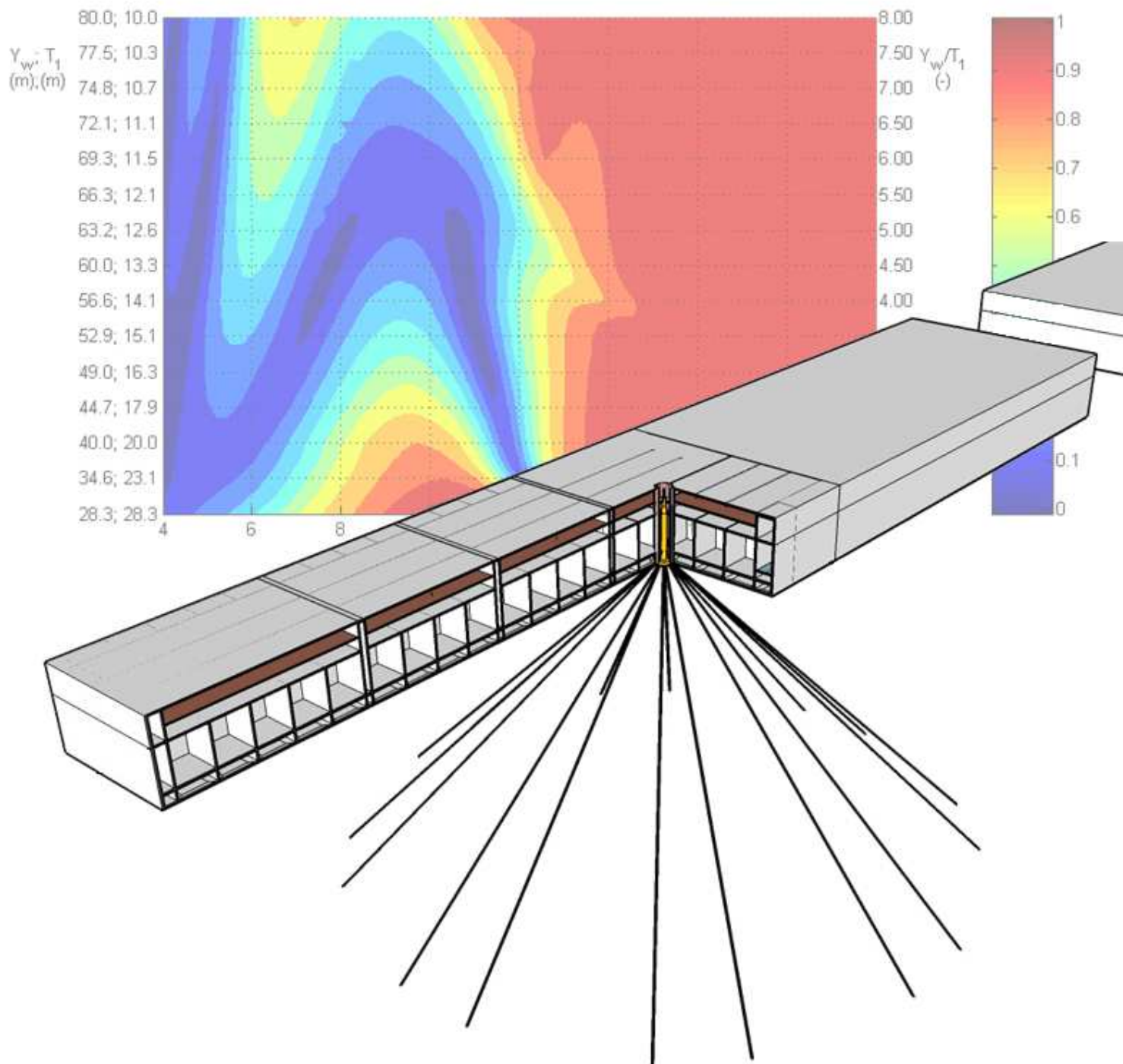


Floating breakwaters

A theoretical study and preliminary design of a dynamic wave attenuating system



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Colophon

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A theoretical study and preliminary design of a dynamic wave attenuating system

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Front cover: Wave transmission coefficients and impression of the final design

Abstract

Humans have been using ships to transport goods and people around the globe for hundreds and thousands of years. In all cases harbours have been critical for the exchange of goods and have played a significant role in the development of countries. Initially, harbours were located in sheltered natural bays; later breakwaters were used to provide shelter. Well-protected harbours act as safe havens for ships during storms, they have been strategic in battle and conquest and therefore were fought over during wars and additionally they are keystones in the economies of many countries.

Modern successful harbours must be deep enough and have enough space to accommodate and facilitate the largest container vessels. A Floating Transshipment Container Terminal (FTCT) is an interesting structure as it can be moored at strategic locations. This would eliminate the need for capital dredging and reducing the number of required harbour calls.

A floating breakwater is an essential element for large scale floating structures, like the FTCT, to become a success if its function is not integrated in the floating structure itself. Floating harbours, such as the Floating Transshipment Container Terminal, can be used more efficiently when the wave attack on these terminals can be reduced has been proved in previous reports [Ali, 2005].

The missing link in designing an effective floating breakwater is a design-tool to make quick preliminary designs which can be developed further. A good step has been made by [Fousert, 2006] by introducing the ReFBreak-model. On basis of the results of this model it was concluded that *"the performance of the floating breakwater is primarily determined by the draft-mass-width relation of the structure"* [Fousert, 2006]. This model should be adapted to overcome its limitations and inaccuracies and expanded to show the effects of the structure's dimensions in irregular wave-fields and the effects of changing wave conditions.

The main objective of this thesis is twofold:

- Determine the influence of the cross-sectional dimensions on the dynamic behaviour of the floating breakwater by adapting and expanding the ReFBreak-model;
- Create a conceptual, structural design of a floating breakwater element on basis of the forces that are expected, in order to prove that a floating breakwater is technically feasible;

In order to reach these objectives an improved calculation method and model is developed to overcome the problems and limitations of the original ReFBreak-model. The ReFBreak2D-model is based on the following assumption:

- Only beam-on regular waves are considered in order to find the transfer function of the optimal floating breakwater;
- Irregular waves are seen as a superposition of regular waves;
- The energy a wave contains below the keel of the element is assumed to pass undisturbed;
- The influence of wind is neglected
- Energy losses due to viscous damping is neglected or user defined;
- Mooring influence on first order motions are neglected or user defined.

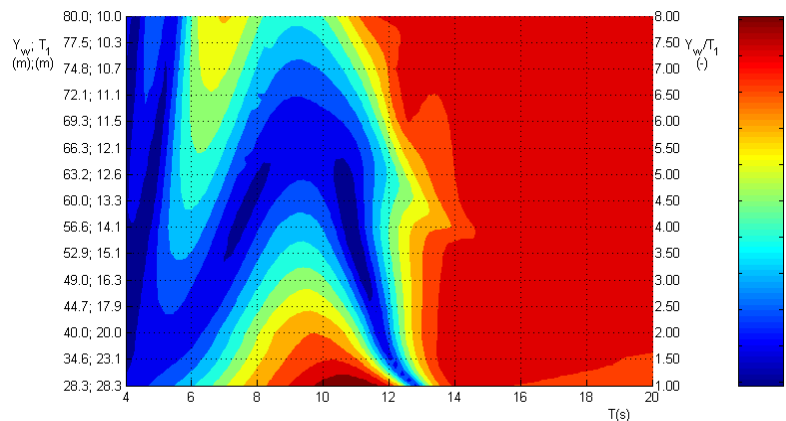


Figure 0.1 The effect of the beam-to-width ratio on the wave transmission amplification factor (RAO), the centre of gravity (COG) is located as high as technical feasibility or stability permits

The ReFBreak2D-model has been used to assess the performance of various floating breakwater designs, see figure 0.1 for some output. The hydrodynamic behaviour as calculated by the model has been validated with DELFRAC, a three dimensional potential model to determine reliability of the calculations. The results of the ReFBreak2D-model correspond well to the output of the DELFRAC simulation.

The most important outcomes from the ReFBreak2D-model are:

- The performance of the FB is primarily determined by the beam/draught ratio (Y_w/T_1 -ratio) and the location of the centre of gravity (COG).
- A rectangular floating breakwater with an Y_w/T_1 -ratio of 5.0 and a high COG is the most optimal design.
- A system of two floating breakwaters fulfils the requirements. The first breakwater, with a beam of 100 metres and a draught of 20 metres, is able to attenuate waves with wave periods up to 14 seconds, but it lacks in performance on attenuating waves with a period from 6 to 10 seconds. The second breakwater, with a beam of 40 metre and a draught of 8 metres, is able to attenuate waves with wave periods of 6 to 10 seconds.

These outcomes have been used to develop a preliminary design of the largest element, see figure 0.2. The length of the element is 500 metres, the beam is 100 metres and draught is 20 metres. This leads to a displacement of 1,000,000 tonnes, of which 270,000 tonnes are ballast. The element is moored using a turret mooring system; this makes it possible to weathervane during storm conditions.

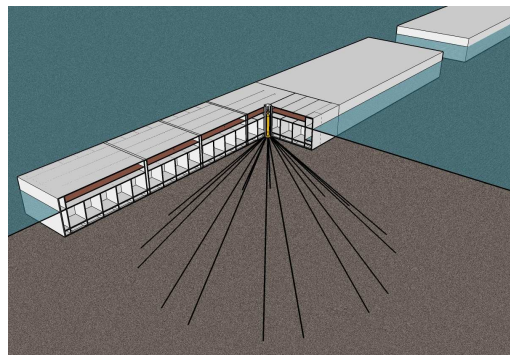


Figure 0.2 Preliminary design of a floating breakwater element

Since the results of this research are based on a purely theoretical model, further research is necessary to study the influence of oblique waves and other three-dimensional effects on the performance of the FB. Also the behaviour of the whole FB-ship-FTCT-system has to be analyzed to determine the exact requirements.

Preface

The research presented in this report has been performed as a final thesis for the Masters program at the TU Delft, faculty of Civil Engineering and Geosciences. The research has been performed under authority of the section of hydraulic engineering and was carried out from July 2007 until April 2008.

This research on the topic of Floating Breakwaters is based on the research of Menno Fousert, who worked on this subject earlier.

The results of the master thesis are presented in this report and can be the basis for further research on the technical and economical feasibility. The developed model can be used as a preliminary design tool to quickly determine the required dimensions in the development of full-scale floating breakwaters. The results of the model have been used for a preliminary design of a floating breakwater element as a proof of the technical feasibility.

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List of symbols

Roman

a_{mn}	: Added mass coefficient	[kg]
b_{mn}	: Hydrodynamic damping coefficient	[kg/s]
c_{mn}	: Hydrodynamic spring coefficient	[kg/s ²]
c_g	: Deep water group velocity	[m/s]
k	: Wave number	[rad/m]
n	: Number of waves underneath the structure	[-]
$x_{n,a}$	: Motion amplitude of the n th motion	[m]
$\overline{BG}$	: Distance between centres of buoyancy and gravity	[m]
C	: Spring stiffness matrix	[kg/s ²]
F_0	: Complex amplitude force matrix	[kgm/s ²]
$F_{w,n}$	: Wave exciting force amplitude for the n th motion	[kgm/s ²]
L_w	: Length of a floating breakwater element	[m]
$\overline{OG}$	: Distance between origin and centre of gravity	[m]
M	: Mass matrix	[kg]
N	: Damping matrix	[kg/s]
P_d	: Power dissipated by damping	[kg
$m^2/s^3]$		
P_d	: Power dissipated by radiated waves	[kg
$m^2/s^3]$		
Q	: Combined complex matrix, defined as $-M\omega^2 + Ni\omega + C$	[kg/s ²]
S_ζ	: Incoming wave spectrum	[m ² /s]
$S_{T,\zeta}$	: Transmitted wave spectrum	[m ² /s]
S_z	: Motion spectrum	[m ² /s]
T	: Wave period	[m]
T_1	: Draught of a floating breakwater element	[m]
Y_w	: Width of a floating breakwater element	[m]

Greek

$\varepsilon_{w,n}$	: Phase change of the wave exciting force relative to the COG	[rad]
$\varepsilon_{n,\zeta}$	: Radiated wave phase change for the n^{th} motion	[rad]
$\varepsilon_{tot,\zeta}$	: Radiated wave phase change	[rad]
λ	: Wave length	[m]
ω	: Circular wave frequency	[rad/s]
ρ	: Specific density of water	[kg/m ³]
ζ_a	: Wave amplitude	[m]
ζ_d	: Viscous damping ratio	[-]
$\zeta_{r,n}$	: Radiated wave amplitude for the n^{th} motion	[m]
$\zeta_{r,tot,a}$	: Transmitted wave amplitude	[m]

Chapter 1 Introduction

Humans have been using ships to transport goods and people around the globe for hundreds and thousands of years. In all cases harbours have been critical for the exchange of goods and have played a significant role in the development of countries. Initially, harbours were located in sheltered natural bays; later breakwaters were used to provide shelter. Well-protected harbours act as safe havens for ships during storms, they have been strategic in battle and conquest and therefore were fought over during wars and additionally they are keystones in the economies of many countries.

Since the first idea of a floating breakwater, documented 200 years ago, for the protection of the British fleet in Plymouth, many studies were performed to develop an effective floating breakwater. The research on this topic became a professional area of study soon after WWII, in which the first floating breakwater was used after the attacks on the harbours of Dunkirk and Dieppe. These attacks showed that in overtaking a harbour it was nearly impossible to gain control without damaging the existing harbour and therefore the need for floating breakwaters became essential.

In recent times, the driving force behind the construction, design and understanding of modern large floating breakwaters (FBs) is the ever-growing transport of goods around the globe in combination with an increase in the size of the merchant (container) vessels. Modern successful harbours must be deep enough and have enough space to accommodate and facilitate the largest container vessels. A Floating Transshipment Container Terminal (FTCT) is an interesting structure as it can be moored in strategic locations, eliminating the need for capital dredging and reducing the number of required harbour calls.

Previous studies have shown that floating harbours, like normal land-based harbours, are affected most significantly by attack from waves. A breakwater therefore is necessary to increase the efficiency of floating harbours, and to create a safe haven for ships.

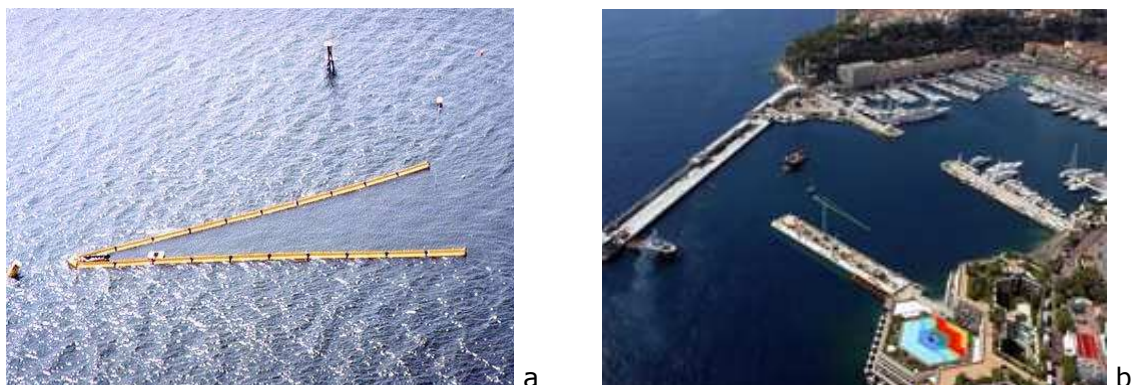


Figure 1.1 A small scale test of the RIBS (a) and the Monaco Floating breakwater (b)

Although there have been many studies performed to assess the performance of various designs of floating breakwaters (FBs), none of these studies has resulted in the development of a floating breakwater that meets all necessary requirements. Despite this, floating breakwaters have been built, but these are all designed for attenuating waves in a moderate wave climate, see also figure 1.1.

A FB has a complex dynamic behaviour and has to mitigate between the boundary conditions imposed by the ocean on one side, and the requirements of the floating

harbour or FTCT on the other side. Many structural and hydraulic factors influence the hydrodynamic behaviour of FBs, and it is this behaviour that influences wave attenuation performance. Determining the relationship between the behaviour of floating breakwaters and the influence of this behaviour on performance is one of the main objectives of this thesis.

Using the knowledge of the hydrodynamic behaviour of floating breakwaters, a model can be developed to determine the required dimensions of the FB element. Based on the results of the model a preliminary design can be made which fulfils the FB requirements, which covers varying dimensions, strength, floatability, and constructability. This design of the FB element is required to determine whether the FB is not only theoretically feasible, but also technically feasible and is the second main objective of this thesis.

The final result of this thesis will include a preliminary design for a floating breakwater. Further work on the subject should focus on a more detailed assessment of the design and required model testing.

The structure of this report is as follows:

- Chapter 2: An overview will be given about the different types of breakwaters, both conventional and unconventional. The different types of floating breakwaters are discussed in this chapter as well.
- Chapter 3: This chapter will give an introduction on the past performance of FBs and an overview of the research that has been conducted in relation to this topic. In addition, potential applications for floating breakwaters in practice will be addressed.
- Chapter 4: The problem definition and objectives of this thesis are presented in this chapter.
- Chapter 5: The relationship between the set boundary conditions and the requirements for an effective floating breakwater are discussed here.
- Chapter 6: In this chapter the ReFBreak-model developed by [Fousert, 2006] is discussed and the ReFBreak2D-model is presented. These tools are used to overcome the problems and limitations of the original ReFBreak-model.
- Chapter 7: An evaluation of the resulting calculations as well as a parameter analysis is the subject of this chapter.
- Chapter 8: This chapter presents a survey of alternative design characteristics and leads to the election of one optimal design which will be elaborated on further in subsequent chapters.
- Chapter 9: The chosen element is designed with regard to the requirements on strength, stability, floatability and constructability. The designed element is presented at the end of this chapter.
- Chapter 10: Finally, in chapter 10 conclusions will be drawn and recommendations on further research will be made.

Chapter 2 Types of breakwaters

Before the subject of breakwaters is dealt with, the question what is a breakwater must be answered. Breakwaters are designed to protect ships in ports as well as port facilities against waves. The way the ships and the port facilities are being protected depends on the type of breakwater that is used. Breakwaters can be divided in two groups, the fixed or conventional breakwaters and the floating breakwaters. Each of these breakwaters is characterised by the method by which it absorbs or reflects wave energy. Before dealing with the subject of floating breakwaters in detail the two groups will be discussed. Special breakwaters, such as the dynamic breakwater [Spek, 2001], the pneumatic breakwater and hydraulic breakwater are left out of consideration because they are out of the scope of this study.

2.1 Conventional breakwaters

Fixed breakwaters work on the basis of wave energy reflection and are bottom founded. There are three types of fixed breakwaters, the rubble mound breakwater, the caisson breakwater or monolithic breakwater, and the composite breakwater, which is actually a combination of a rubble mound sill on which a caisson is placed. The three different types of breakwaters are shown in figure 2.1. [d'Angremond and van Roode, 2001]

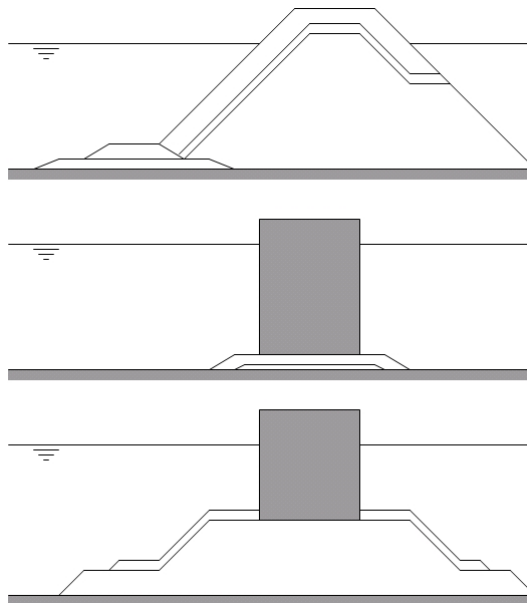


Figure 2.1 Different types of fixed breakwaters, from above to below, the rubble mound, the caisson and the composite breakwater

Due to the wave reflection, which occurs especially with the caisson breakwater, the water in front of the breakwater can become extremely rough. This may have a negative impact on shipping. Fully reflecting breakwaters, such as the caisson and composite breakwaters have to resist the largest loads.

In addition to the main function of protecting ships and port facilities against waves, the fixed breakwater also protects ships against cross-currents.

2.2 Floating breakwater

Floating breakwaters work on the basis of interference to either cancel out wave trains or by using turbulence and friction to remove energy from waves. A combination of both processes is also possible. [Bowley, 1974]

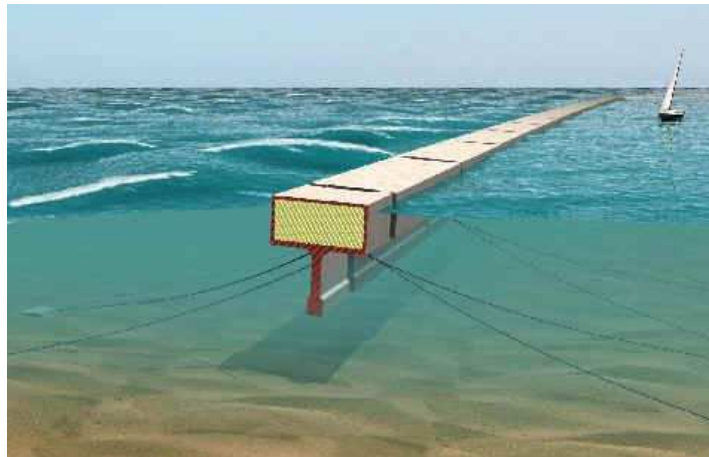


Figure 2.2 An example of a floating breakwater¹

There are two ways to classify floating breakwaters:

1. the wave energy dissipating mechanism [McLaren, 1981];
2. the basic shapes of the breakwaters [Tolba, 1999].

These two ways overlap each other partly, i.e. the wave attenuation is determined by the shape of the breakwater. The list below is based on the wave energy dissipating mechanisms and the shapes are seen as subdivisions.

2.2.1 Reflecting floating breakwaters

This type of breakwater reflects the wave energy by using a thin vertical wall supported by a floating A-frame. The amount of energy that is reflected depends on the draft of the vertical screen and on the breakwaters movements in the lateral direction. The anchor forces can be minimised by designing the structure in such a way that the roll motion compensates for the sway motion.

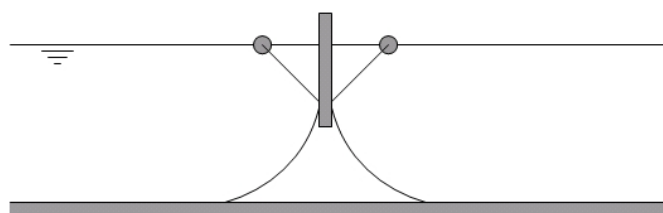


Figure 2.3 An A-frame breakwater

2.2.2 Displacement floating breakwaters

Displacement floating breakwaters attenuate waves by absorbing the wave energy and retransmitting it with a phase-change. Ideally, the retransmitted wave energy should cancel out the wave energy that passes underneath the structure. Parameters involved in reducing the height of the waves are the mass and the radius of gyration, which gives the structure a moment of inertia. This type of floating breakwater can be tuned for an optimal performance when facing regular waves. [Fousert, 2006]

¹ <http://www.noort-innovations.nl/>

The simplest form of a displacement floating breakwater is the single prism type, see figure 2.4, which also offers the best possibilities for multiple purposes. In a previous study [Kato, Noma et al., 1969] an inverse trapezoid, a trapezoid, a rectangular and a triangular cross-section were compared. The inverse trapezoid gave the lowest transmission coefficients, possibly leading to a more effective design. However the triangular cross-section gave also higher anchorage forces, which may limit the usability. [Tolba, 1999]

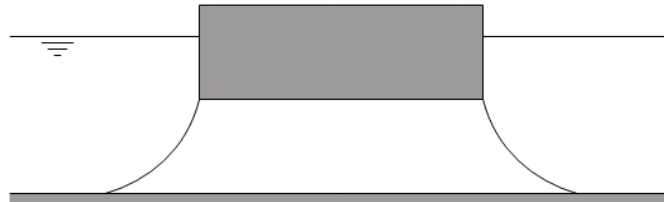


Figure 2.4 A single prism breakwater

To achieve a higher roll period and thus possibly a more stable platform a catamaran type of breakwater can be used, see figure 2.5. By using a catamaran a larger roll period can be achieved by using the same amount of mass as a single prism. The extra corners provide energy losses, while the water trapped between the two pontoons may add to the hydraulic added damping, especially for the sway motion. The more stable platform of a catamaran breakwater might enable the breakwater to be used for multiple purposes.

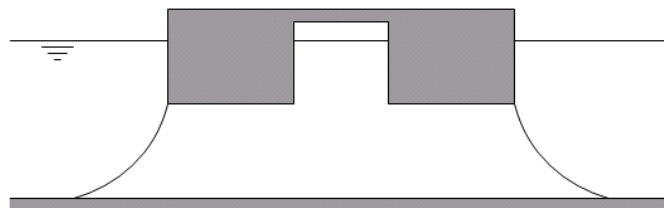


Figure 2.5 A catamaran breakwater

2.2.3 Dissipative floating breakwaters

This type of breakwater relies on dissipating the energy into turbulence rather than absorption, retransmission or reflection. Flexible floating breakwaters are of the dissipative type. They can consist of bags filled with water or air or mats consisting of old tires. Rafts, possibly made of logs, can also be placed in this group.

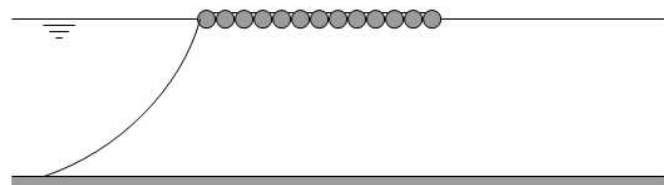


Figure 2.6 A flexible floating breakwater

2.3 Scope of the thesis

This chapter illustrated that there are numerous solutions for breakwaters. Even within the category floating breakwaters there are many alternatives. Each type has its own processes that govern the wave attenuation and various parameters which can be varied to obtain an optimal design. Therefore it is necessary to limit the scope of this master thesis.

In this master thesis a rectangular floating breakwater, which can be categorised as a displacement breakwater will be examined in greater detail. The rectangular floating breakwater was also investigated by [Fousert, 2006]. This report can be seen as an extension to his original research.

Chapter 3

Past performance and research

Since the first idea of a floating breakwater, first documented 200 years ago for protection of the British fleet in Plymouth, many studies were performed to develop an effective floating breakwater. The research on this topic professionalized soon after WWII, in which the first floating breakwater was used. Many designs were tested and checked with numerical simulations. In this chapter first the (dis-)advantages of a floating breakwater (FB) are discussed to determine why and when a floating breakwater can be advantageous over a conventional breakwater. A short review will be given of the FB's that have been built and the possibilities for the future use of them. Finally past (model) testing will be discussed in order to describe the outcomes and problems that have been encountered in the past.

3.1 Why floating breakwaters?

The main reason to apply unconventional types of breakwaters, and floating breakwaters in particular, are the technical and economic restrictions related to the conventional types of breakwater. Besides the restrictions on the conventional types there also other arguments that attract the use and development of a FB solution.

Although the advantages can be distinguished quite easily, some of the disadvantages seem very obvious as well. Performance, costs and human confidence are the primary counter arguments that can be defined. The costs are closely related to the maintenance of the breakwater and the human confidence is lacking due to the progressive design and the failure of some of the applied floating breakwaters in the past, for example in the second World War.

Between the pro-arguments and the counter-arguments there is a tension field in which a solution has to be sought, see also figure 3.1. The optimal solution should mediate between the two sides to have as many advantages combined with the fewest disadvantages.

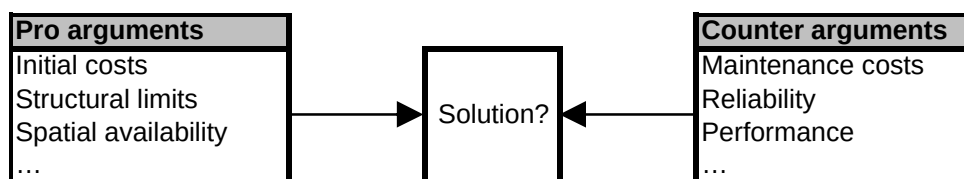


Figure 3.1 Tension field between pro- and counter-arguments, adapted from [Fousert, 2006]

3.1.1 Economic consideration

Both the advantages and disadvantages of the use of floating breakwaters, as was already shown in figure 3.1, have a common origin: economics. Maintenance costs are mentioned as a disadvantage, the independence of the realisation cost to the water depth is a big advantage in favour of the floating breakwater when deep water, i.e. deeper than 150 metres, is considered. In his study, [Fousert, 2006] made a simple calculation to determine the optimal breakwater design for several water depths to justify these assumptions.

Figure 3.2 shows the relation between the water depth and the construction costs found by [Fousert, 2006]. In previous reports [d'Angremond and Tutuarima, 1998] it was already stated that, from an economic point of view:

- a conventional, rubble mount, breakwater is only preferable in water depths up to 8 metres;
- in water depths from 8 to 20 metres a caisson breakwater is the most economic solution;
- For water depths of 20 metres up to water depth of up to 30 metres the composite type of breakwater is preferred.

The estimated costs of a floating breakwater are unknown at this stage, as it is the first design cycle. Since the construction cost of the FB will hardly increase with the water depth the line in figure 3.2 is almost horizontal. Depending on the costs the floating breakwater will become the most economical solution in water depths between 30 and 100 metres.

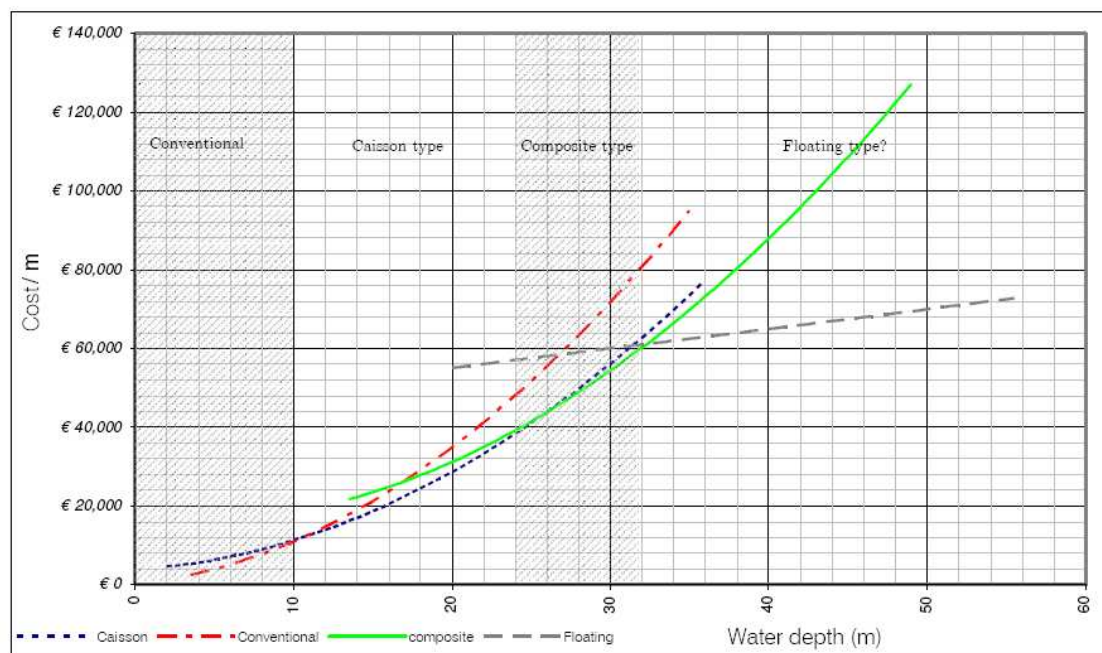


Figure 3.2 Comparison of construction cost per running metre depending on the water depth [Fousert, 2006]

In real practise, the site conditions may have a large influence on the total construction costs. With increasing water depth the sheer weight of a conventional breakwater increases exponentially. The stability of the breakwater depends on the quality of the subsoil. As the gravitational load of a floating breakwater is carried by the water instead of the subsoil a floating breakwater is also feasible in case of poor soil conditions in combination with a great depth. [Ofuya, 1968]

3.1.2 Technical consideration

The technical aspects of the floating breakwater can be divided into four parts:

- the construction;
- the performance;
- the flexibility
- and the failure mechanism.

As has already been described in the previous section the local soil conditions will limit the design of a conventional type of breakwater. A floating breakwater can be

more feasible in poor soil conditions than a heavy conventional breakwater because it does not generate any (vertical) loads on the subsoil. However, the FB does have to be anchored to the sea bottom in some way.

The performance of a floating breakwater is limited to a certain band of frequencies. Within in this band the wave-attenuating properties are good, but the performance will quickly deteriorate when it is loaded by waves with other frequencies. Of course the location of this band and the width depends on a range of factors such as the mass of the structure, mooring line stiffness, etc.

A fixed breakwater, occupying the full depth of the water, can act as a partial dam intersecting littoral drift and hence trap sediment. While silting can occur, scour can affect the base of the fixed breakwater. This problem becomes irrelevant in the application of floating breakwater. [Ofuya, 1968]

Floating breakwaters can be moved and rearranged relatively easily during their lifetimes due to their transportability, making it possible for a floating harbour to be adapted and rearranged as needed. FB's give the opportunity to load and unload ships offshore, being independent of land-based harbours, which can suffer from spatial problems or maintaining the depth of the navigational channels. [Briggs, Ye et al., 2002]

The way the station keeping, or mooring, fails must also be taken into consideration. A damaged bottom-founded structure, such as a conventional breakwater, will show a certain failure warning and will not be suddenly destroyed. During a severe storm the mooring of a FB is susceptible to sudden structural failure. When the mooring fails the dislodged element can become a threat for the harbour facilities.

3.2 Past performance

Already since the beginning of the 19th century people have been trying to develop and build successful floating breakwaters. This section will discuss the designs of the floating breakwater from 1811 until now and is divided into three sections: floating breakwaters before 1900, during WWII and modern floating breakwaters.

3.2.1 Before 1900

The first documented proposal for a floating breakwater dates back to 1811, where the breakwater was intended to protect the British fleet at Plymouth [Readshaw, 1981]. The design was proposed as an alternative to a rubble mound breakwater and the estimated costs were estimated at 10% of the final cost of the rubble mound. The alternative was rejected by the reasoning that if a floating structure could be designed to ride the waves, then a ship could do it as well.

Though it was admitted that the breakwater would have an effect on the waves, they feared that when the protection was really needed the most probable cause was that the breakwater itself would have driven from its moorings and be wrecked. Even in this early stage of the development of the floating breakwater two major disadvantages were already pointed out, the problem of mooring and of public opinion.

Later in the 19 century there were various proposals for floating breakwaters, but there are no records whether they were ever built. More disadvantages of the floating breakwater were discovered, ranging from the materials used to the required maintenance and the related costs.

3.2.2 Second World War

During WWII the development of floating breakwaters was given a new impulse as the Allies needed a harbour for supplying their troops after D-Day, better known as operation Overlord. After a retreat from the beaches of Dunkirk in 1940 and a disastrous raid on the harbour of Dieppe in 1942 it became clear that capturing a harbour was difficult if they were not destroyed shortly after a landing. [Ruppenthal, 1953]

The idea was to pre-fabricate two floating harbour, codenamed Mulberry A and B, and tow them to the shore near Normandy and supply the troops from these harbours until newly captured harbours were restored. A Mulberry harbour consisted of floating piers, sheltered by so-called Phoenix-caissons and blockships which were sunk in place to form a bottom founded breakwater. This breakwater was again sheltered by the floating "Bombardon" breakwater, a crucifix-shaped pontoon.

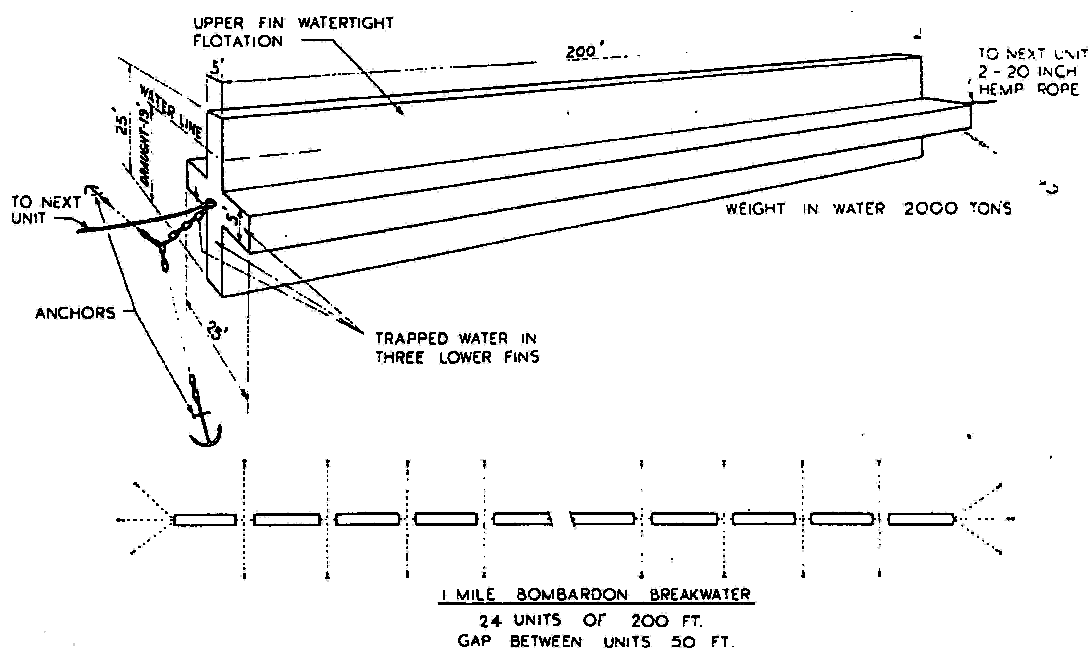


Figure 3.3 The "Bombardon" floating breakwater¹

The Bombardon had overall dimensions of 8x8 metres, with the thickness of the arms 1,5 metres and a length of around 60 metres. The elements were anchored at both ends and they were joined with a manila rope of 0.5 metre thickness, see figure 3.3. During tests [Readshaw, 1981] the breakwater was successful in attenuating waves with a period of 5.8 seconds from a height of 2.6 metres to a height of 0.6 metres, while during their operation in Normandy wave reductions of 50% were measured.

