TP273
		10	84	TP324
10.1	25	2	51	TP245
		4	101	TP324
		6	152	TP406
		8	203	TP457
		10	253	TP508
20.3	51	2	101	TP324
		4	203	TP457
		6	304	TP356
		8	405	TP406
		10	506	TP457
33.8	84	2	169	TP273
		4	338	TP406
		6	506	TP457
		8	675	TP508
		10	844	NO TUBULAR PILE SUFFICES
83.0	208	2	415	TP406
		4	830	NO TUBULAR PILE SUFFICES
		6	1245	NO TUBULAR PILE SUFFICES
		8	1660	NO TUBULAR PILE SUFFICES
		10	2075	NO TUBULAR PILE SUFFICES
165.0	413	2	825	NO TUBULAR PILE SUFFICES
		4	1650	NO TUBULAR PILE SUFFICES
		6	2475	NO TUBULAR PILE SUFFICES
		8	3300	NO TUBULAR PILE SUFFICES
		10	4125	NO TUBULAR PILE SUFFICES
248.0	620	2	1240	NO TUBULAR PILE SUFFICES
		4	2480	NO TUBULAR PILE SUFFICES
		6	3720	NO TUBULAR PILE SUFFICES
		8	4960	NO TUBULAR PILE SUFFICES
		10	6200	NO TUBULAR PILE SUFFICES
330.0	825	2	1650	NO TUBULAR PILE SUFFICES
		4	3300	NO TUBULAR PILE SUFFICES
		6	4950	NO TUBULAR PILE SUFFICES
		8	6600	NO TUBULAR PILE SUFFICES
		10	8250	NO TUBULAR PILE SUFFICES

Table I.88: Support Beam Properties of Screen Breakwater

Support Beam Type	W_{el} (cm^3/m)	M_{Rd} (kNm)
IPE120	53	19
IPE180	146	52
IPE220	252	89
IPE270	429	152
IPE300	658	234

Table I.89: Support Beam Performance of Screen Breakwater

F_H (kN/m)	M_{Ed} (kNm/m)	*Support Beam Type*
3.4	11	IPE120
10.1	32	IPE180
20.3	63	IPE220
33.8	105	IPE270
83.0	259	NO BEAM SUFFICES
165.0	516	NO BEAM SUFFICES
248.0	775	NO BEAM SUFFICES
330.0	1031	NO BEAM SUFFICES

I.11.9. MATERIAL COSTS

For the steel components, the indicative cost of 7020 $€/m^3$ of steel are assumed.

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL ($€/m^3$ or $€/m^2$)

Accordingly, the classified breakwater dimensions result in the material costs as shown in Table I.90, I.91 and I.92.

Table I.90: Plate cost of Screen Breakwater

h_c (m)	V_s (m^3/m)	*Cost of Plates (€/m)*
2	0.10	667
	0.09	649
	0.09	632
4	0.29	2001
	0.28	1948
	0.27	1895
6	0.57	4001
	0.56	3896
	0.54	3791
8	0.95	6669
	0.93	6494
	0.90	6318
10	1.43	10004
	1.39	9740
	1.35	9477

Table I.91: Tubular Pile cost of Screen Breakwater

h_c (m)	L_{pile} (m)	Tubular Pile Type	A_s (m^2)	Cost of Tubular Piles (€/m)
2	6	TP219	0.0066	277
4	12	TP219	0.0066	553
6	18	TP219	0.0066	830
8	24	TP273	0.0083	1397
10	30	TP324	0.0099	2077
2	6	TP245	0.0074	310
4	12	TP324	0.0099	831
6	18	TP406	0.0125	1580
8	24	TP457	0.0140	2359
10	30	TP508	0.0156	3285
2	6	TP324	0.0099	415
4	12	TP457	0.0140	1179
6	18	TP356	0.0109	1377
8	24	TP406	0.0125	2106
10	30	TP457	0.0140	2948
2	6	TP273	0.0083	349
4	12	TP406	0.0125	1053
6	18	TP457	0.0140	1769
8	24	TP508	0.0156	2628
2	6	TP406	0.0125	527

Mind that there are two beams per running meter for the IPE support beams.

Table I.92: Support Beam cost of Screen Breakwater

Support Beam Type	$A_s(m^2)$	Cost of Support Beam (€/m)
IPE120	0.0008	11
IPE180	0.0024	34
IPE220	0.0033	47
IPE270	0.0046	64
IPE300	0.0054	76

I.11.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Transport vessel with hydraulic	250	€/$hour$
Pontoon to carry structure elements	25	€/$hour$
Vibration block	100	€/$hour$
Placement of piles	35	$m^2/hour$
Placement of piles	60	€/m
Placement of screen	2	$units/hour$
Placement of screen	100	€/$hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.93 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.93: Labour and Equipment Costs of Screen Breakwater

Class	L_{embed} (m)	*Placement of piles (hours/m)*	*Placement of piles (€/m)*	*Placement of screen (hours/m)*	*Placement of screen (€/m)*	*Transport vessel with hydraulic (€/m)*	*Pontoon to carry structure elements (€/m)*	*Vibration block (€/m)*	*Total Cost (€/m)*
Screen Breakwater 1	4	0.8	48	0.1	10	225	22.5	90	396
Screen Breakwater 2	8	1.6	96	0.1	10	425	42.5	170	744
Screen Breakwater 3	12	2.4	144	0.1	10	625	62.5	250	1092
Screen Breakwater 4	16	3.2	192	0.1	10	825	82.5	330	1440
Screen Breakwater 5	20	4.0	240	0.1	10	1025	102.5	410	1788

I.11.11. CODES AND GUIDELINES

No codes and guidelines are available to the timber pile breakwater. Instead experience from practice is used.

I.12. GEOTUBE BREAKWATER

I.12.1. GENERAL

Geotextile tubes filled with sand are seen as retaining structure between land and water (Figure I.33). The geotextile consists of high-strength synthetic fabrics. The structure obtains its stability by the weight of the sand and the created friction between the tube and bed. For breakwater and groynes, Longard Tubes are developed.

Figure I.33: Geotube Breakwater
(Pt. Pandu Equator Prima. (2013). *Products Catalog.* Retrieved from http://ptpanduequatorprima.indonetwork.net/. Accessed on November 12, 2014.)

I.12.2. SCHEME

A representation of multiple geotubes as a breakwater is visualized in Figure I.22. A triangular shape in the vertical is used to obtain a stable structure.

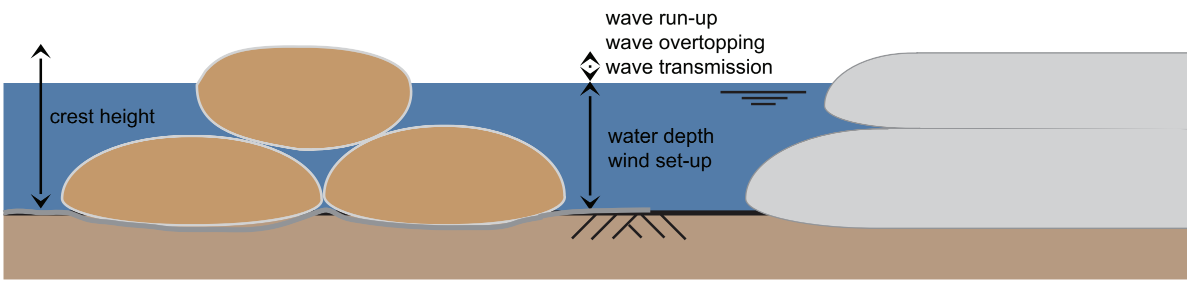


Figure I.34: Cross-sectional View (left) and Side View (right) of Geotube Breakwater

I.12.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.12.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

There are no boundary conditions, which could instantaneously repel this breakwater alternative. What is more, the structure is assumed to be able to cope with the settlements due to weak subsoil or earthquakes. It is expected that ice will not influence the overall stability, but it could damage the geotextile severely.

I.12.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.94: EMVI Criteria Score of Geotubes Breakwater

Criterion	*Score*
System quality	2
Innovation	4
Ecological impact	4
CO2 ambition level	2
Durability	3
Hindrance	3
Noise	3
Risks	2

I.12.6. CLASSIFICATION

The geotube breakwater classes can be found in Table I.96. The configuration shows the number of tubes at the lowest layer to the highest. For example, 2-1 implies that two tubes are at the bed and one is on top.

Table I.95: Geotube Properties

Class	D (m)	a (m)	b (m)	A_0 (m^2)
Geotube A	1.8	0.9	1.4	2.5
Geotube B	2.5	1.25	1.9	4.9

The circular shape of the geotubes deforms when it reaches the bed (Figure I.35).

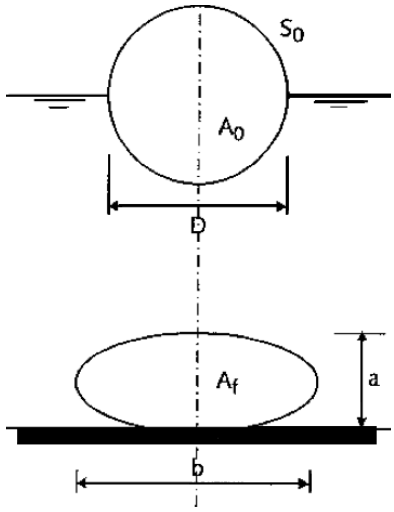


Figure I.35: Relation Surface Area and Fill-ratio

An approximation of the width, b (m) and effective height, a (m), is given by the following relation:

$$b = 1.5a \tag{I.38}$$

The assumed classifiation of the geotubes is enclosed in Table I.96. The capacity of a single geotube varies from 2 to 5 m^3/m. Furthermore, the international standard diameters are 100 cm and 180 cm. The most common sizes are provided by Ten Cate, which is a company specialized in geosynthetics.

Table I.96: Geotube Breakwater Classes

Class	*Total a (m)*	*Diameter (m)*	*Number of Tubes*	*Geotubes Configuration*
Geotube 1	0.9	1.8	1	1A
Geotube 2	1.8	1.8	3	2A-1A
Geotube 3	2.5	2.5	3	2B-1B
Geotube 4	3.4	2.5	5	3B-2B
		1.8	1	1A
Geotube 5	4.3	2.5	7	4B-3B
		1.8	3	2A-1A

The effective height (a) is a function of the fill-ratio (ϕ) and the diameter of the tube (D). The fill-ratio is defined as the ratio between the surface area in a partially filled (A_f) and fully filled (A_0) state:

$$\phi = A_f / A_0 \tag{I.39}$$

The following empirical formula can be applied for the geotube deformation (Pilarczyk, 2000). A conservative assumption for the fill-ratio is 0.75, which is used in the table above.

$$a = D(1 - \sqrt{1-\phi}) \tag{I.40}$$

For detailed computations of the exact shape of the geotube, the geotextile properties, the material and the preferred height should be optimized.

I.12.7. DIMENSIONS

To determine the overall dimensions, the crest height is most important. This can be computed according to the following formula:

Crest height = water depth + wind set-up + run-up

The crest height should be verified with:

Wave transmission and wave overtopping

I.12.8. STRUCTURAL APPROACH

The structural scheme considered is shown in Figure I.36.

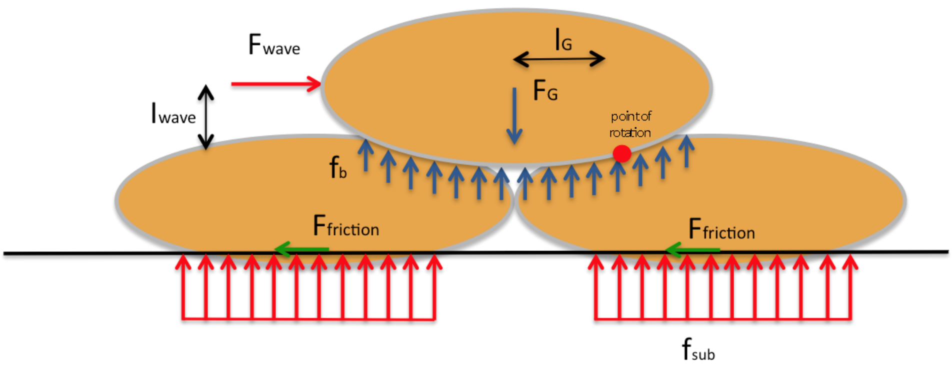


Figure I.36: Structural Model Geotube Breakwater

The requirement for the equilibrium of the horizontal forces, which is preventing sliding of the structure, is:

$$F_{fric} > F_H \tag{I.41}$$

$$F_{fric} = F_G * \delta \tag{I.42}$$

The friction factor between geotextile and sand is assumed to be 0.75.

The requirement for the equilibrium of the rotations is:

$$M_E < M_R \tag{I.43}$$

This can be written as:

$$F_H l_H < F_G l_G \tag{I.44}$$

Where:

F_H = F_{ice} or F_{wave}
l_H = l_{ice} or l_{wave}

In which:

F_H Horizontal load (kN)
l_H Arm of loading (m)
M_R Rotational resistance (kNm)
M_E Design rotation (kNm)
F_{fric} Friction force (kN)

Point of rotation is assumed to be at 1/4 of the width.

Table I.97: Rotational Performance of Geotube Breakwater

Class	F_H (kN)	l_h (m)	M_E (kNm/m)	F_G (kN)	l_G (m)	F_b (kN)	$F_{G,res}$ (kN)	M_R (kNm/m)	*Applicable (rotation)*
A	3.4	0.5	1.5	52	0.3	12	41	13.7	YES
	10.1	0.5	4.6	52	0.3	12	41	13.7	YES
	20.3	0.5	9.1	52	0.3	12	41	13.7	YES
	33.8	0.5	15.2	52	0.3	12	41	13.7	NO
B	3.4	0.6	2.1	101	0.5	23	78	36.6	YES
	10.1	0.6	6.3	101	0.5	23	78	36.6	YES
	20.3	0.6	12.7	101	0.5	23	78	36.6	YES
	33.8	0.6	21.1	101	0.5	23	78	36.6	YES

In which:

A_G Surface area of sand
F_G Self-weight of geotube and sand
F_b Buoyancy force
$F_{G,res}$ Resulting force

Table I.98: Sliding Performance of Geotube Breakwater

Class	A_{sand} (m^2/m)	F_{sand} (kN/m)	F_{fric}(kN/m)	*Applicable (friction)*
Geotube 1	2.5	52.4	39	YES
Geotube 2	7.6	157.3	118	YES
Geotube 3	14.7	303.4	228	YES
Geotube 4	27.1	558.0	419	YES
Geotube 5	42.0	865.1	649	YES

To determine the subgrade reaction, the mass of the core of sand is taken into account. The computed self-weight shall be divided by the surface area of the foundation (A_f) to obtain the pressure applied to the subsoil (Table I.99). One should draw conclusions by comparing the computed and allowed pressure.

$$f_{sub} = (F_G/A_f) - f_b \tag{I.45}$$

In which:

f_{sub} Allowable pressure on subsoil (kN/m^2)
F_G Force from selfweight (kN)
f_b Buoyancy pressure (kN/m^2)
A_f Surface area of structure on foundation (m^2)

Table I.99: Foundation Pressure of Geotube Breakwater

Class	A_f (m^2/m)	q_G (kN/m^2)
Geotube 1	1.4	39
Geotube 2	2.7	58
Geotube 3	3.8	81
Geotube 4	5.6	99
Geotube 5	7.5	115

I.12.9. MATERIAL COSTS

To calculate the total cost of the materials, the following unity costs are used:

Core sand	6 (per barge)	$€/m^3$ (used)
	10 (per truck)	$€/m^3$
Geotube	50	$€/m$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL ($€/m^3$ or $€/m^2$)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.100.

Table I.100: Material Costs of Geotube Breakwater

Class	*Number of Geotubes*	A_{sand} (m^2/m)	A_{rubble} (m^2/m)	*Cost of Sand (€/m)*	*Cost of Rubble (€/m)*	*Cost of Geotubes (€/m)*	*Cost of Geotextile (€/m)*	*Total of Costs (€/m)*
Geotube 1	1	2.5	1.5	15	51.0	50	2	116
Geotube 2	3	7.6	4.6	46	152.9	150	4	349
Geotube 3	3	14.7	8.8	88	295.0	150	6	533
Geotube 4	6	27.1	16.3	163	542.7	300	8	1005
Geotube 5	10	32.5	19.5	195	651.9	500	11	1347

Permanently constructed geotubes, which are submerged, have a crest above the still water level. This part of the breakwater is sensitive to ultraviolet radiation. A layer of rubble can protect the structure against this and increase the amount of wave energy dissipation. Furthermore, excluding the rubble layer, the geotubes are applicable as temporary solution.

I.12.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of geotextile tubes with divers	50	$m/hour$
Placement of geotextile tubes with divers	150	$euro/hour$
Pump system (all-in)	100	$€/hour$
Pump system (sand capacity)	250	$m^3/hour$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€/m) = EQUIPMENT (€/$hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€/m) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.101 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.101: Labour and Equipment Costs of Geotube Breakwater

Class	*Placement of geotextile tubes (€/m)*	*Pumping system ($hour/m^3$)*	*Pumping system (€/m)*	*Placement of rock by hydraulic excavator (hour/m)*	*Transport vessel with hydraulic crane (€/m)*	*Pontoon to carry structure elements (€/m)*	*Total Cost (€/m)*
Geotube 1	3	0.01	1.0	0.01	3.1	0.3	7
Geotube 2	3	0.03	3.1	0.04	9.2	0.9	16
Geotube 3	3	0.06	5.9	0.07	17.7	1.8	28
Geotube 4	3	0.11	10.8	0.13	32.5	3.3	50
Geotube 5	3	0.13	13.0	0.16	39.0	3.9	59

I.12.11. CODES AND GUIDELINES

Design rules of floating breakwaters are to be found in Bezuijen and Vastenburg (2013).

I.13. Synthetic Breakwater

I.13.1. General

The synthetic breakwater is a floating structure. Multiple shapes are found on the market (Figure I.38, I.39 and I.37). The material is designed for all weather conditions.

Figure I.37: WhisprWave Breakwater
(MarineBuzz.com. (2009). *Floating Security Barriers to Protect Coastal Assets.* Retrieved from http://www.marinebuzz.com/. Accessed on November 27, 2014.)

Figure I.38: WaveEater Breakwater
(WaveEater. (2013). *About WaveEater Atteniation Systems.* Retrieved from http://www.waveeater.com/. Accessed on November 10, 2014.)

Figure I.39: WaveBrake Breakwater
(WaveBrake. (2015). *What is WaveBrake.* Retrieved from http://www.wavebrake.com. Accessed on November 27, 2014.)

I.13.2. Scheme

Figure I.40 shows a cross-sectional view of three synthetic breakwaters. One can observe a floating part and a concrete anchor block.

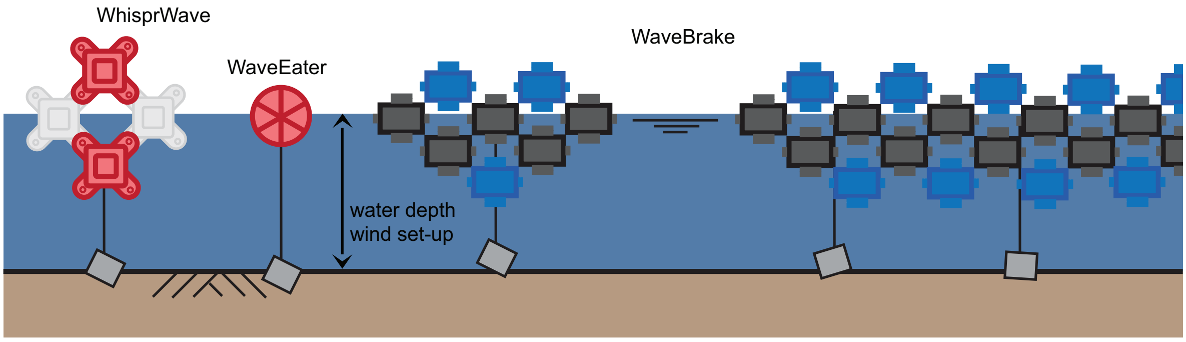


Figure I.40: Cross-sectional View (left) and Side View (right) of Synthetic Breakwaters

I.13.3. CHARACTERISTICS

The breakwater characteristics consisting of the laws, permits, regulations, boundary conditions and design considerations can be elaborated similar to Section I.1 and Section I.2.

I.13.4. BOUNDARY CONDITIONS

The general boundary conditions to be considered are enlisted below.

1. Water depth;
2. Waves;
3. Flow velocities;
4. Ice;
5. Subsoil;
6. Earthquakes.

In case of ice and flow velocities, the synthetic breakwater is assumed to be unable to cope with the load.

I.13.5. EMVI

The following score from 1 to 5 of the EMVI criteria is assumed. 1 and 5 are respectively the lowest and highest score.

Table I.102: EMVI Criteria Score of Timber Pile Breakwater

Criterion	*Score*
System quality	5
Innovation	5
Ecological impact	5
CO2 ambition level	4
Durability	4
Hindrance	3
Noise	4
Risks	5

I.13.6. CLASSIFICATION

The synthetic breakwater class considered are in Table I.103. These are derived from the configurations in Figure I.41.

Table I.103: Classes of Synthetic Breakwater

Style	*Number of units*	*Freeboard (m)*	*Draught (m)*	*Width (m)*
WaveBrake 1	8	0.6	0.9	2.4
WaveBrake 2	11	0.6	0.9	3.3
WaveBrake 3	13	0.6	0.9	4.3
WaveBrake 4	13	0.6	1.4	3.3
WaveBrake 5	17	0.6	1.4	4.3

Information about the WaveBrake breakwater types is available. Therefore, this structure is used for computations about its performance.

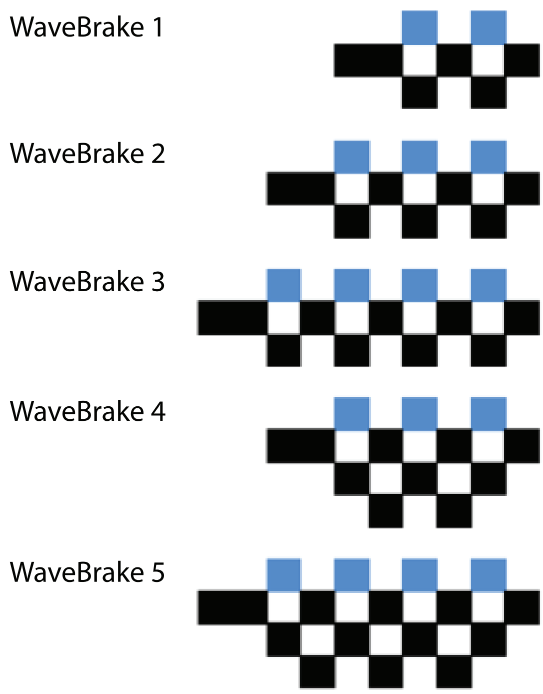


Figure I.41: Configuration of the Synthetic Breakwater Classes
(WaveBrake. (2015). *What is WaveBrake.* Retrieved from http://www.wavebrake.com. Accessed on November 27, 2014.)

I.13.7. DIMENSIONS

The only dimension considered is the width of the tire breakwater. The design aspects wind set-up, wave run-up and wave overtopping are not considered.

Assuming a required clearance of 0.5 m, the minimum water depth is written as:

$$d > D + h_{net} \tag{I.46}$$

In which:

d	Water depth
D	Draught
h_{net}	Clearance

The draught of the synthetic breakwater with respectively 2 and 3 layers of black boxes is shown in Table I.104.

Table I.104: Draught of Synthetic Breakwater

Box Level below Water Surface	*Draught (m)*	*Draught + Clearance (m)*
2	0.9	1.3
3	1.4	1.8

The dimensions depend on the incoming waves. A maximum wave height is provided for typical configuration with a maximum wave dissipation of 85%. The results are enclosed in the Table I.105.

Table I.105: Transmitted Wave Height of Synthetic Breakwater

Class	$H_{s,max}$ *(m)*	K_t $(-)$	$H_{t,max}$ (m)
WaveBrake 1	0.9	0.15	0.1
WaveBrake 2	1.5	0.15	0.2
WaveBrake 3	1.8	0.15	0.3
WaveBrake 4	1.8	0.15	0.3
WaveBrake 5	2.4	0.15	0.4

I.13.8. STRUCTURAL APPROACH

No structural assessment is required. The performance is only expressed in the wave height and wave transmission.

I.13.9. MATERIAL COSTS

The following construction items are found:

Synthetic units	50	€/$unit$
Anchor	100	€/$unit$

Including the unity costs of the breakwater materials, the total cost can be computed by the amounts the materials required. These cost exclusively refer to the price of the material and transport to location. The general formula to be applied, is written as:

COST $(hour/m)$ = QUANTITY $(m^3/m$ or $m/m)$ / PRICE OF MATERIAL (€/m^3 or €/m^2)

Accordingly, the classified breakwater dimensions results in the material costs as shown in Table I.106. It is assumed that a single unit is 0.67 m in width, length and height. What is more, it is required to position three tethers over 9 m.

Table I.106: Material Costs of Synthetic Breakwater

Class	*Cost number of boxes (€/units)*	*Cost of Boxes (€/m)*	*Cost of Anchors (€/m)*	*Total Cost (€/m)*
WaveBrake 1	400	596	25	621
WaveBrake 2	550	820	25	845
WaveBrake 3	650	969	25	994
WaveBrake 4	650	969	25	994
WaveBrake 5	850	1267	25	1292

I.13.10. LABOUR AND EQUIPMENT COSTS

The labour and equipment costs consist of the placement of the elements of the structures, which requires equipment and operators. Because land-based equipment is unable to be on top of small scale breakwaters, the labour and equipment costs will be established with water-borne equipment. Therefore, the following equipment and activities are considered:

Placement of structure	3	$units/hour$
Transport vessel with hydraulic excavator	250	€$/hour$
Pontoon to carry structure elements	25	€$/hour$
Placement of anchors	600	€$/unit$
Connecting structure segments	5	€$/segment$

The unity costs are indicative and approximations. These can differ from company, time and place. The equipment cost consist of machines including personal. What is more, the mobilisation of the equipment and transport of the materials are not incorporated. Therefore, the cost will not result in a relative difference in the total labour and equipment costs.

The labour and equipment costs are a function of the amount of materials to be processed and the cost per unit of time for the equipment. Owing to the fact that the length of a breakwater is not considered, the cost are provided per running meter (COST1). The time-independent costs are divided by the repetition distance of the length of the breakwater (COST2). The total labour and equipment costs consist of the two cost drivers. The general equations are:

- HOURS OF WORK ($hour/m$) = QUANTITY (m^3/m or m/m) / SPEED OF WORK ($m^3/hour$)
- COST1 (€$/m$) = EQUIPMENT (€$/hour$) · HOURS OF WORKS($hour/m$)
- COST2 (€$/m$) = WORKS (€/unit) · UNIT / DISTANCE CTC (m)
- TOTAL COST = Σ COST1 + Σ COST2

Paying strict attention to the units, Table I.107 can be constructed. In the last column, the total labour and equipment costs are found.

Table I.107: Labour and Equipment Costs of Synthetic Breakwater

Class	*Placement of structure (hours/m)*	Transport vessel with hydraulic excavator (€/m)	Pontoon to carry structure elements (€/m)	*Connecting structure segments (€/m)*	*Placement of anchors (€/m)*	*Total Cost (€/m)*
WaveBrake 1	0.03	8.3	0.8	75	200	84
WaveBrake 2	0.03	8.3	0.8	75	200	84
WaveBrake 3	0.03	8.3	0.8	75	200	84
WaveBrake 4	0.03	8.3	0.8	75	200	84
WaveBrake 5	0.03	8.3	0.8	75	200	84

I.13.11. CODES AND GUIDELINES

No codes and guidelines are available to the synthetic breakwater. Instead experience from practice is used.

J

INFEASIBLE BREAKWATER ALTERNATIVES

This appendix consists of the following sections which provide a description of breakwaters to substantiate the reason to neglect these from the method.

J.1. CONCRETE BLOCKS BREAKWATER

J.1.1. GENERAL

A designer can choose to use artificial rocks (e.g. concrete blocks) as a replacement for the rubble mound armour layer. These unreinforced concrete blocks are available in many shapes and formations, examples of which are shown in Figures J.1, J.2, J.3 and J.4.

Figure J.1: Cube Breakwater
(Beeldbank RWS. (2011). *Scheveningen havenhoofd havendam met hoge Golven woeste zee Storm ID313358.* Retrieved from https://beeldbank.rws.nl/. Accessed on November 6, 2014.)