During their service at the Normandy beach they failed after 10 days in, what was said, the worst storm in 40 years. The waves exceeded 4.5 metres with a wave period of 7.7 seconds. The subsequent loads on structure were estimated 8 times in excess of the design load. According to [Readshaw, 1981] the following factors contributed to the failure of the Bombardon floating breakwater:

- The steel plates were bolted together and gave the connection the ability to move. After inspection it is suggested that this was the initial failure.

¹ Mulberries and Gooseberries
http://www.hksw.org/despatches_107_1_a.htm

- The connection between the Bombardon units failed due to the initial failure and subsequent failures.
- The elements were designed to be moored in 12.8 metre of water with a tidal range of 7.3 metre. The actual depth of the location was 12.3 metre. The mooring lines did not enough length to compensate for the larger depth. This in combination with the light anchorage (because of the rapid deployment light anchors were used) caused the elements to drift

3.2.3 Modern applications

Three modern breakwaters will be assessed in this section, which are

- Rapidly Installable Breakwater System (RIBS);
- The U-block system;
- The Monaco floating breakwater.

These three breakwaters are all of the displacement type, as is described in chapter 2, just as the FB proposed and modelled by [Fousert, 2006]. The size, mobility, durability and performance of these breakwaters are different for each of these alternatives. Other types of modern floating breakwaters like the tire breakwater and submerged breakwaters are left out of consideration here because there are out of scope for this research.

The Rapid Installable Breakwater System², or RIBS, consists of a V-shaped breakwater, with the tip orientated in the direction of the waves. In the lee created between the two legs and behind the breakwater ships can (un-) load onto smaller ships during a military operation. The draft of the breakwater can be adapted to the specific wave conditions at the site.



Figure 3.4 A small scale test of the RIBS

The wave transmission of the RIBS is 30-50%. The V-shape of the breakwater, with waves obliquely incoming at the legs, is designed to spread and reflect the waves and to minimise the anchor forces. The structure has been designed to deflect waves and not to absorb them.

The U-block system³, developed by FDN Civil Engineering, consists of a concrete, rectangular float filled with polystyrene. The concrete is pre-stressed to allow tensile forces in the pontoon. The U-block floating breakwater is used in marinas and it is already in use in the Mediterranean Sea.

² CHL – The RIB System Concept
<http://chl.erd.c.usace.army.mil/chl.aspx?p=s&a=ARTICLES;149>

³ FDN – Floating breakwater
<http://www.fdn-engineering.nl/>

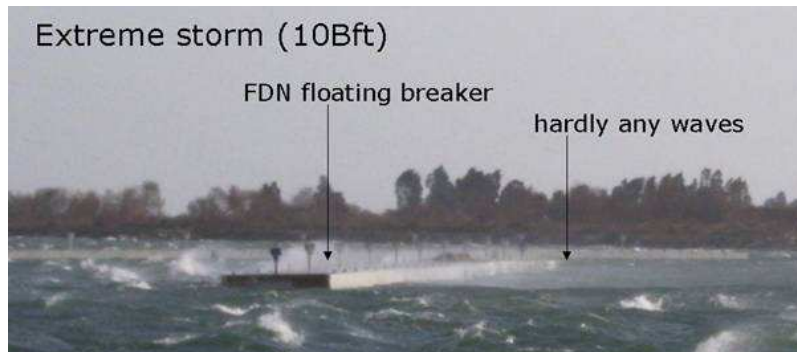


Figure 3.5 The U-block system during a severe storm

The U-block is efficient for waves up to two metres and with a period of 6 seconds. Depending on the wave-height a wave attenuation of 80-90% can be achieved during storm conditions.

The Monaco breakwater⁴, see figure 3.6, is a 352 metre floating breakwater located in the harbour of Monaco with a total weight of 160.000 tons. It is a multi-purpose breakwater as it also houses a parking garage, mooring facilities on the inside, and 25.000 m² of free space, divided in two levels.

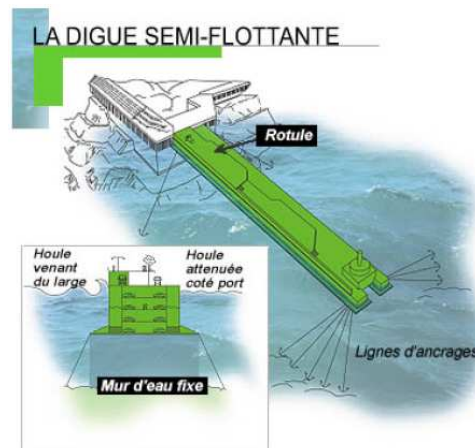


Figure 3.6 The Monaco floating breakwater

The option for a floating breakwater has been chosen because of the combination of great water depth and poor soil conditions. The design life time of the breakwater is 100 years.

The structure has been constructed in Algeria and it has been towed in four weeks from the building dock to Monaco. Although this breakwater is movable it cannot be said to be highly mobile. At the final location the breakwater is moored by a ball joint at the land side and with pre-stressed anchors at the sea side.

The transmission coefficient of the structure is only 20%. Due to the large weight of the structure the waves cannot move the breakwater; the waves are thus attenuated by reflection and not by cancellation with anti-waves.

An overview of the main characteristics of the structures mentioned above is given in table 3-1 on the next page.

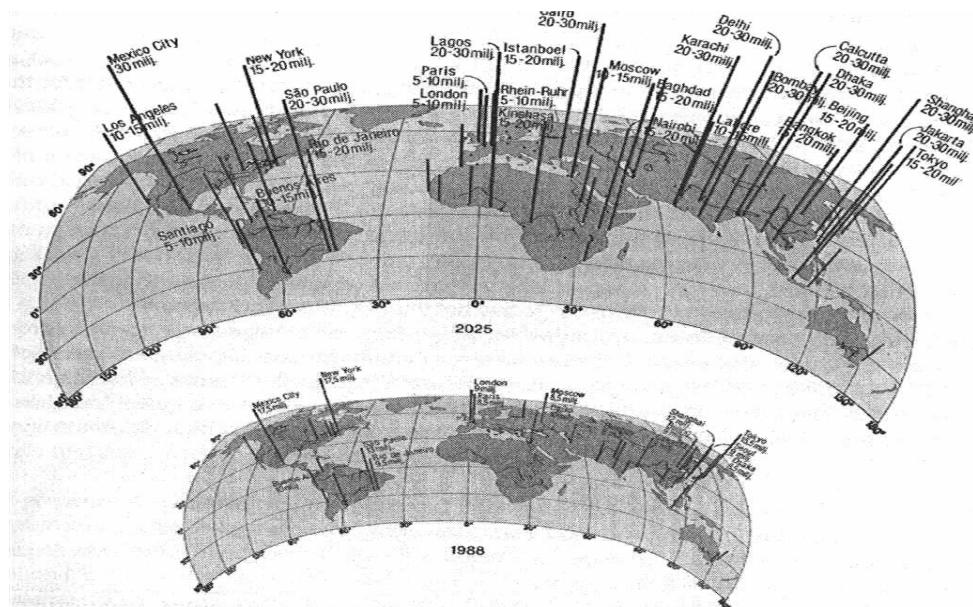
⁴ La digue semi-flottante
<http://www.gouv.mc/devwww/wwwnew.nsf>

Table 3-1 Comparison of floating breakwater designs [Fousert, 2006]

Concept	Monaco Floating Breakwater	Ublock	RIBS
Costs	--	+	++
Flexibility	--	+	-
Construction time	-	+	++
Spectrum width	++	-	--
Durability	+	+	-
Transportability	+/-	+/-	++
Reusability	-	+	++
Designer	Doris Engineering, Paris	FDN Civil Engineering	US Army Engineer Research and Development Center
Materials	Reinforced concrete	Reinforced concrete with core of polystyrene	Steel frame with flexible membranes
Design life	100 years	30 years	Temporary
Length	352 meter	25 meter (each element)	77 m (each leg)
Width	28 meter	4,50 meter	2,40 meter
Draught	16 meter	7,90 meter	4,00 meter
Weight	454 tons/m	5,40 tons/meter	
Station keeping	Ball-and-socket joint Taut mooring lines	Catenary mooring lines	3-point catenary mooring lines
Design wave height		2,00 meter	0,50 meter up to 1,50 meter
Application area	Cruise ship terminal	Small-scale marinas	Military landing operations

3.3 Possible future applications

It is expected that around 80% of the world population <citation needed> will live in the coastal areas in the near future, see figure 3.7. This growth of the world population in the coastal areas will put on growing demand on the scarce space and will eventually compete with other functions of this area. Another function of this area is transportation, in the form of harbours and airports.

**Figure 3.7** World population in 1986 and expected world population in 2025 [d'Angremond, 2001]

3.3.1 Floating harbours

In his literature research, [Ali, 2005] noticed that:

"... in marine world, the increasing cargo traffic makes the expansion of the existing ports imminent, especially the containerized cargo. [...] Forecasts of container traffic predict a yearly growth of 5-10% for the coming years. This growth demands the establishment of new container terminals.

Moreover, the transporting vessels witnessed a continuous increase in their sizes. The introduction of the mega vessels created the need for deep water terminals that could accommodate these vessels. Other issue related to these large vessels, is that the shipping lines are trying to increase their utilization and shorten the turnaround time by reducing the number of port calls along the shipping route.

The increase in coastal activities and the need for new ports which are able to accommodate mega vessel, makes it necessary to innovate new solutions that cope with developments in the marine world."

Also in the report of [Fousert, 2006] the introduction of the more efficient mega-vessels, see figure 3.8, is considered as a call for deep-sea container terminals where the containers will be offloaded onto smaller feeder vessels.

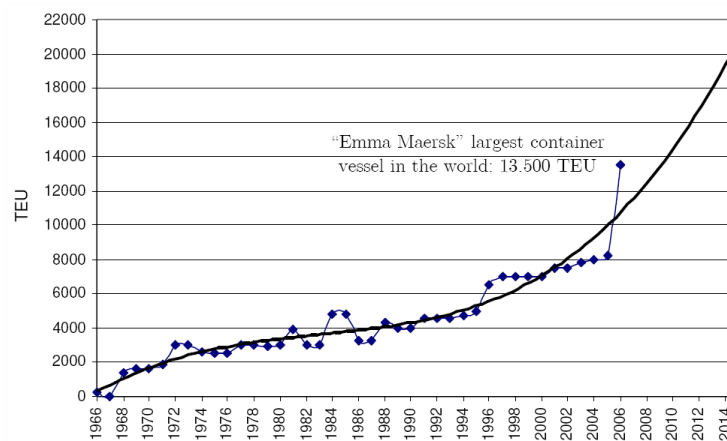


Figure 3.8 Maximum ship size by year of construction (until October 2006) [Fousert, 2006]

As a solution [Ali, 2005] proposed an Floating Transshipment Container Terminal, or FTCT, where mega-vessels could be offloaded offshore onto smaller feeder vessels which will take the containers on the last leg of their route to Europe. The FTCT is designed to act like a harbour and breakwater in one by receiving the waves beam on, with the berths located on the lee-side of the terminal.

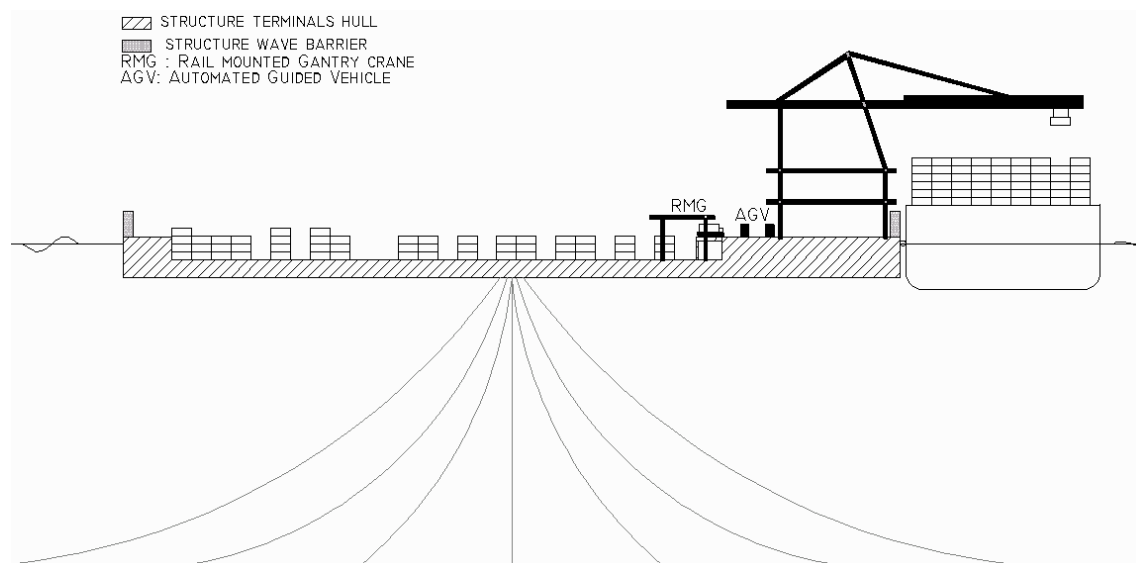


Figure 3.9 The Floating Transshipment Container Terminal [Ali, 2005]

Handling container in a harbour is not possible when the relative motions get too large. To guarantee optimal harbour efficiency, the differential motions of the harbour and vessel may not exceed a certain maximum. This is the reason why [Ali,

2005] located the berths on the lee-side of the breakwater. A floating breakwater may add to the efficiency by reducing the wave heights in the harbour area on the one hand and making berths on both side of the terminal on the other.

3.3.2 Floating airports

Large land-based airports are often located near large metropolises and are a source of pollution and sound hindrance due to departing and landing aircraft. Another important issue in the location of airports is safety, both for the neighbouring citizens for and for the security of the airport and the airplanes.

To overcome the nuisances of an airport as well as the security issues some airports have been moved to an artificial island of the coast, such as the Hong Kong International Airport. For others there are plans to expand the existing airports to an island, such as Schiphol Airport. Both airports are shown in figure 3.10.

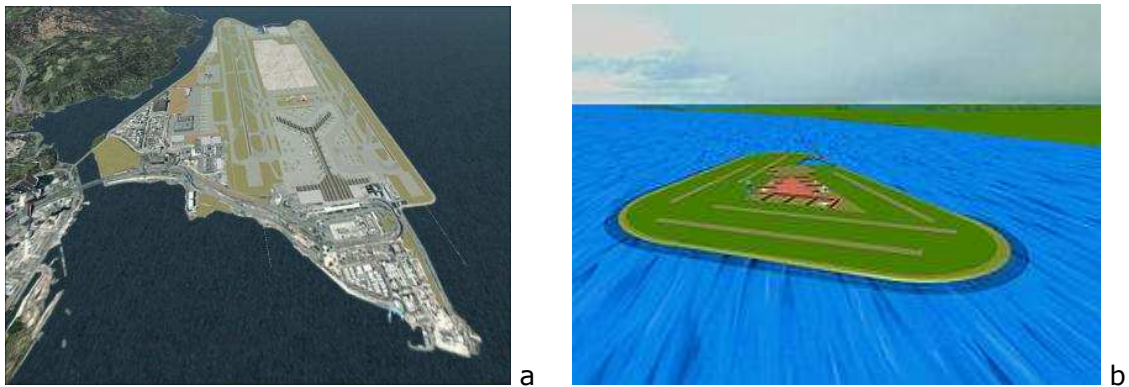


Figure 3.10 Hong Kong international airport (left) and proposed island for Schiphol Airport (right)

The solution of an artificial island is limited to sites with shallow water, i.e. the water depth should not be much larger than 20 metres. Also due to the construction method, the sand body of the island may be subject to liquefaction when it is struck by an earthquake. This means that an artificial island cannot be applied in all cases.

To overcome the problems with an artificial island, a group comprised of 17 shipbuilding and steel companies from Japan developed the Mega-Float concept, see figure 3.11⁵.



Figure 3.11 The Mega-Float Phase 2

⁵ Mega-Float concept:
http://www.srcj.or.jp/html/megafloat_en/index.html

As one can imagine even a structure of this magnitude will have a service limit state, just as the FTCT as has been described in the previous section. In Japan the Mega-Float is sheltered by its location in a bay. If the floating airport is to be used in an open ocean area a floating breakwater may become of use to protect it from the waves, expanding the limits of the structure and decreasing the downtime.

3.4 Performed research

A lot of research has been conducted on the hydrodynamic behaviour of the floating breakwaters since they became more popular as a solution in the 1970's. The focus of all research is to obtain transmission coefficients that are as small as possible. The transmission coefficient can be defined as the ratio between the wave-height on the exposed side, the ocean area, and the protected side, the harbour area, see also figure 3.12.

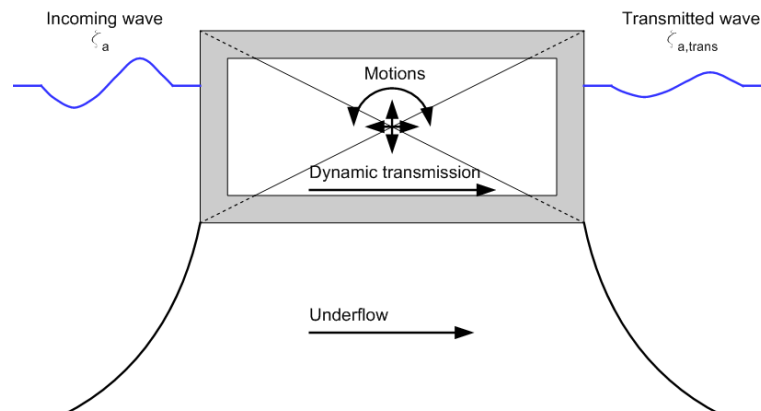


Figure 3.12 The principle of floating breakwater

Although there have been a lot of different research projects over the years, these projects can be divided into two groups. One part of the research has been focused on the effect of the structural design on the wave-attenuating properties of the floating breakwater, while the other group mainly focused on the effect of the structural dynamics on this performance. It should be noted that a high wave-attenuating performance means that the structure has a low transmission coefficient.

The major parameters of the structural design that have been studied are:

- Hull shape;
- Beam of the structure;
- Draught of the structure;
- Mass of the structure, although this is related to the beam and draught;
- Permeability of the structure
- Station keeping

A floating breakwater has, as does any floating breakwater, six degrees of freedom. To fully understand the effect of the motions in these degrees of freedom they were treated separately in some of the researches, such as [Tolba, 1999]. In a two-dimensional case there are only three degrees of freedom, which are sway, heave and roll.

In Appendix A there is a small overview given of interesting studies that were performed.

In their research [Silander, 1999; Tolba, 1999] concluded that a floating breakwater will have a better wave-attenuating performance when it is not limited in the vertical direction. The sway and roll motion however was found to have a

negative impact on the performance. This conclusion results in the idea that the influence of the mooring system on the vertical motion should be kept at a minimum, while it should be tried to limit the horizontal and rotational motions.

In the research of [Tolba, 1999] the sway and the roll motion of the breakwater were limited by the use of spud-poles as mooring, leaving heave as the only degree of freedom for the structure. This provides his design with the opportunity for multiple uses, but makes it impractical when larger water depths are considered.

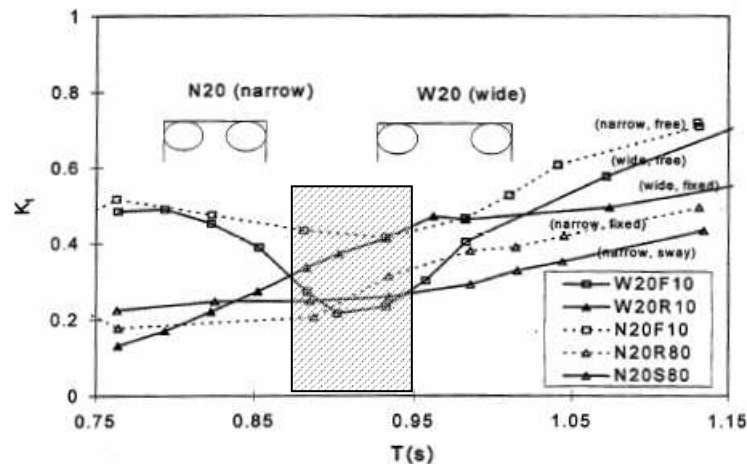


Figure 3.13 Transmission through a normally anchored 0.4 metre wide (W20F) and a 0.3 metre wide (N20F) structures having the same draft. The (***)S stands for structures where sway is allowed, while (***)R stands for fixed structures. [Silander, 1999]

When the heave motion is not affected by the mooring system this motion can have a positive influence on the wave-attenuating performance in a certain frequency range when the width is optimal. This has been proved by [Silander, 1999], see figure 3.13 and it can be verified by the research of [Williams mcdougal].

In the research of [Williams mcdougal] a two dimensional analysis of a FB was conducted. They assumed beam-on waves on an infinitely long structure which was moored with mooring lines which were schematised as linear springs. The waves with which the structure was loaded were assumed to be regular, low-crested waves. The most important conclusions of this research were:

- The FB is the most effective around its heave natural period;
- The performance of the structure is optimal when the diffracted wave, i.e. the wave flowing underneath the structure, and the radiated wave have the same amplitude, but have a phase-difference of π .

All these researches have been conducted under regular wave action. In reality the ocean surface is governed by numerous regular waves co-existing at the same time. This results in a highly irregular wave-field, often described with a wave spectrum instead of the single wave frequency of a regular wave. It is doubtful whether the assumptions made in these researches are still valid when irregular waves are considered, but at least they will provide some representation of the behaviour at different wave conditions.

3.5 Conclusion

Much theoretical and practical research has been done on Floating Breakwater technology, and it is still a subject of interest to many researchers. Despite the amount of research there is still no practical solution found for the general problem of creating a floating breakwater that is able to attenuate waves in a wide frequency range, whilst being able to survive the most severe storms. The floating

breakwaters of the displacement type that have been built until now were designed for specific locations with specific wave conditions. This resulted in the massive structure of the Monaco Floating breakwater and the light-weight, but only temporary, RIBS concept.

From model tests, as have been described in paragraph 3.4 and Appendix A it can be concluded that there is an optimal floating breakwater design for every wave frequency when regular waves are considered. In his report, [Fousert, 2006] noted a striking fact that one common conclusion returns in every report on floating breakwaters:

"A floating breakwater is able to attenuate waves to an acceptable level in a limited wave frequency range."

From this common conclusion [Fousert, 2006] remarked that a FB which can change its appearance would be expected to perform better on a wider wave spectrum. It was suggested that the FB should be able to be tuned into the frequency from the incoming waves. This holds when regular waves are considered, but one has to keep in mind that in real ocean conditions multiple wave frequencies will be present at any given time.

In order for the FB to become the solution to the many problems encountered by large scale offshore floating harbour terminals, as described by [Ali, 2005], the FB concept should not only be further elaborated on, but a design must be sought where the performance of the system is adequate on all frequencies that are harmful to the harbour operations.

Chapter 4 Problem definition and objectives

4.1 Problem definition

There are models or programs, such as DELFRAC, which can model ships and other floating objects like offshore platforms and also floating breakwaters. These programs do have one major drawback: the object dimensions must be known before modelling can begin. The programs are generally used to check the behaviour of the floating breakwater in final design stages. Finally this design can be built or, if necessary, modelled for physical testing in a laboratory.

The missing link in the process of designing a floating breakwater is a tool to make a quick preliminary design which can be developed further. A good step has been made by [Fousert, 2006] by introducing the ReFBreak-model. This is a two-dimensional model which can be used to assess the efficiency of a floating breakwater in regular beam-on waves.

The model is, however, only a small step in the first iteration towards the design of the “relative” most optimal floating breakwater. “Relative” has been put in quotes here because there is never a guarantee that a structured design process leads to the absolute best solution. The model has to be expanded towards irregular waves and also the initial transient process is important, i.e. the time it takes the floating breakwater to react to changing conditions. The ReFBreak-model should be expanded to simulate forces, e.g. anchor forces, as well.

The ReFBreak model is two-dimensional and therefore sufficient as a basic tool in designing FBs. When making a more complex structural design, however, the third dimension should not be forgotten. When designing a single element of the floating breakwater, the global plan for the whole breakwater, its layout, and the connections must be considered.

4.2 Objectives

The dynamic behaviour of a floating breakwater depends on many factors and is directly connected to the wave-attenuating performance. For beam-on waves it has been shown that *“the performance of the floating breakwater is primarily determined by the draft-mass-width relation of the structure”* [Fousert, 2006]. This model must be adapted and expanded to understand the effects of the structure’s dimensions in irregular wave-fields and effect of changing wave conditions.

Based on dimensions and forces produced by the developed model a conceptual design can be created. Various layouts have to be assessed, both on the level of the complete breakwater and on the level of the element itself.

The main objective of this master thesis is therefore twofold:

- Determine the influence of the cross-sectional dimensions on the dynamic behaviour of the floating breakwater by adapting and expanding the ReFBreak-model;
- Create a conceptual, structural design of a floating breakwater element on basis of the forces that are expected, to prove that a floating breakwater is technically feasible;

Chapter 5

Boundary conditions and requirements

The boundary conditions and requirements will be discussed in this chapter. First a general description of the system will be given. Then the boundary conditions and requirements are determined for each subdivision. Finally all results are summarised and listed in the conclusion.

5.1 General

Because of the complexity of the floating harbour system it is divided into three separate parts. Each of these parts does pose boundary condition and requirements on the floating breakwater. For example the harbour actions will require the waves within the harbour to be within certain limits while the sea will determine the wave climate the in which the floating breakwater has to work and survive. The three subdivisions, as depicted in figure 5.1, are defined as:

- the harbour area, where the floating harbour is moored;
- the floating breakwater area, where the FB is positioned;
- the seaside area, where the waves are generated.

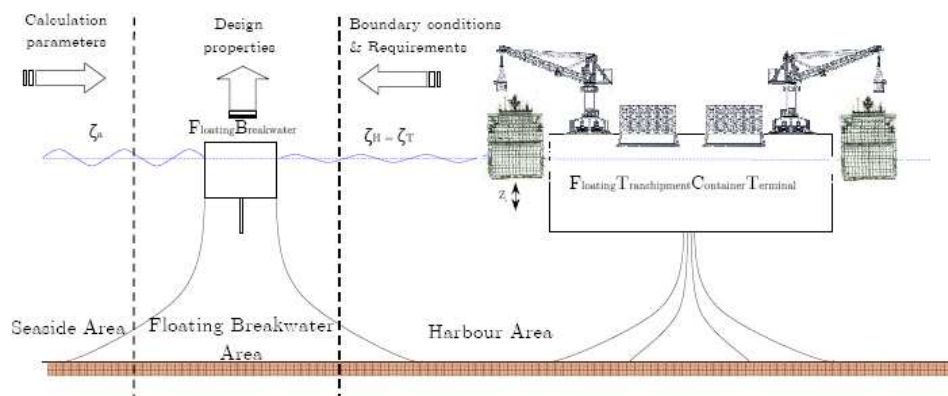


Figure 5.1 The floating harbour system [Fousert, 2006]

For the structures, i.e. the floating harbour and the FB, requirements can be derived from the functions these structures have to fulfil. The functions are first analysed and translated to requirements. The sea-side area only poses the boundary condition, where the FB has to mediate between the boundary conditions on one side and the requirements of the floating harbour on the other side.

5.2 Harbour area

While there are already many floating marinas in use around the world, there are currently no large scale floating harbours. There have been investigations into the feasibility of floating harbours and other mega-floats, such as [Ali, 2005]. In this master thesis a container terminal is assumed because of the rapid growth in this mode of transport and the limits to container vessel motion are most severe.

5.2.1 Floating Transshipment Container Terminal

The Floating Transshipment Container Terminal (FTCT) has been proposed by [Ali, 2005] as a solution for growing capacity problems of harbours around the world. The capacity problem is caused by the increasing flow of goods and the use of larger vessels around the world.

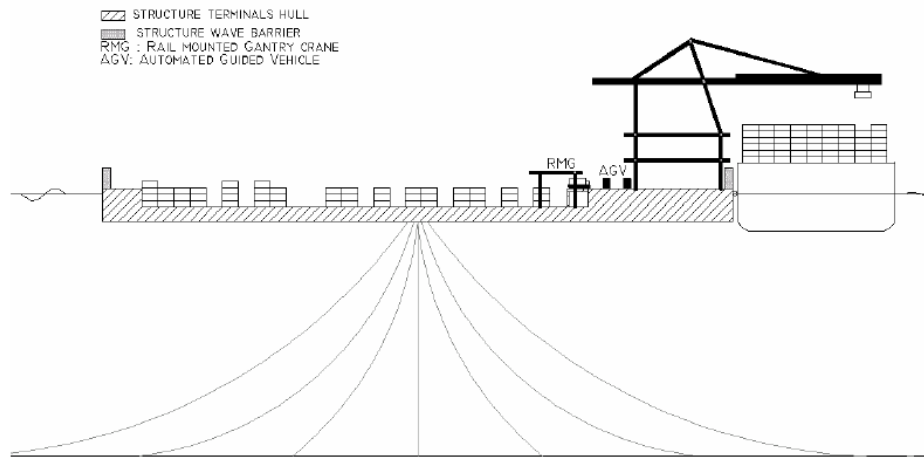


Figure 5.2 The Floating Transshipment Container Terminal as been proposed by [Ali, 2005]

In his design, [Ali, 2005] proposed the FTCT to be a pontoon shaped structure with a rectangular lay-out with a length of 1190 metre, a beam of 240 metre and a draught of 10.40 metre. The FTCT should have the same facilities as a modern container harbour, but located in offshore conditions. The facilities are:

- the loading and unloading of ships with a ship-to-shore (STS) crane;
- the intern transport of containers with automated guided vehicles (AGV's) and rail-mounted gantry (RMG) cranes;
- the storage of containers;
- and the processing of vessels.

The FTCT was designed to act as a breakwater to the ships, which are supposed to moor on the lee-side of the structure. To make the structure able to react on changing wave directions it was decided that the FTCT should have a DP-thruster turret mooring system. With the dynamic positioning thrusters the terminal can be turned in such a way that it always receives beam-on waves, creating a lee where the quay wall is positioned.

During storm conditions all harbour action is suspended and the FTCT should be rotated to face the waves head-on. In this way the pontoon, especially the anchoring system, should survive a maximum significant wave height of 10.25 metre and a maximum wind speed of up to 115 kilometre an hour, or Beaufort 12.

Because the FTCT acts as a breakwater for the moored ships, only one side of the pontoon can be used as a quay-wall. If a floating breakwater is applied around the FTCT it should be possible to position ships on all sides, making other, possible more efficient, design of a FTCT possible.

The function of the FB is to provide shelter for the moored ships during the mooring process and the (un-)loading. During storm conditions, defined as Beaufort 8 and higher, all harbour action has to be stopped due to the inability of operating the STS-cranes. When storm-conditions are expected, the ships have to leave the harbour area, because the ships can be dislodged from the quays due to the exposed harbour location. Consequently the FB does not function as a breakwater during the ULS and has the same ultimate limit state (ULS) as the FTCT.

5.2.2 Harbour operation

It is assumed that during working conditions, i.e. wind speeds lower than 8 Beaufort, the wave-length of the wave is very small compared with the beam of the floating breakwater. Therefore it is assumed the FTCT will not react on the waves, thus it can be schematised as a fixed platform, or as a normal harbour. About the parameters that involve the harbour efficiency [PIANC, 1995] states:

"The governing parameters for operation are vessel movements that are too rapid for the container operator to follow. Spreaders cannot move quickly horizontally in all directions.

The experience of the crane operator is of major importance since experienced drivers are capable to operate safely in more severe conditions and with larger ship motions than a less experienced one. Roll, pitch and heave are governed mainly by the characteristics of the ship itself, whereas the moorings and the fender systems govern surge, sway and yaw to a large extent.

For container vessels heave movements are relatively slow and are not critical. Surge and sway are also rather easy to deal with especially with modern cranes. In surge the container crane cannot follow the ship movements. However the operator can postpone the operation and wait for the return of the vessel to a position where the operation can be undertaken.

Sway is generally less difficult than surge, as only the trolley will have to follow the vessel. Yaw, especially during loading, and roll are critical in quartering seas. Roll is definitely judged to be the most dangerous motion in beam seas and quickly reduces handling efficiency. Roll is particular important where heavy loads have to be raised or lowered, and were during such movements the crane has a chance of bumping against the hatch or cells as a result of the vessel moves.

Motions at berth generally apply to periods of oscillation from 30 s to 120 s for surge, sway and yaw and to the wave periods for heave pitch and roll. Compared to other cargo handling systems, containers vessel operations are rather difficult ones."

An extensive research program was carried out by the Joint Nordic Group on the limits of motion for safe operation of the container vessels. These limits are summarised in table 5-2. These values are for peak-peak values, except for the sway movement; this is the zero-peak distance. Note that [Ligteringen, 2000] gives a value for the maximum allowable heave amplitude of 0.3 metre, i.e. a peak-peak value of 0.6 metre, which is significant smaller than the limit of Nordic. Furthermore in his report, [Goedhart, 2002] concludes that these limits are not strict enough for sway and surge.

Table 5-1 Nordic limits of motion for safe operation of container vessels [Jensen, Viggosson et al., 1990]

Ship type	Cargo handling efficiency [-]	Displacements			Rotations		
		Surge [m]	Sway [m]	Heave [m]	Yaw [°]	Pitch [°]	Roll [°]
Container vessel	100%	1,00	0,60	0,80	1,00	1,00	3,00
	50%	2,00	1,20	1,40	1,50	2,00	6,00

5.2.3 Design vessels

The container transport has grown with an incredible rate the past thirty years. The increased efficiency and the introduction of mega-vessels can be seen as the cause of this growth together with the increased demand in transport.

The FTCT will be visited by mega-vessels as well as smaller feeder vessels. Small feeder container vessels are applied for short sea container transportation while the larger feeder vessels up to 2500 TEU, or Twenty-foot Equivalent Units, will serve the regional harbours and feed the mega-vessels. The ships that can be expected at the FTCT are listed in table 5-2.

Table 5-2 Container ship types [Fousert, 2006], with natural periods (simple spring characteristics)

Ship size	[TEU]	400	600	800	1000	1200	1500	2000	2500
Container ship class		Small	Small	Small	Small	Feeder	Feeder	Feeder	Feeder
Deadweight (design)	[dwt]	5700	8400	11000	13500	16000	20000	26000	31000
Length overall	[m]	105	122	136	150	155	175	185	200
Breadth	[m]	17,2	19,2	21,4	22,7	25	28	30	30
Design draught	[m]	6,5	7,6	8,2	8,7	9,2	10	11,5	11,5
Heave natural period	[s]	4,5	4,8	5,0	5,2	5,3	5,6	6,0	6,0
Roll natural period	[s]	7,2	8,2	9,2	9,8	10,9	12,8	13,7	13,1

Ship size	[TEU]	3000	4000	4500	5000	6000	8000	12000	18000
Container ship class		Panamax	Panamax	Panamax	Post-Panamax	Post-Panamax	Post-Panamax	Suezmax	Post-Suezmax
Deadweight (design)	[dwt]	37000	48000	54000	59000	70000	93000	137000	200000
Length overall	[m]	220	280	293	280	305	355	400	470
Breadth	[m]	32,2	32,2	32,2	40	43	43	52,5	60
Design draught	[m]	12	11,8	12,2	12,5	12,5	13,6	14,6	15,7
Heave natural period	[s]	6,1	6,0	6,1	6,2	6,2	6,5	6,7	7,0
Roll natural period	[s]	14,3	14,0	13,7	18,0	19,2	18,8	22,9	26,1

Every ship that visits has its own characteristics and reacts differently on waves in the harbour area. In table 5-2 the estimated heave natural period and roll natural period are listed. These natural periods have been calculated by the use of a simple mass-spring system. It can be seen that the feeder vessel have a roll natural frequency which is approximately the same as wind-generated waves, while large container vessels will hardly react on the same waves.

Although the maximum motions of the ship for safe harbour operations are well-defined, the limiting significant wave height can not be derived from this. The report of [Pianc invoegen] stated about this:

"Relating the ship motions to a limiting significant wave height is not recommended. It is difficult to give specific limits for the admissible wave height because this is closely connected with the wave period, the angle of incidence, the ship's natural period of oscillation, elastic properties of fenders and hawsers, etc."

The investigation of the motions of a moored ship, relative to the FTCT, is considered to be out of scope of this master thesis, as is the determination of the limiting significant wave height of the floating breakwater.

Because of the difficulty to link the limiting significant wave height to the limits of motion for safe operation, the following assumptions are made for the rest of this thesis:

- Up to Beaufort 7 the limiting significant wave height is taken as 0.40 metre, this is a zero-peak value;
- From Beaufort 8 and higher there is no limiting significant wave height as all harbour action is assumed to be stopped.

5.3 Seaside area

At sea there are four categories of waves that can attack the floating harbour system. These categories are waves generated:

- by moving ships and other water moving objects;

Waves generated by moving ships and other water moving objects cannot be avoided as these waves are also generated within the harbour. When these waves do affect the efficiency of the harbour the solution has to be sought in regulating the maximum speed of the ships.

- by interaction of the water surface with the wind, i.e. wind waves;

These are the waves that have to be attenuated by the floating breakwater due to their period of 6 to 16 seconds and amplitude.

- by the astronomical force, i.e. the tides;

The tides are very long waves; they have a wave period of around 12 hours. These waves do not harm the productivity as the whole harbour system will go up and down slowly. The tides will however affect the water depth in some extent so they cannot be neglected when the mooring of the FB is considered.

- by geomorphic activities, i.e. tsunamis;

In deep water tsunamis are hardly noticeable, so they do not have to be attenuated by the FB. The tsunamis do harness an enormous amount of energy, but this energy is only dangerous when a tsunami gets into shallow water, which will generate massive waves

The wind generated waves are considered to be deep-water waves, also known as short waves, when the water depth is more than $\frac{1}{2}$ times the wave length. The main property of deep water waves is the limited energy penetration into the depth. The energy of a deep water wave is not able to reach the bottom, or analogous the bottom will not have an influence on the behaviour of the wave.

5.3.1 Wind generated waves

Wind generated waves can be divided in two basic categories [Journée and Massie, 2001]:

- Sea

A sea is a train of waves driven by the prevailing local wind field. Sea waves are very irregular; high waves are followed unpredictably by low waves and vice versa. The apparent or virtual wave period varies continuously, as well as the virtual or apparent wave length. The average period of a sea is less than 10 seconds.

- Swell

A swell are waves which have propagated out of the area and local wind in which they were generated. They are no longer dependent upon the wind and can even propagate for hundreds of kilometres through areas where the winds are calm. Individual waves are more regular and the crests are more rounded than those of a sea.

Waves, especially wind generated waves, are very irregular, but can be seen as a superposition of many simple, regular harmonic waves, each with their own wave characteristics, see figure 5.3.

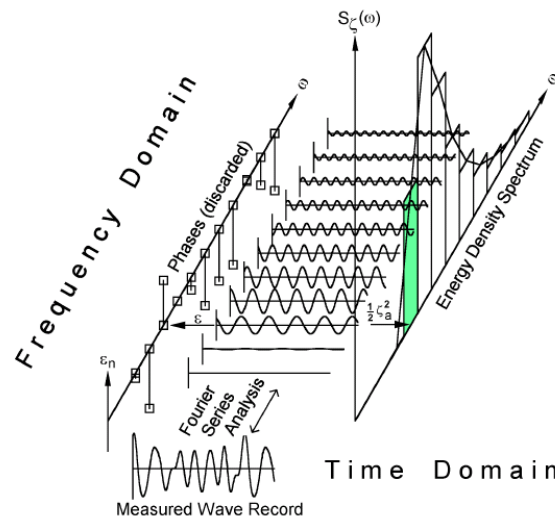


Figure 5.3 Wave record analysis and an energy density spectrum [Journée and Massie, 2001]

Investigators have attempted to describe a wave frequency spectrum in a standard form. One of the oldest and most popular is the wave frequency spectrum given by Brettschneider. This spectrum is most suited for open sea areas. Another spectrum is the JONSWAP spectrum which has been made after extensive research in the North Sea. This spectrum is most suited for situations where there is a limited fetch. A comparison between the two spectra is given in figure 5.4.

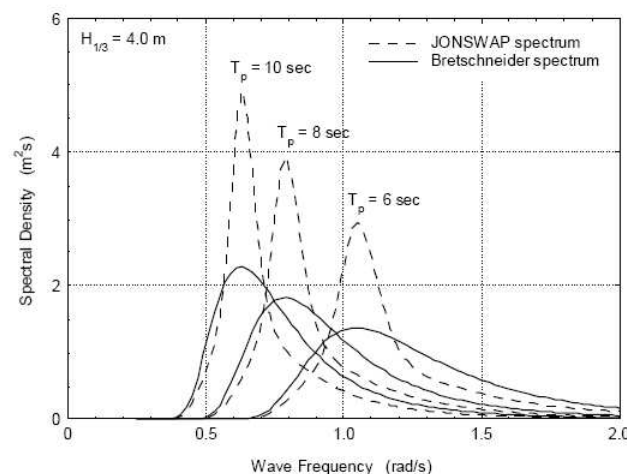


Figure 5.4 Comparison between Brettschneider and JONSWAP spectra [Journée and Massie, 2001]

The position of the considered location of the FB determines which wave spectrum has to be used. Each of the wave spectra has its own application area. There are basically three methods, including the two earlier mentioned methods, to obtain a wave spectrum [Fousert, 2006]:

- The Pierson-Moskowitz wave spectrum:
The spectral density is obtained by linking the significant wave height with a fixed relation.
This wave spectrum is used when fully developed seas are considered.
- The Bretschneider wave spectrum:
This is also known as the modified two-parameter Pierson-Moskowitz wave spectrum since the spectral density is a function of the significant wave height and the mean centroid wave period.

This wave spectrum has to be used when open ocean-conditions are to be expected.

- The JONSWAP, or Joint North Sea Wave Project, wave spectrum:

The spectral density is based upon measurements that were carried out along a line extending over 100 miles into the North Sea.

This wave spectrum is suitable when near shore conditions, especially when the North Sea area is considered.

The floating breakwater is assumed to be situated in deep water. When a wave period of 14 seconds is considered, the result of this assumption is that the minimum water depth has to be 150 metres. Therefore near-shore conditions are not expected and is the Bretschneider wave spectrum the most suitable.

Table 5-3 shows the relation between the Beaufort scale and the wave periods and amplitude for the Bretschneider wave spectrum. From this table it is clear that waves with periods longer than 10 seconds are certainly always swell.

The factors used in table 5-3 are:

$H_{1/3}$: The significant wave height;

T_1 : Mean centroid wave period;

T_2 : Mean zero-crossing wave period;

Table 5-3 Wave conditions in open ocean areas, adapted from [Journée and Massie, 2001]

Scale of Beaufort	Wind speed at 19,5 m [kn]	Open ocean areas (Bretschneider)			Scale of Beaufort	Wind speed at 19,5 m [kn]	Open ocean areas (Bretschneider)		
		$H_{1/3}$ [m]	T_1 [s]	T_2 [s]			$H_{1/3}$ [m]	T_1 [s]	T_2 [s]
1	2,0	1,10	5,80	5,35	7	30,5	3,75	7,80	7,20
2	5,0	1,20	5,90	5,45	8	37,0	4,90	8,40	7,75
3	8,5	1,40	6,00	5,55	9	44,0	6,10	9,00	8,30
4	13,5	1,70	6,10	5,60	10	51,5	7,45	9,60	8,80
5	19,0	2,15	6,50	6,00	11	59,5	8,70	10,10	9,30
6	24,5	2,90	7,20	6,65	12	>64,0	10,25	10,50	9,65

Despite the fact that waves in open-ocean conditions differ from regular waves, regular waves will be used in the process of designing the floating breakwater. Reasons for this simplification are [Fousert, 2006]:

- Regular waves do have one specific wave period T [s]. Irregular waves on the other hand can have more than one wave periods at the same time. To determine the optimal breakwater dimensions, it is essential to know the dynamic behaviour of the floating breakwater in a wide range of wave periods. This is only possible when the specific wave periods can be treated separately.
- A prevailing wave period can often be distinguished in ocean-conditions. The performance of the floating breakwater will be optimal when the floating breakwater is able to attenuate the waves in a range around this peak-period. Knowledge about the performance of the floating breakwater at unique wave periods is a prerequisite to obtain an optimal breakwater design.

5.3.2 Regular waves

A regular wave is defined as a wave with a sinusoidal shape with an amplitude ζ_a [m] and a wave height $H = 2\zeta_a$ [m]. The length between two successive crests is

called the wave length λ [m], see also figure 5.5. The time that is needed for the wave to travel over the distance of one wave-length is called the wave period T [s].

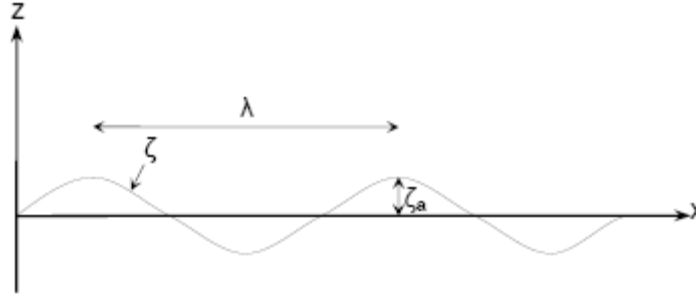


Figure 5.5 Wave parameters

The waves are considered to be deep water waves, i.e. the water depth does not influence the waves, because the energy is not able to reach the bottom. Because the waves are considered to be sinusoidal the wave period and the wave length can be rewritten in an angular form:

$$k = \frac{2\pi}{\lambda} \quad : \text{The wave number [rad/m]} \quad (5.1)$$

$$\omega = \frac{2\pi}{T} \quad : \text{The wave frequency [rad/s]} \quad (5.2)$$

The phase-speed is defined, when deep water is considered, as:

$$c = \frac{L}{T} = \frac{\omega}{k} \quad (5.3)$$

When (5.1) and (5.2) are substituted in (5.3) a relation is obtained for the wave length [Battjes, 1999]:

$$\lambda = \frac{gT^2}{2\pi} = 1.56T^2 \quad (5.4)$$

A combination of (5.3) and (5.4) gives the relation between the wave frequency and the wave number, the so called dispersion relation:

$$\omega^2 = kg \quad (5.5)$$

The surface elevation, velocity and acceleration of the wave in time and space can now be described with the following relation:

$$\zeta = \zeta_a \cos(\omega t - kx) \quad \text{Surface elevation} \quad (5.6)$$

$$\dot{\zeta} = -\zeta_a \omega \sin(\omega t - kx) \quad \text{Surface velocity} \quad (5.7)$$

$$\ddot{\zeta} = -\zeta_a \omega^2 \cos(\omega t - kx) \quad \text{Surface acceleration} \quad (5.8)$$

The floating breakwater has to attenuate waves and is a dynamic system. Therefore not only the wave characteristics are of importance, but the wave pressure, wave energy and wave power as well. The wave pressure is the driving force of the system, while the FB will absorb and transmit wave energy.

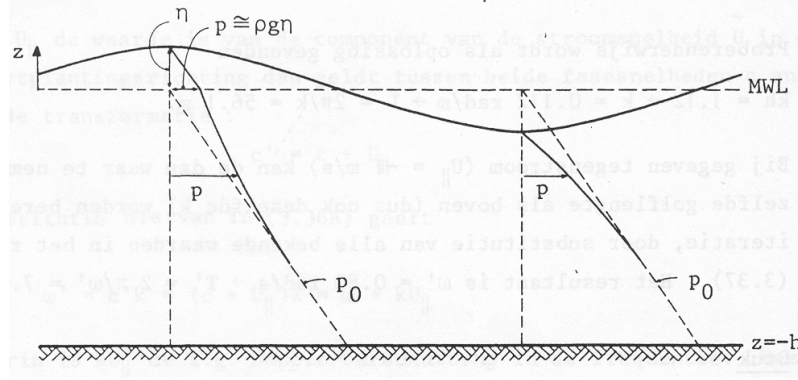


Figure 5.6 Vertical pressure distribution [Battjes, 1999]

The total pressure can be found with the formula of Bernoulli:

$$p = -\rho g z - \rho \frac{\partial \phi}{\partial t} - \frac{1}{2} \rho q^2 \quad (5.9)$$

In equation (5.9) the first term is the hydrostatic pressure. The second term is the pressure fluctuation caused by the waves, see also figure 5.6. The third and last term is the pressure fluctuation caused by the velocity of the water particles. In the linear wave theory this contribution is considered to be very small and therefore is neglected. The wave pressure for deep water is given by:

$$p_+ = \rho g \zeta_a e^{kz} \cos(\omega t - kx) \quad (5.10)$$

Or:

$$p_+ = \rho g e^{kz} \zeta \quad (5.11)$$

The energy contained by the waves consists of two contributions, the potential energy that is stored in the wave and the kinetic energy that is stored in the velocity of the water particles:

$$E_k = \frac{1}{4} \rho g \zeta_a^2 \quad \text{Kinetic energy} \quad (5.12)$$

$$E_p = \frac{1}{4} \rho g \zeta_a^2 \quad \text{Potential energy} \quad (5.13)$$

The total energy stored in the wave is the summation of (5.12) and (5.13):

$$E_{tot} = \frac{1}{2} \rho g \zeta_a^2 \quad \text{Kinetic energy} \quad (5.14)$$

In the previous part the energy that is contained within the waves has been determined. More interesting is the problem of the transfer of the energy, or the transmitted wave power. The wave power is transmitted by two mechanisms; the stored energy of water particles and the work done by the wave pressure. The transferred wave power can be found by solving the integral:

$$P = \int \overline{p_+ u} \cdot dz \quad (5.15)$$

When this integral is taken over the complete water depth, i.e. when there is no FB, the power transfer is:

$$P = \left(\frac{1}{2} \right) \left(\frac{1}{2} \rho g \zeta_a^2 \right) \left(\frac{\omega}{k} \right) = \frac{1}{2} E_{tot} c \quad (5.16)$$

All formulas that have been discussed in this section are summarized in table 5-4.

Table 5-4 Formulas linear wave theory

Symbol	Unit	Description	Formula
ζ	[m]	Wave amplitude	
T	[s]	Wave period	
λ	[m]	Wave length	$\lambda = \frac{gT^2}{2\pi}$
k	[rad/m]	Wave number	$k = \frac{2\pi}{\lambda}$
ω	[rad/s]	Wave frequency	$\omega = \frac{2\pi}{T}$
p_+	[N/m ²]	Wave pressure	$p_+ = \rho g e^{kz} \zeta$
E_k	[J/m ²]	Stored kinetic energy	$E_k = \frac{1}{4} \rho g \zeta_a^2$
E_p	[J/m ²]	Stored potential energy	$E_p = \frac{1}{4} \rho g \zeta_a^2$
E_{tot}	[J/m ²]	Total wave energy	$E_{tot} = \frac{1}{2} \rho g \zeta_a^2$
P	[Js/m]	Transferred power	$P = \frac{1}{2} E_{tot} c$

5.4 Floating breakwater area

The floating harbour loses its efficiency when there is too much wave action within this area. Especially hoisting operations with containers is affected by the motions of the ships due to the waves. To make a floating harbour feasible and economic attractive there has to be a structure that mediates between the boundary conditions, i.e. the waves, imposed by the ocean on one side and the limited allowed significant wave height on the other side.

The floating breakwater should be able to attenuate waves with wave periods between 6 and 12 seconds, as these waves will hamper the harbour operations the most. During extreme storm conditions longer wave periods are to be expected, but there is no need to attenuate these waves as all harbour operation are assumed to be stopped when wind speeds are expected of Beaufort 8 and higher.

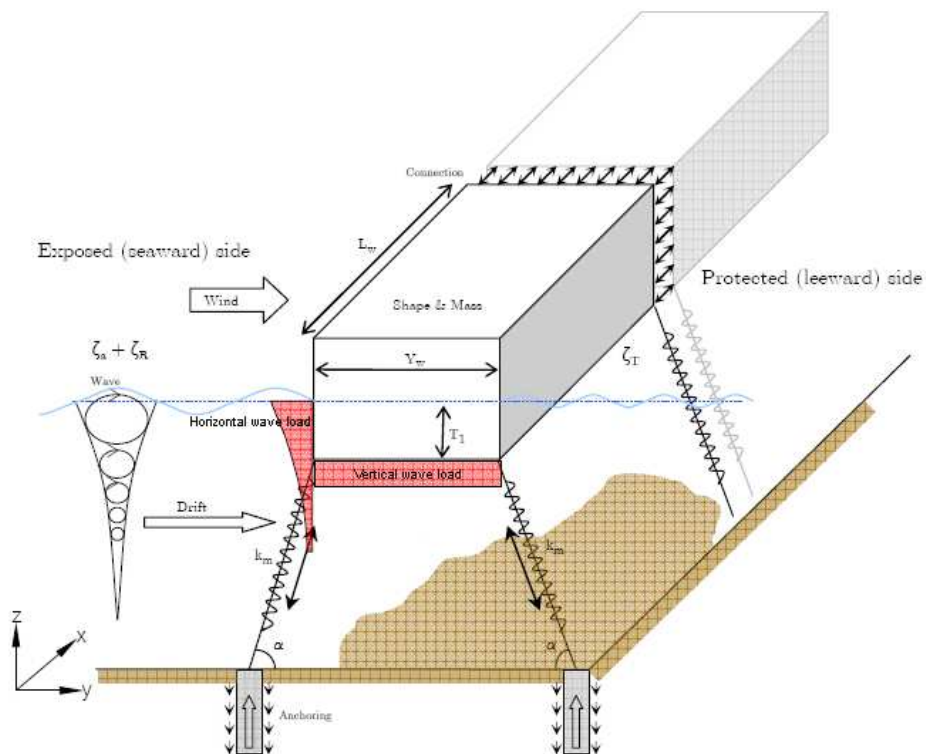


Figure 5.7 Interaction with the environment, the mooring and another elements [Fousert, 2006]

The floating breakwater area can be seen as a system that interacts with three independent systems, see also figure 5.7:

- interaction with the environment;
- interaction with the mooring system;
- interaction between the elements.

Each of these interactions will be elaborated on separately.

5.4.1 Interaction with the environment

The floating breakwater is forced by three forces:

- Hydrostatic pressure;
- Wave pressure;
- Currents.

Every object that is submerged in water is inevitably subjected to hydrostatic pressure. This force is responsible for keeping the FB afloat as long the displaced volume is more than the total weight of the FB system. When a one of the dimensions can be varied in the final design, for example the draught or the beam, the smallest displacement determines the maximum light ship mass, i.e. the mass of the materials use and the machinery if any.

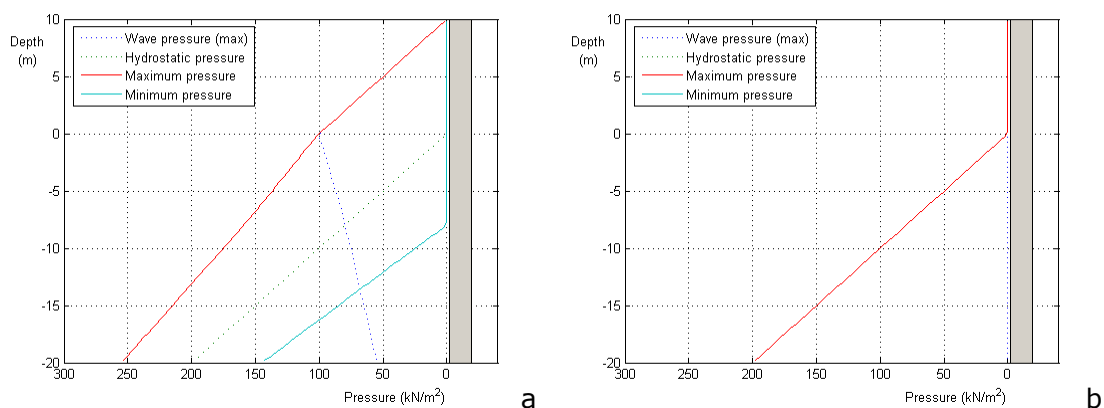


Figure 5.8 Wave pressures on the FB for very short waves (a) and very long waves (b)

In this thesis the wave-attenuating performance of the floating breakwater is researched with the linear wave theory and beam-on waves. The waves force the breakwater to move and therefore the FB produces waves in its turn. There are two extreme situations, see also figure 5.8:

- The waves are very small in comparison with the beam of the FB

When the FB is large compared with the incoming waves it will react as a concrete wall; the Monaco Floating Breakwater is based on this principle. Besides the hydrostatic pressure the structure is loaded with the wave pressure, which can be twice as high as the incident wave pressure because of the reflection.

- The waves are very large in comparison with the beam of the FB

When the FB is very small compared with the incoming waves it will react like a water particle. It will move along with the water and it will only be loaded with the hydrostatic water pressure.

Besides these two extreme situations it has to be ensured that the hull can withstand the actual pressures which can be higher due to the motion of the FB relative to the water particles.

Currents will drag the structure along, causing bending moments in the structure and possible significant forces in the mooring system. The (tidal) currents are assumed not to have an effect on the hydrodynamic properties of the floating breakwater.

For the interaction with the environment the following requirements have to be fulfilled:

- The mass of structure has to be less than the minimum water displacement, or in other words, the structure has to float;
- The forcing of the system are the waves;
- The hull of the structure has to withstand the summation of hydrostatic pressure, wave pressure and forces due to acceleration;
- The hull is rectangular and has to be optimised for an optimal wave-attenuating performance.

From a combination of the boundary conditions imposed by the ocean and the requirements about limited wave height inside the harbour area the following requirements has to be fulfilled as well:

- Up to a wind force of Beaufort 7 the maximum significant wave height inside the harbour area due to wind generated waves is 0.30 metre;
- When the wind speed if Beaufort 8 or higher the FB does not have to attenuate waves anymore because all harbour action will be ceased;
- The FB should have the same ultimate limit state (ULS) as the floating harbour. It should survive, and especially the anchoring system and connections, a maximum significant wave height of 10.25 metre and a maximum wind speed of up to 115 kilometre an hour, or Beaufort 12.

5.4.2 Interaction with the mooring system

A mooring system is needed to keep the floating breakwater on its positions where it is supposed to be. Besides the station keeping property, the mooring system will have an influence to some extent on the dynamic behaviour of the FB. The tide will have in its turn an influence on the properties on the mooring system, see also figure 5.9.

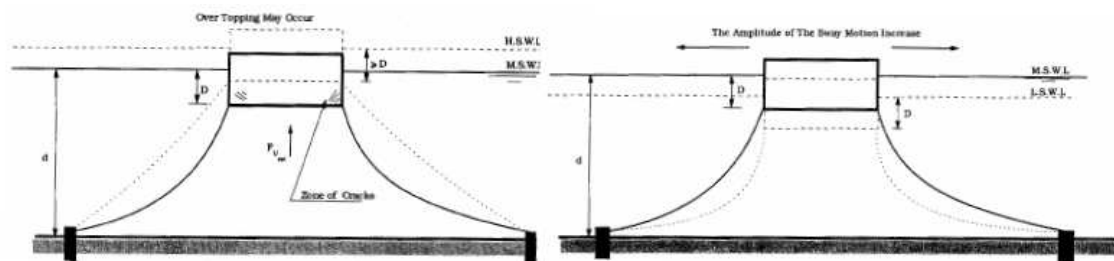


Figure 5.9 Tidal influence on the mooring line [Tolba, 1999]

The calculations on the floating breakwater will be carried out by using the linear wave theory. As a consequence of this theory there is no mooring system needed. According to the linear wave theory the FB will oscillate around its equilibrium position which is not moving. In the real world the breakwater the equilibrium position will move. The forces responsible for this movement are:

- second order drift forces;
- currents.

These forces do have to be absorbed by the mooring system in order to keep station.

For the calculation of the dynamic behaviour it assumed that the mooring system has a limited influence on the short term dynamic behaviour of the floating breakwater system. The effects of the tides, second order drift forces and the currents on the wave-attenuating performance of the system are not considered in this research. When designing the FB, and especially its mooring system and connections, these forces may not be neglected.

For the interaction with the mooring system the following requirements have to be fulfilled:

- The mooring system is responsible for the station keeping of the floating breakwater;
- The second order drift forces and the forces due to (tidal) currents have to be absorbed by the mooring system and transferred to the FB by means of a connection;
- The mooring system should not limit the dynamic behaviour of the floating breakwater when this behaviour is beneficial to the wave-attenuating performance;
- The mooring system may limit, but does not necessarily have to, the dynamic behaviour when this behaviour is not beneficial to the performance of the FB.

5.4.3 Interaction between the elements

Although beam-on waves are considered the interaction between the elements may not be omitted. When there is chosen for a certain design this will have consequences for the connections between the elements.

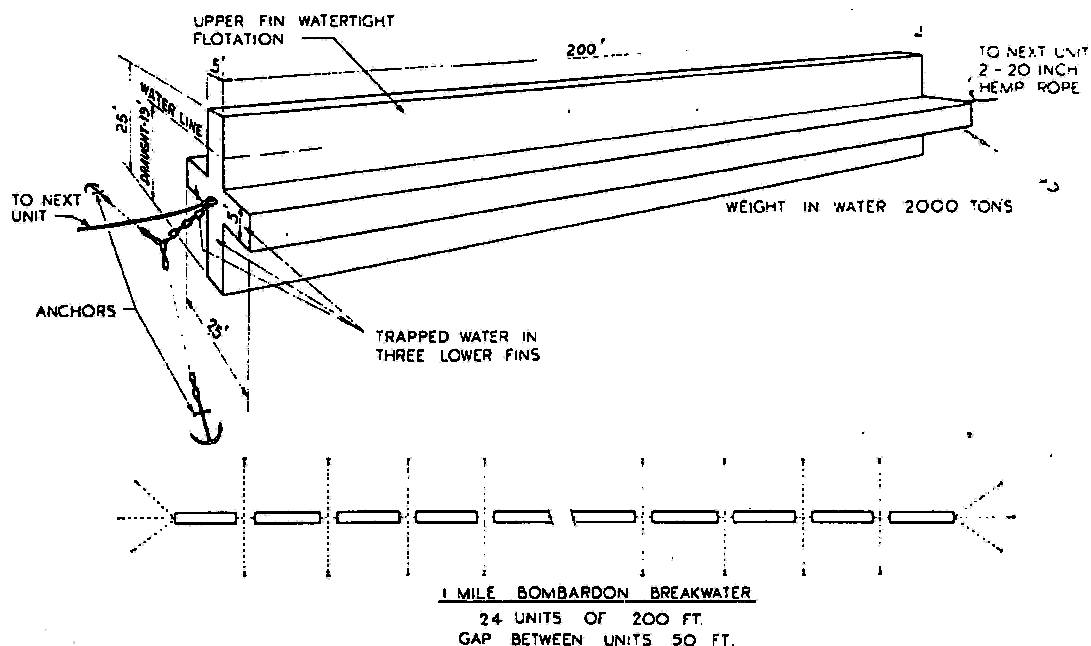


Figure 5.10 The "Bombardon" floating breakwater¹

During WWII the Bombardon floating breakwater, see figure 5.10, failed after just nine days in a one in forty year storm when the connections between the elements failed together with the mooring system [Readshaw, 1981]. It can be concluded that the connections between the elements are of great importance.

¹ Mulberries and Gooseberries
http://www.hksw.org/despatches_107_1_a.htm

There are basically two types of connections:

- Non-rigid connection
- Rigid connection

When there is a non-rigid connection damage has to be expected at the end of the elements due to collision [McLaren, 1981]. Also the waves which are generated by one element affect the neighbouring elements and thus can affect the efficiency of the breakwater. When there is rigid connection damage has to be expected due to fatigue failure of the connection.

For the interaction between the elements the following requirements have to be fulfilled:

- There should be no damage allowed, neither due to collision nor fatigue;
- The breakwater elements have to be coupled at the open sea, i.e. the in same environment which they have to protect the ships in the harbour from;
- The connections should not limit the dynamic behaviour of the floating breakwater when this behaviour is beneficial to the wave-attenuating performance;
- The connections may limit, but do not necessarily have to, the dynamic behaviour when this behaviour is not beneficial to the performance of the FB.

5.5 Conclusion

All boundary conditions and requirements that are discussed above are listed here. First the boundary conditions are listed, followed by the requirements for the service limit state (SLS) and ultimate limit state (ULS). Finally some complementary requirements are listed for the mooring system and connections between the elements.

The boundary conditions where the FB is subjected to:

- The FB is loaded with waves in an open sea area. The wave frequency spectrum is described with a Bretschneider spectrum as stated in table 5-5;
- To assess the wave-attenuating performance of the FB the linear wave theory will be used.

Table 5-5 Wave conditions in open ocean areas, adapted from [Journée and Massie, 2001]

Scale of Beaufort	Wind speed at 19,5 m [kn]	Open ocean areas (Bretschneider)			Scale of Beaufort	Wind speed at 19,5 m [kn]	Open ocean areas (Bretschneider)		
		$H_{1/3}$ [m]	T_1 [s]	T_2 [s]			$H_{1/3}$ [m]	T_1 [s]	T_2 [s]
1	2,0	1,10	5,80	5,35	7	30,5	3,75	7,80	7,20
2	5,0	1,20	5,90	5,45	8	37,0	4,90	8,40	7,75
3	8,5	1,40	6,00	5,55	9	44,0	6,10	9,00	8,30
4	13,5	1,70	6,10	5,60	10	51,5	7,45	9,60	8,80
5	19,0	2,15	6,50	6,00	11	59,5	8,70	10,10	9,30
6	24,5	2,90	7,20	6,65	12	>64,0	10,25	10,50	9,65

The factors used in table 5-3 are:

- $H_{1/3}$: The significant wave height;
- T_1 : Mean centroid wave period;
- T_2 : Mean zero-crossing wave period;

For the service limit state (SLS) and the ultimate limit state (ULS) the requirements are:

- Up to a wind force of Beaufort 7 the maximum significant wave height inside the harbour area due to wind generated waves is 0.30 metre;
- When the wind speed is Beaufort 8 or higher the FB does not have to attenuate waves anymore because all harbour action will be ceased;
- The FB should have the same ultimate limit state (ULS) as the floating harbour. It should survive, and especially the anchoring system and connections, a maximum significant wave height of 10.25 metre and a maximum wind speed of up to 115 kilometre an hour, or Beaufort 12.

For the mooring the following complementary requirements have to be fulfilled:

- The water depth is 150 metres;
- There should be no damage allowed, neither due to collision nor fatigue;
- The breakwater elements have to be coupled and moored at the open sea, i.e. the in same environment which they have to protect the ships in the harbour from;
- The mooring system and connections should not limit the dynamic behaviour of the floating breakwater when this behaviour is beneficial to the wave-attenuating performance;
- The mooring system and connections may limit, but do not necessarily have to, the dynamic behaviour when this behaviour is not beneficial to the performance of the FB.

Chapter 6 ReFBreak2D-model

The ReFBreak-model was developed by [Fousert, 2006] in order to assess the influence of the structural dimension of the floating breakwater. The ReFBreak2D model is based on the ReFBreak-model, but takes in account:

- The coupling of the sway and the roll motion;
- The effect of diffraction on the wave exciting forces and moments.

First the dynamic behaviour a FB and the basics of the forces and motions are discussed as these are the same for both models. The equations of motions and further elaboration of the models is discussed separately for both models as from this point the models are differ from each other.

6.1 General

The basis of both models is the flowchart that is depicted in figure 6.1. Incoming waves are responsible for exciting forces and moments on the structure. These forces and moments will put the floating breakwater in an oscillatory motion, which in turn will cause waves radiating from both sides of the breakwater. The total transmitted wave towards the harbour-side of the floating breakwater is found by the summation of all radiated waves and the wave that passes underneath the breakwater.

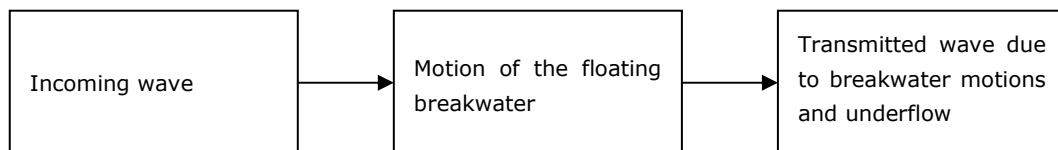


Figure 6.1 Model flowchart

6.1.1 Dynamic behaviour

A floating body in a three dimensional space has 6 degrees of freedom:

1. Surge
2. Sway
3. Heave
4. Roll
5. Pitch
6. Yaw

These motions are defined by the translation of the centre of gravity in the x-, y- and z-direction and the three rotations about those axes. This is shown in figure 6.2, where the numbers are corresponding to the respective motion. [Journée and Massie, 2001]

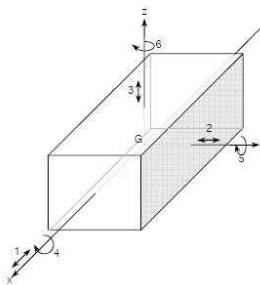


Figure 6.2 Degrees of freedom for a floating body in a three-dimensional space [Fousert, 2006]

With respect to the axes in figure 6.2 the formulas for the three translations and the three rotations are defined. With the motions of the centre of gravity (COG) known the motions in any point on the structure can be calculated by the use of superposition. The motions are:

$$\begin{aligned}
 x(t) &= x_a \cos(\omega t + \varepsilon_{x\zeta}) \\
 y(t) &= y_a \cos(\omega t + \varepsilon_{y\zeta}) \\
 z(t) &= z_a \cos(\omega t + \varepsilon_{z\zeta}) \\
 \phi(t) &= \phi_a \cos(\omega t + \varepsilon_{\phi\zeta}) \\
 \theta(t) &= \theta_a \cos(\omega t + \varepsilon_{\theta\zeta}) \\
 \psi(t) &= \psi_a \cos(\omega t + \varepsilon_{\psi\zeta})
 \end{aligned} \tag{6.1}$$

Where:

n_a	: Motion amplitude	[m], [rad]
$\varepsilon_{n\zeta}$	: Phase angle	[rad]
ω	: Circular wave frequency	[rad/s]

The floating breakwater is loaded by waves. The wave elevation can be defined as:

$$\zeta(x, y, t) = \zeta_a \cos(\omega t - \sin(\mu)kx - \cos(\mu)ky) \tag{6.2}$$

Where

ζ_a	: Wave amplitude	[m]
ω	: Wave frequency	[rad/s]
k	: Wave number	[m ⁻¹]
μ	: Angle of attack	[rad]

For a 2 dimensional case with beam-on waves equation (6.2) can be simplified to:

$$\zeta(y, t) = \zeta_a \cos(\omega t - ky) \tag{6.3}$$

The phase shifts of the previous mentioned motions are related to the harmonic wave elevation at the origin, i.e. the centre of gravity, even though no wave can be measured at this location. The wave elevation in the centre of gravity ($y = 0$):

$$\zeta_{COG}(t) = \zeta_a \cos(\omega t) \tag{6.4}$$

6.1.2 Forces and motions

It is assumed that a floating breakwater will not generate waves by itself due to structural movements when floating in still water. The only hydro-mechanical reaction will occur when the structure is loaded by waves, causing wave exciting forces and moments. In other words the floating breakwater does not generate energy; it can only absorb energy of waves and re-transmit this energy. It is assumed that this relation is linear, i.e. doubling the input wave amplitude results in doubled output amplitude while possible phase changes between input and output will not change.

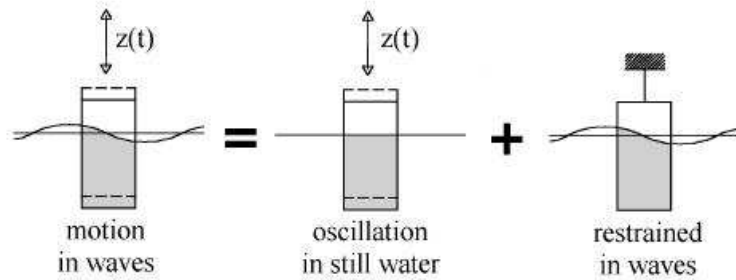


Figure 6.3 Superposition of hydro-mechanical and wave loads [Journée and Massie, 2001]

Since the system is assumed to be linear, the resulting motion in waves can be seen as a superposition of the body in still water and the forces on a restraint body, as can be seen in the picture equation in figure 6.3. This leads to two important assumptions [Journée and Massie, 2001]:

1. The hydro-mechanic forces and moments are induced by the harmonic oscillations of the rigid body in the undisturbed surface of the fluid;
2. The wave exciting forces and moments are produced by the waves coming in on the restraint body.

6.2 ReFBreak

The ReFBreak-model has been developed by [Fousert, 2006] in order to assess the influence of the structural dimension of the floating breakwater. This paragraph is a summary of the work that has been done.

A complete evaluation of the ReFBreak-model can be found in Appendix B. Here a short description with the outline of the model is given.

6.2.1 Equation of motions

The motions are not independent with respect to each other. However, since only beam-on waves are considered, surge, pitch and yaw will not occur when a symmetric structure is assumed.

Lateral forces or moments can not be produced by the vertical motion of floating breakwater due to symmetry of the structure. Also, [Vugts, 1968] proved that the Φ - and y -components in the wave exciting forces are relatively small in respect to the z -components. Therefore heave can be considered as an uncoupled motion. [Fousert, 2006] considered heave, sway and roll as uncoupled motions.

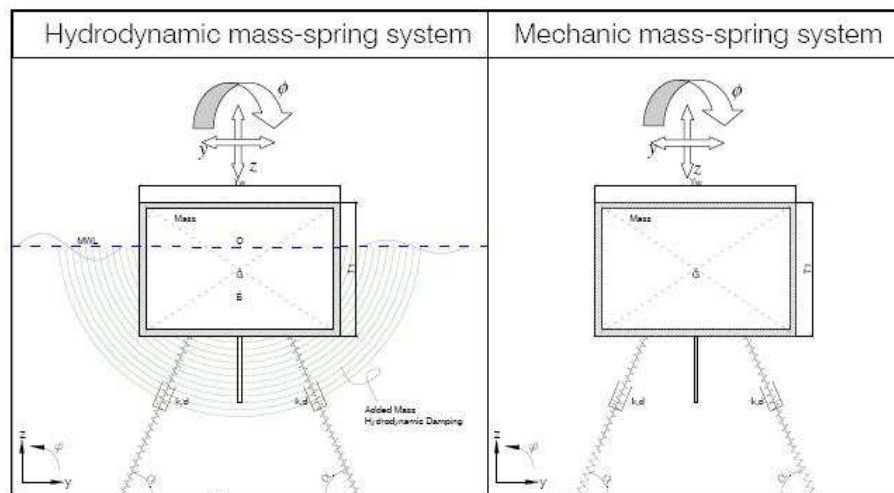


Figure 6.4 A 2-dimensional hydrodynamic and mechanical mass-spring system [Fousert, 2006]

The floating breakwater can now be described with a hydrodynamic mass-spring system, as depicted in figure 6.4, and the corresponding second-order linear differential equation:

$$\begin{aligned} (m + a_{22})\ddot{y} + (d_{cy} + b_{22})\dot{y} + (k_{cy} + c_{22})y &= F_{w;hor} \\ (m + a_{33})\ddot{z} + (d_{cz} + b_{33})\dot{z} + (k_{cz} + c_{33})z &= F_{w;ver} \\ (I_x + a_{44})\ddot{\phi} + (d_{c\phi} + b_{44})\dot{\phi} + (k_{c\phi} + c_{44})\phi &= M_{w;g\phi} \end{aligned} \quad (6.5)$$

Where:

- y, z, ϕ : Displacement in the y-, z-, and ϕ -direction
- a_{nn} : Mass or moment of inertia, including the hydrodynamic component;
- b_{nn} : Damping, including the hydrodynamic component
- c_{nn} : Spring constants
- F_w, M_w : Wave induced forces and moment

6.2.2 Hydrodynamic coefficients

There are three hydrodynamic coefficients that govern the motions of the FB element:

- Added mass

When an object is accelerated in a fluid the force that is required is larger than expected when the same object is accelerated in air. This force is due to the added mass of the object, or the amount of water that is pushed away by the object when it moves.

- Hydrodynamic damping

When an object is moving in a fluid it generates waves. These waves do transport energy away from the object, thus removing energy. This removal of energy, sometimes called wave resistance, can be seen as a source of damping. Another type of damping, the viscous damping, is left out of consideration in this model.

- Spring stiffness

The spring stiffness is determined by the law of Archimedes. The force the element is subjected to is equal to the displacement of the object. A deviation from the equilibrium position will result in a counteracting force for the heave and roll motion. It does not activate any opposing force in the horizontal direction, because the sway motion does not result in an extra displacement of the FB element.

In the ReFBreak-model the curves for the additional mass and the hydrodynamic damping have been approached by the use of polynomials. Between the polynomial curves and outside the area that was investigated by [Vugts, 1968] the coefficients are interpolated and extrapolated respectively. For roll it was chosen to derive the hydrodynamic coefficients with a three dimensional potential program, DELFRAC.