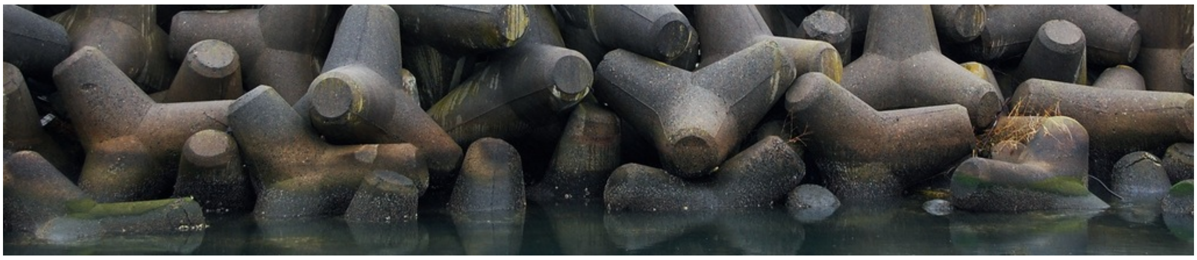


Figure J.2: Tetrapod Breakwater
(Free Association Design. (2010). *Tetrapods Entropy and Excess.* Retrieved from http://freeassociationdesign.wordpress.com/. Accessed on November 6, 2014.)

Figure J.3: Xbloc Breakwater
(Delta Marine Consultants. (2011). *Caladh Mor, Ireland.* Retrieved from http://www.xbloc.com/. Accessed on November 6, 2014.)

Figure J.4: Dolos Breakwater
(Alcyone. (2011). *Did you know?: The Dolos.* Retrieved from http://www.alcyone.co.za/. Accessed on November 6, 2014.)

A summation of the most familiar blocks is: Gasho Block, n-shaped Block, Toskane, Tribar, Tetrapod, Xbloc, Core-Loc, Accropod, Dolos, Crablock, n-shaped Block, Tribar, Cube Antifer Cube, Crablock, Cubipods and Bipod (Schiereck et al., 2012).

Outdated blocks applied to a lesser extend are: Akmon, Gasho Block and Kolos. Also on the market, but less known, are the Modified Cube, Stabit, Cob, Seabee, Shed, Haro, Hollow Cube, Diahitis, Samoa Block, Grobbelar, Hexaleg A-jack with rocks are to be found (Bakker et al., 2003).

Cubes and Antifer Cubes are frequently applied in practice for the reason that these are always applicable. These blocks belong to the category massive units, which are constructed as a multi-layer armour unit. While a double layer provides stability of the blocks by the self-weight and interlocking, the single layer blocks obtain their stability by the self-weight and friction on the under layer. In contrast, XBlocs, Accropods II are to be applied to cases of heavy wave loading, where the interlocking of a single layer is important. (PIANC/Marcom 36, 2005)

J.1.2. EXPULSION

When natural stones of 10 tons or higher are required, these can be replaced by the above-mentioned concrete blocks. Since concrete is more expensive than rock, the concrete elements are only effective and profitable in coastal areas with large wave heights, where rubble mound does not suffice. Due to the special moulds, the intensiveness of the labour and the handling during construction, these blocks are also time consuming. This makes them ineffective to use in inland waterways with limited wave loads. In addition, in section I.1 the maximum stone weight of approximately 4.7 tons is determined. This is less than half the weight of the requirement, which refers to concrete blocks. Therefore, concrete blocks will not be part of the method.

The concrete block breakwater is not considered.

J.2. CONCRETE L-WALL BREAKWATER

J.2.1. GENERAL

When due to limited space and water depth a mound breakwater is not possible, an L-wall breakwater is a favourable option (Takahashi, 2002). The wall consists of prefab panels of reinforced concrete.

Figure J.5: Concrete L-wall Breakwater
(Archi Expo. (2014). *Prefab retaining wall / reinforced concrete / prefabricated-L*. Retrieved from http://www.archiexpo.com/. Accessed on November 8, 2014.)

J.2.2. SCHEMATIZATION

A typical cross-section of the L-wall is shown in Figure J.6. The vertical part counteracts waves and currents. Furthermore, the horizontal part provides the structure vertical, horizontal and overturning stability. An important requirement is that the subsoil provides a solid foundation. Therefore, a rubble foundation is included. To obtain a more stable structure, loose rock can be dumped on top of the foot to increase its self-weight.

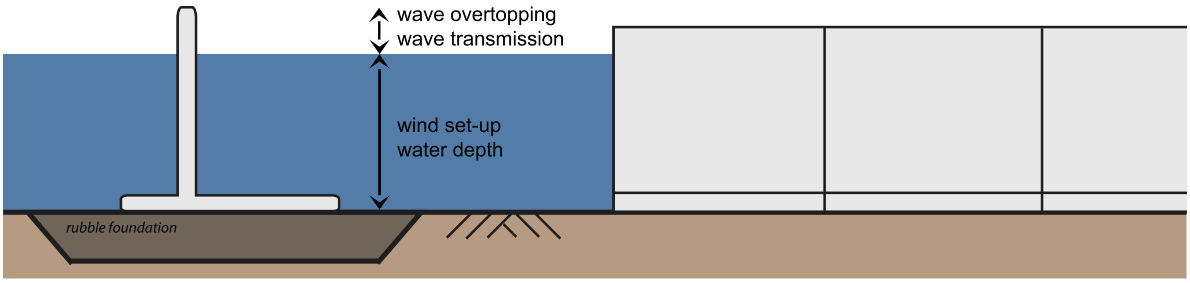


Figure J.6: Cross-sectional View (left) and Side View (right) of L-wall Breakwater

J.2.3. BOUNDARY CONDITIONS

This type of breakwater is restricted by three boundary conditions, namely: water depth, subsoil and earthquakes. For the reason that the L-shaped elements have a maximum length of 3 m, for water depths larger than this the breakwater is not applicable. Moreover, it would allow significant wave transmission, which is not preferred.

Table J.1: Classification of Design Water Depth for L-wall

Property	*Water Depth (m)*	*Applicability (-)*
Lowest	1	Yes
Mean Low	1.5	Yes
Low	2	Yes
Mean	2.5	Yes
High	3	No
Mean High	3.5	No
Highest	4	No

The subsoil should provide a sufficient base for an appropriate foundation. Therefore, for weaker subsoils this breakwater alternative is not applicable.

Table J.2: Classification of Design Soil for L-wall

Name	*Deformation*
Loam	High
Light Clay	High
Light Sand	High
Organic Sand	High
Peat	High
Clean Sand	Low
Heavy Clay	Low
Heavy Sand	Low

Similar to the placed block breakwater, deformation of the structure is expected due to earthquakes. For the reason that maintenance of L-walls is rather difficult of L-walls, regions with stronger seismic activity are not preferred.

Table J.3: Classification of Design Earthquake for L-wall (CIRIA, CUR, CETMEF, 2007)

Zone	*Horizontal Acceleration (cm/s^2)*	*Risk of Collapse*
A	*10*	Low
B	*22*	Low
C	*50*	High
D	*100*	High

J.2.4. CLASSIFICATION

The L-wall classess are presented in Table J.4.

J.2.5. STRUCTURAL APPROACH

The L-shaped structure is subjected to multiple forces which work on both the stability and the strength. For reasons of simplicity, it is assumed that the bending moment resistance and the shear strength suffice at the section where the vertical and horizontal plates are connected. On the contrary, the rotational and horizontal stability are a point of attention (Figure J.7).

Table J.4: Classification of L-Wall Types

(O'Reilly Oakstown. (2014). *L-walls*. Retrieved from http://www.oakstownseptictanks.com/. Accessed on May 10, 2015.)

Type	*Height (m)*	*Width (m)*	*Weight (kg/m)*	*Costs (€/m)*
L-wall 1	1.5	1.7	1.25 ton	500
L-wall 2	2.0	1.7	1.50 ton	600
L-wall 3	2.5	1.7	1.75 ton	700
L-wall 4	3.0	1.7	2.00 ton	800

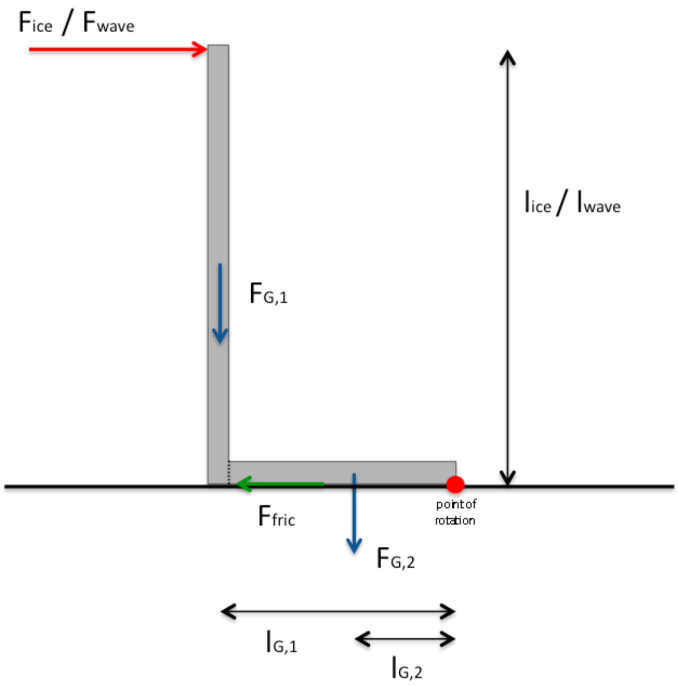


Figure J.7: Model of Forces of L-wall Breakwater

Rotational stability is verified by the design rotation moment, M_E (kNm), as a result of the wave, ice and vessel collision forces and the magnitude of the rotation resistance, M_R (kNm). The general formula sounds:

$$M_{Ed} << M_{Rd} \tag{J.1}$$

Which can be written as:

$$F_H l_H << F_{G,1} l_{G,1} + F_{G,2} l_{G,2} \tag{J.2}$$

The external force, F_H (kN), represents the ice force, F_{ice} (kN) or the wave force, F_{wave} (kN). The arms, l_{ice} (m) and l_{wave} (m), is the distance between the point of impact and rotation (Figure J.7).

Moreover, the horizontal stability should be secured by the friction force which originates from the rubble-concrete interaction. The friction factor is assumed to be 0.5. This is checked by verifying the balance between the external force and the friction force. The following equation holds:

$$F_H << F_{frict} \tag{J.3}$$

$$F_{fric} = Gf \tag{J.4}$$

$$f = tan(\delta) = tan(2/3\phi) \tag{J.5}$$

The variables are:

G mass of the structure (kg)
f friction coefficient ($-$)
δ friction angle subsoil-structure (deg)
ϕ angle of international friction (deg)

J.2.6. DIMENSIONS

The required crest height of the L-wall can be computed as follows:

crest height = water depth + wind set-up + wave set-up + wave amplitude

Subsequently, the rotational and horizontal stability can be checked. Therefore, the friction force and bending moment resistance for element A through D are determined in Table J.6. One can observe a relatively low self weight.

Table J.5: Properties of L-wall Breakwater

Class	*G1 (kg)*	F_{G1} *(kN)*	l_{G1} *(m)*	*G2 (kg)*	F_{G2} *(kN)*	l_{G2} *(m)*
L-wall 1	586	6	1.7	664	7	0.85
L-wall 2	811	8	1.7	689	7	0.85
L-wall 3	1042	10	1.7	708	7	0.85
L-wall 4	1277	13	1.7	723	7	0.85

Table J.6: Shear and Bending Moment Resistance of L-wall Breakwater

Class	F_{fric} *(kN/m)*	M_R *(kNm)*
L-wall 1	6	15
L-wall 2	7	19
L-wall 3	9	23
L-wall 4	10	27

The impacts of wave heights and ice are determined in the Table J.7 and J.8. Almost all results are larger than the resistance.

Table J.7: Wave Forcing on L-wall Breakwater

Class	*d (m)*	H_s *(m)*	F_{wave} *(kN/m)*	*Applicable*	$M_{E,wave}$ *(kNm)*	*Applicable*
L-wall 1	1.00	0.50	3.4	YES	3	YES
L-wall 2	1.00	0.50	10.1	NO	10	YES
L-wall 3	1.50	1.00	20.3	NO	30	NO
L-wall 4	2.00	1.00	33.8	NO	68	NO

Table J.8: Ice Forcing on L-wall Breakwater

Class	*d (m)*	t_{ice} *(m)*	F_{ice} *(kN/m)*	*Applicable*	$M_{E,ice}$ *(kNm)*	*Applicable*
L-wall 1	1.00	0.10	83.0	NO	83	NO
L-wall 2	1.50	0.20	165.0	NO	248	NO
L-wall 3	2.00	0.30	248.0	NO	496	NO
L-wall 4	2.50	0.40	330.0	NO	825	NO

J.2.7. EXPULSION

According to the analysis in the previous section, the L-wall breakwater is not a feasible option, because it fails the stability check. What is more, uneven settlement, which are likely to occur, can easily lead to collapse of the breakwater. In conclusion, the structure is:

- water depth limited;
- subsoil dependent;
- earthquake sensitive;
- collapsing due to wave force;
- collapsing due to ice force.

Forces of waves and ice can be resisted when the thickness of the elements is increased and when the L-wall is combined with rubble on the bottom slab. The advantages are an increased self-weight and an increased friction component. On the other hand, since it is a combination of concrete and rubble, the structure will be more expensive and complex to construct compared to breakwaters consisting of a single material.

The L-wall breakwater is not considered.

J.3. MASONRY BREAKWATER

J.3.1. GENERAL

The masonry breakwater consists of bricks and mortar. The mortar is required to glue the smaller stones together in order to obtain a monolithic structure (Figure J.8).

Figure J.8: Masonry Breakwater
(Jersey. (2014). *St Catherine's*. Retrieved from http://www.jersey.com/. Accessed on November 10, 2014.)

J.3.2. EXPULSION

Masonry breakwaters are not considered in the design projects nowadays. The construction is not only time consuming, but also information about these structures is in short supply. The reason for this is that masonry structures are relatively weak, where damage occurs very easily. When cracks develop and scraps loosen, it will affect stability and strength of structure significantly. When it weakens from the inside, the required repair works become quite difficult or even impossible.

The masonry breakwater is not considered.

J.4. VERTICALLY COMPOSITE BREAKWATER

J.4.1. GENERAL

The vertically composite breakwater consists of a rubble layer on the seabed with a caisson on top. Hindrance of a waterway by large mound structures in relatively deep water is avoided and mooring opportunities are created.

J.4.2. EXPULSION

When a combination of concrete and loose rock in the vertical is considered, one can conclude that a large caisson would be too expensive or impossible to construct. Also a wide rubble mound breakwater would provide hindrance for vessels and mooring facilities are not possible. In contrast, in inland waterways, a composite breakwater is neither profitable nor required, which is due to the lower wave loads and water depths. In this matter, the rubble mound breakwater has significantly smaller dimensions and the caisson breakwaters will not have non-achievable dimensions. Therefore, in coastal areas these structures could be an advantage.

The vertically composite breakwater is not considered.

J.5. HORIZONTALLY COMPOSITE BREAKWATER

J.5.1. GENERAL

The horizontally composite breakwater consists of the same materials as the vertically composite breakwater. The difference is found in the horizontal rather than the vertical configuration. So, a caisson or block wall is constructed at the sea bed and is extended with a slope of rubble mound at the sea side (Figure J.9).

Figure J.9: Horizontally Composite Breakwater
(The Liverpool Thessaloniki Network, 1996)

J.5.2. EXPULSION

For less deep water than for the vertically composite breakwater, the horizontal composite breakwater could provide a feasible option. For example, the client requests a breakwater with road, off-loading facility and mooring opportunities. A caisson is than a familiar option to the designer. A problem occurs when the sea side of the breakwater is subjected to large waves. These waves would require a high and heavy structure to prevent large amounts of wave overtopping and stability issues. To reduce the wave forcing on the wall, loose rock or concrete blocks can be placed in front of the structure to dissipate wave energy. This scenario mainly exists in coastal zones with large fetch lengths and deep waters. The complexity and costs of this breakwater does not add value for inland waterway conditions. Therefore, it is excluded from the method.

The horizontally composite is not considered.

J.6. PNEUMATIC BREAKWATER

J.6.1. GENERAL

Around 1900 a screen of bubbles (so-called 'air bubble curtain') was conceived as a breakwater alternative (Figure J.10). The vertical motion of the bubbles intersect with the horizontal motion of the waves resulting in breaking and turbulence which dissipates wave energy.

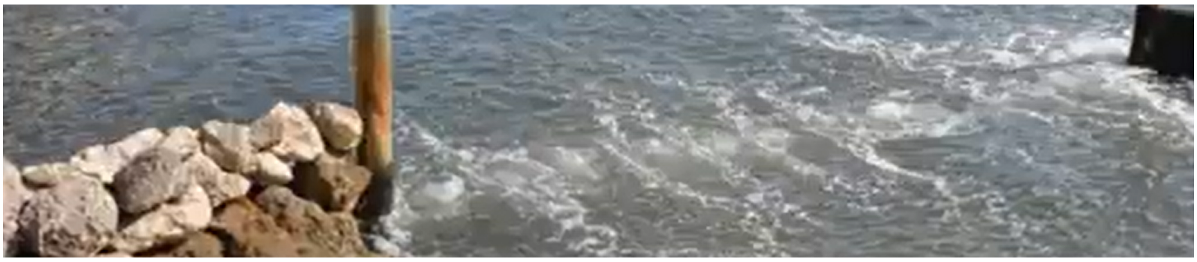


Figure J.10: Pneumatic Breakwater
(Canadian.ca. (2014). *Discover Air Bubble Curtain Applications with Bubble Tubing.* Retrieved from http://canadianpond.ca/. Accessed on November 10, 2014.)

J.6.2. EXPULSION

These breakwaters are not widely used in practice. Also related studies are restricted and practical design guidelines are absent. In addition, a pump station working on electricity is required. This aspect delivers more risks of malfunctioning. Moreover, repair works and maintenance should be performed by a specialised company. Issues in the pipes would with lead to excavation and diving activities. As a result, this breakwater alternative is excluded from the method.

The pneumatic breakwater is not considered.

J.7. PLATE BREAKWATER

J.7.1. GENERAL

The horizontal plate breakwater consists of inclined piles with a horizontal plate on top, where a maximum orbital motion can be found. Generally, these breakwaters are designed to decrease the wave height of significant offshore waves.

J.7.2. EXPULSION

The wave heights in inland waters have a maximum of 2 m, while plate breakwaters are very effective in coastal areas where wave heights of 10 m occur. In case of waves in inland waterways, the transmission of waves will be significant. Also the height of the plate breakwater cannot be adapted to varying water depth. For these reasons, this type of breakwater is excluded from the method.

The plate breakwater is not considered.

K

CONSTRUCTION ELEMENT DATA

K.1. LOOSE ROCK

Table K.1: Rock Sizes of Coarse Grading (CIRIA, CUR, CETMEF, 2007)

NLL/NUL (mm)	D_{50} **(mm)**
45/125	85
45/180	113
63/180	122
90/180	135
90/250	170

Table K.2: Rock Sizes of Light Grading (CIRIA, CUR, CETMEF, 2007)

NLL/NUL (kg)	D_{n50} **(mm)**	M_{50} **(kg)**	M_{em} **(kg)**	M_{50}/M_{em} **(-)**
5-40	199	21	15	1.386
10-60	241	37	28	1.352
40-200	363	127	100	1.570
15-300	376	141	90	1.243
60-300	417	193	155	1.193

Table K.3: Rock Sizes of Heavy Grading (CIRIA, CUR, CETMEF, 2007)

NLL/NUL (kg)	D_{n50} **(mm)**	M_{50} **(kg)**	M_{em} **(kg)**	M_{50}/M_{em} **(-)**
300-1000	646	715	615	1.163
1000-3000	924	2090	1900	1.099
3000-6000	1214	4745	4500	1.054
6000-10000	1457	8195	8000	1.024
10000-15000	1677	12500	12500	1.002

The lowest and the highest size of the rock per subgrading are provided by Nominal Lower Limit (NLL) and Nominal Upper Limit (NUL).

The relation between the nominal diameter (D_{n50}) and the mass for a single stone which is exceeded by 50% of the total stones mass (M_{50}) is as follows:

$$D_{n50} = (M_{50}/\rho_{stone})^{1/3} \tag{K.1}$$

Two other variables mentioned are the diameter for a single stone which is exceeded by 50% of the total stones diameters (D_{50}) and the effective mean mass (M_{em}).

The bulk density of rock is assumed to be 1590 kg/m^3 (including 40% pores), which is a result of a grain density of 2650 kg/m^3.

K.2. STEEL SHEET PILES

Section	Dimensions				A	G_{sp}	G_w	I_y	$W_{el,y}$
	b	h	t	s					
	mm	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^4/m	cm^3/m
AZ 12-770	770	344	8,5	8,5	120	72,6	94,3	21 430	1 245
AZ 13-770	770	344	9,0	9,0	126	76,1	98,8	22 360	1 300
AZ 14-770	770	345	9,5	9,5	132	79,5	103,2	23 300	1 355
AZ 14-770-10/10	770	345	10,0	10,0	137	82,9	107,7	24 240	1 405
AZ 12-700	700	314	8,5	8,5	123	67,7	96,7	18 880	1 205
AZ 13-700	700	315	9,5	9,5	135	74,0	105,7	20 540	1 305
AZ 13-700-10/10	700	316	10,0	10,0	140	77,2	110,2	21 370	1 355
AZ 14-700	700	316	10,5	10,5	146	80,3	114,7	22 190	1 405
AZ 17-700	700	420	8,5	8,5	133	73,1	104,4	36 230	1 730
AZ 18-700	700	420	9,0	9,0	139	76,5	109,3	37 800	1 800
AZ 19-700	700	421	9,5	9,5	146	80,0	114,3	39 380	1 870
AZ 20-700	700	421	10,0	10,0	152	83,5	119,3	40 960	1 945
AZ 24-700	700	459	11,2	11,2	174	95,7	136,7	55 820	2 430
AZ 26-700	700	460	12,2	12,2	187	102,9	146,9	59 720	2 600
AZ 28-700	700	461	13,2	13,2	200	110,0	157,2	63 620	2 760
AZ 24-700N	700	459	12,5	9,0	163	89,7	128,2	55 890	2435
AZ 26-700N	700	460	13,5	10,0	176	96,9	138,5	59 790	2600
AZ 28-700N	700	461	14,5	11,0	189	104,1	148,7	63 700	2765
AZ 36-700N	700	499	15,0	11,2	216	118,6	169,5	89 610	3 590
AZ 38-700N	700	500	16,0	12,2	230	126,4	180,6	94 840	3 795
AZ 40-700N	700	501	17,0	13,2	244	134,2	191,7	100 080	3 995
AZ 42-700N	700	499	18,0	14,0	259	142,1	203,1	104 930	4 205
AZ 44-700N	700	500	19,0	15,0	273	149,9	214,2	110 150	4 405
AZ 46-700N	700	501	20,0	16,0	287	157,7	225,3	115 370	4 605
AZ 48-700 New	700	503	22,0	15,0	288	158,5	226,4	119 650	4 755
AZ 50-700 New	700	504	23,0	16,0	303	166,3	237,5	124 890	4 955
AZ 52-700 New	700	505	24,0	17,0	317	174,1	248,7	130 140	5 155

Figure K.1: Steel Sheet Pile Properties
(ArcelorMittal. (2014). *Z Section.* Retrieved from http://sheetpiling.arcelormittal.com/. Accessed on December 4, 2014.)

A	Cross sectional steel area
G_{sp}	Mass per single pile
G_w	Mass per m/ft of wall
I_y	Moment of inertia about the main neutral axis y-y
$W_{el,y}$	Elastic section modulus

K.3. PLASTIC SHEET PILES

		Opneembaar moment rep.	Opneembaar moment rek.	Weerstands moment	Traagheids moment	Buigstijfheid	Dikte	Hoogte	Breedte	Profiel	Gewicht	Gewicht	Doorsnede oppervlakte	Omtrek
	type	Mmax	Mmin	Wx	Ix	E*I	t	h	b		per plank	per m2	A	enkelzijdig
		kNm/m¹	kNm/m¹	cm³/m¹	cm⁴/m¹	kNm²/m¹	mm¹	mm¹	mm¹	type	kg/m¹	kg/m²	cm²/m¹	m²/m²
		A	B											
	Shore Guard													
Vinyl	SG-225	17,08	8,54	387	2.458	64,40	5,7	127	457	Ω	6,02	13,17	91,47	1,60
	SG-300	27,76	13,88	629	5.599	146,70	6,4	178	305	Z	4,82	15,8	109,77	1,66
	SG-325	26,34	13,17	597	5.326	139,54	6,4	178	610	Ω	8,99	14,74	102,41	1,57
	SG-425	33,92	16,96	769	7.784	203,94	7,2	203	610	Ω	10,38	17,02	118,26	1,64
	SG-525	41,74	20,87	946	10.788	282,65	7,4	229	610	Ω	11,62	19,05	132,34	1,77
	SG-550	46,26	23,13	1.048	10.652	279,08	9,4	203	305	Z	7,42	24,33	169,05	1,83
	SG-625	57,88	28,94	1.312	16.660	436,49	9,8	254	762	Ω	17,86	23,44	162,86	1,67
	SG-650	70,22	35,11	1.591	20.212	529,55	9,8	254	457	Z	11,8	25,82	179,45	1,80
	SG-750	86,36	43,18	1.957	24.854	651,17	11,7	254	305	Z	10,86	35,61	247,48	2,17
	SG-825	87,05	43,53	1.973	30.043	787,12	12,2	305	762	Ω	23,21	30,46	211,70	1,73
	SG-850	88,24	44,12	2.000	25.400	665,48	13,1	254	457	Z	15,44	33,78	231,00	1,80
	SG-950	134,74	67,37	3.054	46.567	1.220,06	16,5	305	457	Z	20,76	45,43	315,60	1,99
	C-Lock													
	CL-9000	38,44	19,22	871	9.969	261,19	7,1	229	610	Ω	10,72	17,57	121,39	1,70
	CL-9900	47,44	23,72	1.075	12.290	322,00	8,9	229	610	Ω	13,31	21,82	151,62	1,72
	Sheer Scape													
	FP-475	48,64	24,32	1.102	6.145	161,00	6,4/6,1	178	610	F	6,99	11,45	79,57	2,03
	FP-575	74,02	37,01	1.677	12.430	325,67	7,4/12,1	229	610	F	14,31	23,46	163,00	2,06
	Ultra Composite													
Composiet	UC-30	96,38	48,19	698	7.101	1.958,38	6,4/6,7	203	457	Z	9,2	20,13	99,58	1,44
	UC-50	154,20	77,10	1.118	14.200	3.916,22	8,3/9,0	254	914	Ω	22,99	25,15	130,39	1,48
	UC-75	281,70	140,85	2.043	36.325	10.018,07	10,2/10,9	356	610	Z	19,88	32,59	173,13	1,60
	UC-95	433,68	216,84	3.145	67.870	18.717,87	13,7	432	762	Z	31,37	41,17	218,70	1,56

Figure K.2: Synthetic Sheet Pile Properties
(Sheet Pile Europe, 2015)

Design stress	Vinyl	$\sigma_d = 22.06 N/mm^2$
Design stress	Ultra Composite	$\sigma_d = 68.95 N/mm^2$
Modulus of elasticity	Vinyl	$E = 0.02620 * 10^5 N/mm^2$
Modulus of elasticity	Ultra Composite	$E = 0.27579 * 10^5 N/mm^2$

K.4. REINFORCED CONCRETE

The properties of concrete and steel, which are used to obtain reinforced concrete, are discussed separately.