6.2.3 Forces

The vertical wave force is calculated with the Froude-Krylov assumption. This assumption holds that the wave is not disturbed by the presence of the floating breakwater and that the vertical wave exciting force can be obtained by integrating

the pressures that are working on the bottom. The obtained force however has to be corrected because of the diffraction that is present. This results in the total wave exciting force for the heave motion:

$$F_{w;ver} = a_{33}\ddot{\zeta}_a^* + b_{33}\dot{\zeta}_a^* + C_3c_{33}\zeta \quad (F.6)$$

The horizontal wave force can be written as the sum of a so-called inertia force and a drag force as has been proposed by [Grotmaack and Meylan, 2006] for small floating bodies. The drag force is not included in the ReFBreak-model; the horizontal wave force will therefore be:

$$F_{w;hor} = (a_{22}\ddot{\zeta}_a^* + b_{22}\dot{\zeta}_a^*) \quad (F.7)$$

The wave exciting moment for roll can be determined by calculating the horizontal and vertical wave exciting forces and multiplying them respectively with vertical and horizontal lever arms, see figure 6.5. The formula for the wave exciting moment is:

$$M_{w;\phi} = F_{w;hor} \cdot z_b + F_{w;ver} \cdot y_b \quad (F.8)$$

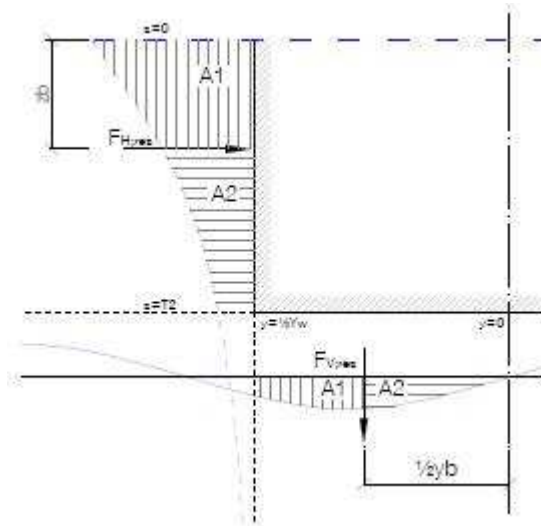


Figure 6.5 Determination of z_b and y_b [Fousert, 2006]

6.2.4 Results

The ReFBreak-model makes it possible to analytically determine the dynamic reaction of a floating breakwater and the transmitted wave height towards the harbour side. With the aid of the model it can be quickly assessed which factors do influence the performance and to which extent. The factors that determine the performance of the floating breakwater are:

- The structural width;
- The structural draft;
- The screen draft;
- The structural mass;
- The mooring stiffness;
- The mooring damping.

The model can be used to produce a graph of the RAO, the so-called Response Amplitude Operator, as can be seen in figure 6.6 on the next page. This RAO can be used to quickly determine the most optimal floating breakwater for different wave frequencies.

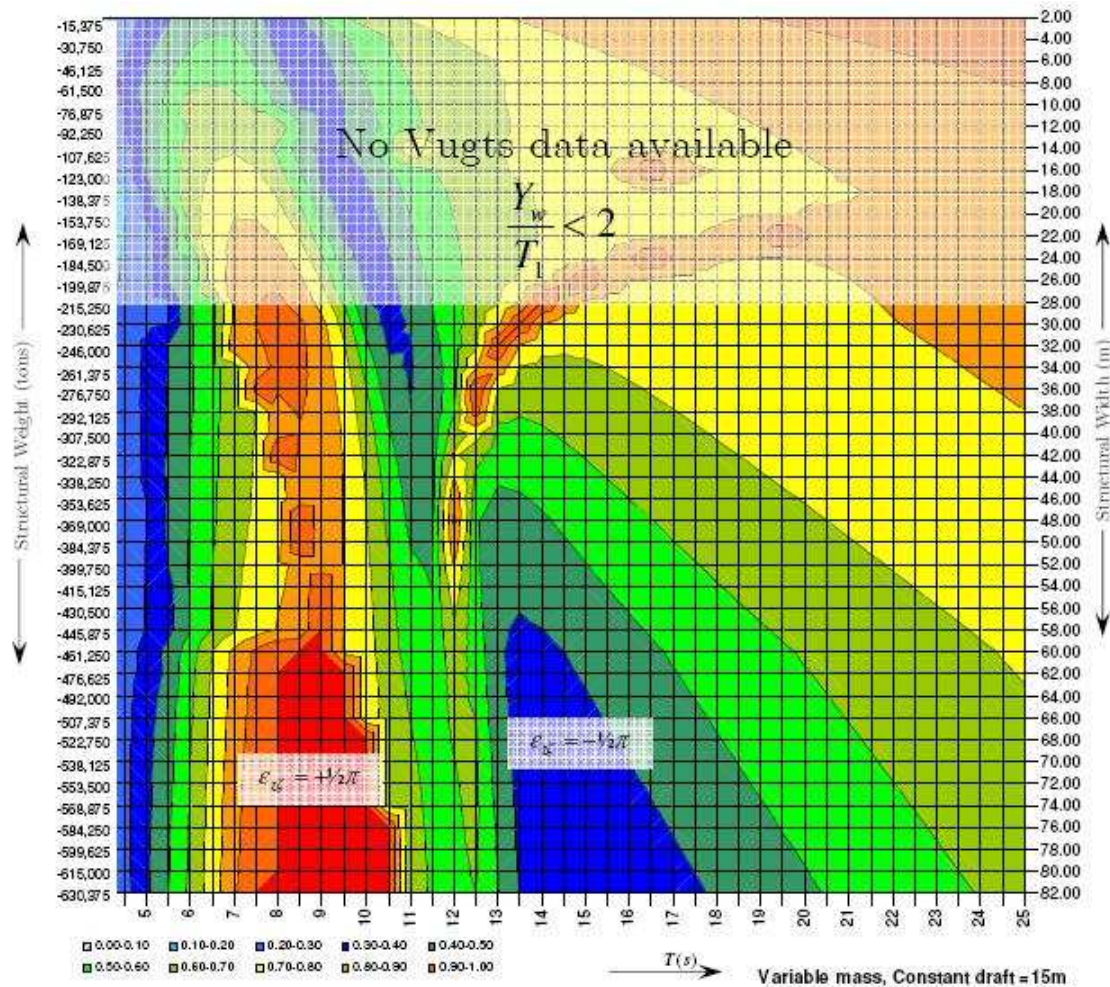


Figure 6.6 Example of a RAO for the transmitted wave height [Fousert, 2006]

6.2.5 Model limitations and inaccuracies

While translating the ReFBreak-model from MS Excel to MATLAB some discrepancies between the model and the documentation were discovered. The model was deduced again on basis of [Fousert, 2006] and some model limitations and inaccuracies were found. These limitations and inaccuracies included, but are not limited to:

- Uncoupled motions

It was assumed that in the two dimensional case the heave, sway and roll motions are uncoupled. It is true that the heave motion is uncoupled from sway and roll motions due to symmetry, however the roll and sway motions are not uncoupled.

When the centre of gravity is translated the floating breakwater there is a force opposing this translation. If and only if the centre of gravity is exactly on the same height as the resulting force due to the translation there will be no moment causing roll.

- Wave exciting forces

In the ReFBreak model formulas have been used that are only valid when the structures dimensions are very small compared to the wave length. When waves with a period of 10 seconds are considered, the wave length of these waves is approximately 150 metres. The proposed construction has a beam of 60 metres, so the assumption of a construction which is very small

compared to the waves is not valid. Therefore it is expected that the used formulas are not correct. Inconsistent units in some of the formulas reinforce this presumption.

- Hydrodynamic coefficient polynomials

The polynomials for the hydrodynamic coefficients are based on the curves of the research of [Vugts, 1968] and these are limited to a breadth-to-draught ratio of 2.0 and higher. In his research [Fousert, 2006] indicated that the best performance of a floating breakwater is to be expected with breadth-to-draught ratios of 1.0 to 2.0.

To have a more accurate prediction on the performance of a floating breakwater more polynomials are required. As these cannot be based on the research of [Vugts, 1968], these have to be determined by the three dimensional potential model DELFRAC.

6.3 ReFBreak2D

In order to overcome the limitations and inaccuracies of the ReFBreak-model a new model has been developed, ReFBreak2D. This new model uses the same basic principles as the ReFBreak model developed by [Fousert, 2006], but deals also with coupled motions, improved wave forces and adds hydrodynamic parameters for another hull shape.

A complete description of the ReFBreak2D-model can be found in Appendix C. Here a short description with the outline of the model is given.

6.3.1 Equations of motions

In the previous section it was concluded that the motions are coupled and that this coupling between the motions has to be processed in the ReFBreak2D-model. In the three-dimensional case there are 6 degrees of motion and these are coupled into two groups [Journée and Massie, 2001]:

1. Surge – Heave – Pitch, the so-called symmetric motions
2. Sway – Roll – Yaw, the so-called asymmetric motion.

Note that this distinction between symmetric and anti-symmetric motions can disappear when the ship is anchored. Then, for instance, the pitch motions can generate roll motions via the anchor lines. This also holds for a floating breakwater element as it is held in its place by a kind of anchoring system.

Because of symmetry, the floating breakwater is a rectangular box with two axis of symmetry, it can be deduced that heave is also an uncoupled motion in a three dimensional situation loaded with beam-on waves when it is assumed that the anchoring system does not influence the coupling between the motions.

The coupled second order differential equation, governing all six degrees of motion is given by [Journée and Massie, 2001]:

$$M\ddot{\vec{X}} + N\dot{\vec{X}} + C\vec{X} = \vec{F} \quad (6.9)$$

Where:

- | | |
|-----------|--|
| $\vec{X}$ | : Translation and rotation vector; |
| $\vec{F}$ | : Wave exciting forces and moments vector; |
| C | : Hydrodynamic spring stiffness matrix; |

In the two dimensional case equation (6.9) reduces to:

$$\begin{bmatrix} M_{22} & 0 & M_{24} \\ 0 & M_{33} & 0 \\ M_{42} & 0 & M_{44} \end{bmatrix} \begin{bmatrix} \ddot{y} \\ \ddot{z} \\ \ddot{\phi} \end{bmatrix} + \begin{bmatrix} N_{22} & 0 & N_{24} \\ 0 & N_{33} & 0 \\ N_{42} & 0 & N_{44} \end{bmatrix} \begin{bmatrix} \dot{y} \\ \dot{z} \\ \dot{\phi} \end{bmatrix} + \begin{bmatrix} c_{22} & 0 & 0 \\ 0 & c_{33} & 0 \\ 0 & 0 & c_{44} \end{bmatrix} \begin{bmatrix} y \\ z \\ \phi \end{bmatrix} = \begin{bmatrix} F_{w;hor} \\ F_{w;ver} \\ M_{w;\phi} \end{bmatrix} \quad (6.10)$$

Where

- M_{nn} : The hydrodynamic mass;
- N_{nn} : The hydrodynamic dampening;
- c_{nn} : The hydrodynamic spring stiffness;
- x, z, ϕ : Translations and rotations.

From (6.10) it can be seen that the asymmetric motions are uncoupled from the symmetric motions, as has been described in earlier.

6.3.2 Hydrodynamic coefficients

There are three hydrodynamic coefficients that govern the motions of the FB element:

- Added mass
- Hydrodynamic damping
- Spring stiffness

The added mass and the hydrodynamic damping have been determined for all motions with the three dimensional potential program DELFRAC, as well as the added mass coupling coefficients and the hydrodynamic damping coupling coefficient. These parameters have been made dimensionless and will be scaled in the ReFBreak2D-model. In this way numerous alternatives can be analysed quickly, while limiting the time consuming DELFRAC simulations to four.

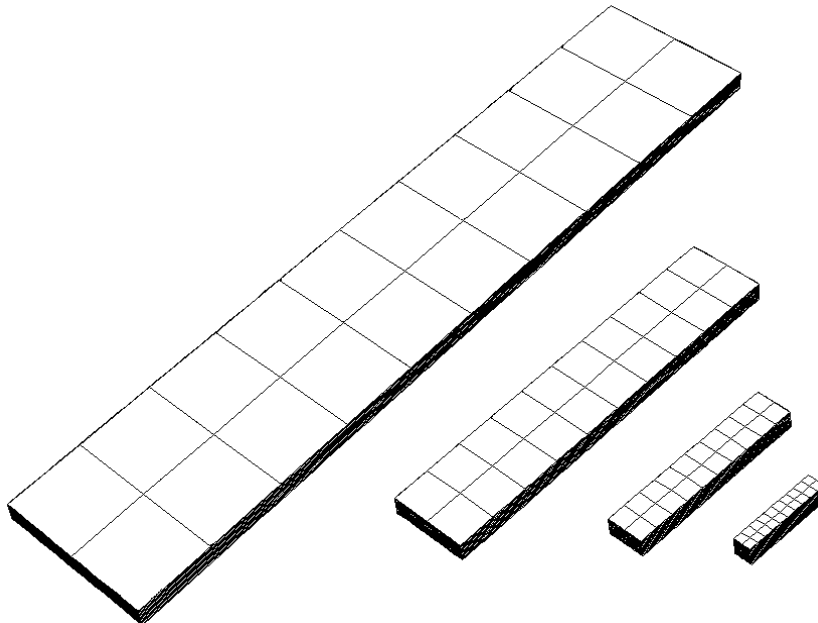


Figure 6.7 The four DELFRAC panel-models, $W/D = 8.0, 4.0, 2.0$ and 1.0 respectively

In addition to the three designs with beam-to-draught-ratios, or Y_w/T_1 -ratio, of 8.0, 4.0 and 2.0 which were investigated by [Vugts, 1968], a fourth design, with an Y_w/T_1 -ratio of 1.0, has been researched with DELFRAC. All panel-models which were used can be seen in figure 6.7. Information on the panel sizes is listed in table 6-1.

Table 6-1 Number of panels for the different W/D-ratios per quarter of an element

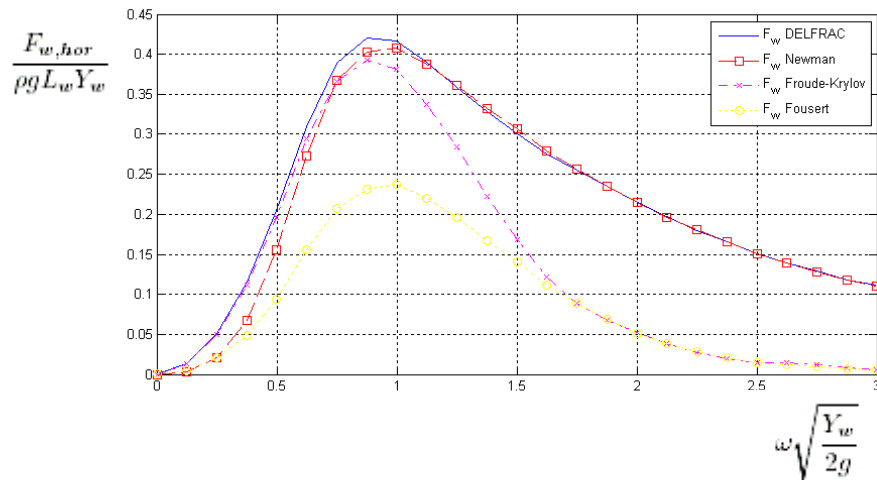
W/D-ratio	1	2	4	8
Draught	5 m	5 m	5 m	5 m
Breadth	5 m	10 m	20 m	40 m
Length	25 m	50 m	100 m	200 m
Min. wave-length	1,75 m	3,50 m	7,00 m	14,00 m
Panel-length	0,35 m	0,70 m	1,40 m	2,80 m
Number of panels	859 [-]	859 [-]	859 [-]	859 [-]

6.3.3 Forces

The wave exciting forces that are acting on the structure can basically be determined with three methods:

- Interpolating and extrapolating the data which was obtained from the DELFRAC simulation;
- The method that has been described by [Newman, 1977];
- On basis of the Froude-Krylov assumption, as has been described by [Journée and Massie, 2001].

Figure 6.8 gives a comparison of the three methods that are listed above for the wave exciting force in the horizontal (sway) direction. The Froude-Krylov assumption means the floating object is very small compared to the wave length. It can be seen that for very long waves Froude-Krylov can be used indeed. For high frequencies however, it can be seen that there is a large discrepancy between the forces from the DELFRAC and the calculated wave forces according to Froude-Krylov.

**Figure 6.8** Horizontal wave exciting forces for heaving by using the DELFRAC output

The method that has been described by [Newman, 1977] gives a good result on the complete frequency domain. This method describes a relation between the energy that is dissipated by the radiated waves and the energy that is contributed to the system by the force. This results in a relationship which is directly depending on the hydrodynamic damping:

$$F_{w,n} = \zeta_a \sqrt{\frac{\rho |b_{nn}| g^2}{L_w \omega}} L_w \quad (6.11)$$

Where:

- b_{nn} : Hydro-dynamic damping coefficient for the motion;
 ω : Circular wave frequency;

L_w : Length of the element;
 ζ_a : Wave elevation.

6.3.4 Energy consideration

There are three contributions to the transmitted wave towards the harbour area:

- Waves radiated due to the heave motion;
- Waves radiated by the combined sway/roll motion;
- Waves transmitted by underflow.

The amplitude of the radiated waves depends on the velocity amplitude of the floating breakwater. The phase-change depends on whether a positive velocity causes a “positive” wave, or a “negative wave”. This is for both the heave motion and the combined sway/roll motion different, see also figure 6.9a and figure 6.9b.

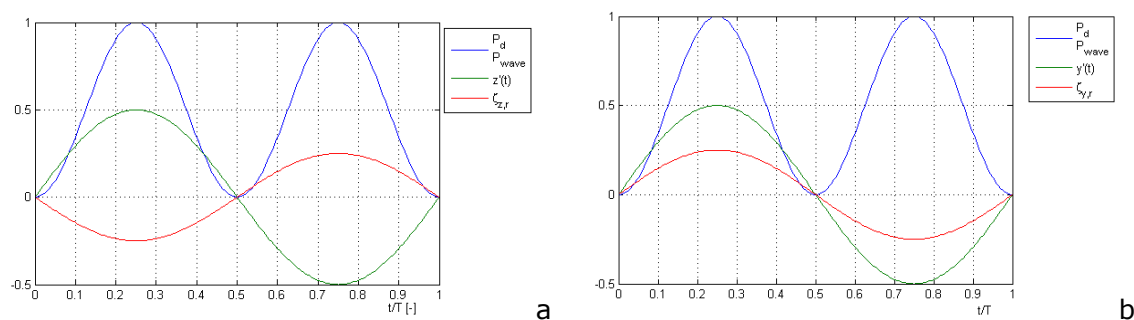


Figure 6.9 Dissipated power, structural velocity and generated wave versus time for heave (a) and sway/roll (b)

The underflow is governed by the energy that passes undisturbed underneath the FB element. Relative to the incoming wave the waves caused by the underflow phenomena do not have any phase-change, while the waves generated by the motions of the floating breakwater do have.

All contributions can be summed taking in account their phase-change to obtain the total transmitted wave, see Appendix C.7.4. The same can be done to calculate the reflected wave height towards the ocean.

6.3.5 Results

With the ReFBreak2D-model it is possible to determine the reaction of a floating breakwater and the transmitted wave height towards the harbour side analytically and quick when compared to a DELFRAC simulation. With the aid of this new model it can be quickly assessed which parameters do influence the performance and to which extent. Parameters which can be varied in the ReFBreak2D-model are:

- The structural width;
- The structural draft;
- The structural mass;
- The location of the centre of gravity;
- The mooring stiffness (only linear and user defined);
- The mooring damping (only linear and user defined).

The model can be used to produce a graph of the RAO, the so-called Response Amplitude Operator, as can be seen in figure 6.10 and figure 6.11 on the next page. This RAO can be used to quickly determine the most optimal floating breakwater for different wave frequencies.

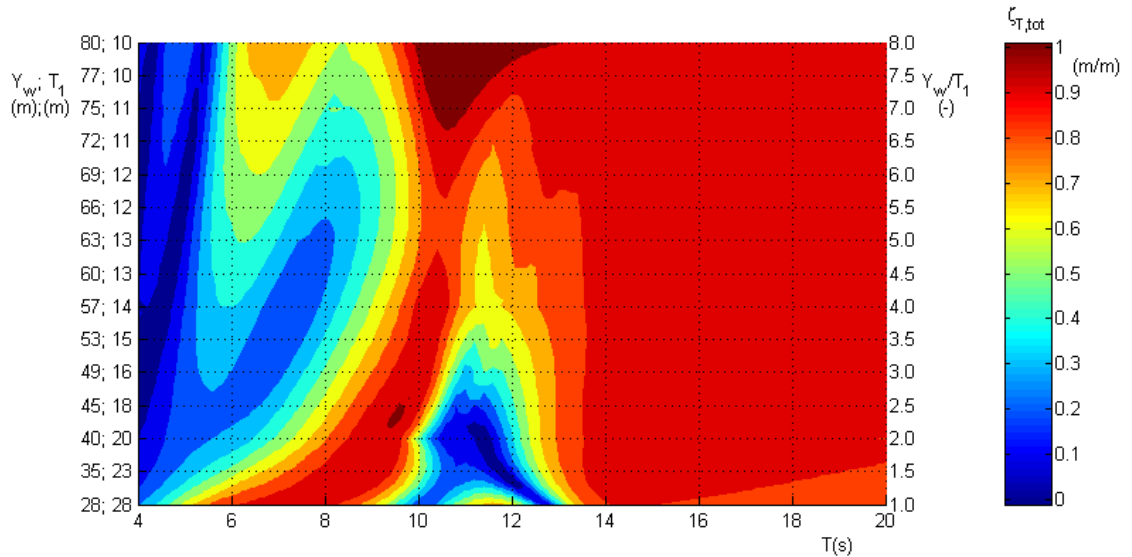


Figure 6.10 An example of the model output. The effect of the beam-to-width ratio (Y_w/T_1 -ratio) on the wave-attenuating performance, with the COG as low as technically feasible

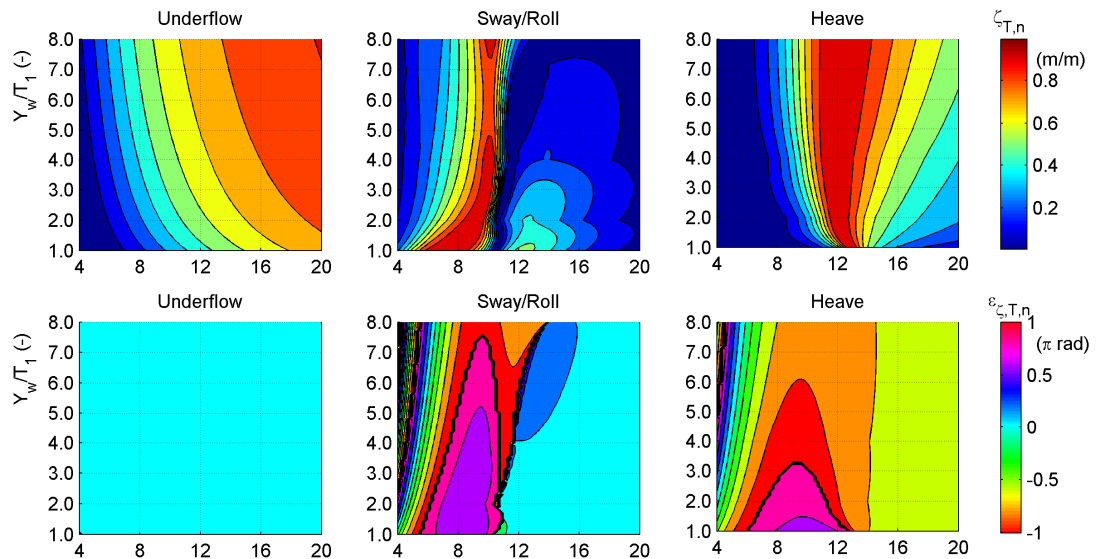


Figure 6.11 An example of the model output. The effect of the Y_w/T_1 -ratio, with the COG as low as technical feasibility or stability permits, on the wave-attenuating properties of the FB, split in different contributions of the separate processes, with their respective phase changes

6.4 Irregular waves

As described in paragraph 5.3.1 waves, especially wind generated waves, are very irregular, but can be seen as a superposition of many simple, regular harmonic waves, each with their own wave characteristics. As every wave component has its own specific energy an energy distribution spectrum can be made. An example of a wave energy spectrum and the way how it is derived can be seen in figure 5.3. A complete description how the transmitted wave spectrum can be derived is found in Appendix D.

The original model was not able to determine the performance of the floating breakwater in an irregular wave field. Because the waves are assumed to be linear, the principle of superposition can be used to solve this. When a wave spectrum is known this can be translated in to a motion spectrum by combining information on the wave spectrum and the behaviour of the structure. This principle is explained in figure 6.12.

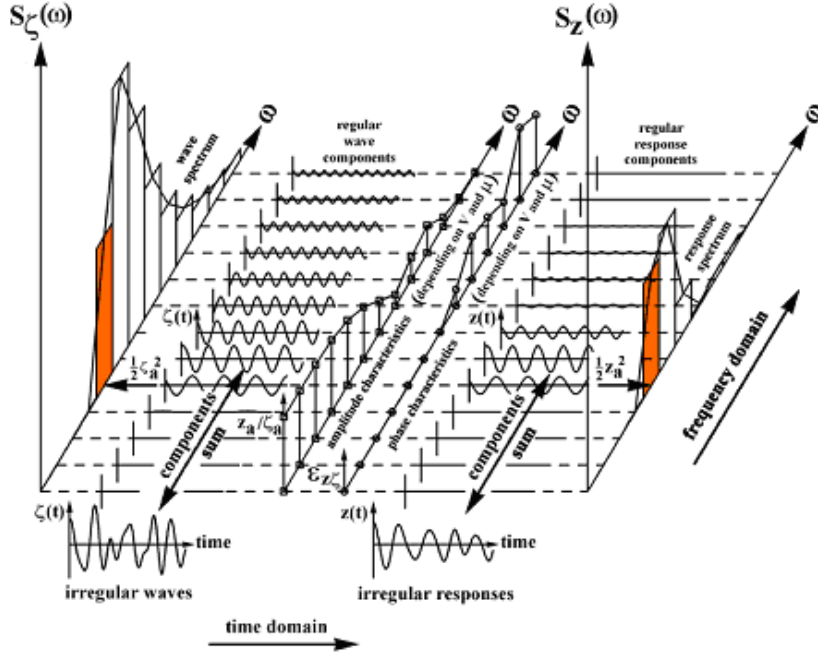


Figure 6.12 The transfer of a wave spectrum into a response spectrum [Journée and Massie, 2001]

The ReFBreak2D-model gives as output amplification factors for the three motions that are considered, i.e. the sway, heave and roll motion. With these amplification factors and the incoming wave spectrum the motion spectrum can be calculated with [Journée and Massie, 2001]:

$$S_z(\omega) = \left| \frac{x_{n,a}}{\zeta_a}(\omega) \right|^2 S_\zeta(\omega) \quad (6.12)$$

Where:

$$\frac{x_{n,a}}{\zeta_a}(\omega) \quad : \text{The transfer function of the motion;} \\ S_\zeta \quad : \text{The wave spectrum.}$$

Analogous to the way the motion spectrum is calculated the transmitted wave spectrum can be obtained as well:

$$S_z(\omega) = \left| \frac{\zeta_{T,a}}{\zeta_a}(\omega) \right|^2 S_\zeta(\omega) \quad (6.13)$$

Where:

$$\frac{\zeta_{T,a}}{\zeta_a}(\omega) \quad : \text{The transfer function of the transmitted wave height;} \\ S_\zeta \quad : \text{The wave spectrum.}$$

6.5 Validation

The ReFBreak2D is validated with an extra DELFRAC-simulation with an Y_w/T_1 -ratio of 5.0. The validation is presented in Appendix E.6 and shows a good agreement between the DELFRAC-simulation and the outcome of the ReFBreak2D-model.

Chapter 7 Parameter analysis

To find the optimal dimensions of a FB system the influence of the parameters is determined with the use of the developed ReFBreak2D-model. An optimal performance is defined as a structure with low transmission coefficients over a broad band of wave conditions. For this first analysis regular waves are considered, for further research on the optimal design irregular waves are used and the effects of the application of a system of breakwaters is assessed.

7.1 Behaviour in regular waves

The behaviour in regular waves is dependant on five parameters:

- the beam;
- the draught;
- the beam-to-draught ratio (Y_w/T_1 -ratio);
- the position of the centre of gravity;
- the viscous roll damping.

With the use of DELFRAC the added mass and hydrodynamic damping coefficients have been determined for several rectangular hull-shapes. The beam-to-draught ratio (Y_w/T_1 -ratio) dictates which coefficients have to be used as it is the coefficient which describes the hull-shape.

The effect of the position of gravity will be discussed first, as this will have a significant effect on the wave-attenuating performance of the FB. Then the effect of the hull-shape, or Y_w/T_1 -ratio, is assessed. Finally the effect of the beam and the draught will be treated, as these are mere scale-factors rather than real independent variables. The viscous roll damping is not treated, as this does not have a large impact on the wave-attenuating performance of the FB.

A complete oversight of the effects of changing the parameters of the FB is given in appendix E. In figure 7.1 it is shown how the graphs in this chapter should be read.

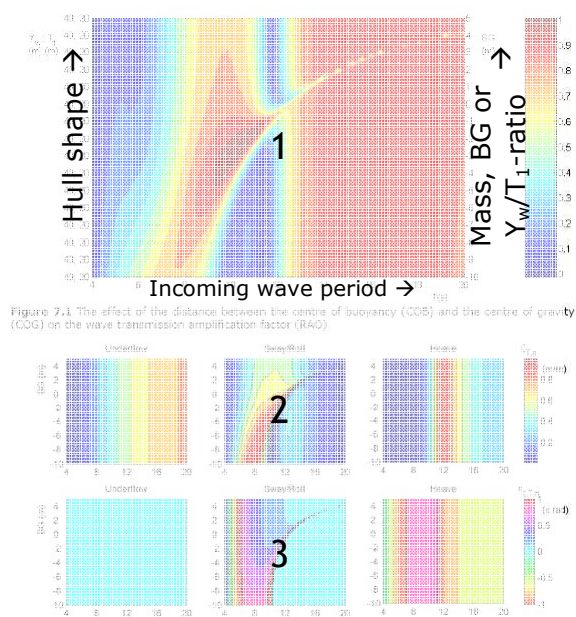


Figure 7.1. The effect of the distance between the centre of buoyancy (COB) and the centre of gravity (COG) on the wave transmission amplification factor (RAO)

Figure 7.2. The effect of the distance between the COB and COG on the wave-attenuating properties of the FD, split in different contributions of the separate processes, with their respective phase changes

1. Wave amplification factor (RAO) of the transmitted wave towards the harbour area. A RAO of 0.1 means a wave transmission of 10%, i.e. a 1 metre high wave is attenuated to 0.1 metre. A lower RAO leads to a more effective design at that wave frequency.

This figure is a summation of the three RAOs, presented in (2), with respect to their phase-changes, presented in (3). See also C.7.4.

2. Transmitted wave RAO due to the modes:
 - a. Underflow
 - b. Sway
 - c. Heave
3. Phase change relative to the incoming waves for the considered modes

Figure 7.1 Example of the presentation of the model results

7.1.1 Effect of the centre of gravity

In the maritime industry ships are said to be tender when they have long roll natural period and stiff when they have a very short roll natural period. A stiff ship is subjected to larger internal forces, possibly leading to fatigue, and its crew is prone to sea-sickness. The roll natural period is mainly determined by the position of its centre of gravity (COG).

In the DELFRAC calculations the centre of gravity (COG) was originally chosen to coincide with the centre of buoyancy (COB), i.e. the distance between the COG and COB, called BG, is zero. The added mass and hydrodynamic damping do not depend on the location of the centre of gravity, they can be rewritten to any position of the COG, see also Appendix C.3.3. By choosing a different location for the COG, i.e. varying BG, the behaviour of the FB can be changed.

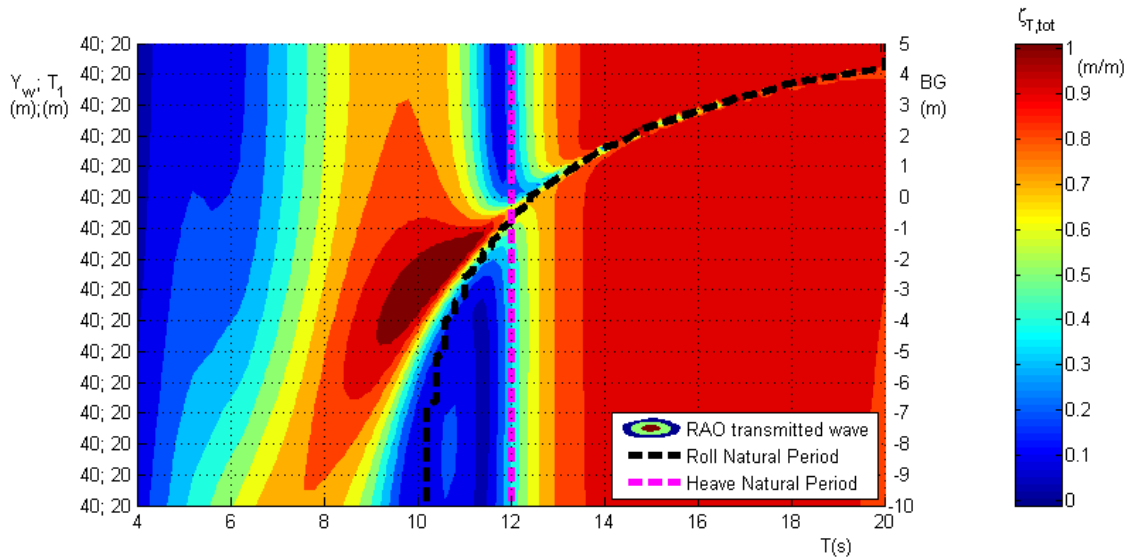


Figure 7.2 The effect of the distance between the centre of buoyancy (COB) and the centre of gravity (COG), called BG, on the wave transmission amplification factor (RAO)

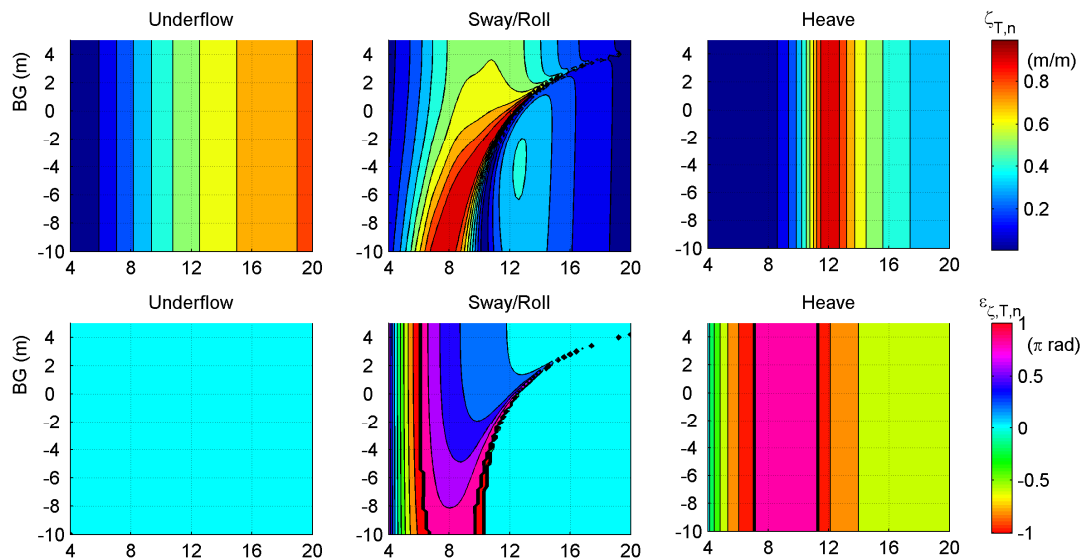


Figure 7.3 The effect of the distance BG on the wave-attenuating properties of the FB, split in different contributions of the separate processes, with their respective phase changes

From figure 7.2 it can be seen that the originally chosen location of the COG ($BG = 0$) results in an ineffective design of the FB. In the case of $BG = 0$ the roll natural period coincides with the heave natural period. When the centre of gravity

is located higher the roll natural period becomes larger and a tender design is obtained. In the opposite case, when the COG is located lower, the roll natural period becomes smaller and a stiff design is obtained. In both cases the roll natural period does not coincide with the heave natural period and the roll motion does not affect the wave-attenuating performance.

From this it can be concluded that:

- the roll natural period can be tuned by shifting the location of the COG;
- the roll movement of the FB does have a negative impact on the wave-attenuating performance of the FB;
- the COG should be as low as possible or as high as possible to prevent roll natural period to be equal to the heave natural period;

7.1.2 Effect of the hull-shape

The hull-shape does have an influence on the wave-attenuating performance of the floating breakwater as each hull-shape, in this case all rectangular with different Y_w/T_1 -ratios, will have different hydrodynamic parameters. The processes that govern the wave transmission are:

- wave generation by the combined roll/sway motion;
- wave generation by the heave motion;
- underflow.

The mass of the structure is assumed to be an important factor in the cost of the total design. Therefore the optimal hull-shape should be determined with structures that are comparable in mass.

Structures with a high Y_w/T_1 -ratio will have a small draught and thus the underflow will cause a significant wave transmission. However the structure will not experience a big excitation in sway and roll as it is very stable, but also the heave motion will be limited. Structures with a low Y_w/T_1 -ratio will have a large draught and thus a limited wave transmission by the underflow phenomena. This kind of structure will be subjected to large sway and roll motions which have a negative impact on the wave attenuation.

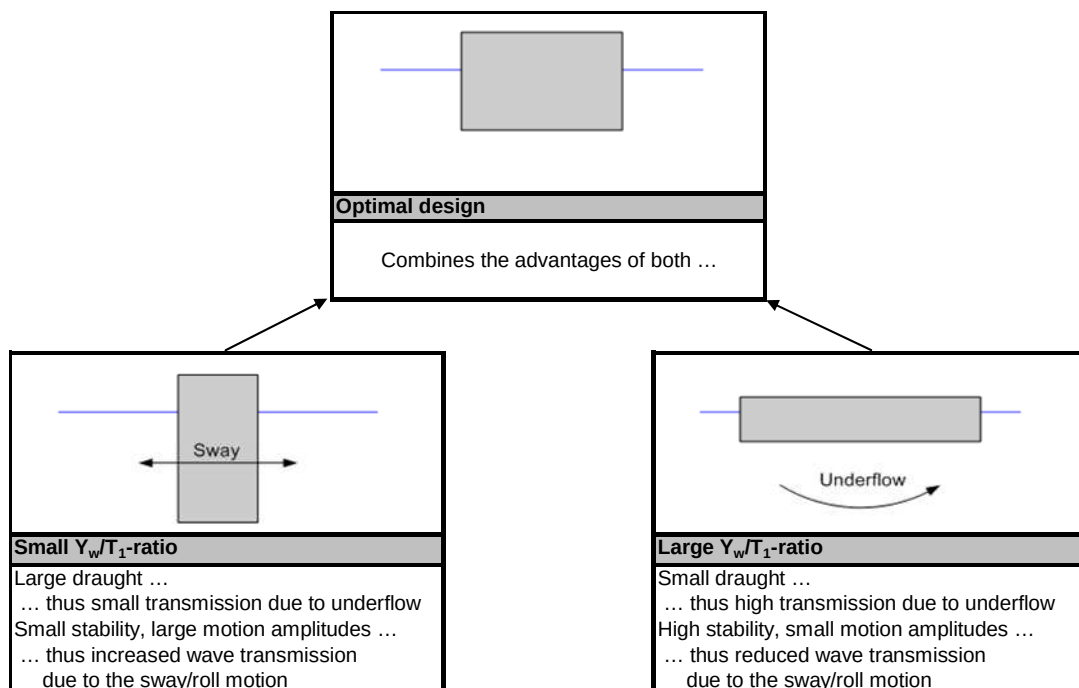


Figure 7.4 The most optimal design combines the advantages of a small and a large Y_w/T_1 -ratio

The most optimal design for a FB will be between the two extremes, which can also be seen in figure 7.5 and figure 7.6. This leads to an optimal design for a FB, i.e. a FB which is able to attenuate wave in a broad range of frequencies, which has an Y_w/T_1 -ratio of 5.0.

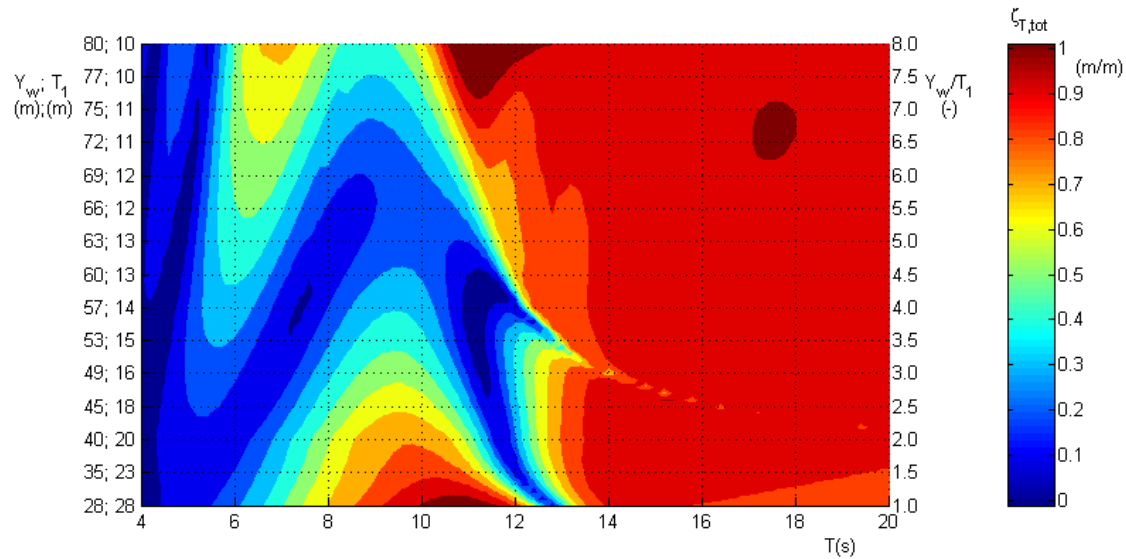


Figure 7.5 The effect of the beam-to-width ratio (Y_w/T_1 -ratio) on the wave-attenuating performance, with the COG as high as technical feasibility or stability permits, on the wave transmission RAO

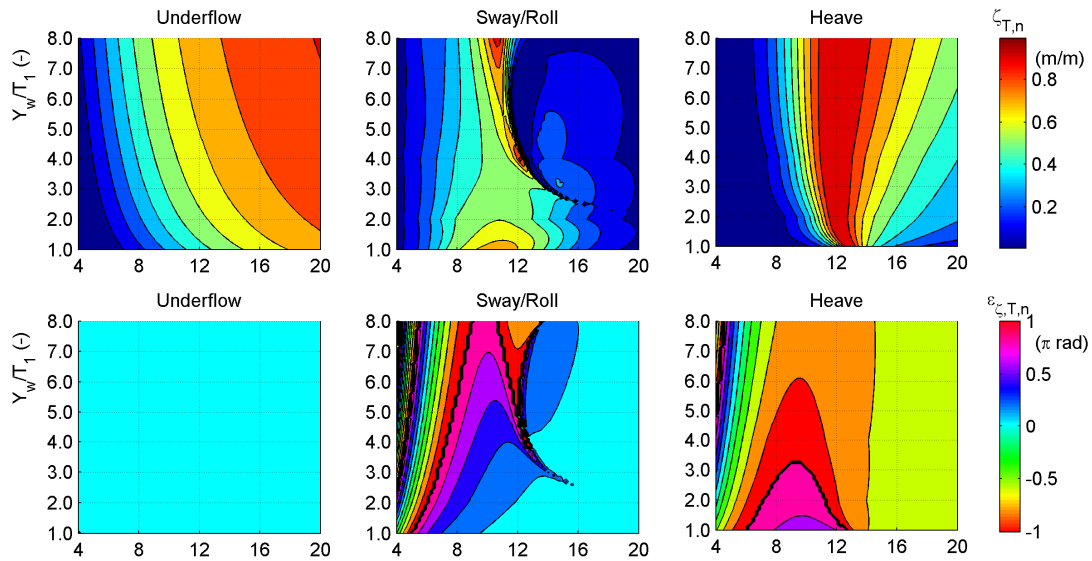


Figure 7.6 The effect of the Y_w/T_1 -ratio, with the COG as high as technical feasibility or stability permits, on the wave-attenuating properties of the FB, split in different contributions of the separate processes, with their respective phase changes

When the COG is placed as low as is technically feasible a very stiff structure is obtained with a roll period in the order of 8 to 10 seconds. Due to a large structural width a FB with a high Y_w/T_1 -ratio will be more stable and will have a higher roll natural period. This leads to that the heave natural frequency coincides with the roll natural frequency, which eliminates all positive effect of the heave motion on the wave transmission, which can be seen in figure 7.7 and figure 7.8 on the next page.

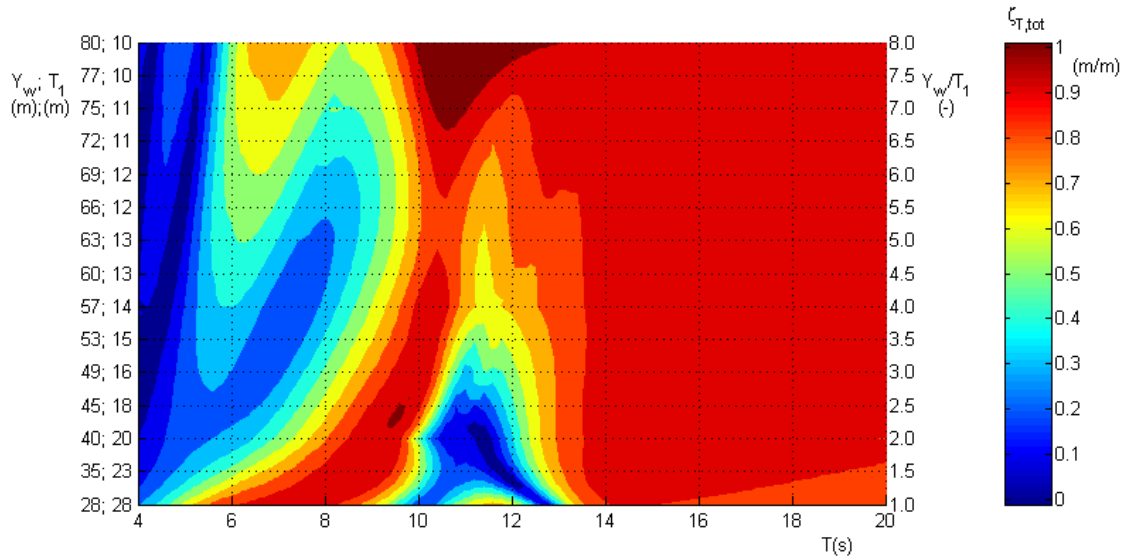


Figure 7.7 The effect of the beam-to-width ratio (Y_w/T_1 -ratio) on the wave-attenuating performance, with the COG as low as technically feasible, on the wave transmission RAO

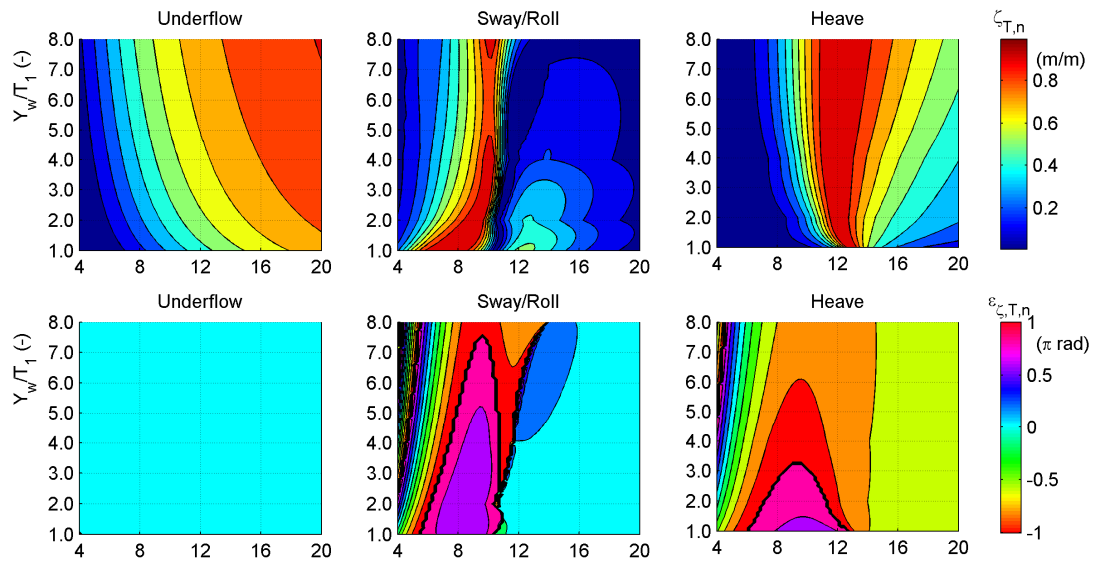


Figure 7.8 The effect of the Y_w/T_1 -ratio, with the COG as low as technical feasibility or stability permits, on the wave-attenuating properties of the FB, split in different contributions of the separate processes, with their respective phase changes

From this it can be concluded that:

- The FB should have Y_w/T_1 -ratio of 5.0 to have an optimal wave-attenuating performance on a wide number of frequencies;
- A tender FB, or a FB with a high centre of gravity, has less wave transmission than a stiff FB with a low COG.

7.1.3 Effect of the beam and draught

The effect of a change in the beam and draught of the FB with an Y_w/T_1 -ratio that is constant is a mere scale effect. The hydrodynamic parameters are scaled from the dimensionless parameters as described in [Vugts, 1968] with respect to the beam and draught of the element. The element is scaled with the use of Froude.

This results in the following scale-factors:

- Length : X ;
- Area : X^2 ;
- Volume : X^3 ;
- Force : X^3 ;
- Time : $\sqrt{X}$;
- Speed : $\sqrt{X}$;
- Acceleration : 1.

From figure 7.9 on the next page it can be seen that to attenuate waves with a period twice as large the FB has to be four times as large. The wave-attenuating properties are not scaled, they are only shifted.

From this it can be concluded that actual size of the elements is dictated by the longest waves. When the allowed wave transmission is 40 percent for a wave with a period of 14 seconds, one can see that the required beam is at least 85 metre with a draught of 17 metres.

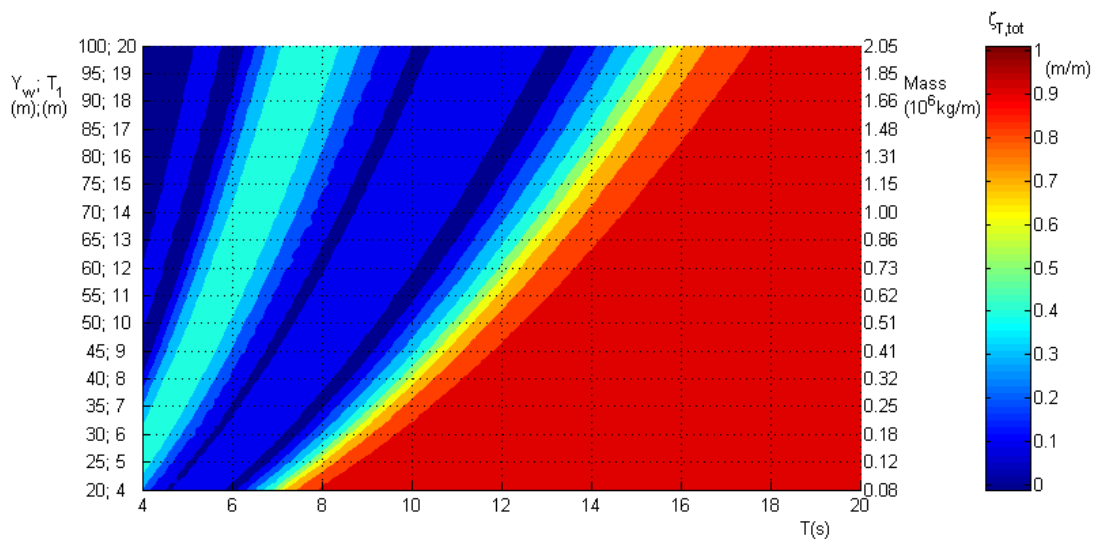


Figure 7.9 The effect of the beam and draught on the wave-attenuating properties, $Y_w/T_1 = 5.0$ and the COG is as high as assumed to be technical feasible, on the wave transmission RAO

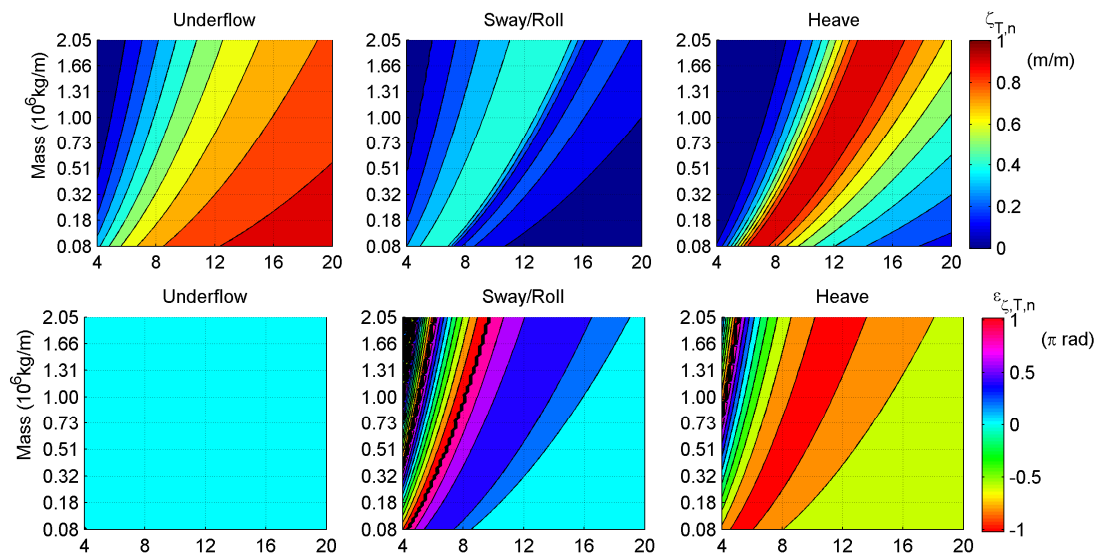


Figure 7.10 The effect of the scale of the structure, with the COG as high as technical feasibility or stability permits, on the wave-attenuating properties of the FB, split in different contributions of the separate processes, with their respective phase changes

7.1.4 Conclusion

From the analysis of the wave-attenuating properties in regular waves it can be concluded that:

- it is possible to attenuate waves with a floating object;
- a floating breakwater with a Y_w/T_1 -ratio of 5.0 is the most effective in attenuating waves within a broad band of frequencies;
- the COG should be located as high as possible to avoid the negative influence of the combined sway/roll motion on the wave-attenuating character;
- the longest wave that has to be attenuated dictates the size of the elements.

7.2 Behaviour in irregular waves

In the previous section it has been concluded that a floating breakwater can actually work when regular waves are considered. However in real ocean conditions there is no single wave frequency, but there is a spectrum. When the FB is only capable in attenuating waves in a small band it will not work.

First the boundary condition and requirements are discussed. The floating breakwater has to mediate between the boundary condition imposed by the ocean and the requirements that come from the floating harbour area. This will lead to a maximum RAO of the FB. With this maximum RAO the minimum dimension of an element can be determined

7.2.1 Mediating between boundary condition and requirement

In 'Chapter 5 Boundary conditions and requirements' it was assumed that all harbour action will be stopped when wind with a force of 8 or higher on the scale of Beaufort is present. During this situation the waves do not have to be attenuated, thus the service limit state is Beaufort 7. In open ocean areas a Bretschneider wave spectrum can be used to describe the motions of the ocean surface. The data for the wave spectra up to Beaufort 7 is given in table 5-3 and figure 7.9

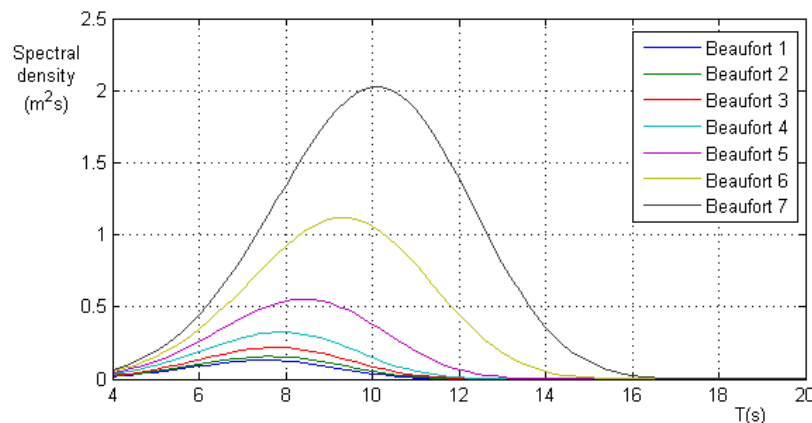


Figure 7.11 Wave spectra for Beaufort 1 up to Beaufort 7

The wave spectrum which can be allowed in the harbour can be calculated from the requirement that the wave height should not exceed 0.4 metres in the harbour during operation. The transformation of a wave height into a wave spectrum is described in Appendix D.1. In figure 7.12 this wave spectrum is shown. This spectrum is approximation because the difficulty of relating the ship motions to a limiting wave height. The determination of this relation is considered to be out of the scope of this master thesis.

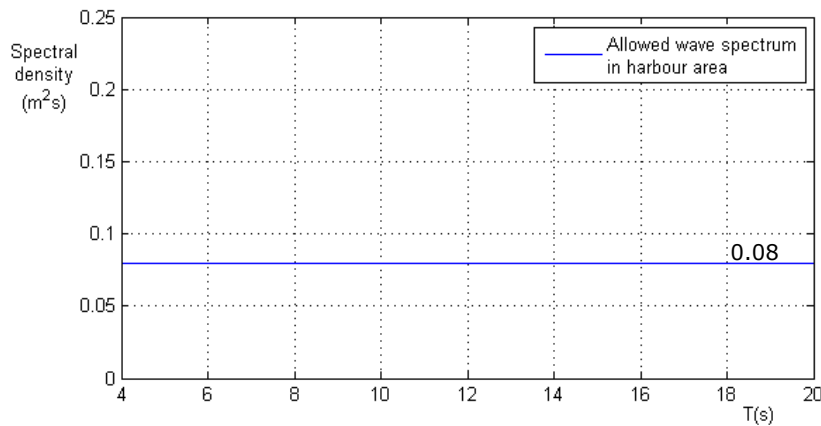


Figure 7.12 The allowed wave spectrum in the harbour area

From the incoming wave spectrum as shown in figure 7.11 and the maximum allowed wave spectrum in the harbour as shown in figure 7.12 the minimum wave-attenuating performance of the floating breakwater can be determined. This minimum is given in figure 7.13 on the next page. It can be seen that the limiting case is Beaufort 7, where a maximum wave transmission of 20% is allowed for wave periods of 8 up to 12 seconds.

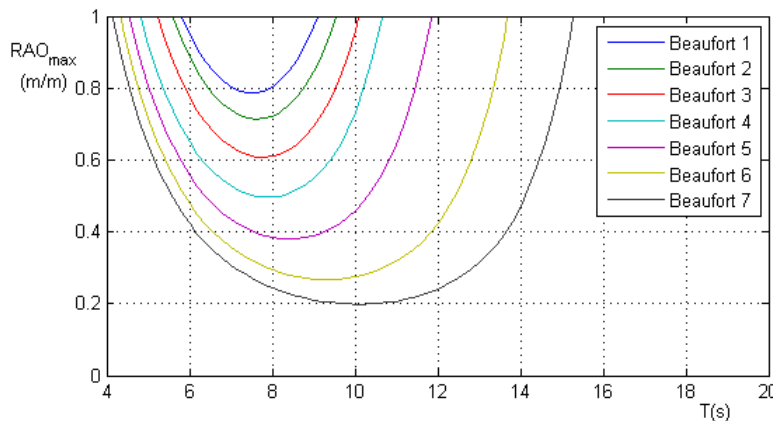


Figure 7.13 Maximum allowed wave transmission

7.2.2 Minimum dimensions

In the previous subsection the maximum allowed wave height is determined. It has already been concluded that the most optimal floating breakwater, i.e. a FB that can attenuate waves in a wide spectrum, has a Y_w/T_1 -ratio of 5.0. In figure 7.14 the wave-attenuating properties of different sized FB elements are given with respect to the boundary condition.

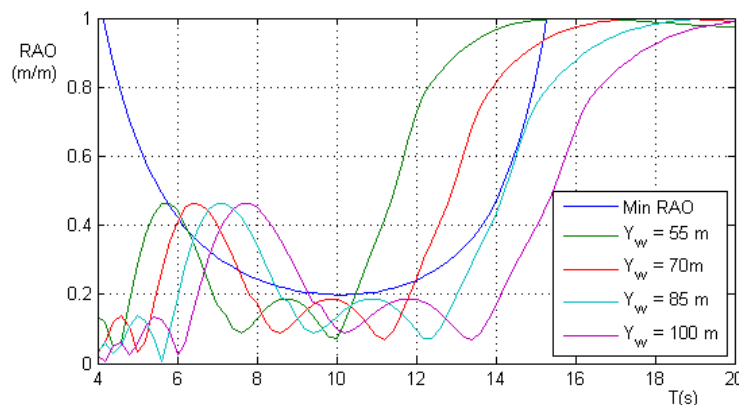


Figure 7.14 Wave attenuating properties of a FB with Y_w ranging from 55 metre to 100 metre

From figure 7.14 it can be seen that the wave-attenuating properties are in wave periods ranging from 6 to 8 seconds. These waves will hamper the harbour action as these wave periods are similar to the estimate heave natural wave periods of the ships moored, see also table 5-2.

There is no way of avoiding this peak in the wave transmission with a single floating breakwater, except increasing the size considerably. The size that is needed would be in the order of the size of the FTCT.

7.3 Multiple floating breakwaters

When a single FB is unable to attenuate waves in the whole frequency range of the incoming wave spectrum a system of floating breakwaters can be considered. In a system not only the wave transmission will be important but the wave reflection as well. The first FB will react to the incoming waves but also on the waves reflected and generated by the other FB as well.

First a simple hypothesis behind transmission and reflection will be discussed. It is assumed that the wave transmission calculated with this hypothesis is an upper limit, as no positive influence of the reflective waves is taken into account. In future research this hypothesis should be verified with a DELFRAC simulation, or a simulation with another model, where the hydrodynamic coupling of the two floating bodies is taken into account.

Second the wave attenuating properties of a system of FBs, calculated with the hypothesis, will be tested against the boundary condition.

7.3.1 Transmission and reflection

When the incoming waves hit the first floating breakwater a part of the wave will be transmitted towards the lee side of the element and a part will be reflected back to the seaside. The processes that govern the transmission and the reflection are given in table 7-1.

Table 7-1 Processes governing the reflection and transmission of the waves

Reflection	Transmission
Waves generated by the sway/roll motion	Waves generated by the sway/roll motion
Waves generated by the heave motion	Waves generated by the heave motion
Reflected wave against hull	Underflow

The part of the wave that is transmitted towards the lee-side is the part of the wave that has to be attenuated by the second floating breakwater. The same processes as with the first FB will take place, but the waves will not be reflected towards the ocean but towards the first FB. This first FB will react to the waves that are reflected by the second floating breakwater.

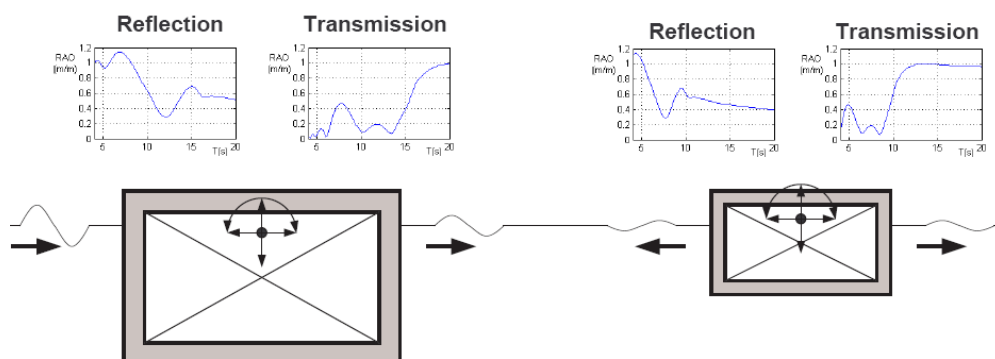


Figure 7.15 Wave transmission and reflection, an iterative process

In its turn the first FB will transmit a part of the reflected waves towards the ocean, this part of wave energy leaves the system. Another part however is reflected back towards the second FB again, which will react again and so on. This iterative effect is shown in figure 7.15. The amount of energy that is contained in one wave which is reflected back and forth is reduced, so a limited number of iterations are required to obtain the total transmitted wave towards the harbour area, see figure 7.16.

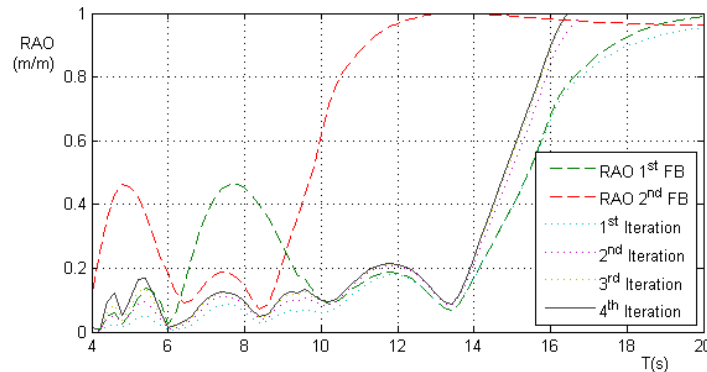


Figure 7.16 An example of the wave transmission of two FBs with the converging iterations

The distance between the floating breakwaters is of importance, as the reflected wave can have a positive or negative effect on the motions and thus on the waves that are being produced. The reflected wave of the second breakwater will cause the first breakwater to move and generate waves. These motions and generated waves have to be added, on the basis of superposition, to the motions and generated waves due to the incoming waves from the ocean.

The addition of the two generated waves, i.e. the generated wave due to the incoming wave and due to the reflected wave, can lead to a lower transmitted wave height towards the harbour area when they are in anti-phase. When the two generated waves are in phase with each other the transmitted wave is amplified instead of damped.

The wave lengths where the positive effect on the wave attenuating performance of the FB system is present differ with each wave period. Therefore it can not be relied on in a wave spectrum, consisting of several wave periods. Also due to low frequency motions, caused by the drift forces, it is virtually impossible to maintain the desired distance between the two FB elements.

Therefore it is assumed that reflection causes only a negative effect on the wave-attenuating performance. The transmission coefficients obtained in this way are assumed to be maximum coefficients; depending on the wave period and actual distance between the two FBs a lower transmission coefficient may be obtained.

7.3.2 Dimensions of the system

In figure 7.17 it can be seen that a system of two floating breakwaters is needed to fulfil the requirements. The dimensions of the floating breakwaters are stated in table 7-2. It is of great importance that the COG of both FBs is located as high as possible. A low COG will lead to an ineffective design.

Table 7-2 Required dimensions of the system of floating breakwaters

Parameter	1 st FB	2 nd FB	Unit
Y_w/T_1 -ratio	5	5	[-]
Beam	100	40	[m]
Draught	20	8	[m]
BG	>5	>2	[m]
Displacement	2000	320	[m ³ /m]
Mass	2050	328	[ton/m]

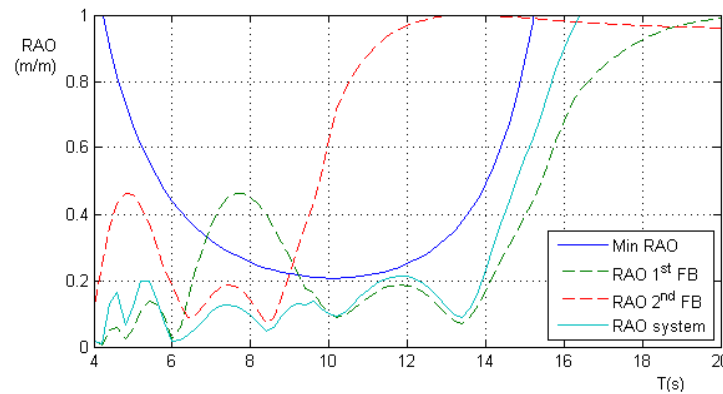


Figure 7.17 The RAO's of the individual breakwaters and of the combined system

7.4 Conclusion

A floating breakwater can attenuate regular waves with any wave period as long as its heave natural frequency is tuned to coincide with the incoming wave frequency. The Y_w/T_1 -ratio is not of any importance to this behaviour, but the COG is. In order to obtain optimal wave attenuation the COG should be as low as possible or as high as possible. This is to avoid the roll natural frequency to be the same as the heave natural frequency.

For an optimal design in irregular waves the range of frequencies where the wave transmission is in the order of 20% should be as wide as possible. This behaviour can be obtained by designing the FB element with an Y_w/T_1 -ratio of 5 with the COG located as high as possible, see also figure 7.5.

Despite the wide range of frequencies the performance of the FB in irregular waves is insufficient. A system consisting of two FBs is proposed. The first FB attenuates all low frequency wave components. The second FB is tuned for an optimal performance in the frequency range where the wave transmission of the first is too high. The dimensions of the system that fulfils the requirements are listed in table 7-2.

Chapter 8 A survey of solutions

In the previous chapter an analysis has been given on the minimal required dimensions of the floating breakwater. Although these main dimensions may be fixed, there is still enough room for alternatives. The parameters that are still undefined are:

- *The material;*
- *Station keeping;*
- *The connections between elements;*
- *The element section length;*
- *The division into sections.*

These parameters are discussed in this chapter. Several variants are presented with the goal to produce an alternative which is elaborated further. First structures of comparable sizes that have been built in the past are discussed.

8.1 General

Recently three large scale floating structures have been built:

- The N’Kossa concrete barge;
- The Monaco floating breakwater;
- Mega-Float phase 2.

These constructions will be discussed first because they have a size comparable to that of a FB element. Points that will be stressed are the use of materials, safety against flooding of compartments and construction method to stress the possible challenges in the design of the FB elements.

8.1.1 N’Kossa concrete barge

The hull of the N’Kossa barge is a rectangular shape, 220 metres long, 46m wide and 16m high. The bottom is 45cm thick, the sides 50cm and the deck 40cm thick. All concrete elements are highly pre-stressed. The hull is stiffened by a series of bulkheads: a central bulkhead with openings, two watertight longitudinal bulkheads on either side of the central bulkhead and 12 transverse watertight bulkheads¹.

The N’Kossa barge is a Floating Production and Storage Unit (FPSU) and is used on the N’Kossa oilfield off the coast of Congo. The water depth where it is moored is 150 metres.



Figure 8.1 The N’Kossa barge under construction in a dry-dock

¹ France Concrete barge set for 30-year assignment on N’Kossa
http://www.offshore-mag.com/articles/save_screen.cfm?ARTICLE_ID=23204

It was built in one piece in a dry-dock in Marseille (France) and towed to its location in Congo. To speed up construction the bulkheads were pre-fabricated and the hull was made by using a slipform, see figure 8.1. This barge is the largest concrete barge built at this moment.

8.1.2 Monaco floating breakwater

The hull of the Monaco floating breakwater has a rectangular shape, 352 metres long, 28 metres wide and 19 metres high. On the sides of the breakwater there are two 8 metres wide concrete wings, bringing the total submerged width to 44 metres. The hull is designed as a double shell and is entirely made out of pre-stressed concrete and has a design-lifetime of 100 years.²



Figure 8.2 The dry-dock in which the Monaco floating breakwater was built

It is built in a dry-dock near La Linea in Spain. The dock had the following dimensions 400 metres long and 60 metres wide and 15 metres deep. When the construction was completed, the breakwater was brought afloat by flooding the basin. The tow to its installation site took four weeks. At the site the structure was ballasted to the required draught and anchored with chains to pre-installed anchors.

8.1.3 Mega-Float

The Mega-Float phase 2 involved the construction of an airport model, consisting of a runway with a length of 1000 metres, a width of 60 metres and a draught of 3 metres. This model is tested with planes taking off and landing on the runway prior to designing of the full scale Mega-Float with a runway length of 4000 metres.

The hull of is made of steel and is divided in compartments as a provision for a collision with a ship. The platform is moored by the means of mooring dolphins and is located in a bay with water depths up to 30 metres.



Figure 8.3 Construction method of the Mega-Float structure. Fabrication in a shipyard (a), towing out to the site (b), work on the mooring dolphins (c) and connecting the elements at sea (d)

² La Digue Semi-Flottante
<http://www.gouv.mc>

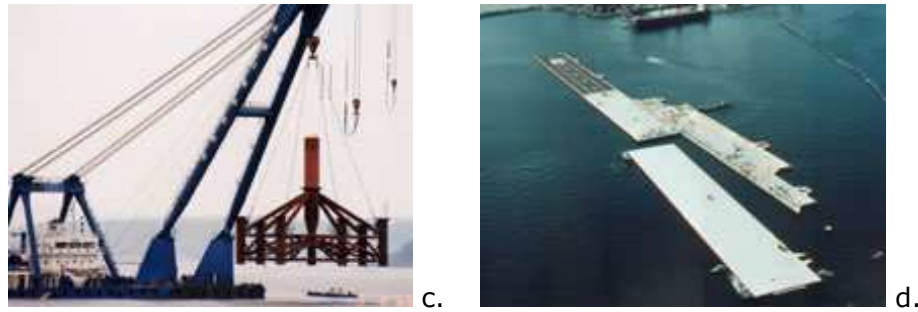


Figure 8.3 (continued) Construction method of the Mega-Float structure. Fabrication in a shipyard (a), towing out to the site (b), work on the mooring dolphins (c) and connecting the elements at sea (d)

The structure has been built in a conventional shipyard in units, each 300 metres long and 60 metres wide. These dimensions were determined during the design as the maximum class for a conventional ship, only a few shipyards are able to manufacture larger vessels. After completion in the dry-dock the elements are towed to the installation site one by one and joined together at sea. Although the Mega Float phase 2 is located in a sheltered bay, relative movements are inevitable. See figure 8.3a-d for photos of the construction of Mega Float phase 2³.

8.2 Materials

The principal materials for offshore structures are steel and concrete. These must perform in a harsh environment. They are subjected to corrosive and erosive processes, cyclic loading of waves and impact conditions. Large temperature differences may occur over the life-time of the structure.

Another type of material that is often used in the shipping industry, especially the yachting industry, are composites, however concrete is strictly speaking also a composite.

The quality of fabrication is critical in order to assure the structure to perform properly under both service and ultimate limit state. Cracks tend to propagate relatively fast due to the cyclic loading and the corrosive action of the ocean. They can grow into serious problems when the structure is not detailed or fabricated properly.

8.2.1 Steel

Steel is a ductile material, i.e. it will undergo a large elastic deformation before it yields and will undergo a large plastic deformation before it breaks, see figure 8.4 on the next page. The material is well suited to take up tensile loads. Under compression the ultimate strength of the material is seldom reached as the element will buckle or fail by another instability phenomena.

Steel has been used for many years in the shipping industry. All commercial ships are fabricated from this material as it results, due to its high strength, in an economical design with high carrying capacities of the ships.

³ Mega-Float floating airport - Construction
http://www.srcj.or.jp/html/megafloat_en/data/da_index.html

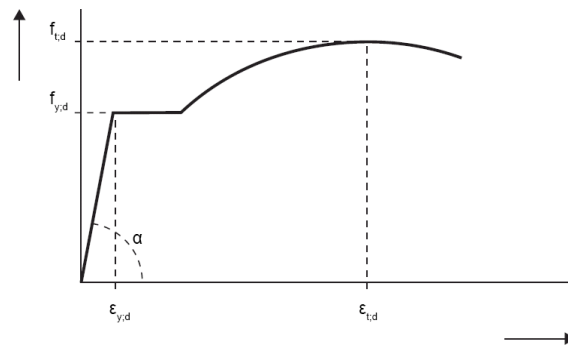


Figure 8.4 Stress-strain diagram for steel

The material has an excellent fatigue resistance if the details, i.e. the welds, are designed and produced properly. In contrast to concrete, steel will react with the environment in the form of rust. Therefore the structure has to be equipped with a protective coating, sacrificial anodes or impressed current cathodic protection [Gerwick, 1986]. The first two methods require maintenance in the form of replacing and repairing the coating and the dissolved anodes. The last method, impressed current, does have the risk of hydrogen generation, which can cause embrittlement of the steel and is dangerous in confined spaces. [Chakrabarti, 2005]

All characteristics of steel are summarised in table 8-2.