K.4.1. CONCRETE

Table K.4: Concrete Properties (NEN, 2004)

Concrete Class	$f_{ck,cil}$ (MPa)	f_{ck} (MPa)	f_{cm} (MPa)	f_{ctm} (MPa)	$f_{ctk,0.05}$ (MPa)	$f_{ctk,0.95}$ (MPa)	E_{cm} (GPa)
C12/15	12	15	20	1.6	1.1	2.0	27
C20/25	20	25	28	2.2	1.5	2.9	30
C30/37	30	35	38	2.9	2.0	3.8	33
C35/45	35	45	43	3.2	2.2	4.2	34
C45/55	45	55	53	3.8	2.7	4.9	36
C55/67	55	67	63	4.2	3.0	5.5	38

$f_{ck,cil}$	characteristic compressive cylinder strength after 28 days (MPa)
f_{ck}	characteristic compressive cube strength (MPa)
f_{cm}	mean value of the concrete cylinder compressive strength after 28 days (MPa)
f_{ctm}	mean value of axial tensile strength (MPa) $f_{ctm} = 0.30 f_{ck} < C50/60$ $f_{ctm} = 2.12 ln(1 + (f_{ck}/10))) > C50/60$
$f_{ctk,0.05}$	characteristics axial tensile strength $f_{ctk,0.05} = 0.7 f_{ctm} 5\% fractile > C50/60$ (MPa)
$f_{ctk,0.95}$	characteristics axial tensile strength $f_{ctk,0.95} = 1.3 f_{ctm} 95\% fractile > C50/60$ (MPa)
E_{cm}	second modulus of elasticity $E_{cm} = 22(f_{cm}/10)^{0.3}$ (GPa)

Table K.5: Partial Safety Factors of Ultimate Limit State (NEN, 2004)

Load Type	γ_c	γ_s
Persistent and transient	1.5	1.15
Accidental	1.2	1.0

γ_c	Partial safety factor for concrete $(-)$
γ_s	Partial safety factor for reinforcement steel $(-)$

K.4.2. REINFORCEMENT STEEL

Table K.6: Steel Properties

Reinforcement steel classes	⌀ (mm)	R_e (MPa)	R_m/R_e (-)	A_{gt} (%)
B500A	4-16	500	1.05 (1.03 for diameter <5.5mm)	3.0
B500B	6-50	500	1.08	5.0
B500C	6-50	500	1.15 (1.13 for diameter <12mm)	7.5

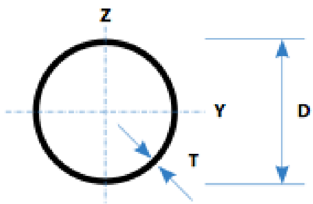
(NEN, 2008)

⊘	nominal diameter (mm)
R_e	characteristic yield strength of reinforcement (MPa)
R_m	characteristic tensile strength of reinforcement (MPa)
R_m/R_e	minimum ratio tensile (yield) strength $(-)$
A_{gt}	minimum percentage total elongation at maximum force $(-)$
A	indicates a smooth, dented or ribbed profile $(-)$
B	indicates a dented or ribbed profile $(-)$
C	indicates ribbed profile $(-)$

The Young's modulus of steel is $2.0 \cdot 10^5 N/mm^2$ and the general bar diameters of reinforcement steel are 12, 16, 20, 25, and 32 mm.

K.5. TUBULAR PILES

The cross-section shows the variables, which characterize the various tubular piles offered on the market.



The typical dimensions are presented in the following tables.

Outside Diameter D mm	Thickness T mm	Mass M kg/m	Sectional area A cm^2	Moment of inertia I cm^4	Radius of gyration i cm	Elastic modulus W_{el} cm^3	Plastic modulus W_{pl} cm^3
101.6	3.2	7.77	9.89	120	3.48	23.6	31.0
	3.6	8.70	11.1	133	3.47	26.2	34.6
	4.0	9.63	12.3	146	3.45	28.8	38.1
	4.5	10.8	13.7	162	3.44	31.9	42.5
	5.0	11.9	15.2	177	3.42	34.9	46.7
	5.6	13.3	16.9	195	3.40	38.4	51.7
	6.3	14.8	18.9	215	3.38	42.3	57.3
	8.0	18.5	23.5	260	3.32	51.1	70.3
	10.0	22.6	28.8	305	3.26	60.1	84.2
114.3	3.2	8.77	11.2	172	3.93	30.2	39.5
	3.6	9.83	12.5	192	3.92	33.6	44.1
	4.0	10.9	13.9	211	3.90	36.9	48.7
	4.5	12.2	15.5	234	3.89	41.0	54.3
	5.0	13.5	17.2	257	3.87	45.0	59.8
	5.6	15.0	19.1	283	3.85	49.6	66.2
	6.3	16.8	21.4	313	3.82	54.7	73.6
	8.0	21.0	26.7	379	3.77	66.4	90.6
	10.0	25.7	32.8	450	3.70	78.7	109
139.7	3.2	10.8	13.7	320	4.83	45.8	59.6
	3.6	12.1	15.4	357	4.81	51.1	66.7
	4.0	13.4	17.1	393	4.80	56.2	73.7
	4.5	15.0	19.1	437	4.78	62.6	82.3
	5.0	16.6	21.2	481	4.77	68.8	90.8
	5.6	18.5	23.6	531	4.75	76.1	101
	6.3	20.7	26.4	589	4.72	84.3	112
	8.0	26.0	33.1	720	4.66	103	139
	10.0	32.0	40.7	862	4.60	123	169
	12.5	39.2	50.0	1020	4.52	146	203
168.3	5.0	20.1	25.7	856	5.78	102	133
	5.6	22.5	28.6	948	5.76	113	148
	6.3	25.2	32.1	1053	5.73	125	165
	8.0	31.6	40.3	1297	5.67	154	206
	10.0	39.0	49.7	1564	5.61	186	251
	12.5	48.0	61.2	1868	5.53	222	304
193.7	5.0	23.3	29.6	1320	6.67	136	178
	5.6	26.0	33.1	1465	6.65	151	198
	6.3	29.1	37.1	1630	6.63	168	221
	8.0	36.6	46.7	2016	6.57	208	276

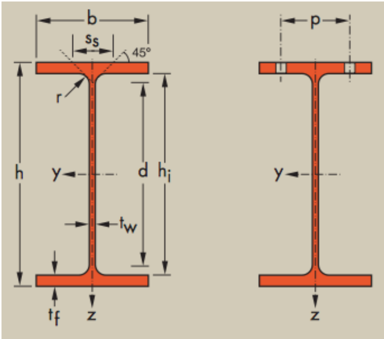
	10.0	45.3	57.7	2442	6.50	252	338
	12.5	55.9	71.2	2934	6.42	303	411
	16.0	70.1	89.3	3554	6.31	367	507
219.1	4.5	23.8	30.3	1747	7.59	159	207
	5.0	26.4	33.6	1928	7.57	176	229
	5.6	29.5	37.6	2142	7.55	195	255
	6.3	33.1	42.1	2386	7.53	218	285
	8.0	41.6	53.1	2960	7.47	270	357
	10.0	51.6	65.7	3598	7.40	328	438
	12.5	63.7	81.1	4345	7.32	397	534
	14.2	71.8	91.4	4820	7.26	440	597
	16.0	80.1	102	5297	7.20	483	661
Outside Diameter	**Thickness**	**Mass**	**Sectional area**	**Moment of inertia**	**Radius of gyration**	**Elastic modulus**	**Plastic modulus**
D	**T**	**M**	**A**	**I**	**i**	**W_{el}**	**W_{pl}**
mm	**mm**	**kg/m**	**cm^2**	**cm^4**	**cm**	**cm^3**	**cm^3**
244.5	5.0	29.5	37.6	2699	8.47	221	287
	5.6	33.0	42.0	3000	8.45	245	320
	6.3	37.0	47.1	3346	8.42	274	358
	8.0	46.7	59.4	4160	8.37	340	448
	10.0	57.8	73.7	5073	8.30	415	550
	12.5	71.5	91.1	6147	8.21	503	673
	14.2	80.6	103	6837	8.16	559	754
	16.0	90.2	115	7533	8.10	616	837
273.0	5.0	33.0	42.1	3781	9.48	277	359
	5.6	36.9	47.0	4207	9.46	308	400
	6.3	41.4	52.8	4696	9.43	344	448
	8.0	52.3	66.6	5852	9.37	429	562
	10.0	64.9	82.6	7154	9.31	524	692

	12.5	80.3	102	8697	9.22	637	849
	14.2	90.6	115	9695	9.16	710	952
	16.0	101	129	10707	9.10	784	1058
323.9	5.0	39.3	50.1	6369	11.3	393	509
	5.6	44.0	56.0	7094	11.3	438	567
	6.3	49.3	62.9	7929	11.2	490	636
	8.0	62.3	79.4	9910	11.2	612	799
	10.0	77.4	98.6	12158	11.1	751	986
	12.5	96.0	122	14847	11.0	917	1213
	14.2	108	138	16599	11.0	1025	1363
	16.0	121	155	18390	10.9	1136	1518
355.6	6.3	54.3	69.1	10547	12.4	593	769
	8.0	68.6	87.4	13201	12.3	742	967
	10.0	85.2	109	16223	12.2	912	1195
	12.5	106	135	19852	12.1	1117	1472
	14.2	120	152	22227	12.1	1250	1656
	16.0	134	171	24663	12.0	1387	1847
406.4	6.3	62.2	79.2	15849	14.1	780	1009
	8.0	78.6	100	19874	14.1	978	1270
	10.0	97.8	125	24476	14.0	1205	1572
	12.5	121	155	30031	13.9	1478	1940
	14.2	137	175	33685	13.9	1658	2185
	16.0	154	196	37449	13.8	1843	2440
457.0	6.3	70.0	89.2	22654	15.9	991	1280
	8.0	88.6	113	28446	15.9	1245	1613
	10.0	110	140	35091	15.8	1536	1998
	12.5	137	175	43145	15.7	1888	2470
	14.2	155	198	48464	15.7	2121	2785
	16.0	174	222	53959	15.6	2361	3113
508.0	6.3	77.9	99.3	31246	17.7	1230	1586
	8.0	98.6	126	39280	17.7	1546	2000
	10.0	123	156	48520	17.6	1910	2480
	12.5	153	195	59755	17.5	2353	3070
	14.2	173	220	67199	17.5	2646	3463
	16.0	194	247	74909	17.4	2949	3874

Figure K.3: Tubular Pile Properties
(Tata Steel, 2013)

K.6. SUPPORT BEAMS

The cross-section shows the variables, which characterize the various support beams offered on the market.



The typical dimensions are presented in the following tables.

Désignation Designation Bezeichnung	G kg/m	h mm	b mm	t_w mm	t_f mm	r mm	A mm^2 $x10^2$
		Dimensions Abmessungen					
IPE AA 120*	8,4	117	64	3,8	4,8	7,0	10,7
IPE A 120•	8,7	117,6	64	3,8	5,1	7,0	11,0
IPE 120	10,4	120	64	4,4	6,3	7,0	13,2
IPE AA 180*	14,9	176,4	91	4,3	6,2	9,0	19,0
IPE A 180•	15,4	177	91	4,3	6,5	9,0	19,6
IPE 180	18,8	180	91	5,3	8,0	9,0	23,9
IPE O 180+	21,3	182	92	6,0	9,0	9,0	27,1
IPE AA 220*	21,2	216,4	110	4,7	7,4	12,0	27,0
IPE A 220•	22,2	217	110	5,0	7,7	12,0	28,3
IPE 220	26,2	220	110	5,9	9,2	12,0	33,4
IPE O 220+	29,4	222	112	6,6	10,2	12,0	37,4
IPE A 270•	30,7	267	135	5,5	8,7	15,0	39,2
IPE 270	36,1	270	135	6,6	10,2	15,0	45,9
IPE O 270+	42,3	274	136	7,5	12,2	15,0	53,8
IPE A 300•	36,5	297	150	6,1	9,2	15,0	46,5
IPE 300	42,2	300	150	7,1	10,7	15,0	53,8
IPE O 300+	49,3	304	152	8,0	12,7	15,0	62,8

Désignation Designation Bezeichnung	G	Valeurs statiques / Section properties axe fort y-y strong axis y-y starke Achse y-y					axe faible z-z weak axis z-z schwache Achse z-z			
		I_y	$W_{el.y}$	$W_{pl.y}$♦	i_y	A_{vz}	I_z	$W_{el.z}$	$W_{pl.z}$♦	i_z
	kg/m	mm^4 $x10^4$	mm^3 $x10^3$	mm^3 $x10^3$	mm x10	mm^2 $x10^2$	mm^4 $x10^4$	mm^3 $x10^3$	mm^3 $x10^3$	mm x10
IPE AA 120	8,4	244	41,7	47,6	4,79	5,36	21,1	6,59	10,4	1,41
IPE A 120	8,7	257	43,8	49,9	4,83	5,41	22,4	7,00	11,0	1,42
IPE 120	10,4	318	53,0	60,7	4,90	6,31	27,7	8,65	13,6	1,45

IPE AA 180	14,9	1020	116	131	7,32	9,13	78,1	17,2	26,7	2,03
IPE A 180	15,4	1063	120	135	7,37	9,20	81,9	18,0	28,0	2,05
IPE 180	18,8	1317	146	166	7,42	11,3	101	22,2	34,6	2,05
IPE O 180	21,3	1505	165	189	7,45	12,7	117	25,5	39,9	2,08
IPE AA 220	21,2	2219	205	230	9,07	12,8	165	29,9	46,5	2,47
IPE A 220	22,2	2317	214	240	9,05	13,6	171	31,2	48,5	2,46
IPE 220	26,2	2772	252	285	9,11	15,9	205	37,3	58,1	2,48
IPE O 220	29,4	3134	282	321	9,16	17,7	240	42,8	66,9	2,53
IPE A 270	30,7	4917	368	413	11,2	18,8	358	53,0	82,3	3,02
IPE 270	36,1	5790	429	484	11,2	22,1	420	62,2	97,0	3,02
IPE O 270	42,3	6947	507	575	11,4	25,2	514	75,5	118	3,09
IPE A 300	36,5	7173	483	542	12,4	22,3	519	69,2	107	3,34
IPE 300	42,2	8356	557	628	12,5	25,7	604	80,5	125	3,35
IPE O 300	49,3	9994	658	744	12,6	29,1	746	98,1	153	3,45

Figure K.4: IPE Beam Properties
(ArcelorMittal. (2014). *IPE*. Retrieved from http://sheetpiling.arcelormittal.com/. Accessed on November 30, 2015.)