Table 8-1 Steel characteristics

Type		S235	S275	S355	S460
E-modulus	E_b	210.000 N/mm ²	210.000 N/mm ²	210.000 N/mm ²	210.000 N/mm ²
Design yield strength	$f_{y,d}$	235 N/mm ²	275 N/mm ²	355 N/mm ²	420 N/mm ²
Design tensile strength	$f_{t,d}$	360 N/mm ²	430 N/mm ²	510 N/mm ²	-- N/mm ²
Yield limit	$\epsilon_{y,d}$	1,12 %	1,31 %	1,69 %	-- %
Volumic mass	ρ	7850 kg/m ³	7850 kg/m ³	7850 kg/m ³	7850 kg/m ³
Fatigue resistance		When detailed properly, good			

8.2.2 Concrete

Concrete is a stony material, i.e. it is very strong under compression but relatively weak under tension. The failure mechanism of concrete may be considered as brittle, allowing only a small deformation before it gives away, see also figure 8.5. To overcome these shortcomings it is almost always used in combination with steel (pre-stressed) reinforcement. Reinforced concrete combines both the advantages of steel and concrete.

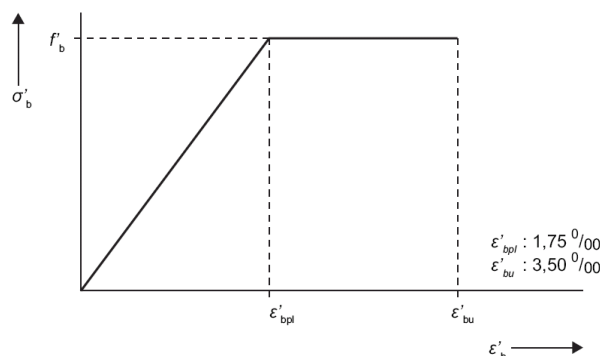


Figure 8.5 Stress-strain diagram for concrete

Reinforced concrete and pre-stressed concrete has been used for some 20 off-shore platforms in the North-Sea and has also been used in Arctic platforms. Although it has been used for some yachts as building material, it is not used as a building material in the shipping industry due to its high structural weight. This high structural weight would limit the carrying capacity of the ships too much.

Reinforced concrete and especially pre-stressed concrete shows excellent fatigue endurance in the air, as long as the concrete is not cycled in its tensile range more than half its static tensile strength, nor in its compressive range more than half its compressive strength. When submerged conventional concrete shows a reduction of fatigue resistance. This is possibly due to the high pore pressures in micro-cracks, which are opened and closed constantly due to the cyclic loading.

To avoid cracks in the concrete, and thus a deterioration of the fatigue resistance, the design of the pre-stressing force can be adapted so that tensile forces will not occur during the structures life-time. The bending capacity of the elements will not be reduced in a great extent due to this method because of the low tensile capacity of concrete.

All characteristics of concrete are summarised in table 8-2.

Table 8-2 Concrete characteristics

Type		B45	B55	B65	B75
E-modulus	E_b	33500 N/mm ²	36000 N/mm ²	38500 N/mm ²	40000 N/mm ²
Char. Compression strength	$f_{c,k}$	45 N/mm ²	55 N/mm ²	65 N/mm ²	75 N/mm ²
Design compression strength	f_b	27 N/mm ²	33 N/mm ²	39 N/mm ²	45 N/mm ²
Design tensile strenght	f_b	1,65 N/mm ²	1,9 N/mm ²	2,25 N/mm ²	2,4 N/mm ²
Volumic mass	ρ	2400 kg/m ³	2400 kg/m ³	2400 kg/m ³	2400 kg/m ³
Fatigue resistance		When not loaded more then half the range of the static strength, good			

8.2.3 Composite material

Composite material, polyester, have been used in yachting to produce boats which are strong, yet light-weight. The weight saved onboard these boats by the use of these high-tech materials can be used as ballast in the keel to get a ship which is as stable as possible. With large commercial vessels there is very little to no experience on the use of this material.

Weight is not an issue for a floating breakwater when one looks at similar structures such as submersed tunnels. These structures are transported to their location afloat despite the use of heavy materials such as concrete. The performance of the FB depends on its draught and location of the COG and thus on its mass and mass distribution. When this fact is taken in account one can conclude that the light-weight composite materials, for example in the form of polyester, is not a suitable material for a floating breakwater.

8.2.4 Selection

It has already been concluded that composite materials like polyester are not a suitable material for a structure with the magnitude of an FB element. The choice is therefore limited to the use of concrete or steel.

The main disadvantage of steel is the amount of maintenance that is required to prevent corrosion. Concrete does not wear due to the presence of water as long as the concrete remains under compression. When cracks form due to tensile stresses, water ingress into these cracks will diminish the fatigue resistance significantly.

The main reason why concrete is not used in the shipping industry is the weight of the concrete that is needed in a cross-section of a ship, reducing the carrying capacity of the ship. This disadvantage is not applicable to a floating breakwater as this structure does not need a carrying capacity. Therefore concrete is chosen as the preferred material.

8.3 Station keeping

It is essential that the FB has a fit to purpose mooring system. The mooring system traditionally consists of lines connecting the structure to anchors, deadweights or piles at the bottom. It can also consist of a dynamic positioning system with thrusters. The types of mooring that are discussed in this section are:

- Catenary line mooring
- Taut line mooring
- Tension leg mooring
- Dynamic position

These types of moorings all come with their own (dis-)advantages. They can also be combined to overcome some of the problems, for example dynamic positioning can be used to decrease the loads on the traditional line moorings. These solutions are also discussed.

8.3.1 Catenary line mooring

This is the oldest and still the most common mooring system, see figure 8.6a. It derives its restoring force primarily by lifting and lowering the weight of the mooring line, at least in a static sense. Catenary line mooring is a hard spring system and it is non-linear, i.e. the force increases more than directly proportional to the displacement, see figure 8.6. To ensure that the anchors are kept in position it is necessary that a significant part of the anchor line lies on the sea bed. [Journée and Massie, 2001]

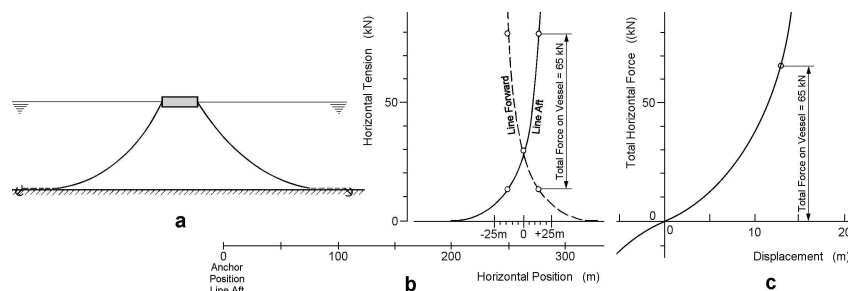


Figure 8.6 Horizontal forces on a FB as function of horizontal displacements [Journée and Massie, 2001]

8.3.2 Taut line mooring

This mooring system has a pattern of taut, light-weight (synthetic) lines radiating outwards. The catenary action has been eliminated due to the low submerged weight of the mooring lines. The system gets its restoring force as a function of horizontal displacement primarily from elastic stretch of the line itself. The anchors, which can be either conventional anchors, deadweight or suction piles, have to provide either enough weight or holding power to counteract the tensile forces in the mooring lines. Synthetic fibres are the most common for this type of mooring. [Journée and Massie, 2001]



Figure 8.7 Seafloor footprint of a catenary mooring (left) and a taut line mooring (right)⁴

⁴ Mooring concepts - <http://www.offshoreengineering.org>

Taut-line mooring has significant advantages over a catenary line or chain or wire because of the reduced weight. These ropes can stretch 1.2 times up to 20 times as much as steel. The stiffness of synthetic ropes however depends with the load range and the mean load and is not constant over the life-time of the rope. Due to their flexibility they can absorb dynamic motion through extension instead of lifting and lowering the weight of the mooring line.

Additional advantages include the smaller footprint on the seabed, lower tensions in the fairlead and a smaller vertical load on the vessel. This lower vertical load on the vessel results in a smaller limitation of the heave motion, which has a positive effect on the wave-attenuating performance of the FB.

The disadvantage of using synthetic fibre lines is that their material and mechanical properties are more complex and not as well understood as the traditional wires and chains. There is only limited in-service experience as well. This leads to a design which will be over-conservative. This can annihilate some of the advantages. When synthetic lines were subject to dynamic loads, the limited in-service experience also led to designs with very large factors of safety.

8.3.3 Tension leg mooring

This special mooring system is normally used for tension leg platforms, see figure 8.8a, which are used in the offshore oil industry to perform as a stable platform for several applications. The buoyancy of the platform exceeds its weight and a net downward force is supplied by the vertically tensioned mooring, secured by deadweight or anchor piles [Journée and Massie, 2001].

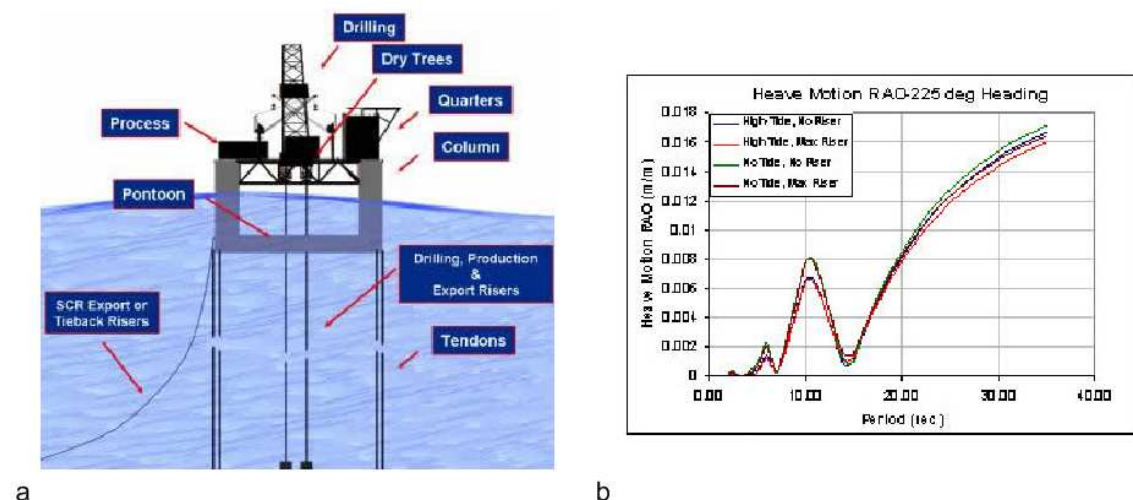


Figure 8.8 Example of a TLP (a) and the RAO for heave for a TLP (b)⁵

The mooring lines provide a total restraint against vertical movements of the structure; the vertical amplitude for a TLP is in the order of millimetres, see figure 8.8b. This limits the positive contribution of the heave motion to the wave-attenuating performance of the FB. When the structure is horizontally displaced from its equilibrium position the restoring force comes from the horizontal component of the mooring line tension. Horizontal motions are therefore still possible.

Due to the dynamic behaviour of the tension leg mooring this is considered as not appropriate for a solution for the station keeping of a FB.

⁵ <http://www.floatec.com/tlp.htm>

8.3.4 Dynamic positioning

Dynamic positioning has been applied first in 1961 on the 'Eureka', a semi-submersible built by Shell Oil Company. Since the first application the dynamic positioning has been improved, resulting in an automatic, computer controlled system with multiple redundancies. The present day mean time between failure (MTBF) is about three years for the best systems. [Journée and Massie, 2001]

The application of dynamic positioning is limited to vessels which can not be moored with a conventional mooring system, either due to the water depth at the working site or the need to reposition the vessel on a regular basis. The downside of the system is its energy demand because engines are needed to drive the thrusters.

When dynamic positioning is applied to the FB concept some other draw-backs, beside the energy consumption, have to be taken into account. The thrusters will cause jets around the element, possibly causing hindrance to the moored ships.

8.3.5 Combination: Line mooring with dynamic positioning

Dynamic positioning thrusters are sometimes used to assist catenary line mooring or taut line mooring rather than replacing them. The thrusters are used to relieve the maximum line tension during design conditions and damps out oscillations. The station keeping is done primarily by the line mooring.

In normal operation the dynamic positioning thrusters are not used, in order to avoid fuel consumption. Only during storm conditions the system is started up and the thrusters will limit the maximum stress in the mooring lines.

When calculating the ultimate limit state of the system of mooring lines and thrusters the failure of the dynamic positioning system has to be treated with care. The failure of this component may not lead to a progressive failure of the floating breakwater section.

8.3.6 Combination: Turret mooring with dynamic positioning

When all mooring lines are attached to one point on the floating breakwater it can pivot around this point with the help of thrusters. This type is called turret mooring or single point mooring (SPM) and is used for Floating Production and Storage Units (FPSU) , see figure 8.9, and was proposed in the design of [Ali, 2005] for the Floating Transshipment Container Terminal. Due to the fact that they can pivot around their turret mooring, these ships are able to act as a weathervane to avoid excessive loads on the mooring lines and structure.

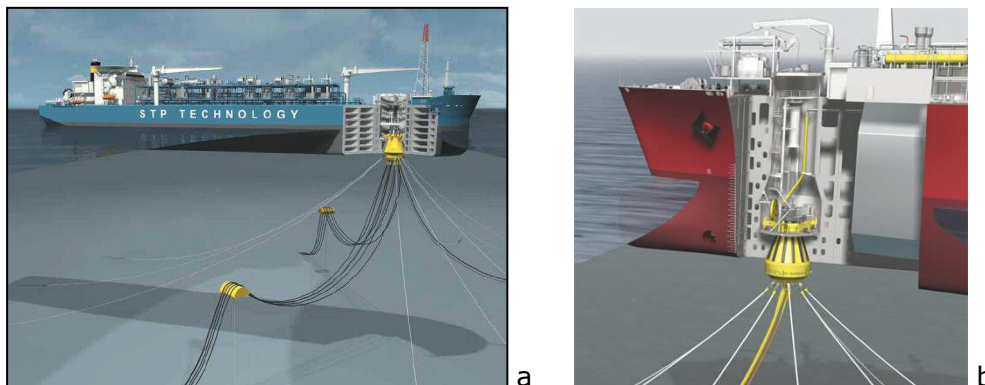


Figure 8.9 A FPSU with turret mooring (a)⁶ and a detail of a turret (b)⁷

⁶ Internal turret gallery

For the floating breakwater the turret mooring can be used to pivot the elements in such a way that the wind generated waves are always beam on. Although it can not be checked with ReFBreak2D, it was assumed that the wave-attenuating performance is better when beam on waves are considered. During storm conditions the elements have to weathervane to receive the waves head-on, increasing the survivability of the system, see also figure 8.10.

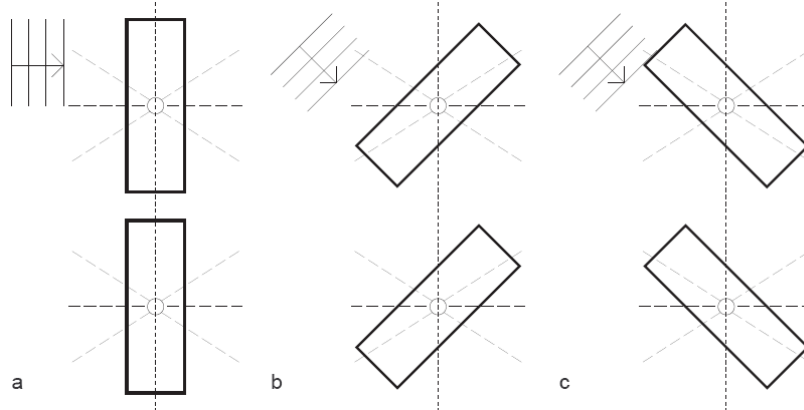


Figure 8.10 FB with turret mooring loaded with beam on waves (a), oblique waves (b) and in ULS (c)

8.3.7 Selection

It was already concluded that transforming the FB element into a tension leg platform is not a wise solution because the TLP completely restricts any motion in the heave direction. It was shown earlier that heave does have a positive effect on the wave-attenuating performance and thus should not be limited.

The space on the seafloor may be too limited for the enormous foot print of the catenary mooring line, because of the combination of many floating breakwaters located next to or behind each other. The smaller footprint of the taut line mooring gives more flexibility in the overall design.

During a storm the forces on the mooring lines will be immense due to the combined action of the waves, current and wind. A turret mooring gives the FB the possibility to minimise its area perpendicular to the waves by turning itself to receive the waves head-on, i.e. weathervane. The function as a safe haven however will be lost as the breakwater will not protect the harbour any more during storm conditions.

Despite the disadvantage of losing the safe haven function of the FTCT, the forces on the mooring lines will be limited. The result for a structure that is able to weathervane is the more economical design of the mooring, in the way that it is not designed for one unique extreme load. Therefore the turret mooring system is chosen in combination with dynamic positioning to give the element the possibility to weathervane.

8.4 Connections

The floating breakwater will be build out of elements which are significantly smaller than the total length of the breakwater. Every connection between the elements has six degrees of freedom, see figure 8.11.

www.offshoreengineering.org
⁷ APL STP system for Montara
<http://marinenorway.etp.no>

These are degrees of freedom are:

- Relative surge;
- Relative sway;
- Relative heave;
- Relative roll;
- Relative pitch;
- Relative yaw.

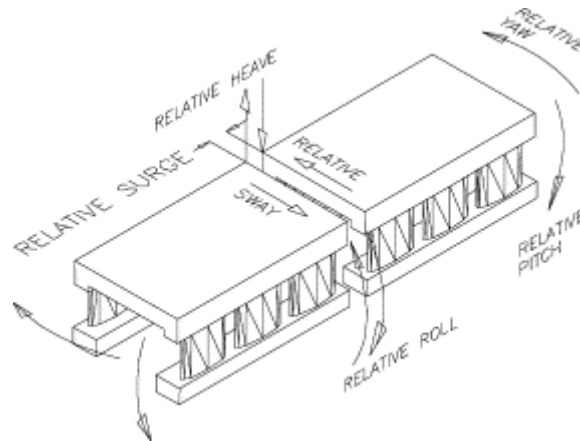


Figure 8.11 Joint degrees of freedom for an Mobile Offshore Base (MOB) [Derstine, 2000]

Every degree of freedom has to be dealt with in the connection. There are basically three solutions for each of degree of freedom. Either it is:

- Rigid connected, i.e. there is no movement possible;
- Soft connected, i.e. there is movement possible with a spring coefficient;
- Not connected, i.e. there is movement possible without a spring coefficient;

The force the connection has to withstand decreases with an increasing flexibility of the connection, but the relative movement between the elements will increase.

One of the most important aspects is the area where the floating breakwater will be assembled. The connections have to be made in the same environment as they are used as protection for the (un-) loading of the ships. This means that it has to be possible to establish the connection with a significant wave height of 4 metres as this is the sea state which is only exceeded 20% of the time. When a smaller significant wave height is chosen as requirement it will lead to a too small chance of success in coupling the elements.

8.4.1 Completely rigid

With completely rigid connections, i.e. completely fixed in all degrees in freedom, a continuous structure is obtained which has the characteristics of a structure without any connections. The connections however have to be designed well as they have to transfer all forces from one element to the other. Due to the repetitive wave action damage due fatigue is to be expected. [McLaren, 1981]

When the elements are rigidly interconnected it is most unlikely that these connections can be released and connected again. This makes it impossible for the element to weathervane. The mooring system should be strong enough to withstand the forces from beam on waves during storm conditions.

8.4.2 Restricted movements

When the connections are not completely rigid, but the elements are still connected to each other the movements of the elements are restricted. The connection should

prevent the elements from hitting each other because this would result in damage [McLaren, 1981]. The wave-attenuating performance of the breakwater differs from the calculated performance due to the reaction of an element on the motions and generated waves of the other elements. An example of a small scale six-degree-of-freedom coupling is found in figure 8.12, showing its complexity.

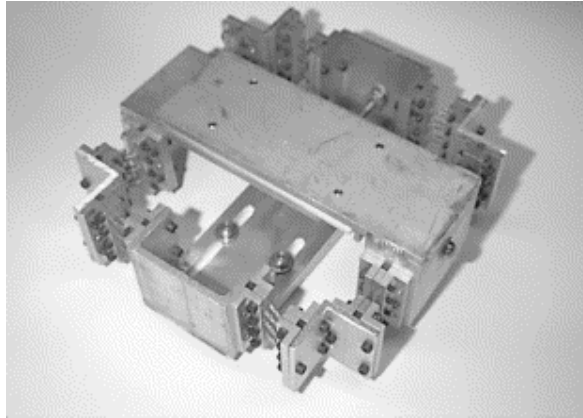


Figure 8.12 A small scale six-degree-of-freedom coupling [Derstine, 2000]

As with a completely rigid connection it is not likely that the connections can be released when storm conditions are expected. Therefore also in this case the mooring system should be strong enough to withstand the beam-on waves during storm conditions.

8.4.3 Free floating

A free floating element does not have any connection with its neighbouring elements. The required distance between two elements is determined by the relative motions. The first order motions of an element are on the order of the incoming wave-height, but the second order motions depend on the applied mooring system. The second order motions will be determining on the required distance to avoid collision damage.

The distance between two elements has a negative impact on the wave-attenuating performance of the FB system, as all energy passes the breakwater at these points. The distance, when compared to the length of an element, should be kept minimal.

The forces on the mooring system can be balanced. The length of the element can be chosen such, that the mooring forces in SLS, with waves beam-on, are on the same order as the forces in ULS, with the waves head-on.

8.4.4 Selection

The connections of the floating breakwater are a dilemma. On one hand a breakwater consisting of interconnected segments offers an uninterrupted structure, leading to a smaller wave force compared to one with free-floating elements on the other hand. The larger forces on the mooring system due to the inability to weathervane and the design and manufacturing of connections that can transfer all forces between the elements is the price that has to be paid for the lower coefficients.

The requirements on the connections will be very high because they have to be able to be connected in ocean-conditions with waves up to 4 metres. When this limiting wave height is chosen smaller, the chance on a successful coupling is too small.

The forces and the requirements on the connections are the major counter-arguments against the application of connections between elements. Therefore it is chosen for free-floating elements. In this way the lay-out as described in figure 8.10 is obtained.

8.5 Element length

When the FTCT is considered as the harbour area that has to be protected it is roughly estimated that around 12 kilometres of breakwater required, see figure 8.13. This length cannot be produced and transported in one piece; therefore the breakwater is divided into separate elements.

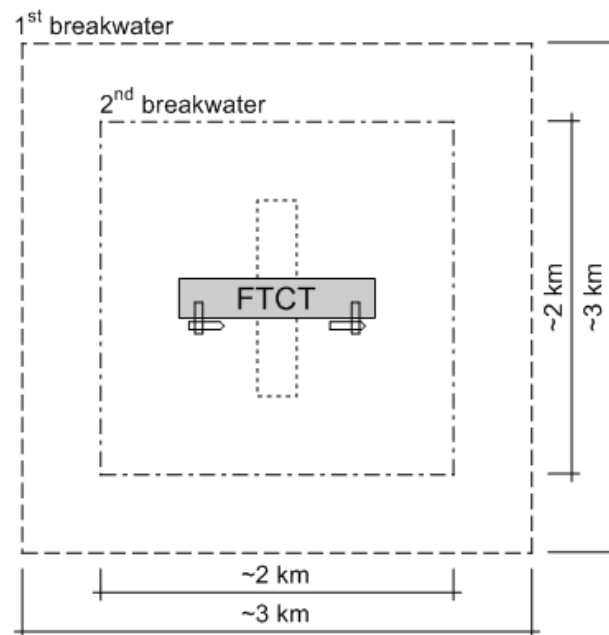


Figure 8.13 Required breakwater length in order to protect the FTCT

First the most important parameters involving the optimal length are discussed before a selection of the element length is made.

8.5.1 Optimising the element length

The driving force behind the optimal length of a FB length is the force that has to be absorbed by the mooring system. When an element is able to weathervane the length should be chosen such, that the force on the mooring system is equal in SLS, with waves beam-on, and in ULS, with waves head-on. See figure 8.14 for the conflicting, opposed requirements.

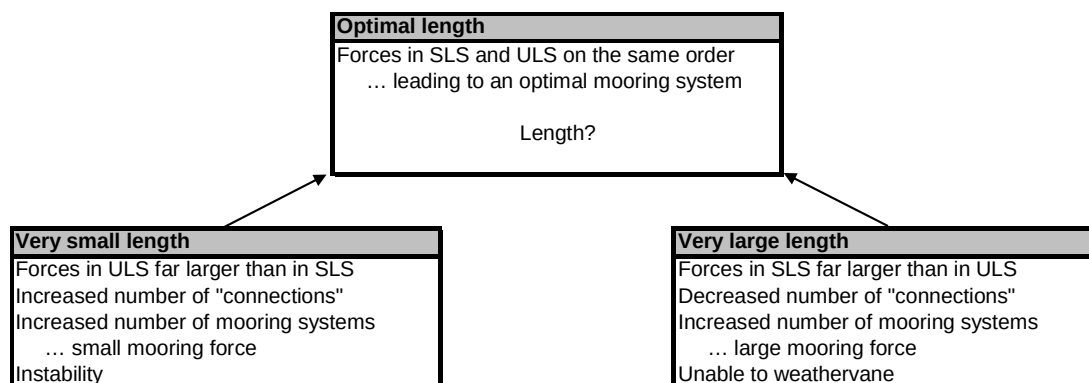


Figure 8.14 Tension field between a small element length and a large element length

A very large element will yield in high mooring forces and forces in SLS that are larger than in ULS. In the extreme case the element becomes too long to be able to weathervane; the required space will become too large.

A very small element on the contrary will have small mooring forces that will be larger in the ULS than in the SLS, because the length and the beam will have approximately the same dimensions. In the extreme case the element can become unstable in the longitudinal direction.

The optimal design should combine the advantages and have a mooring force in SLS that is approximately similar to the force in ULS.

8.5.2 Selection

At this stage of the design the optimal length of a FB element is estimated to be 500 metres. When all forces on the element are calculated it should be checked whether this estimation is correct. When the values of the mooring force in SLS and ULS differ too much from each other another element length should be chosen and a new design cycle is required.

8.6 Division in sections

Basically three variants of the division of the element can be defined, see also figure 8.15:

- No sections;
- Longitudinal sections;
- Transverse sections.

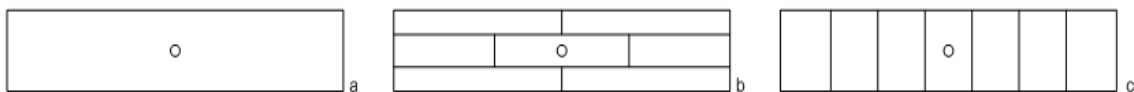


Figure 8.15 No sections (a), longitudinal sections (b) and transverse sections (c)

The element does not necessarily have to consist of sections, as it is only slightly bigger than the Monaco Floating Breakwater, which was built in a custom-made dry-dock. However building a complete element in a dry-dock will hamper the serial production of large breakwater elements and is thus not a solution for the FB elements, as more than one is required for the protection of the FTCT.

The division of the elements into sections can be done in a longitudinal and a transverse direction. The longitudinal division, as has been done with the Mega-Float, will lead to sections with a small beam, which, in combination with the larger draught, can lead to instability problems. The transverse division does not have this problem of instability as the width can be chosen more freely, avoiding the problems.

When the element is divided into transverse section some of the section may have the same layout, leading to a more efficient building process. Basically there are only three types of sections required, the bow and stern sections, the normal sections and the section housing the turret mooring system. The length of the connections can also be limited.

It is chosen to divide the element in transverse section for the reasons mentioned in this section. These reasons are the avoidance of possible stability problems of the sections and the more efficient building process.

The number of different sections should be minimised in order to speed up construction and to come to a more economically sound design. There can be three different sections defined:

- The bow/stern section;
- The turret mooring section;
- A normal section, linking the bow/stern section with the turret mooring section.

The final design should be limited to the listed section types.

8.7 Conclusion

In this chapter possible solutions have been determined for the four components of the design:

- the material;
- station keeping;
- the connections;
- the element length;
- the division in sections.

All solutions are summarised in figure 8.16.

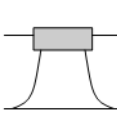
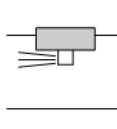
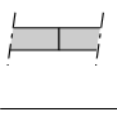
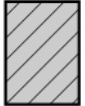
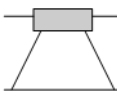
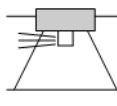
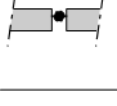
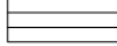
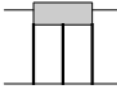
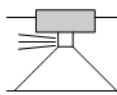
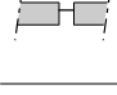
Material	Station keeping		Connections	Element length	Sections
				~100 metres	
1) steel	1) catenary	4) DP	1) rigid	1) small	1) none
				~500 metres	
2) concrete	2) taut moored	5) DP assist	2) flexible	2) optimal	2) longitudinal
				~1000 metres	
3) composite	3) TLP	6) SPM (turret)	3) free-floating	3) large	3) transverse

Figure 8.16 A survey of solutions

Although alternatives can be composed from the solutions discussed in this chapter, one alternative is chosen to be elaborated on. This alternative is a concrete element with a length of 500 metres. It is moored with a turret mooring system with dynamic positioning and all elements are free-floating. The design of the two elements forming the breakwater system is summarised in table 8-3.

Table 8-3 Details on the elements forming the FB system

Parameter	1 st Breakwater	2 nd Breakwater
Length	500 [m]	500 [m]
Beam	100 [m]	40 [m]
Draught	20 [m]	8 [m]
Mass	10.000.000 [kN]	1.600.000 [kN]
Material	Pre-stressed concrete	
Mooring system	SPM (turret mooring) with dynamic positioning	
Connections	None, free-floating elements	
Other details	Ability to weathervane to reduce mooring forces	

Although the breakwater design consist of two element types placed in series, only one of the elements is elaborated on in this master thesis to further investigate the concept. The element that will be designed is the larger, 1st breakwater element with a length of 500 metres, a beam of 100 metres and a draught of 20 metres.

Chapter 9 Structural design

The main dimensions of the floating breakwater are determined on the basis of the requirements of the wave attenuating performance. The beam and draught of the vessel are determined in Chapter 7, together with the mass and centre of gravity. The preferred materials, element length and station keeping are chosen in Chapter 8. In this chapter the design of the structure will be elaborated upon up to the level of a preliminary design.

The construction method will have a large impact on the loads on the structure; therefore this is discussed before the structural calculations.

9.1 Recapitulation and preview

The design process of the FB system is schematised in figure 9.1. In this section recapitulates the designs steps which are already made and an introduction is given on further steps in the design process.

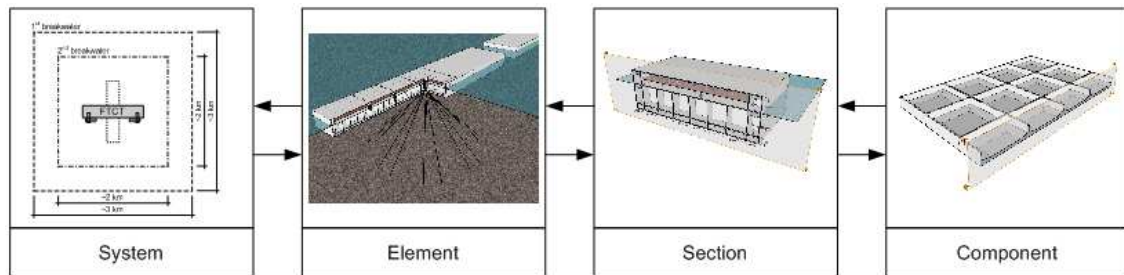


Figure 9.1 The design at system-, element-, section- and component level

9.1.1 System

In Chapter 7 it has been concluded that a single FB did not have an adequate enough wave attenuating characteristic to protect the FTCT against wave action from the ocean area. The requirements which have to be met by the system are listed in Chapter 5. To fulfil the requirements, a system consisting of two floating breakwaters in series was proposed, see figure 9.2.

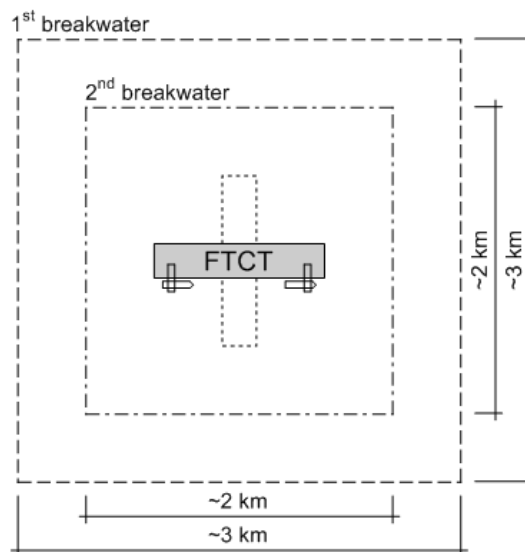


Figure 9.2 Required breakwater length in order to protect the FTCT

A survey of solution, given in Chapter 8, resulted in a global design of the FB system. The most important characteristics of the system are:

- Free floating elements, to avoid connections;
- Ability to weathervane through a turret mooring system;
- Length of the elements optimised, leading to forces in SLS and ULS on the same order.

All characteristics of the FB system are listed in table 9-1.

Table 9-1 Details on the elements forming the FB system

Parameter	1 st Breakwater	2 nd Breakwater
Length	500 [m]	500 [m]
Beam	100 [m]	40 [m]
Draught	20 [m]	8 [m]
Mass	10.000.000 [kN]	1.600.000 [kN]
Material	Pre-stressed concrete	
Mooring system	SPM (turret mooring) with dynamic positioning	
Connections	None, free-floating elements	
Other details	Ability to weathervane to reduce mooring forces	

Although the breakwater design consists of two element types placed in series, only the 1st breakwater element is elaborated on in this master thesis as a proof of concept.

9.1.2 Element

The first FB element should be able to attenuate waves with a wave period of 14 seconds. In Chapter 7 a parameter analysis has been performed, which resulted in design charts, see figure 9.3. In order to meet with the requirements the 1st breakwater should have a draught of 20 metres and a beam of 100 metres. The centre of gravity (COG) is located 5 metres above the centre of buoyancy (COB) or 15 metres above the keel. This design is also indicated in figure 9.3.

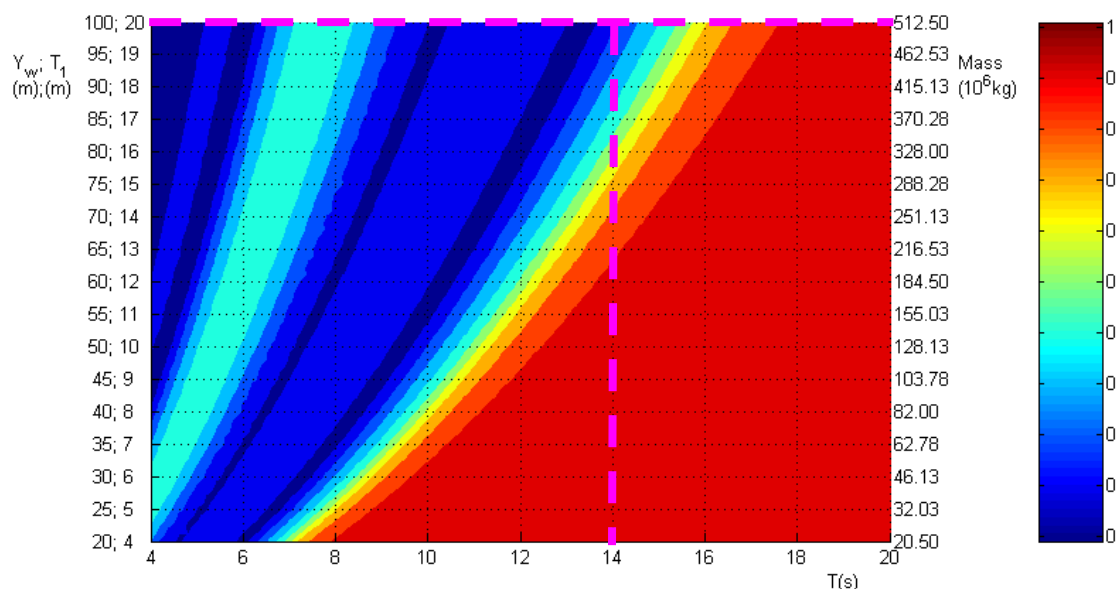


Figure 9.3 Transmitted wave RAO of a FB with a fixed Y_w/T_1 -ratio of 5.0. The draught and the beam are varied, while the COG is positioned 5 metres above the centre of buoyancy. The considered design is shown with a dashed line.

The element is divided into sections for a more economical design and reduced building time. An impression of the element is given in figure 9.4 the three types of sections can also be seen. In figure 9.5 a generic transverse cross-section is depicted, showing important design considerations.

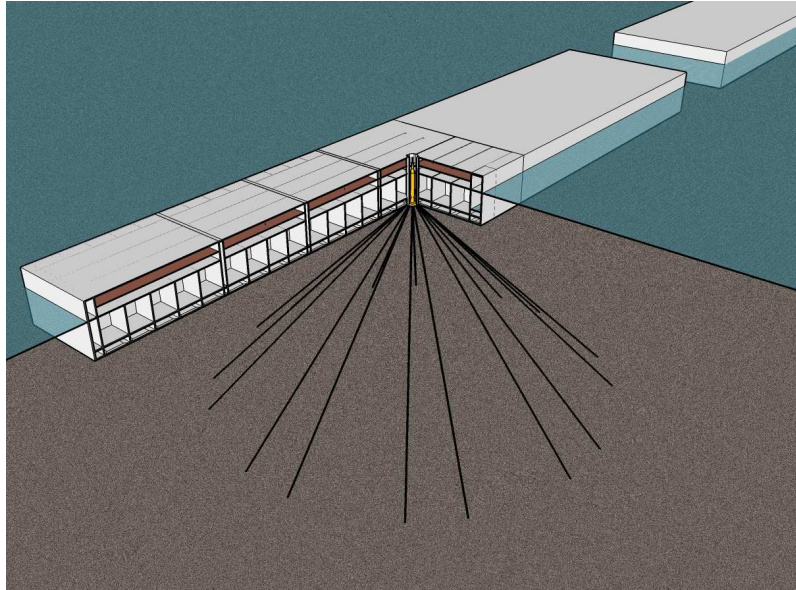


Figure 9.4 Impression of the designed element

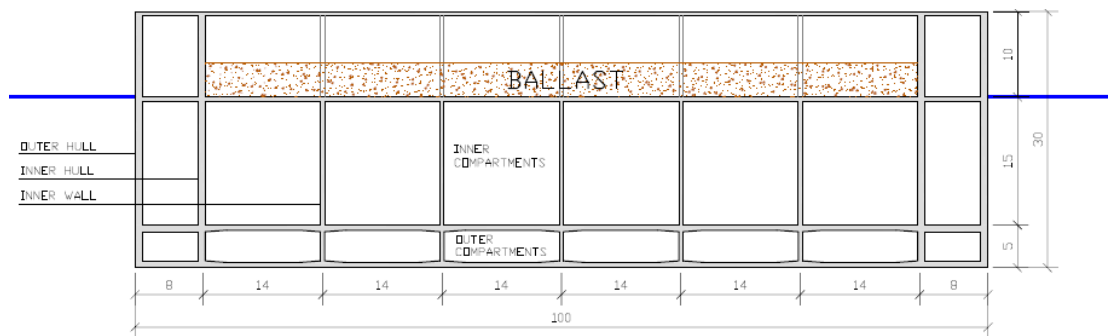


Figure 9.5 Generic cross-section of an element

The whole element is divided into separate compartments. Because of the function and the location of the structure the likelihood is high of the structure having to withstand a collision with a ship. Although concrete is very resilient, one or more compartments may be damaged in a collision, resulting in the flooding of that compartment. When a compartment of the FB element is damaged it must not lead to the loss of the function or to a complete loss of the FB element.

The element has a double hull, similar to modern oil carriers. The function of the double hull on oil carriers is to limit the oil spill in the unfortunate case of a collision or when the ship is run aground. The function of the double hull on the FB element is different from that one on an oil carrier. The double hull has to retain water in the outer compartments when these are flooded due to an unfortunate event. The small size of the outer compartments leads to a reduced impact of the flooding of a compartment on the stability and location of the COG and thus to a reduced impact on the function of the FB element.

Another advantage of a double hull is the reduced load on the inner walls, as they do not have to withstand any hydrodynamic or hydrostatic pressure. The bottom compartments can be built in a dry-dock and when finished be towed out. This part should provide enough buoyancy for the rest of the construction alongside a jetty. In this way the construction time can be minimised.

The function of the inner walls is twofold. Firstly, they have to transfer the load of the sand ballast, which is located high in the element, to the bottom. Secondly, they provide stiffness in the transverse direction to the bottom, reducing the bending moments.

9.1.3 Sections

The types of elements that can be distinguished are:

- Normal elements

The normal sections are the sections numbered 2, 3, 5 and 6 in figure 9.6. These elements are always coupled to two other elements, removing the need to design the bulkheads to withstand the extreme wave loads.

- Turret mooring section

The turret mooring section is the section numbered 4 in figure 9.6 and is located in the middle of the FB section. The design of this section is similar to the normal section, except for the space that is required for the turret mooring system.

- Bow and stern section

The bow and stern section are numbered 1 and 7 in figure 9.6. The section is only coupled on one side to another section. The other side of the section is exposed to wave-attack when the FB element is attacked by head-on waves and has to be dimensioned for the occurring pressures. In Appendix F it was also shown that this element needs a stronger bottom and sides than a normal section due to the higher wave induced pressures.

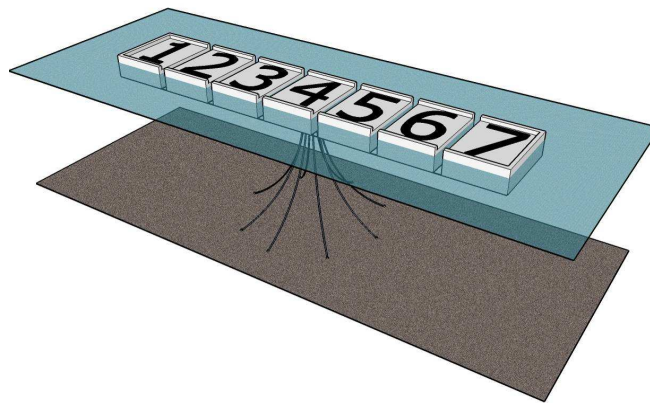


Figure 9.6 The different types of sections

These three sections will be elaborated on in this chapter component by component, resulting in a complete preliminary design. The components that have to be designed are listed in the next section.

9.1.4 Components

Every type of section consists of a number of components which have to be designed:

- Bottom of the double hull;
- Sides of the double hull;
- Inner walls;
- Ballast.

For the turret mooring section an additional component has to be designed, that is the turret, for the bow/stern section the bow is an extra component.

Besides the components of the sections, the following components have to be elaborated on as well in order to obtain a complete design of the FB element:

- Mooring system;
- Connections between the sections.

9.1.5 Chapter outline

The construction method has a large impact on the forces on the components, especially the bottom part of the section; therefore this is discussed first, before determining the forces on the sections. Secondly, the forces and pressures on the sections are addressed. For each component the determining load case is identified.

The sections are elaborated on component by component. The normal section is elaborated on extensively, as it is the concept design method of the other sections. Finally the two remaining components, i.e. the mooring system and the connections, are discussed.

9.2 Construction method

The FB system does not consist of one single element, but out of a large number. When the FTCT has to be protected from waves possibly originating from all directions around 25 elements are needed. This calls for an efficient fabrication plant where several construction phases co-exist in order to reduce the building time of the complete system.

Because the segment is divided into sections, which leads to a high repetition factor, the production process of an element can be divided into three stages:

- Construction of the segments;
- Joining the segments to an element;
- Transport and installation.

The sections can be built completely in a dry-dock, imposing severe requirement on the depth of the dry-dock. The draught of a FB element during its use is 20 metres, but it will be ballasted at this stage. When the element is towed out of the dry-dock it will have an estimated draught of about 15 metres.

The depth of the dry-dock can be minimised by producing the bottom part only and to such an extent that it can float. The element is finished along a quay-wall to speed up the construction. When a cross-section is assumed as shown in figure 9.7 a water depth of 5 metre will suffice. The bottom part will be box-shaped and should have a height of around 5 metre. These are dimensions which are similar to the design of submerged tunnels used in The Netherlands.

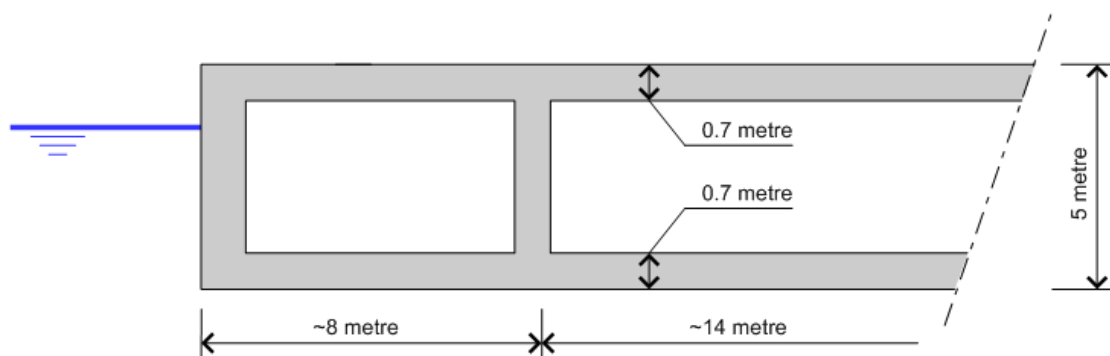


Figure 9.7 Part of the cross-section of the bottom slab when ready to be towed out of the dry-dock

The box that is built in the dry-dock will be towed out and moored along a jetty where the building process will be completed. At this stage it is important that all work is carefully planned. When construction takes only place at one side of the section this will cause bending moments in the bottom slab. Also the stability of the bottom part has to be guaranteed at every stage.

Finally the finished sections have to be mated to form one FB element. This stage can be omitted when the element is made in one piece. The process of mating is important as the connection has to transfer the shear forces and moments from one element to the other. It also has to cushion the impact of the two elements when they join.

The finished FB elements have to be stored until they can be towed out to their final location where they will be moored by use of a turret mooring system which is assisted by thrusters. The anchor lines and anchors can be pre-installed to ease installation at the site, as was done with the Monaco Floating breakwater.

9.3 Failure mechanisms and forces

There are several ways the FB element can fail. Failure is defined as the loss of the function of an element after an unwanted event. Several load conditions can induce a failure mechanism:

- Construction;
- Waves during service combined with a collision;
- Extreme waves;
- Drift forces;
- Internal moments.

Each of these load conditions will be discussed and finally a conclusion will be drawn which load condition is decisive.

9.3.1 Construction

When the mass of the structure is equally distributed over its length and beam there will be no bending moments in still water as the hydrostatic pressure of the water counteracts the mass. During construction the bottom part is manufactured in a dry-dock and will be towed out when it is completed. It can be assumed that the mass of the bottom part is more or less equally distributed over its surface.

Along the finishing jetty the sides of the breakwater will be poured, and thus mass will be added to the structure. This mass however, is not distributed equally over the surface, but is concentrated near the sides of the bottom part, see figure 9.8. The bottom part should have enough bending moment capacity to take up the pressures that will occur.

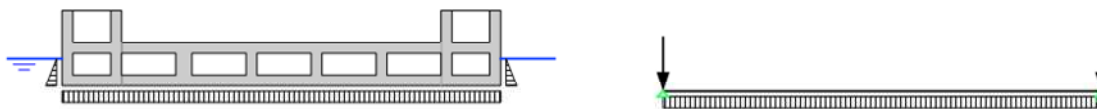


Figure 9.8 Internal (static) moments due to the construction scheme and the corresponding mechanics schematisation

9.3.2 Waves during service combined with a collision

The FB will be located in an area where there is a lot of shipping traffic. The chance of an element being hit by a ship cannot be neglected. The damage of a ship hitting the breakwater, i.e. the flooding of one compartment, should not result in the loss of the complete element.

Even when one compartment is damaged the element still has to fulfil its function, i.e. the incoming irregular wave-field should be attenuated to an acceptable level. The pressures on the hull of the element are calculated in Appendix F.1 with the use of a probabilistic method. The pressures with a chance of exceedance of 0.0001 are shown in figure 9.9 on the next page. These wave induced pressure have to be taken up by the structure in addition to the (enlarged) hydrostatic pressure.

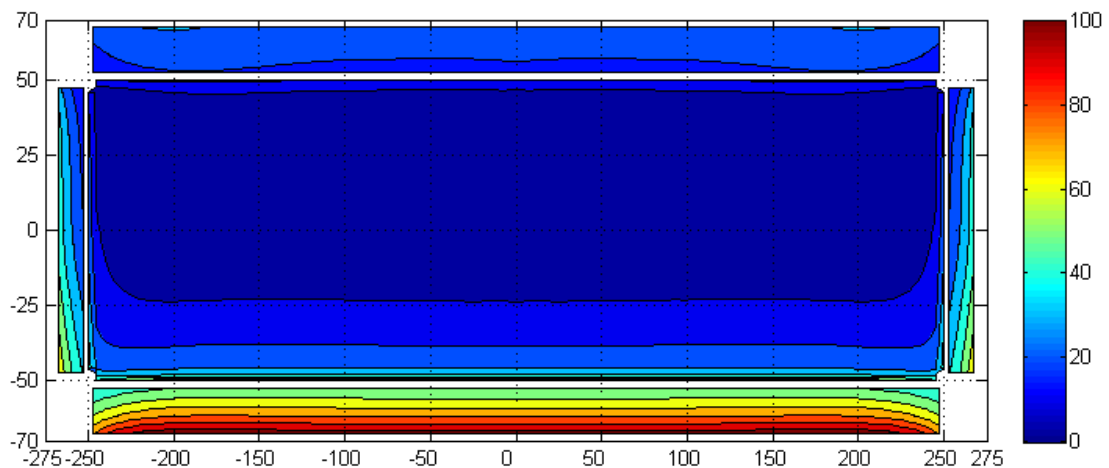


Figure 9.9 Design wave pressure on a FB element loaded by beam-on waves in kN/m^2

9.3.3 Extreme waves

During its life-time the structure can be loaded by an extreme storm. To avoid large forces on the turret mooring the element is able to weathervane and receives the waves head-on. The pressures on the hull are calculated similar to how they have been calculated for beam on waves. The design wave pressures are shown in figure 9.10. These wave induced pressure have to be taken up by the structure in addition to the hydrostatic pressure.

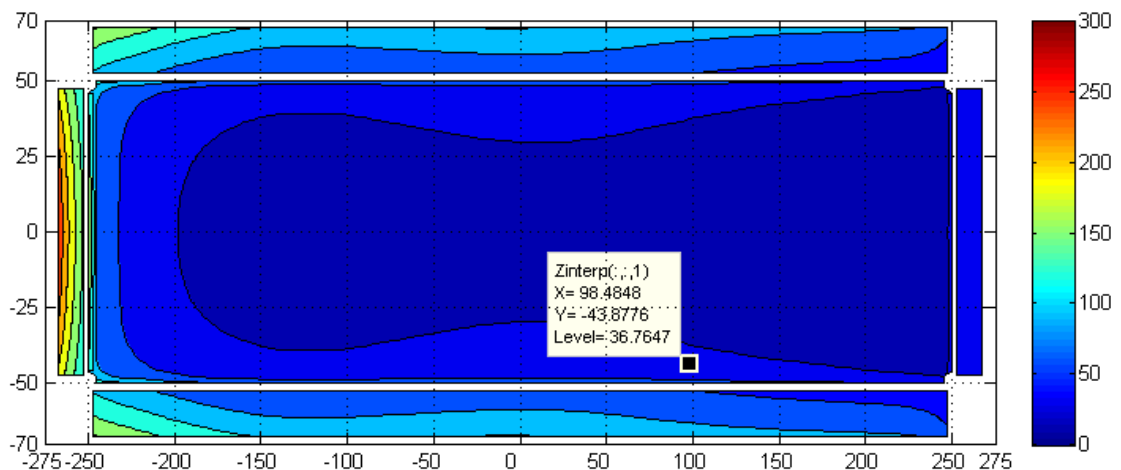


Figure 9.10 Design wave pressure on a FB element loaded by head-on waves in kN/m^2

9.3.4 Drift and drag forces

The ReFBreak2D model and the pressures which have been described earlier do consider the first order motions of and pressures on a structure that is attacked by waves. Beside these first order effects there are also second order effects, especially when an irregular wave-field is considered. In [Journée and Massie, 2001] it is stated that:

"The effects of second order wave force are most apparent in the behaviour of moored floating structures. [...] these are horizontally restrained by some form of mooring system."

The floating breakwater system consists of several moored elements, so the second order wave forces will be apparent in the behaviour of a FB. The analysis of the

forces and the motions of a FB element include three important components [Journée and Massie, 2001]:

1. An oscillating displacement of the structure with a frequency corresponding to the incoming waves.

These are the motions due to the first order wave loads. The time-averaged wave load on the structure and the resulting average displacement are zero. The ReFBreak2D model deals with the first order behaviour of the FB. This will not be further discussed here.

2. A mean displacement of the structure, resulting from a constant load.

Clear sources of a constant load are the wind and (tidal) currents. In addition to these loads there is also a so-called mean wave drift force caused by non-linear (second-order) wave potential effects. This load will, influenced by the mooring system, determine the new equilibrium around which it will oscillate with its first order displacements.

3. An oscillating displacement of the structure with a frequency much lower than those of the incoming irregular waves.

These motions are caused by non-linear elements in the wave load, the low frequency wave forces. The low frequency wave forces are caused by the differential frequencies that exist in the components of an irregular wave field. Generally a ship, and thus a FB-system, will have a low natural frequency in the horizontal directions as well as a very little damping at such frequencies. These low-frequency wave forces can result in very large motion amplitudes and corresponding loads in the mooring system.

The force mentioned in point 2 and 3 are calculated in Appendix F.2 and are presented in table 9-2. It can be seen that the drift and drag forces in SLS and ULS do have the same order of magnitude due to the ability of the FB element to weathervane in storm conditions.

Table 9-2 Drift and drag forces working on a FB element

	SLS	ULS
Wave direction	Beam-on	Head-on
Spectrum	Bretschneider	Bretschneider
Magnitude	Beaufort 7	Beaufort 12
Drift forces		
Mean wave drift force	4800 [kN]	4700 [kN]
Low frequency drift force	5414 [kN]	4720 [kN]
Currents		
Speed	1 [m/s]	2 [m/s]
Drag force	5125 [kN]	4100 [kN]
Wind		
Speed	17 [m/s]	32 [m/s]
Drag force	903 [kN]	640 [kN]
Total	16242 [kN]	14160 [kN]

With respect to table 9-2, it has to be reminded that in Appendix F.2 it was concluded that the low frequency mooring force depends on the response of the FB on the low frequency drift forces. In their turn they are depending on the mooring system. The determination of the mooring force is very complex and is regarded as out of scope for this preliminary design study.

To deal with the low frequency wave force it is assumed that the low frequency oscillatory force which acts on the mooring system is equal to halve of the sum of the steady forces [Ali, 2005].

9.3.5 Internal moments

Waves are causing pressures but will also cause bending moments in the floating structure. These bending moments are working on the complete structure, and not only on a certain member, distinguishing these bending moments from the local bending moments induced by own weight, pressures on panels, etc. The internal moments can be calculated in three ways:

- Froude-Krilov;
- Froude-Krilov with reduced wave-elevations;
- DELFRAC design pressures.

The results of these calculations, which are explained in Appendix F.3, are summarised in table 9-3 below.

Table 9-3 Design moments according to various methods

Method	Design moment per metre width	Total design moment
Froude-Krilov	12,1 [MNm/m]	1210 [MNm]
Froude-Krilov with reduced wave-elevation	8,1 [MNm/m]	810 [MNm]
DELFRAC	N/A [MNm/m]	N/A [MNm]

The dynamic moments can be investigated using the principle of dynamic equilibrium, using Newton's second law. This method is explained in Appendix F.3.4 and it should be used in future research. For the preliminary design this method has not been used.

The Froude-Krilov also overestimates the bending moments as the presence of the structure is neglected. The wave pressures are exponentially decaying with the depth. When this is taken into account the design global bending moment is $8.1 \cdot 10^6 \text{ Nm}$ or 8.1 MNm per metre width of the FB element. This value will be used for elaborating on the design of the elements.

9.3.6 Conclusion

There are five sources of forces acting on the structure; these will all be discussed in Appendix F. Each of them can induce a failure mechanism:

- Construction;
During construction bending moments may exist in the bottom slab causing it to crack during construction.
- Waves during service combined with a collision;
In SLS, with waves beam-on and an increased draught, the pressures on the hull may cause tensile stresses in the hull shell.
- Extreme waves;
In ULS, with waves head-on, the pressures on the hull may cause tensile stresses and thus cracks in the hull.
- Drift and drag forces;
The drift and drag forces may cause the mooring system to fail and set the FB element adrift, with possible damage to surrounding structures and ships.
- Internal moments.
Internal moments may break the element or may cause the connections between separate sections to fail. The separate sections may sink or cause damage in the harbour area.

In table 9-4 the decisive load cases are given for the different components of the FB element. In the further design of the element the loads as has been discussed will be used to determine the required cross-sections and strength.

Table 9-4 Load cases on the different structural components

Component	Location (x-coordinate)	Desive load case	Check needed
Bottom	-180 meter to 180 meter	Extreme waves	Construction
Bottom bow/stern	250 meter to -180 meter and 180 meter to 250 meter	Extreme waves	Construction
Side	-180 meter to 180 meter	Waves during service combined with a collision	
Side bow/stern	250 meter to -180 meter and 180 meter to 250 meter	Extreme waves	
Bow/stern	At -250 meter		
Connections		Global wave induced moments	
Turret mooring	At 0 meter	Drift forces	

9.4 Section design

The complete element will be built in sections and will be assembled in sheltered water before it is towed out to the installation site. First the division in sections will be discussed, after which the different types of sections that can be distinguished in an element are stated with their design criteria. The structural design of the elements is done in Appendix G, of which the results are presented in this section.

9.4.1 Normal section

The normal section, i.e. the sections numbered 2, 3, 5 and 6 in figure 9.6 does have the following components of which the cross-section has to be determined:

- Bottom
- Sides
- Inner walls

In Appendix G.3 all calculations are given, in this section an overview of the design will be given.

For the bottom part two basic designs have been calculated, the first with a cell size of 14x14 metre and the other with a cell size of 14x35 metre, see also figure 9.11. Due to the increased length of the cell size the maximum moment increased as well, leading to an increased cross-section of the second design. This increase of the bottom slab thickness eliminates the material that is saved by the omitted dividing walls. Therefore design 1, with cell sizes of 14x14 metre is chosen.

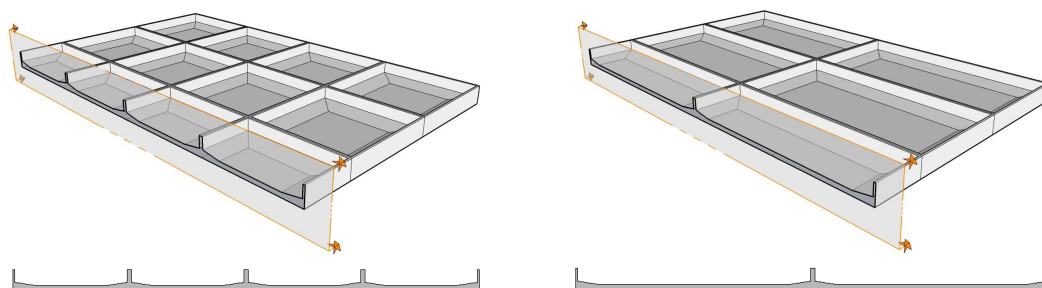


Figure 9.11 Design 1 with 14x14 metre cells (left) and design 2 with 14x35 metre cells (right)

The bottom part of the hull does consist of a double shell, as described in paragraph 9.1.2. The inner hull does have to withstand the load of the outer hull when one of the compartments is flooded. Therefore the inner shell does have the same dimensions and strength as the outer shell.

The required distance between the inner and outer shell forming the hull depends on:

- The bottom part of the FB section has to be floated out of a dry-dock and towed to a jetty to be finished. It will have to have a displacement that is large enough to float with a high enough freeboard for transport if necessary. This determines the minimum height of the cells.
- It has to withstand the bending moments due to the not equal distributed loads from the walls and deck. This also determines the minimum height of the cells.
- When one-compartment is flooded it may not lead to stability problems or sinking. This determines the maximum height of the cells.

The height of the bottom part of the hull is determined to be 5 metres. In this way a structure with a size comparable to that of an immersed tunnel element is obtained. An overview of the bottom part of the section is given in figure 9.12; only four cells are displayed in the displayed cross-section. The beam of the FB element is divided in totally in 8 cells.

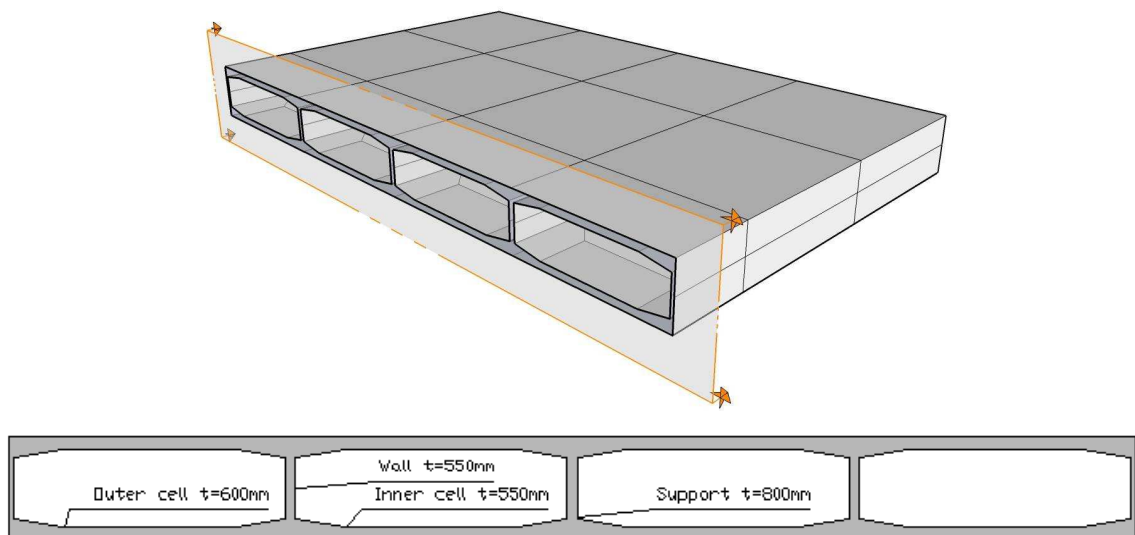


Figure 9.12 Cross-section of the inner and outer hull of the bottom part of the section

The sides of the floating breakwater are loaded with a hydrostatic pressure that is increasing with the depth and the hydrodynamic pressures that are caused by the waves. The hydrodynamic pressures are oscillatory of nature, when the pressures are used which were found by DELFRAC suction may occur around the water line, see figure 9.13a. In reality the pressure distribution on the breakwater will look more like as has been depicted in figure 9.13b.

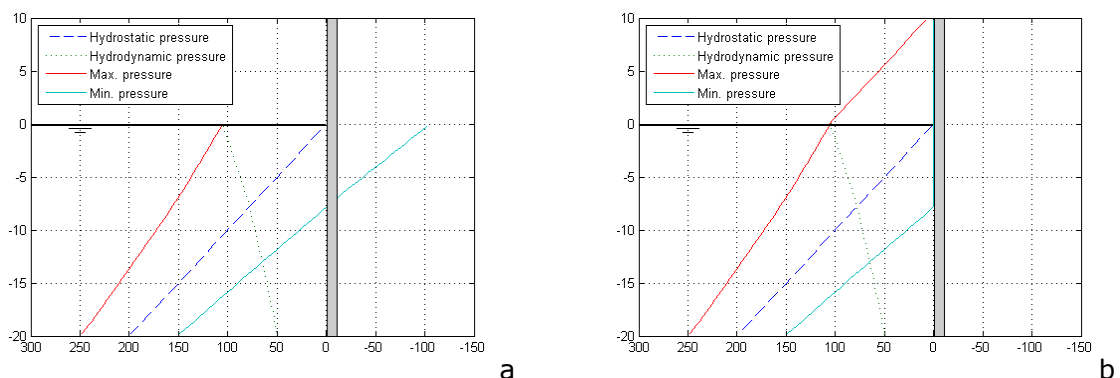


Figure 9.13 Pressures according to DELFRAC (a) and idealised pressures on a wall (b)

The sides of the section consist of two parts, a smaller section near the bottom which is heavily loaded and a larger section in the upper part. A larger section will lead to an increase in the moments, but this is partly compensated by the lower occurring pressures. See for the dimensions of the panels figure 9.14 on the next page. The thickness of the panels is 750 millimetres, as well for the outer shell as for the inner shell.

The inner walls of the section do not have a water retaining purpose, other than retaining water ballast when needed. When one or more compartments are flooded it will be limited to the compartments located between the inner and outer shell of the hull. The inner walls do have a thickness of 500 millimetres.

The section has to be ballasted for two purposes:

- To obtain the required draught

The mass of the concrete hull is not enough to obtain the design draught of 20 metre.

- To position the centre of gravity above the centre of buoyancy

The mass of the hull is concentrate near the heavy bottom, giving it a centre of gravity which is located lower as the centre of buoyancy. This may be beneficial to the stability of the FB element but it has a negative impact on the wave-attenuating properties. The combined own weight and ballast should have a COG located 5 metre above the COB.

There are various variants to ballast the element, see table 9-5, each results in a different location of the COG and draught during transport. The required ballast is calculated in Appendix G.3.4.

Table 9-5 Variants on ballasting the section

	No ballast	Water ballast	Sand ballast	Optimal
Mass concrete	896400 kN	896400 kN	896400 kN	896400 kN
Mass ballast water	0 kN	503600 kN	0 kN	132904 kN
Mass ballast sand	0 kN	0 kN	503600 kN	370696 kN
Total mass	896400 kN	1400000 kN	1400000 kN	1400000 kN
Draught	12,81 m	20 m	20 m	20 m
KG	11,69 m	11,44 m	15,45 m	15,00 m
Draught at float out	- m	12,81 m	20 m	18,10 m
KG at float	- m	11,69 m	15,45 m	14,58 m

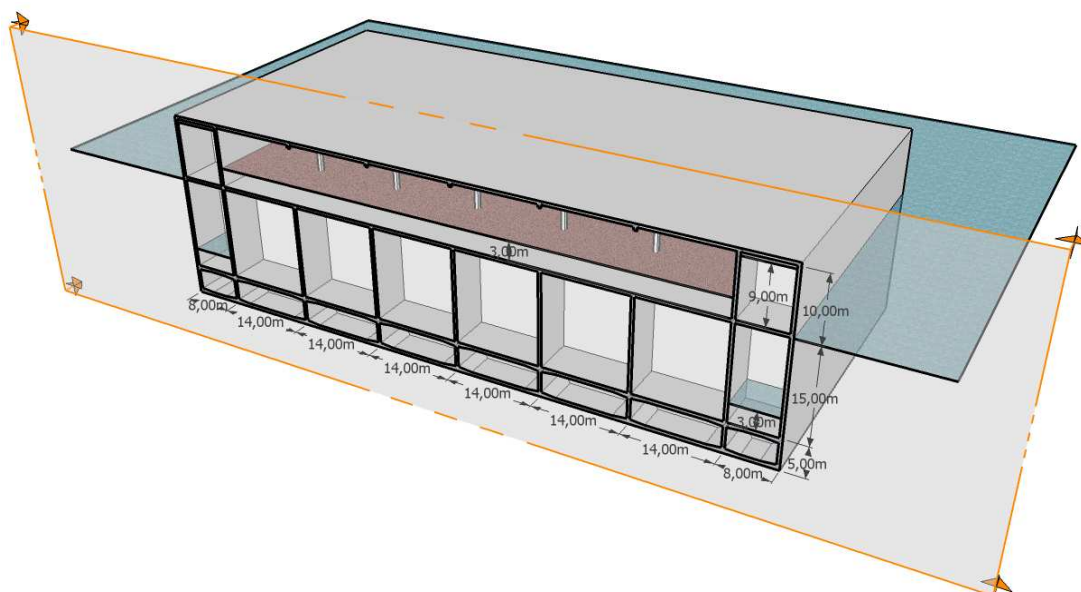


Figure 9.14 Transverse cross-section of the normal section

It is assumed that the sand ballast is applied in the harbour and is not removable before transport, this results in a higher draught during transport. Due to the amount of ballast that is required water ballast solely is not an option as the centre of gravity cannot be positioned on the required height. An optimal solution therefore is found in combining sand ballast around the waterline and water ballast in the outer compartments, see also figure 9.14.

The application of water ballast in the voids between the inner and outer hull has an additional advantage that the (effective) pressures on the outer hull is reduced. This reduction comes from the fact that a part of the pressure is transferred from the outer hull to the inner hull by the ballast water.

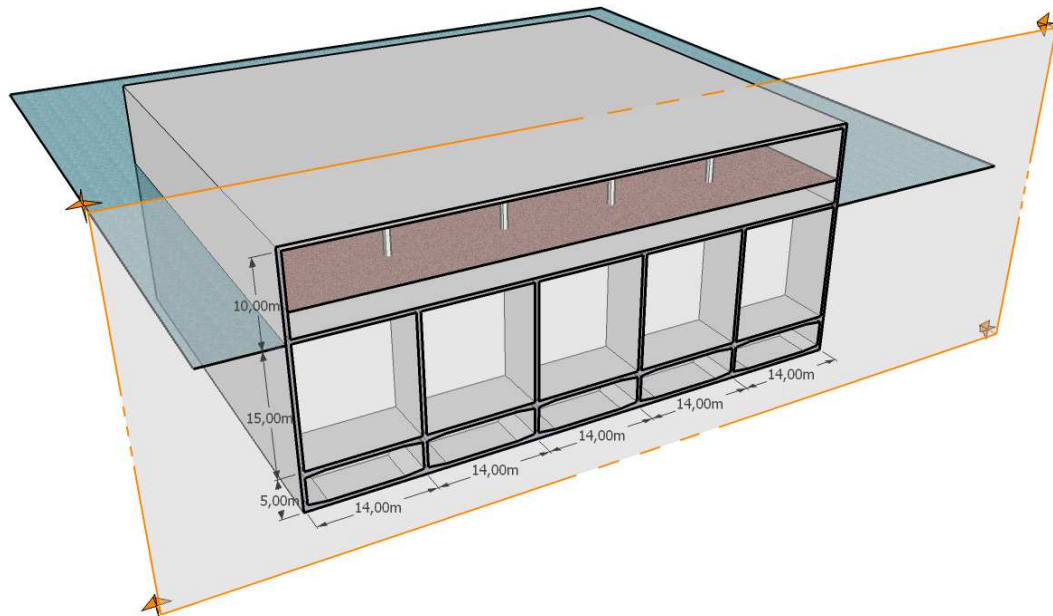


Figure 9.15 Longitudinal cross-section of the normal section

9.4.2 Turret mooring section

The turret mooring section is similar to the normal sections and will have the same dimension as shown in figure 9.14 and figure 9.15. The main difference is that the turret mooring section is equipped with the mooring system. The mooring system itself will be elaborated in section 9.5.

In figure 9.16 the principle of a turret mooring system is explained. It shows the components of the system in a Floating Production and Storage Unit (FPSU). The mooring forces are transferred from the system onto the hull in two places. One is located near the bottom or keel; the other is located near the top at deck level. The largest mooring forces are directly transferred to the hull by the lower connection structure. The keel of the vessel is a strong point in the hull design.

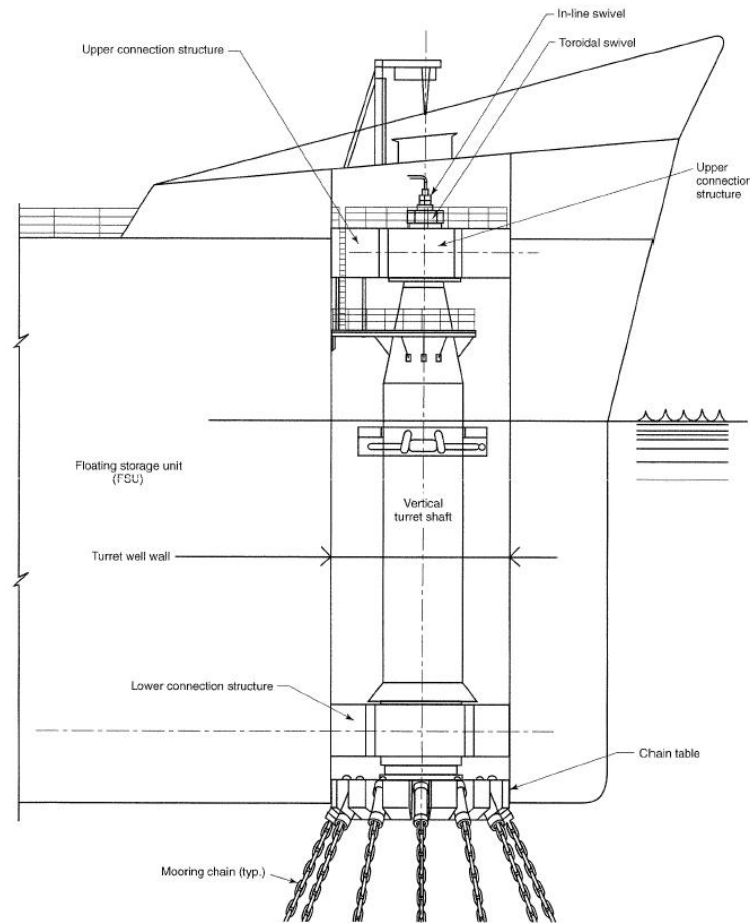


Figure 9.16 Turret mooring system of a Floating Production and Storage Unit (FPSU)¹

When the turret mooring system is applied to the FB element, the lower connection structure can be integrated in the bottom part of the section. In this way the mooring forces are directly transferred to the structure. The upper connection should be placed above the waterline to ease the maintenance on this structure.

The mooring force will be transferred for the biggest part from the lower connection structure onto the bottom part of the double hull shell. From there it is transferred in the form of shear-forces to the rest of the structure.

An impression of the turret mooring in the FB section is given in figure 9.17.

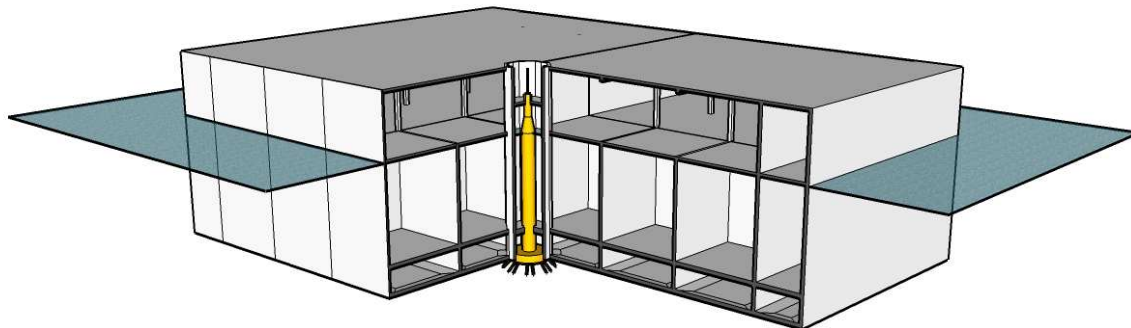


Figure 9.17 Impression of the turret mooring section

¹ TU Delft Offshore Engineering
<http://www.offshoreengineering.org>

9.4.3 Bow/stern section

The normal section, i.e. the sections numbered 1 and 7 in figure 9.6, does have the following components of which the cross-section has to be determined:

- Bottom
- Sides
- Bow/Stern
- Inner walls

In Appendix G.5 all calculations are given, the same design steps as for the normal section have been followed, but with higher hydrodynamic pressures. The only addition for this element is the double shell that forms the bow of the element.