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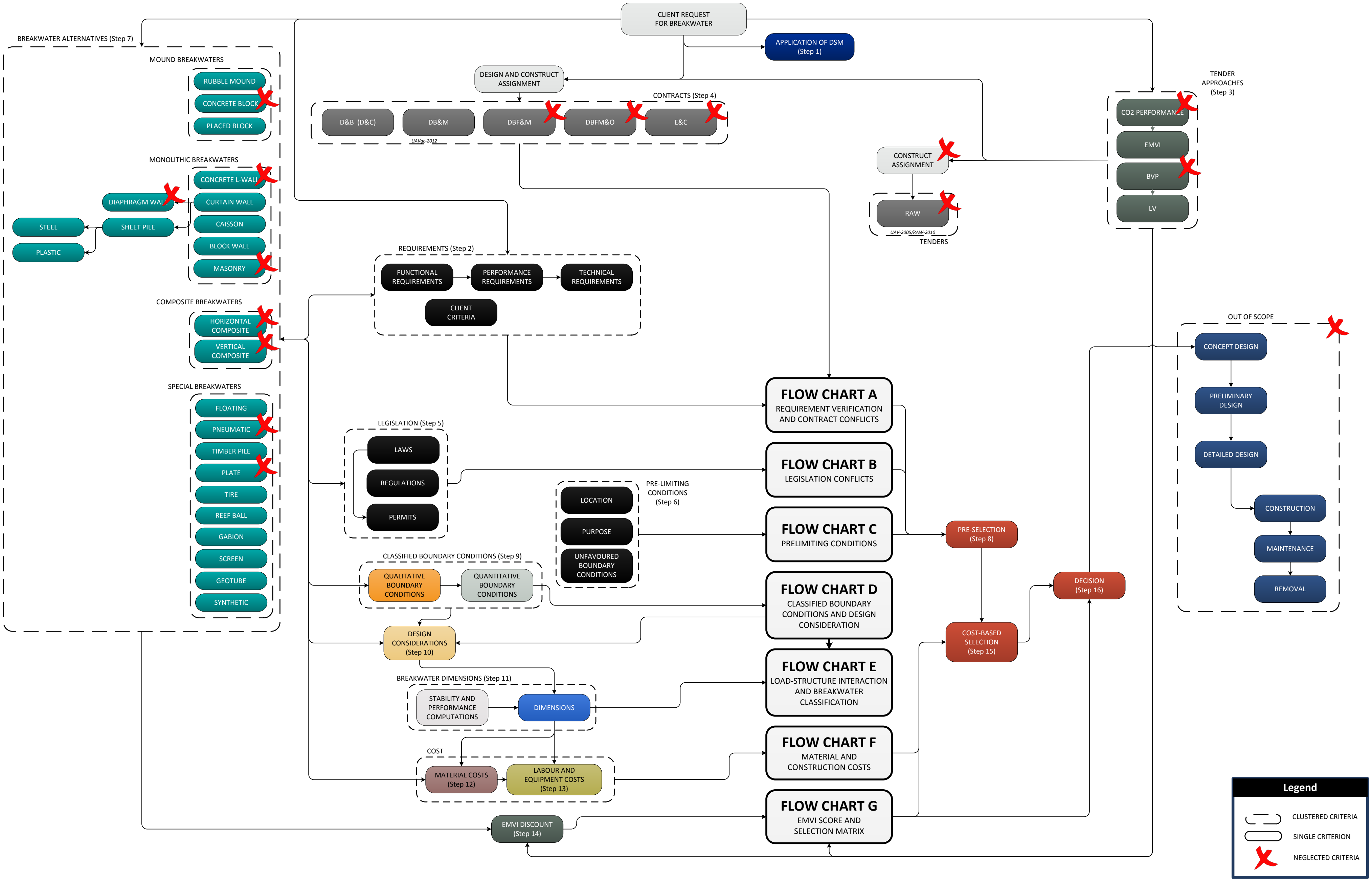
MASTER FLOW CHART: OVERVIEW OF THE DECISION SUPPORT METHOD

A more detailed elaboration of Figure 4.1 is found in this appendix. It consists of design steps (1 through 16) with underlying flow charts (A through F). The charts are required to obtain the input to perform the assessment. The assessment consists of the pre-selection (step 8) and cost-based selection (step 15). These two selection steps will support the decision making (step 16) in which one or more breakwater alternative(s) are recommended to further design.

MASTER FLOW CHART

OVERVIEW OF THE DECISION SUPPORT METHOD



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Sub Flow Chart A: Requirement Verification and Contract Conflicts

In this appendix the interaction between the requirements and contracts, and the breakwater alternatives is verified. By using green, red and yellow shapes in the several flow charts, a breakwater alternative could respectively cooperate, conflict or be situation dependent. The verification is performed by engineering judgement.

SUB FLOW CHART A

REQUIREMENT VERIFICATION AND CONTRACT CONFLICTS

REQUIREMENTS

CODE	FUNCTIONAL REQUIREMENT CODE	REQUIREMENT	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
R1	F1, F2 AND F3	REDUCTION WAVE ACTION	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green
R2	F4	REDUCTION FLOW VELOCITIES	Green	Green	Green	Green	Green	Red	Green	Red	Red	Green	Red	Green	Red
R3	F5	NOT SEDIMENTATION IN CHANNEL	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange
R4	F6	OBSERVABLE BY NAVIGATION	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange
R5	F7, F8, F23 AND F25	AFFECTING LANDSCAPE	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange
R6	F9	ADJUSTABLE IN SIZE	Green	Green	Red	Green	Green	Green	Green	Green	Green	Green	Red	Red	Green
R7	F10, F24	NO HINDRANCE OF FAIRWAY	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange
R8	F11	MOORING OPPORTUNITY	Red	Red	Green	Green	Green	Green	Red	Red	Red	Red	Orange	Red	Red
R9	F12	NO DAMAGE TO VESSELS	Red	Red	Orange	Orange	Orange	Green	Red	Green	Red	Red	Orange	Green	Green
R10	F13	ENABLE WALKING	Orange	Orange	Green	Green	Green	Green	Red	Red	Red	Red	Orange	Orange	Red
R11	F14	STAY OVERNIGHT FACILITIES	Orange	Orange	Orange	Green	Green	Green	Red	Red	Red	Red	Red	Orange	Red
R12	F15	NOT INTERRUPTING AQUATIC ECOLOGY	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green
R13	F16	FREE OF MAINTENANCE	Red	Red	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green
R14	F17	LONG LIFETIME	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Green	Orange	Orange	Orange	Green	Green
R15	F18	NOT AFFECTING WATER QUALITY	Orange	Orange	Green	Orange	Orange	Green	Green	Green	Orange	Orange	Green	Green	Green
R16	F19	ENTIRELY REMOVABLE	Red	Red	Green	Red	Red	Green	Green	Green	Red	Red	Green	Green	Green
R17	F20	NO HINDER TRAFFIC AND RESIDANTS	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange	Orange
R18	F21	TAKES LIMITED SPACE	Red	Red	Green	Red	Green	Green	Green	Green	Green	Green	Green	Green	Green
R19	F22	LOW LIFE CYCLE COSTS	Red	Red	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green
R20	F26	PARTIAL WAVE REFLECTION	Green	Green	Red	Red	Red	Red	Green	Green	Green	Green	Green	Green	Green
R21	F27	NO WAVE TRANSMISSION	Orange	Orange	Orange	Orange	Orange	Red	Red	Red	Red	Orange	Red	Orange	Red

Remarks:

R1 Wave energy is reduced by the structure to have reduced wave action in the sheltered area.
R2 Flow velocities are obstructed by the breakwater.
R3 The structure does not trap sediment, which can block a fairway.
R4 High crest, striking colours and mark points accomplish an observable breakwater.
R5 The materials and colours fit the landscape.
R6 Simplicity of adjusting the length of the structure.
R7 Passage of vessels is not hindered by the size or shape of the breakwater.
R8 Vessel are able to more at the breakwater.
R9 In case of vessel-structure interaction vessels are not damaged easily.
R10 Pedestrians can access the top of the structure.
R11 Mooring facilities, electricity and public lighting is provided.
R12 The construction elements do not affect the water quality of, biological process in and chemical balance in inland water systems.
R13 In its lifetime, the breakwater is to be free of maintenance.
R14 The general lifetime is extensive.
R15 Water quality could be affected by suspended load released from the bed during construction.
R16 After removal there will be no trace of the breakwater.
R17 During construction the traffic and local residents are not hindered.
R18 The space required for the breakwater is limited compared to the other alternatives.
R19 The life cycle cost found in maintenance during the structures lifetime is low.
R20 Wave reflection is partially allowed.
R21 The wave is not transmitted through or over the breakwater.

CONTRACTS

CODE	CONTRACT	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
C1	D&B	Green	Green	Green	Green	Green	Orange	Green	Orange	Orange	Orange	Green	Orange	Orange
C2	DB&M	Green	Green	Orange	Orange	Orange	Green	Green	Green	Orange	Orange	Orange	Green	Orange

Remarks:

C1* The significant amount of time for design and construction should get special attention.
C2* The intensity of inspection and maintenance of the structure should get special attention.
***experience of the contractor in the phase of design, construction and maintenance should be considered**

Legend

REQUIREMENTS

- Green: NO CONFLICT WITH REQUIREMENT
- Red: CONFLICT WITH REQUIREMENT
- Orange: DEPENDS ON SITUATION

CONTRACTS

- Green: MOST PREFERRED
- Orange: LESS PREFERRED

N

SUB FLOW CHART B: LEGISLATION CONFLICTS

In this appendix the interaction between the laws, permits and legislation, and the breakwater alternatives is verified. By using green, red and yellow shapes in the several flow charts, a breakwater alternative could respectively cooperate, conflict or be situation dependent. The verification is performed by engineering judgement.

SUB FLOW CHART B

LEGISLATION CONFLICTS

LAWS

CODE	LAW	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
L1	LAW OF WATER	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED
L2	LAW OF FLORA AND FAUNA	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
L3	LAW OF SUBSOIL PROTECTION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	REQUIRED	NOT REQUIRED
L4	LAW OF NATURE PROTECTION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
L5	LAW OF NATURE PRESERVATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
L6	LAW OF MARITIME TRAFFIC	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION
L7	LAW OF EXCAVATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	REQUIRED	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
L8	COMPENSATION OF NATURE	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION
L9	EUROPIAN WATER DIRECTIVE	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	REQUIRED	NOT REQUIRED

Remarks:
L1 Quality of the surface waters is guaranteed.
L2 No sound disturbance and damage to vegetation.
L3 Disturbance of the bed and released polluted soil. (See also L1.)
L4 Protection of animals, vegetation, biodiversity and natural resources. (See also L2.)
L5 Prevent hinder, damage or danger of surrounding nature. (Environmental Permits, WABO and MER is obliged.) (See also L2 and L4)
L6 Safety of transport and construction vessels.
L7 Excavation to win or remove sediment.
L8 Removed nature is compensated in a different area. (See also L2, L3 and L5.)
L9 Quality of the surface waters is guaranteed. (See also L1, L2 and L3.)

PERMITS

CODE	PERMIT	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
P1	PERMIT OF SURROUNDINGS	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED
P2	ENVIRONMENTAL PERMIT	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
P3	WATER PERMIT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT
P4	WATER NOTIFICATION	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED
P5	EXCAVATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
P6	DECISION UNIFORM SOIL RESTORATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION
P7	PERMITS OF SOIL RESTORATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
P8	STORAGE REQUEST SILT DEPOSIT	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION
P9	CENTRE OF INFORMATION ABOUT CABLES AND PIPES	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	NOT REQUIRED	NOT REQUIRED	REQUIRED	REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED
P10	CONTACT POINT OF PUBLIC AREA	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT	IRRELEVENT
P11	PERMIT OF CONSTRUCTION	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED
P12	PERMIT OF DEMOLITION	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED

Remarks:
P1 Required for all construction projects.
P2 Originates from the Law of Nature Preservation.
P3 Required for extraction of ground water and infiltration of water.
P4 Required for all construction projects around surface waters.
P5 Required when excavation is planned.
P6 Required to remove polluted subsoil.
P7 Required to remove pollutions from the subsoil.
P8 Required to enable construction traffic by road.
P9 Required when excavation is planned.
P10 Required for all construction projects.
P11 Required to start construction.
P12 Required to disassemble or demolish a structure.

REGULATIONS

CODE	REGULATION	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
R1	DECISION OUTSIDE INTERIOR	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED
R2	DECISION SOIL QUALITY	NOT REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	NOT REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION	NOT REQUIRED	NOT REQUIRED	DEPENDS ON SITUATION	DEPENDS ON SITUATION	DEPENDS ON SITUATION
R3	MANAGE AND DEVELOPMENT PLAN OF WATERWAYS	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED
R4	DECISION OF ACTIVITIES	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED	REQUIRED

Remarks:
R1 The subsoil quality is controlled.
R2 Structure fits the landscape (urban or natural).
R3 Attention is there for recreational, fishermen's and nature.
R4 Activities should be notified.

Legend

- NOT REQUIRED
- REQUIRED
- DEPENDS ON SITUATION
- IRRELEVENT

O

SUB FLOW CHART C: PRE-LIMITING CONDITIONS

In this appendix the interaction between the location, purpose and unfavoured boundary conditions, and the breakwater alternatives is verified. By using green, red and yellow shapes in the several flow charts, a breakwater alternative could respectively cooperate, conflict or be situation dependent. The verification is performed by engineering judgement.

SUB FLOW CHART C

PRE-LIMITING CONDITIONS

LOCATION

CODE	STRUCTURE AND LOCATION	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
S1	LAKE BREAKWATER	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green
S2	RIVER GROYNE	Red	Red	Green	Red	Red	Red	Yellow	Red	Yellow	Yellow	Yellow	Yellow	Red

Remarks:
S1 Breakwater is applicable at lakes.
S2 Breakwaters is applicable as river groyne.

PURPOSE

CODE	PURPOSE	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
Q1	SMALL PORTS AND MOORING AREAS	Green	Green	Green	Green	Green	Green	Yellow	Green	Yellow	Green	Green	Green	Green
Q2	SWIMMING AREAS, BANKS, HABITATS	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green	Green

Remarks:
Q1 Breakwater is optional for marinas. (economical purpose)
Q2 Breakwater is optional for bank and habitat protection. (non-profitable purpose)

BOUNDARY CONDITIONS

CODE	BOUNDARY CONDITIONS	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
B1	HIGH WAVES	Green	Green	Green	Green	Green	Yellow	Green	Red	Red	Yellow	Green	Green	Red
B2	HIGH WATER DEPTH	Green	Green	Green	Green	Green	Green	Green	Green	Red	Green	Green	Red	Green
B3	HIGH FLOW VELOCITIES	Green	Green	Green	Green	Green	Red	Green	Red	Green	Green	Green	Green	Red
B4	ICE	Green	Green	Green	Green	Green	Red	Red	Red	Red	Yellow	Yellow	Green	Red
B5	WEAK SUBSOIL	Yellow	Yellow	Yellow	Red	Red	Green	Green	Green	Green	Yellow	Yellow	Yellow	Green
B6	EARTHQUAKES	Yellow	Yellow	Yellow	Yellow	Yellow	Green	Green	Green	Green	Yellow	Green	Green	Green

Remarks:
B1 Breakwater can be attenuated or withstand high waves.
B2 Breakwater is applicable in high water depths.
B3 Breakwater can counteract flow velocities.
B4 Breakwater can withstand ice forces.
B5 Breakwater is applicable in case of soft soils.
B6 Breakwater is not sensitive to earthquakes.

Legend

Green	NOT APPLICABLE
Red	APPLICABLE
Yellow	DEPENDS ON SITUATION

P

Sub Flow Chart D: Classified Boundary Conditions and Design Considerations

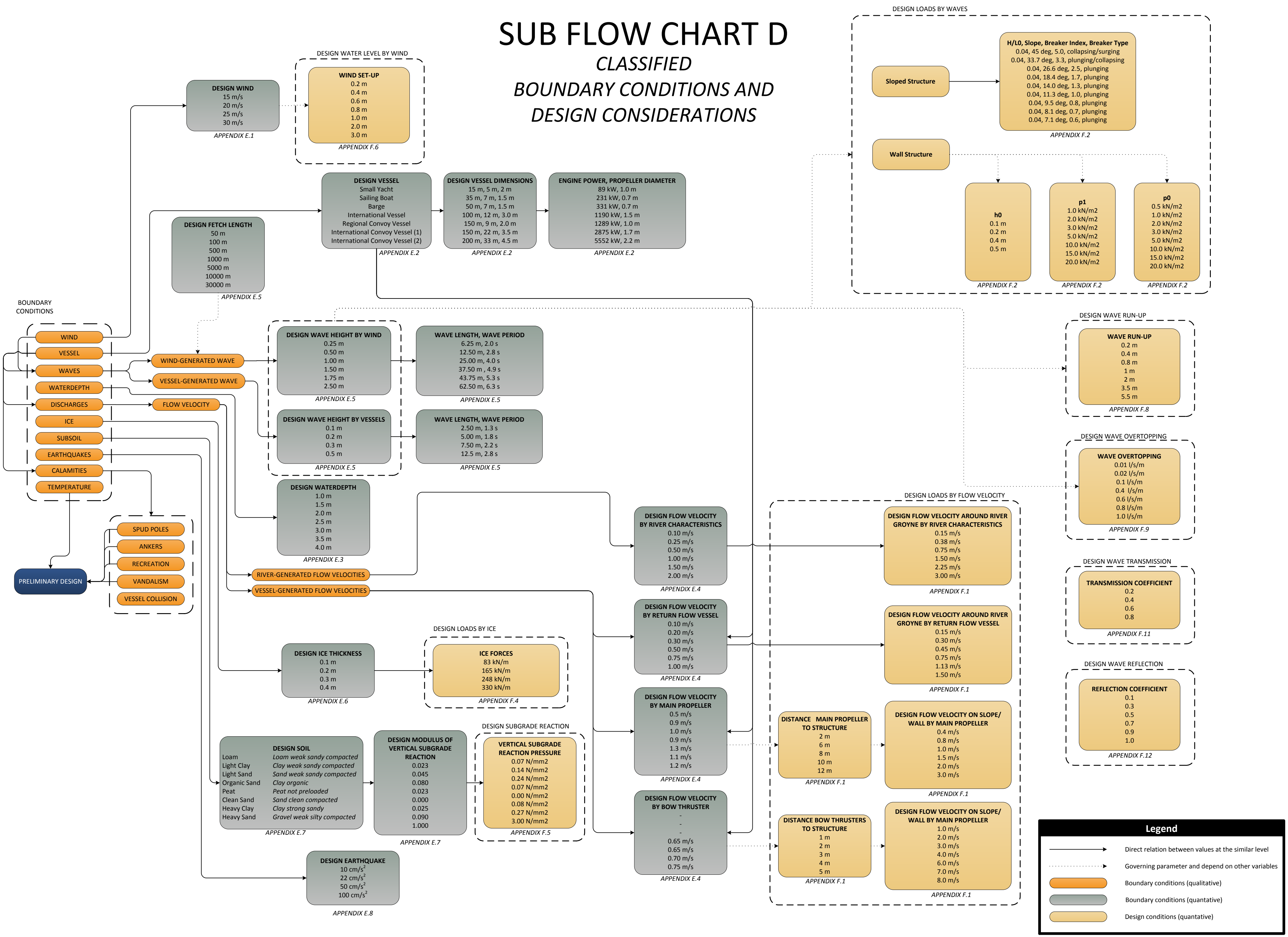
In this appendix, the boundary conditions are quantified. In the orange shapes the name is written, after which the classification is provided in grey shapes. The yellow shapes show the classification of the design considerations, which is often only connected to the most dominant dependent variable. It is advised to consult the referred appendices, which provide the necessary background information, design formulas and computations.

What is more, the boxes are connected by solid and dashed lines. The solid lines indicate a direct relation between the values from top to bottom. For example, the Small Yacht in the vessel classification is related to the vessel dimensions of 15 m (length) by 5 m (beamwidth) by 2 m (draught). The opposite is true for the dashed lines, which implies that the elaboration in the appendices should be consulted.

SUB FLOW CHART D

CLASSIFIED

BOUNDARY CONDITIONS AND DESIGN CONSIDERATIONS



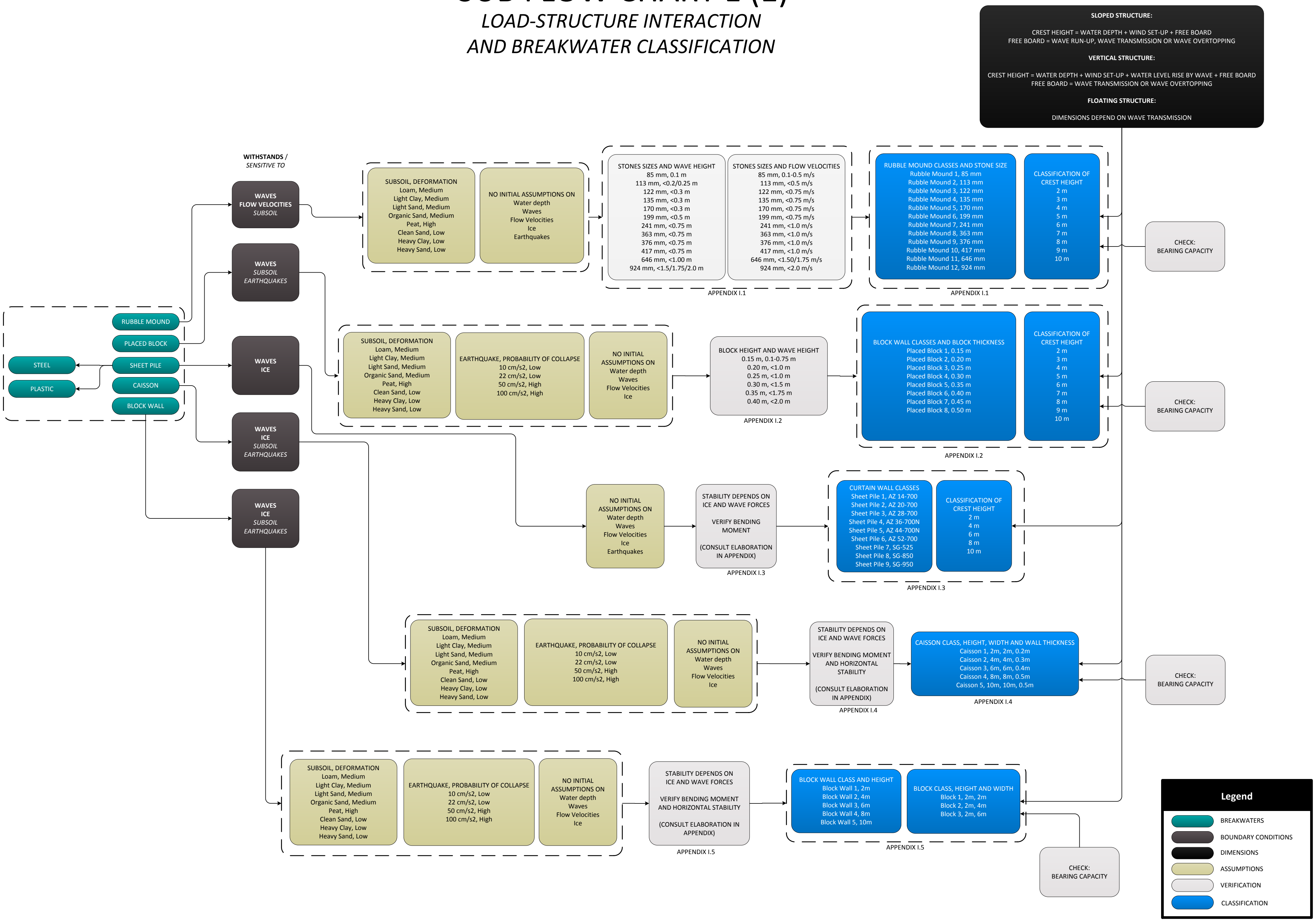
Q

Sub Flow Chart E: Load-Structure Interaction and Breakwater Classification

In this appendix, the breakwater alternatives are connected to the classified dimensions. The breakwaters are found in the blue shapes on the left. Subsequently, these should withstand and/or are sensitive to certain boundary conditions, which are shown in the black shapes. In the yellow brown shapes, the classified sensitive boundary conditions are provided with an assumed risk of instability. The black shape in the right corner shows how the dimensions of a breakwater can be determined. It is advised to consult the appendices when dimensions (blue shape) cannot be determined by using the flow chart itself, and to assess the structural performance (grey shape) for the boundary conditions to be withstood.

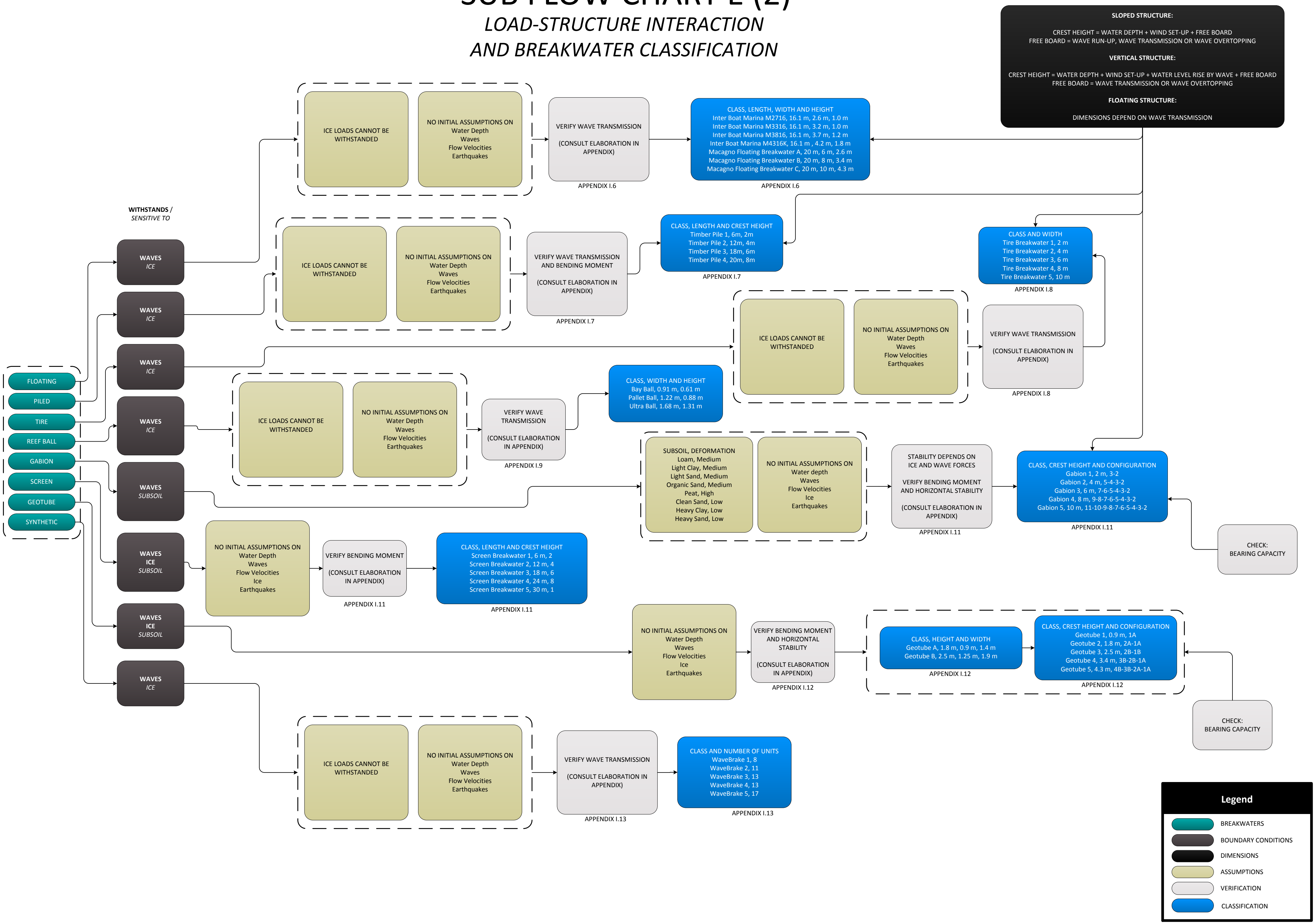
SUB FLOW CHART E (1)

LOAD-STRUCTURE INTERACTION
AND BREAKWATER CLASSIFICATION



SUB FLOW CHART E (2)

LOAD-STRUCTURE INTERACTION AND BREAKWATER CLASSIFICATION



R

Sub Flow Chart F: Material, Labour and Equipment Costs

In this appendix, the classified breakwater alternatives obtain a cost estimate of the direct costs. This is divided in the material costs (brown shapes) and the labour plus equipment costs (yellow shapes). It is advised to consult the appendices for the all necessary computations.

SUB FLOW CHART F (1)

MATERIAL, LABOUR AND EQUIPMENT COSTS

ORIENTATION

DETACHED

NON-DETACHED

SUBMERGED

IMMERSED

EQUIPMENT

LAND BASED EQUIPMENT

- EXCAVATORS
- CRANES
- DUMP TRUCKS
- BULLDOZERS

FLOATING EQUIPMENT

- DUMP VESSELS
- BARGES

RUBBLE MOUND

PLACED BLOCK

SHEET PILE

- STEEL
- PLASTIC

CAISSON

BLOCK WALL

STONES SIZES AND COST
85 mm, 32 euro/m3
113 mm, 32 euro/m3
122 mm, 32 euro/m3
135 mm, 32 euro/m3
170 mm, 32 euro/m3
199 mm, 32 euro/m3
241 mm, 32 euro/m3
363 mm, 33 euro/m3
376 mm, 33 euro/m3
417 mm, 35 euro/m3
646 mm, 38 euro/m3
924 mm, 43 euro/m3

CREST HEIGHT AND MATERIAL COST

h_c (m)	Cost RM 1-7 (€/m)	Cost RM 8-9 (€/m)	Cost RM 10 (€/m)	Cost RM 11 (€/m)	Cost RM 12 (€/m)
2	254	267	280	305	343
3	525	551	577	630	708
4	890	935	979	1068	1202
5	1352	1419	1487	1622	1825
6	1908	2003	2099	2290	2576
7	2560	2688	2816	3072	3456
8	3307	3473	3638	3969	4465
9	4150	4357	4565	4980	5602
10	5088	5342	5597	6106	6869

APPENDIX I.1

LABOUR AND EQUIPMENT COST ITEMS
Placement of rock by hydraulic excavator, 125 m3/hour
Transport vessel with hydraulic excavator, 250 euro/hour
Pontoon to carry structure elements, 25 euro/hour

CREST HEIGHT AND CONSTRUCTION COST
2 m, 18 euro
3 m, 36 euro
4 m, 62 euro
5 m, 94 euro
6 m, 132 euro
7 m 177 euro
8 m, 229 euro
9 m, 287 euro
10 m, 352 euro

APPENDIX I.1

MATERIAL COST ITEMS
Core sand 6 (per barge) euro/m3 (used)
Core sand 10 (per truck) euro/m3
Geotextile 0.5 euro/m2
Filter layer 32 euro/m2/m
Basalton/Haringman/Armorflex 200 euro/m3

CREST HEIGHT AND MATERIAL COST

h_c (m)	Cost of Sand (€/m)	Cost of Filter Layer (€/m)	Cost of Geotextile (€/m)
2	84	27	7
3	180	40	10
4	313	53	13
5	481	65	16
6	685	78	20
7	926	91	23
8	1202	103	26
9	1515	116	29
10	1864	129	32

APPENDIX I.2

CREST HEIGHT AND MATERIAL COST

	Cost of Placed Blocks (€/m)	d_b (m)							
h_c (m)	Length of Slopes with Crest Width (m)	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
2	14	410	547	684	820	957	1094	1231	1367
3	20	600	800	1000	1201	1401	1601	1801	2001
4	26	790	1054	1317	1581	1844	2108	2371	2634
5	33	980	1307	1634	1961	2288	2614	2941	3268
6	39	1171	1561	1951	2341	2731	3121	3512	3902
7	45	1361	1814	2268	2721	3175	3628	4082	4535
8	52	1551	2068	2584	3101	3618	4135	4652	5169
9	58	1741	2321	2901	3482	4062	4642	5222	5803
10	64	1931	2574	3218	3862	4505	5149	5793	6436

APPENDIX I.2

LABOUR AND EQUIPMENT COST ITEMS
Placement of sand by hydraulic excavator, 125 m3/hour
Placement of rock by hydraulic excavator, 125 m3/hour
Placement of placed block by hydraulic excavator, 15 m2/hour
Transport vessel with hydraulic excavator, 250 euro/hour
Pontoon to carry structure elements, 25 euro/hour
Placement of geotextile, 12 euro/m2
Slope finishing, 50 euro/m2

CREST HEIGHT AND CONSTRUCTION COST
2 m, 1131 euro/m
3 m, 1676 euro/m
4 m, 2233 euro/m
5 m, 2804 euro/m
6 m, 3388 euro/m
7 m, 3985 euro/m
8 m, 4595 euro/m
9 m, 5219 euro/m
10 m, 5856 euro/m

APPENDIX I.2

MATERIAL COST ITEMS
Steel 7020 euro/m3

MATERIAL COST OF SHEET PILES
(consult elaboration appendix)

APPENDIX I.3

LABOUR AND EQUIPMENT COST ITEMS
Placement of sheet piles, 35 *m2/hour*
Transport vessel with hydraulic, 250 euro/*hour*
Pontoon to carry structure elements, 25 euro/*hour*
Vibration block, 100 euro/*hour*

CREST HEIGHT AND CONSTRUCTION COST
2 m, 34 euro/m
4 m, 69 euro/m
6 m, 103 euro/m
8 m, 137 euro/m
10 m, 171 euro/m

APPENDIX I.3

MATERIAL COST ITEMS
Core sand 6 (per barge) euro/*m*3
Core sand 10 (per truck) euro/*m*3
Reinforced concrete with formwork 750 euro/*m*3
Rubble 32 euro/*m*3

CLASS AND MATERIAL COST
Caisson 1, 1895 euro/m
Caisson 2, 4579 euro/m
Caisson 3, 7902 euro/m
Caisson 4, 11864 euro/m
Caisson 5, 15136 euro/m

APPENDIX I.4

LABOUR AND EQUIPMENT COST ITEMS
Excavation of subsoil 40 m3/hour
Placement of foundation 40 m3/hour
Equalizing foundation 35 euro/m2
Transport vessel with crawler crane 225 euro/hour
Pontoon to carry structure elements 25 euro/hour
Placement of caisson 2 units/hour
Connection of caisson 2 units/hour
Connecting structure segments 500 euro/hour

CLASS AND CONSTRUCTION COST
Caisson 1, 232 euro/m
Caisson 2, 401 euro/m
Caisson 3, 584 euro/m
Caisson 4, 770 euro/m
Caisson 5, 958 euro/m

APPENDIX I.4

MATERIAL COST ITEMS
Concrete with formwork 125 euro/m3
Rubble 32 euro/m3

CLASS AND MATERIAL COST
Block Wall 1, Block 1, 628 euro/m
Block Wall 1, Block 2, 1256 euro/m
Block Wall 1, Block 3, 1884 euro/m
Block Wall 2, Block 1, 1128 euro/m
Block Wall 2, Block 2, 2256 euro/m
Block Wall 2, Block 3, 3384 euro/m
Block Wall 3, Block 1, 1628 euro/m
Block Wall 3, Block 2, 3256 euro/m
Block Wall 3, Block 3, 4884 euro/m
Block Wall 4, Block 1, 2128 euro/m
Block Wall 4, Block 2, 4256 euro/m
Block Wall 4, Block 3, 6384 euro/m
Block Wall 5, Block 1, 2628 euro/m
Block Wall 5, Block 2, 5256 euro/m
Block Wall 5, Block 3, 7784 euro/m

APPENDIX I.5

LABOUR AND EQUIPMENT COST ITEMS
Excavation of subsoil 40 m3/hour
Placement of foundation 40 m3/hour
Equalizing foundation 35 euro/m2
Transport vessel with crawler crane 225 euro/hour
Pontoon to carry structure elements 25 euro/hour
Placement of blocks 6 units/hour

CLASS AND CONSTRUCTION COST
Block Wall 1, Block 1, 200 euro/m
Block Wall 1, Block 2, 285 euro/m
Block Wall 1, Block 3, 572 euro/m
Block Wall 2, Block 1, 401 euro/m
Block Wall 2, Block 2, 770 euro/m
Block Wall 2, Block 3, 1147 euro/m
Block Wall 3, Block 1, 601 euro/m
Block Wall 3, Block 2, 1156 euro/m
Block Wall 3, Block 3, 1720 euro/m
Block Wall 4, Block 1, 770 euro/m
Block Wall 4, Block 2, 1541 euro/m
Block Wall 4, Block 3, 2294 euro/m
Block Wall 5, Block 1, 960 euro/m
Block Wall 5, Block 2, 1926 euro/m
Block Wall 5, Block 3, 2867 euro/m

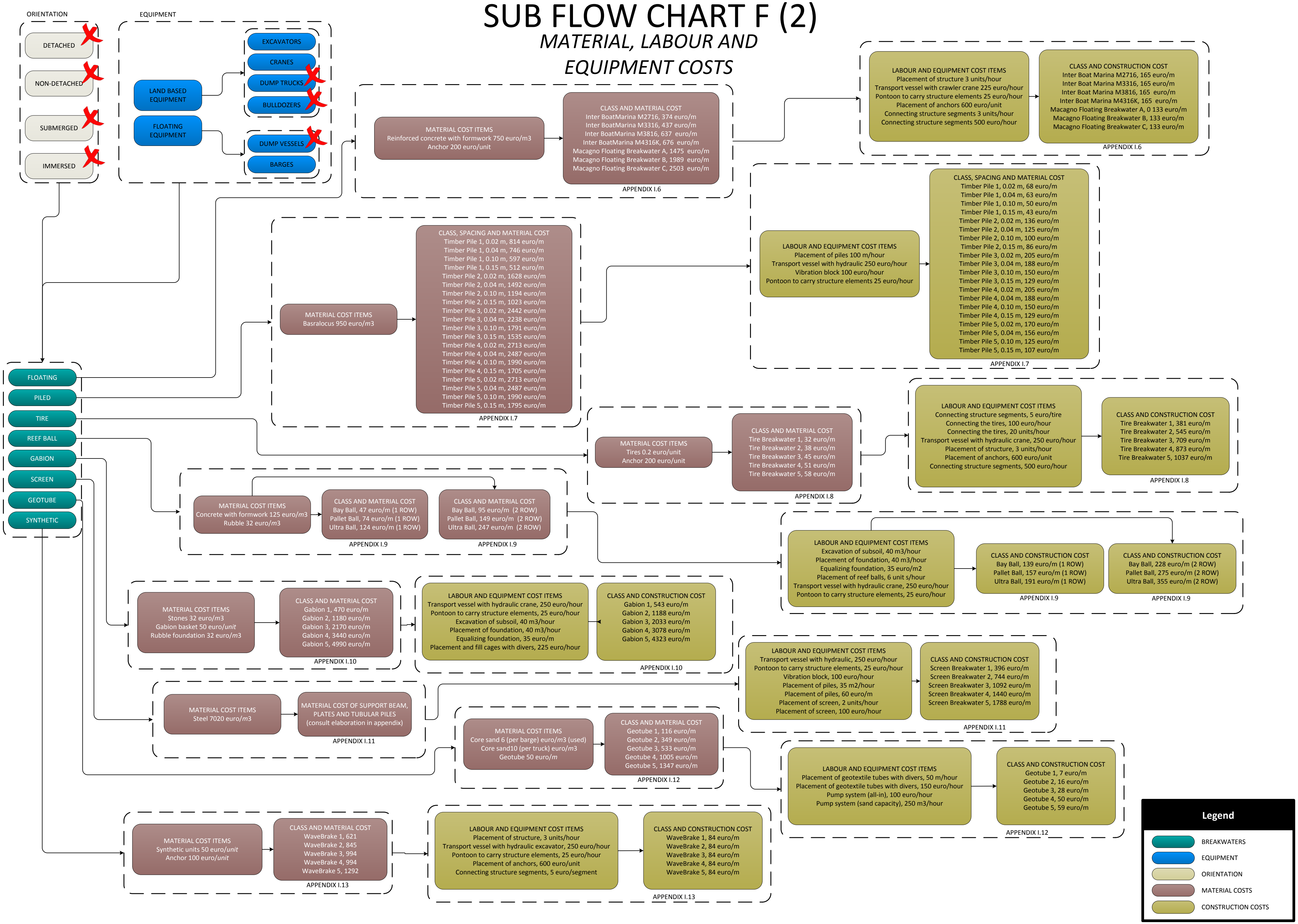
APPENDIX I.5

Legend

- BREAKWATERS
- EQUIPMENT
- ORIENTATION
- MATERIAL COSTS
- CONSTRUCTION COSTS

SUB FLOW CHART F (2)

MATERIAL, LABOUR AND EQUIPMENT COSTS



S

SUB FLOW CHART G: EMVI SCORE AND SELECTION MATRIX

In this appendix, the EMVI score per breakwater alternative is assumed in the black table. The blue table presents the pre-selection and cost-based selection matrix combined. The grey shapes can be exchanged with green, red, blue or yellow shapes to perform the pre-selection. And the cost items will be valued in monetary units per running meter.

SUB FLOW CHART G

EMVI SCORE AND SELECTION MATRIX

EMVI SCORE

	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
SYSTEM QUALITY	4	3	3	5	2	4	5	5	3	4	3	2	5
DURABILITY	5	4	4	4	4	4	3	5	4	5	4	4	5
INNOVATION	1	2	3	1	1	4	1	3	2	1	3	4	5
ECOLOGICAL IMPACT	4	3	3	2	2	4	2	1	4	4	2	2	4
CO2 AMBITION LEVEL	4	2	2	2	2	2	3	4	2	4	3	3	4
HINDRANCE	3	3	3	3	3	3	3	3	3	3	3	3	3
NOISE	3	3	1	3	3	3	3	3	3	3	3	3	4
RISKS	4	3	3	5	2	4	5	5	3	4	3	2	5

SCORE:
1 lowest, 20% discount
2 low, 40% discount
3 mean, 60% discount
4 high, 80% discount
5 highest. 100% discount

APPENDIX I.

PRE-SELECTION AND COST-BASED SELECTION

CODE	ITEM	RUBBLE MOUND	PLACED BLOCK	SHEET PILE	CAISSON	BLOCK WALL	FLOATING	TIMBER PILE	TIRE	REEF BALL	GABION	SCREEN	GEOTUBE	SYNTHETIC
I1	REQUIREMENTS													
I2	CONTRACT													
I3	LAWS & PERMITS													
I4	BOUNDARY CONDITIONS													
I5	FEASIBLE													
I6	HYDRAULIC FULFILMENT													
I7	COSTS OF MATERIAL (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I8	COSTS OF EQUIPMENT AND LABOUR (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I9	EMVI DISCOUNT (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0
I10	TOTAL (€/m)	0	0	0	0	0	0	0	0	0	0	0	0	0

Remarks:
I1 According to the requirement not feasible.
I2 According to the contract not advisable to choice.
I3 According to the laws and permits, not preferable.
I4 According to the boundary conditions, not feasible.
I5 According to I1, I2, I3 and I4 a possible alternative.
I6 Forcing conditions verification.
I7 Material cost estimate.
I8 Construction cost estimate.
I9 EMVI discount estimate.
I10 Summation of I7, I8 and I9.

SECTION 5.7 AND 5.16 (STEP 8 AND 15)

Legend

- POSSIBLE/PREFERABLE
- IMPOSSIBLE/NOT PREFERABLE
- LEGISLATION CONFLICT
- DEPENDS ON SITUATION