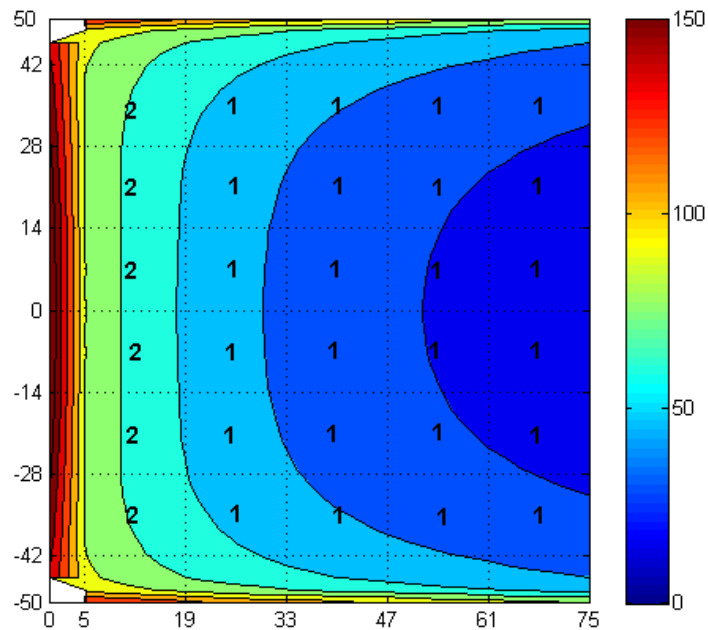


Figure 9.18 Hydrodynamic pressures on the bow/stern section in kN/m^2 (from Appendix F.1.5)

In figure 9.18 the hydrodynamic pressure distribution on the bottom of the bow/stern section is given. Due to the head-on waves and the diffraction of these waves by the structure the hydrodynamic pressure is high in the cells located near the bow, in figure 9.18 denoted with a 2. Therefore these cells have a thicker bottom slab, 700 millimetres against 600 millimetres for cells denoted with a 1. In the final design it can be further elaborated for optimizing the material usage and required labour.

For the sides the same situation as for the bottom exists, see figure 9.19. The thickness of the side of the FB section is here 750 millimetres, as with the normal sections, but it requires heavier reinforcement. Also the outer compartments cannot be used for ballast as the combination of the applied pre-stress (especially the geometry of the tendons), the pressure from the ballast water and a wave trough will cause tensile forces in the concrete.

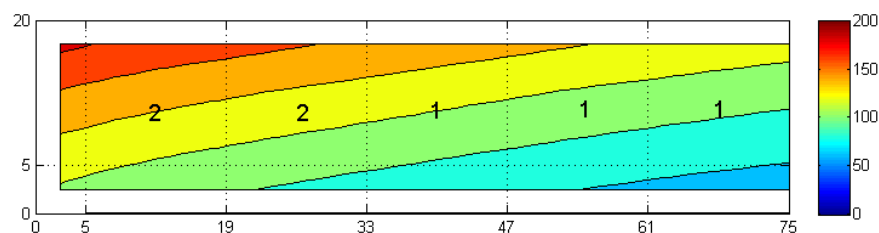


Figure 9.19 Hydrodynamic pressures on the side of the bow/stern section in kN/m^2 (enlarged)

The bow, or front end of the FB element, is loaded by the highest hydrodynamic pressures, see figure 9.20. The required thickness of the bow is 900 millimetres and for the same reason as for the sides, no ballast water can be contained in the outer compartments located at the bow.

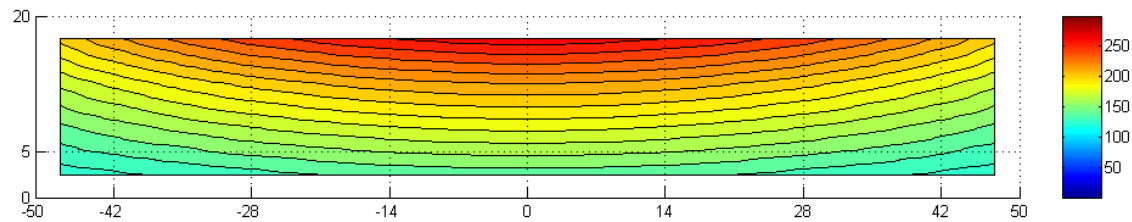


Figure 9.20 Hydrodynamic pressures on the side of the bow/stern section in kN/m² (enlarged)

The ballast of this element will completely exist out of sand as no water can be stored between the outer hull and the inner hull. A part of the ballast water can also be located in other sections in order to minimise the draught during the transport. However it has to be remembered that an uneven division of the (ballast) weight over the length of the element will lead to internal (static) moments. In figure 9.21 a longitudinal cross-section of the section is given. The transverse cross-section is similar to figure 9.14.

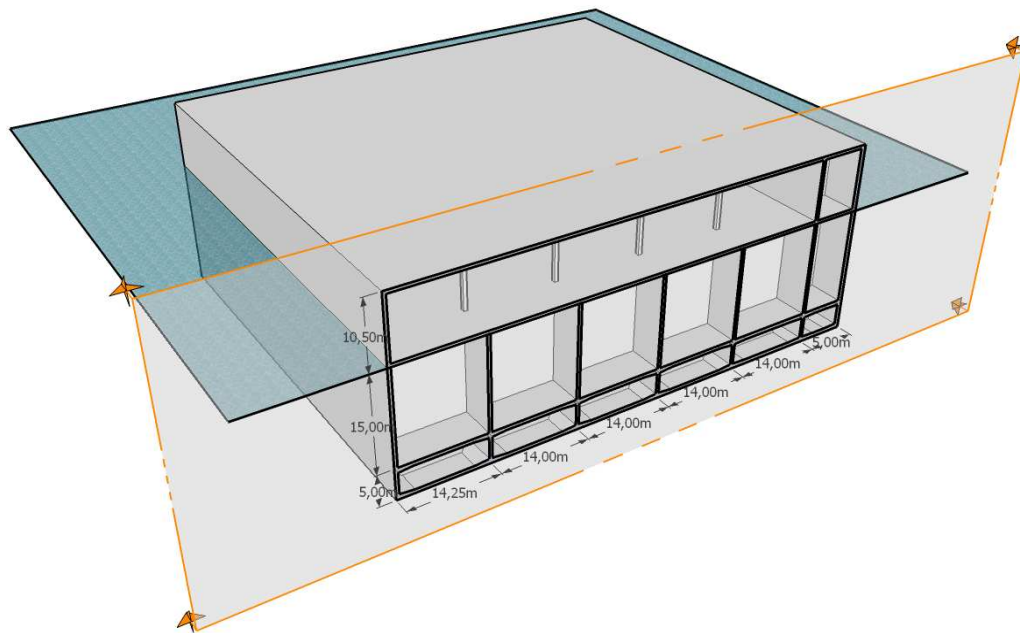


Figure 9.21 Longitudinal cross-section of the bow/stern element

9.5 Mooring system

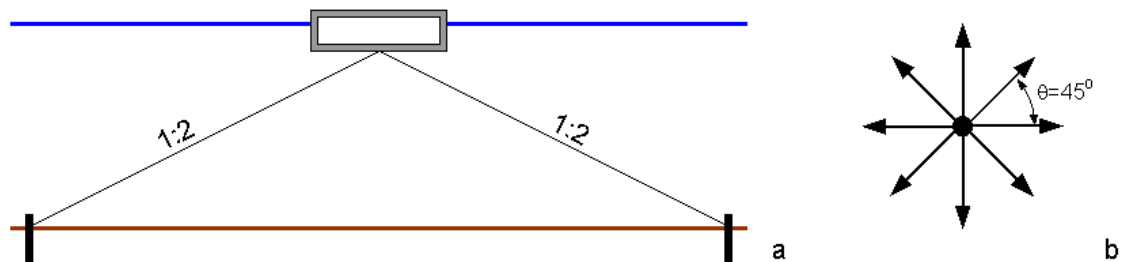
The mooring system is responsible for holding the FB element on its place. An equilibrium has to be sought between a very stiff system, with small deflections and large forces, and a very soft system, with large deflections and small forces. In the design of taut moored structures there are three types of synthetic ropes that can be used, see table 9-6. Each of these ropes does have a higher elasticity than steel, see also figure 8.4 and table 8-1.

Table 9-6 Material properties of mooring lines [Vryhof, 1999]

	Polyester	HMPE	Circular braided nylon
Material	High tenacity polyester	High modulus polyethylene	Nylon
Specific weight	1,38 [kg/m ³]	0,99 [kg/m ³]	1,14 [kg/m ³]
Elongation			
At 20% of MBL	3 [%]	0,8 [%]	16 [%]
At 50% of MBL	6 [%]	2 [%]	22 [%]
At break	12 [%]	4 [%]	>40 [%]
Tensile strength (MBL)			
Diameter 120 mm	6057 [kN]	3924 [kN]	3345 [kN]
Diameter 144 mm	8529 [kN]	5396 [kN]	4679 [kN]
Diameter 168 mm	11301 [kN]	6867 [kN]	6023 [kN]

Polyester has a high ultimate tensile strength combined with a moderate elasticity, HMPE and nylon have both a lower break strength. The elasticity of HMPE is so low that it can be considered as too stiff, resulting in higher forces and thus not suitable for this application. When a water depth of 150 metres is considered, and the mooring line has an angle of 1:2, see figure 9.22, the FB element can translate 40 metres when polyester is applied and more than 100 metres with nylon mooring lines.

The large possible translation that is possible with nylon will reduce the mooring forces, but will impose requirements on the distance between two free-floating elements as they may not hit each other. A larger possible translation will lead to a larger required distance and thus to a higher transmission of the whole system. All wave energy will pass the breakwater on places where no element is present. Therefore it is chosen to limit the motions of the breakwater and polyester is chosen.

**Figure 9.22** Turret mooring system, side view (a) and plan view (b)

Only a limited number of mooring lines are active, i.e. carrying the load. The amount of each mooring line contributing to the transmission of the forces to the subsoil is determined in Appendix G.6. The required number of mooring lines is 16, divided in 8 pairs radiating away from the FB element in successive angles of 45°, see figure 9.22.

9.6 Connections

The sections will be combined to an element in a sheltered area. The connections between the sections should be completely stiff, i.e. the FB should not be influenced by the fact it is made out of several sections. Therefore extra attention has to be paid to the design of the connections between the sections. First a description is made how a watertight connection is made between tunnel sections in the Netherlands. Secondly it will be discussed how this technology can be used in the design of the connection between FB sections. Finally the connection is designed, the calculations can be found in Appendix G.7.

9.6.1 Connections between tunnel elements

In the Dutch immersed tunnel building industry, separate concrete elements are sunk and positioned on the bottom of the waterway. To get a watertight seal between two successive elements a rubber Gina-gasket, see figure 9.23, is applied around the circumference of the element. The Gina-gasket consists of a soft rubber nose, required for the initial water sealing, and a hard rubber body.

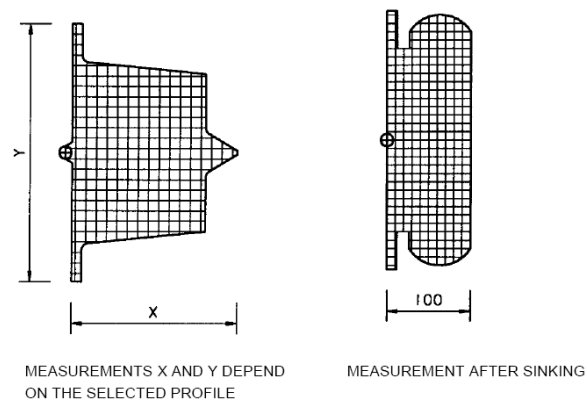


Figure 9.23 The Gina-gasket [Bakker, Huijben et al., 2006]

When the elements are positioned correctly the water that is trapped between the two elements is drained and the elements are pressed against each other due to the hydrostatic pressure. The Gina-gasket now completely seals the tunnel. A secondary rubber seal is applied to the connection to retain any water what is seeping through the Gina-gasket, see figure 9.24. The space between the two elements is now finished with a concrete lining.

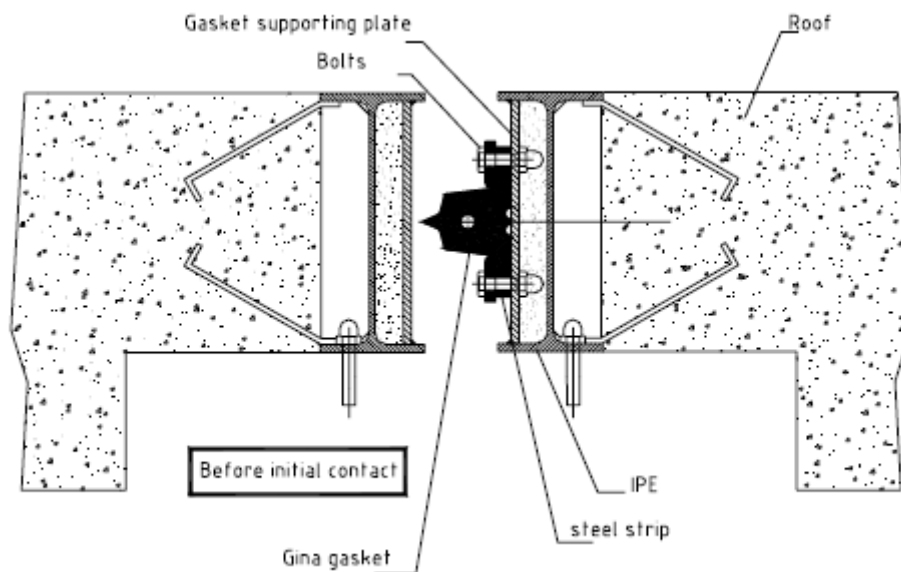


Figure 9.24 Closure using a Gina-gasket, before initial contact²

² Trellborg – Engineered seals
<http://www.trelleborg.com>

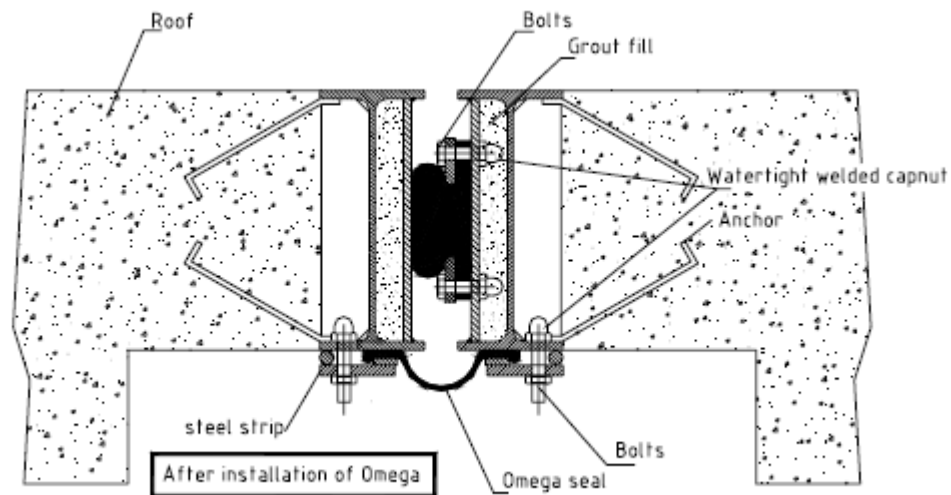


Figure 9.25 Closure using a Gina-gasket, after installation of Omega-seal³

9.6.2 Connections between FB sections

There are some difference between a tunnel section and a FB element:

- A tunnel is completely submerged and the hydrostatic pressure can be used to compress the Gina-gasket. At and above the waterline no hydrostatic pressure is present, so it has to be checked whether a compressive force is present.
- A tunnel section is placed on the bottom and its location has to be guided only in two dimensions. The FB element is a floating object so the sections have to be guided in three dimensions. A guidance structure will be needed that centres the sections and extra (temporary) ballast tanks may be necessary to get a good vertical alignment.

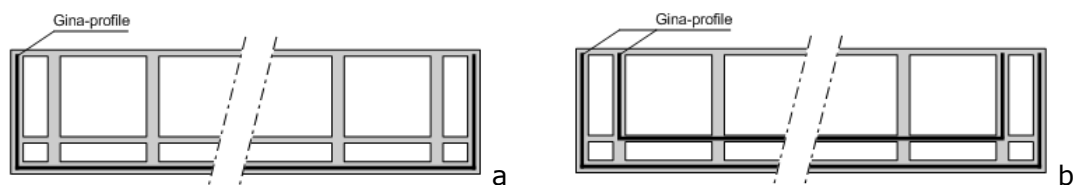


Figure 9.26 Two designs for the Gina-gasket, one gasket (a) and a double gasket (b)

For two designs the minimum and maximum pressures on the Gina-gasket have been determined. The first design includes a single Gina-gasket which is located on the outer hull. The second design has a double Gina-gasket, one on the outer hull and one on the inner hull, see also figure 9.26. All data on both designs is given in table 9-7.

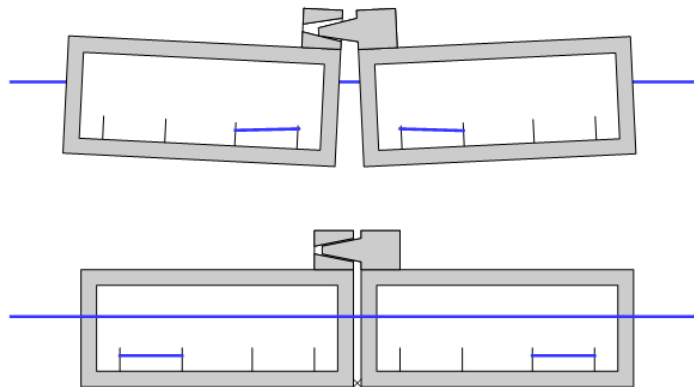
From table 9-7 it can be seen that the application of a double gasket leads to tensile stresses at the top of the Gina-gasket. With a single Gina-gasket no tensile stresses are present. Because no tensile stresses are present, no mitigating measures have to be taken to guarantee a watertight connection. Therefore it is chosen to apply a single gasket as a temporary connection.

³ Trellerborg – Engineered seals
<http://www.trelleborg.com>

Table 9-7 Minimum pressures and maximum pressures on the Gina-gasket for the two designs

	Single Gina-gasket	Double Gina-gasket
Element (all values from keel)		
Height	30 [m]	30 [m]
Beam	100 [m]	100 [m]
Location waterline	15 [m]	15 [m]
Gina profile		
Thickness	0,2 [m]	0,2 [m]
Neutral line	5,63 [m]	7,47 [m]
I_{zz}	2587,5 [m ⁴]	2696,0 [m ⁴]
Forces, moments and stresses		
Hydrostatic force	450000 [kN]	450000 [kN]
Pressure line	5 [m]	5 [m]
Moment	281250 [kNm]	1109694 [kNm]
Min stress (compression at top)	-11,41 [N/mm ²]	<i>tension</i> [N/mm ²]
Max stress (compression at bottom)	-14,67 [N/mm ²]	-10,73 [N/mm ²]

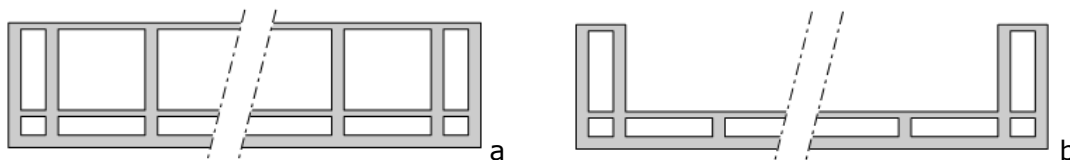
To guide the two sections to desired location a guidance structure is placed on the deck of the elements, see figure 9.27. During the mating process the sections will be tilted, so the top side of the sections will be mated first. The guidance structure ensures the correct relative location of the two elements and will be locked. Now the ballast is shifted in the elements to compress the bottom Gina-gaskets, resulting in a watertight seal. The mating process is finalised by draining the joint and constructing the permanent connection


Figure 9.27 Example of a guidance system (exaggerated) and the mating process

When the sections are mated and the space between them a secondary gasket (Omega Seal) is applied, see also figure 9.25. The permanent connection can now be constructed between the sections. The construction consists of joining the concrete bulkheads between the sections and the installation of pre-stressing tendons.

9.6.3 Required pre-stress

Two designs, both presented in figure 9.28, are evaluated in Appendix G.7.2.


Figure 9.28 Designs for the connection, inner walls and decks connected (a) and not connected (b)

The required pre-tension stresses in the concrete are low:

- For the design where the inner walls of two sections are interconnected, see figure 9.28a, the required stress is 0.46 N/mm². This is equal to a pre-stress force of 168 MN in total.

- For the design where the inner walls of two sections are not interconnected, see figure 9.28b, the required stress is 0.95 N/mm^2 . This is equal to a pre-stress force of 215 MN in total.

To ease the construction it is chosen not to connect the inner walls.

The ducts for the pre-stress cables are installed and the walls are poured in the dry working space. When the concrete has cured the pre-stress tendons are installed in the ducts and tensioned up to the required level. In this way a complete element, which is completely under compression in all conditions, can be assembled by individual elements.

9.7 Conclusion

In this chapter a FB element has been designed with a length of 500 metres, a beam of 100 metres and a draught of 20 metres. As it is impossible to construct a structure with such dimensions, it is divided into 7 transverse sections.

Each section is designed for the forces that may occur due to:

- Construction;
- Waves during service combined with a collision;
- Extreme waves;
- Drift forces;
- Internal moments.

The dimensions of the sections and the total dimensions are given in table 9-8. An impression of the completed element is shown in figure 9.29.

Table 9-8 Minimum pressures and maximum pressures on the Gina-gasket for the two designs

Section	Normal	Turret mooring	Bow/stern	Total element
Dimensions				
Length	70 [m]	70 [m]	75 [m]	500 [m]
Beam	100 [m]	100 [m]	100 [m]	100 [m]
Draught	20 [m]	20 [m]	20 [m]	20 [m]
Displacement	1.400.000 [kN]	1.400.000 [kN]	1.500.000 [kN]	10.000.000 [kN]
Mass				
Concrete	896.400 [kN]	896.400 [kN]	1.197.000 [kN]	6.876.000 [kN]
Sand ballast	370.696 [kN]	370.696 [kN]	303.000 [kN]	2.324.088 [kN]
Water ballast	132.904 [kN]	132.904 [kN]	0 [kN]	398.712 [kN]
Total	1.400.000 [kN]	1.400.000 [kN]	1.500.000 [kN]	10.000.000 [kN]

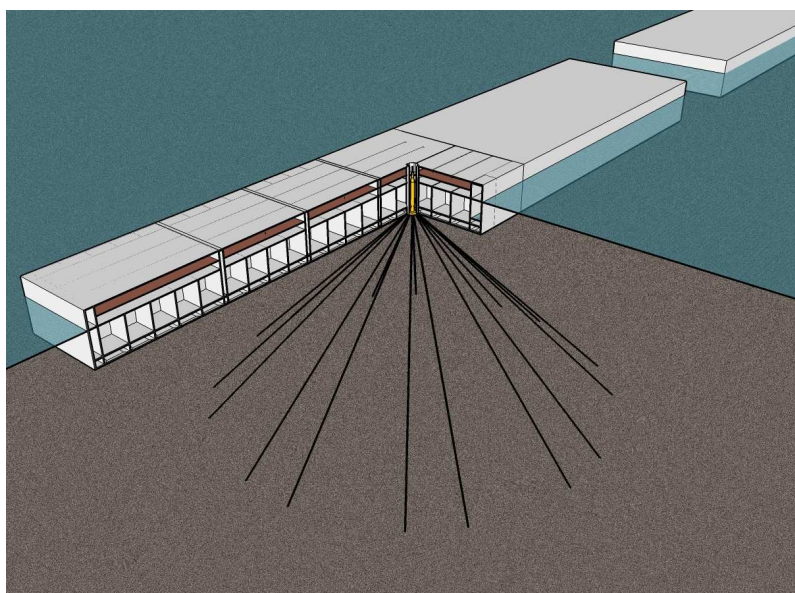


Figure 9.29 Impression of the designed element

Chapter 10 Conclusions and recommendations

This chapter will summarize the most important conclusions that have been made in this master thesis. Although the contribution of the most important parameters in a two dimensional situation to the wave attenuating performance of the floating breakwater have been studied in this research, many question have raised that will need further explanation. Recommendations on further research are discussed in this chapter as well.

A floating breakwater is an essential element for large scale floating structures to become a success, if its function is not integrated in the floating structure itself. That floating harbours, such as the Floating Transshipment Container Terminal, can be used more efficient when the wave attack on these terminals can be reduced has been proved in previous reports.

The missing link in designing an effective floating breakwater is a design-tool to make quick preliminary designs which can be developed further. A good step has been made by [Fousert, 2006] by introducing the ReFBreak-model. The dynamic behaviour of a floating breakwater depends on a lot of factors and is directly connected to the wave-attenuating performance. For beam-on waves it has been shown that *"the performance of the floating breakwater is primarily determined by the draft-mass-width relation of the structure"* [Fousert, 2006]. This model should be adapted and expanded to understand the effects of the structure's dimensions in irregular wave-fields and the effects of changing wave conditions.

The main objective of this thesis is twofold:

- Determine the influence of the cross-sectional dimensions on the dynamic behaviour of the floating breakwater by adapting and expanding the ReFBreak-model;
- Create a conceptual, structural design of a floating breakwater element on basis of the forces that are expected, to prove that a floating breakwater is technically feasible;

In order to reach these objectives an improved calculation and an improved model is developed to overcome the problems and limitations of the original ReFBreak-model. The ReFBreak2D-model is based on the following assumption:

- Only beam-on regular waves are considered in order to find the transfer function of the optimal floating breakwater;
- Irregular waves are seen as a superposition of regular waves;
- The energy a wave contains below the keel of the element is assumed to pass undisturbed;
- The influence of wind is neglected
- Energy losses due to viscous damping is neglected or user defined;
- Mooring influence on first order motions is neglected or user defined.

The ReFBreak2D-model has been used to assess the performance of various floating breakwater designs. The optimal design has been detailed to come to an preliminary design that meets with the requirements on wave-attenuating performance, floatability, constructability and structural strength.

10.1 Conclusions

The conclusions that can be drawn from this research are:

- The performance of the FB is primarily determined by the beam/draught ratio (Y_w/T_1 -ratio) and the location of the centre of gravity.
- It is theoretically possible to attenuate waves transmitted by the underflow and the combined sway/roll motion with the waves generated by the heave motion. This occurs when the FB is tuned to have its heave natural frequency near the wave frequency and the structure is loaded with regular waves.
- For an effective floating breakwater in irregular waves a low transmission coefficient is required on a wide range of frequencies.
- A rectangular floating breakwater having an Y_w/T_1 -ratio of 5.0 and a high COG is the most optimal design.
- A system of two floating breakwaters is required to meet the requirements. The first breakwater, with a beam of 100 metre and a draught of 20 metres, is able to attenuate waves with wave periods up to 14 seconds, but it lacks performance on attenuating waves with a period from 6 to 10 seconds. The second breakwater, with a beam of 40 metre and a draught of 8 metres, is able to attenuate waves with wave periods of 6 to 10 seconds.
- A system of floating breakwaters enables vessels to berth at all sides of the floating breakwater until Beaufort 7. Without the system, the ships will only be able to use the lee-side of the harbour.
- From Beaufort 8 and higher the ships have to leave the harbour and all harbour action has to be stopped. In these conditions the FB elements and FTCT will weathervane to minimise the force on the mooring.
- The breakwater is best to be constructed in sections; each of them can be manufactured in a factory-wise method. This will speed up the construction and will lead to a more economical design.
- The displacement of the 1st FB, which has been elaborated, is 1,000,000 tonnes, of which 270,000 tonnes are ballast, consisting of sand and water ballast. The displacement of the 2nd FB, which has not been elaborated, is 160,000 tonnes.

10.2 Recommendations for further research

Many assumptions were used during this theoretical research to make it possible. The assumptions can be the basis for further research on this topic. The recommendations for further research are:

- The model has been calibrated with the use of an extra DELFRAC-simulation. This calibration showed that in the two-dimensional case all hydrodynamic parameters are correct when compared to the simulation. Further model-tests have to be performed to verify the results of the simulation and the ReFBreak2D-model on the hydrodynamic parameters and wave transmission coefficients.
- The effect of oblique waves and other three-dimensional effects, such as diffraction, were not involved in this report as it is considered a two-dimensional case. Calculations in the three-dimensional space will provide essential information on the performance of the floating breakwater in real ocean conditions.

- A floating harbour, in this case the FTCT, was considered as the area that had to be protected. An allowed wave spectrum has been assumed, where the influence of the moored ship and the floating harbour were neglected. The total FB-ship-FTCT system has to be analyzed in order to determine the exact requirements on wave-attenuating performance that the floating breakwater must meet.
- A system of floating breakwaters is chosen as the optimal design. Its performance has been determined with a simple model, accounting for transmission and reflection. The mutual influence of the FB elements needs further research to assure that the required level of wave attenuation can be reached also in a three-dimensional case.
- The effect of the width on the underflow is not considered in this research while the width of the chosen alternative is considerably large. In previous reports, [Jones, 1971] determined a positive influence of an increasing width relative to the wave-length. Practical research is required to determine the width-underflow relation in deep water and the influence of this relation of the wave-attenuating performance.
- The effect of a screen has not been considered in this report. In his report, [Fousert, 2006] determined that a screen underneath the structure may have a positive influence on the wave-attenuating performance when wave periods shorter than 10 seconds are considered. A screen will limit the energy transmission by the underflow phenomena, but it will possibly increase the transmission due to the combined sway/roll motion. It has to be investigated whether a screen may be beneficial and what the optimal length will be.
- The design presented in this report does not have connections between the FB elements; they are all free-floating. The optimal width of the gap between the elements has to be determined and is a compromise between wave transmission and the risk of damage when the elements hit each other.
- Overtopping is not considered in this study while it may occur. Deck-wetness will influence the hydrodynamic behaviour and the water washed over the top will influence the wave transmission. In further research this subject should be investigated.
- The detailing of the structure deserves more attention as only a preliminary design was presented in this report. A more detailed design can be made after further research shows that the floating breakwater concept is feasible, both in a technical and economical way in real ocean conditions.

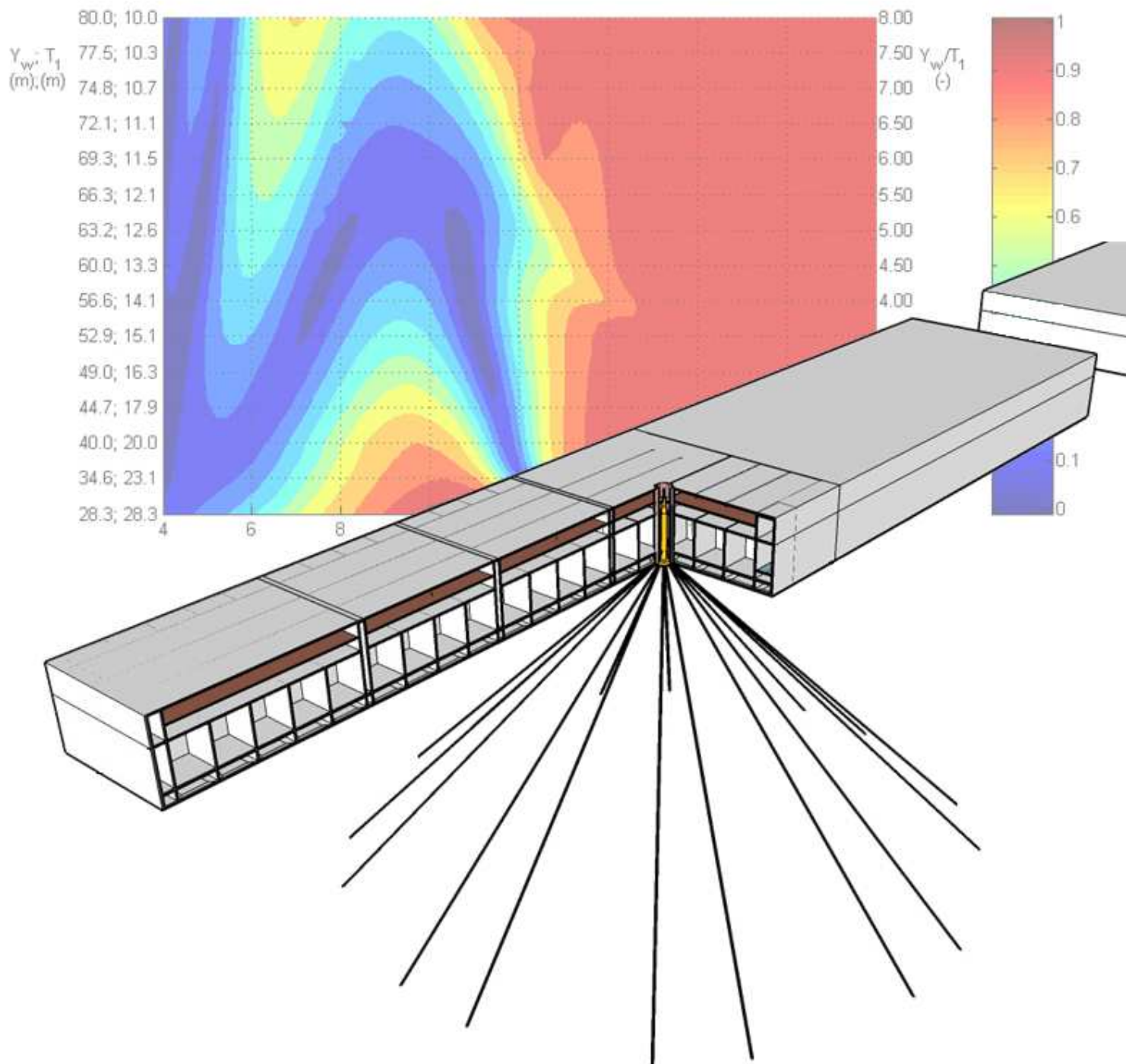
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Floating breakwaters

A theoretical study and preliminary design of a dynamic wave attenuating system



P.T.G. van Tol
Delft, 25 April 2008

Appendices

Colophon

Floating breakwaters - Appendices

A theoretical study and preliminary design of a dynamic wave attenuating system

Appendices belonging to the final report of the master thesis

Delft, 2008

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Front cover: Wave transmission coefficients and impression of the final design

Appendix A Performed research

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Appendix A Performed research

From 1970 until now the interest in floating breakwaters has increased and lots of research has been done on this topic. The results of the studies about these displacement type floating breakwaters are summarised in this appendix. Research on other types of floating breakwaters is not discussed as it is considered as out of scope for this thesis.

A.1 Model tests

The performed research is divided into:

- the effects of the degrees of freedom;
- the effects of the structural dimensions;
- and the effects of oblique waves.

The performance of the floating breakwaters is expressed in a wave transmission coefficient, C_t or K_t . A structure with a transmission coefficient of zero is the ideal floating breakwater as all waves are attenuated. A structure with a transmission coefficient of one does not have an influence on the wave-field. The reflected coefficient is denoted as C_r or K_r and in the ideal case with no viscous damping the summation of C_r and C_t should result in one, as no energy is lost.

A.1.1 Effects of the degrees of freedom

In the study of [Tolba, 1999] the effects of the degree of freedom were tested to develop a breakwater which could also function as a pier. The degrees of freedom that were tested:

- No degrees of freedom;
- Only roll;
- Only heave;

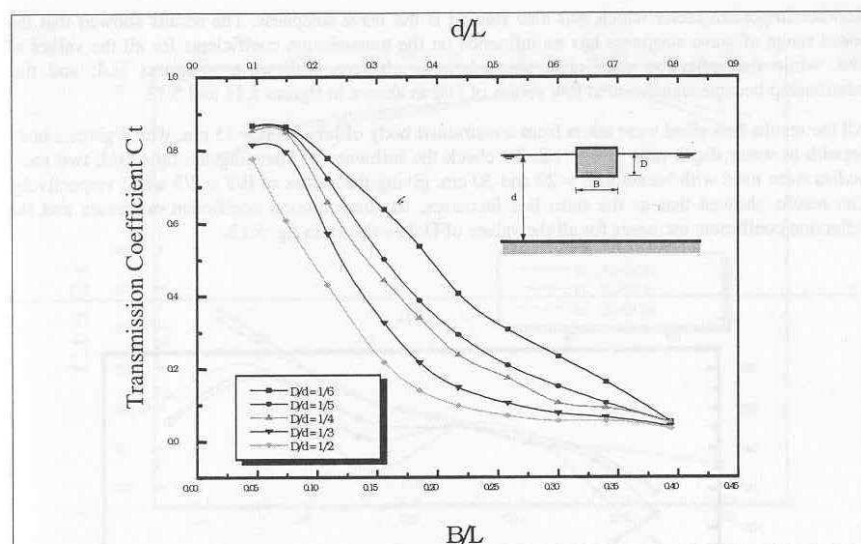


Figure A.1 Transmission coefficients for a restraint body [Tolba, 1999]

With no degrees of freedom, i.e. a completely restraint model, the rate of wave attenuation is depending on the relative draft of the model and the relative width, i.e. the ratio between the length of the model and the wave length. The relative draft of the model determines to amount of wave energy that can pass under the

structure. The relative width of the model determines the disturbance of the orbital motion of the water particles.

For a relative width, or B/L , of 0.15 and higher the breakwater is efficient for all values of the relative draft, see figure A.1. This is due to the exponential decrease of the wave-energy with increasing depth. For very short waves, and hence thus a high relative width, most wave energy is contained in the upper part of the water column. This part is completely reflected by the restraint model.

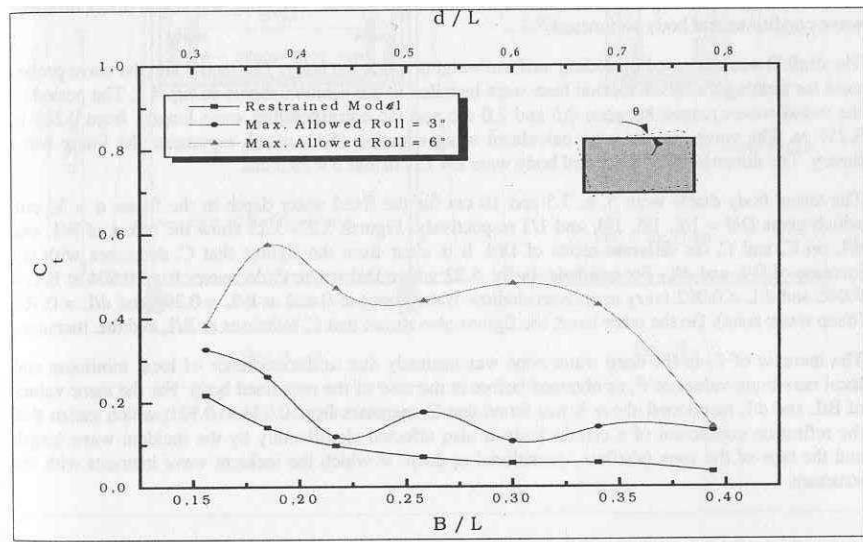


Figure A.2 Effect of the allowed angle of roll on the transmission coefficient [Tolba, 1999]

With only roll it was noted by [Tolba, 1999] that the transmission coefficients were increased in comparison to the restraint model. This is due to the fact that the roll-motion of the breakwater generates waves which are in phase with the transmitted wave under the structure, thus leading to higher transmitted waves, see figure A.2. Therefore it has been concluded that roll should be avoided, resulting in a heave-only model.

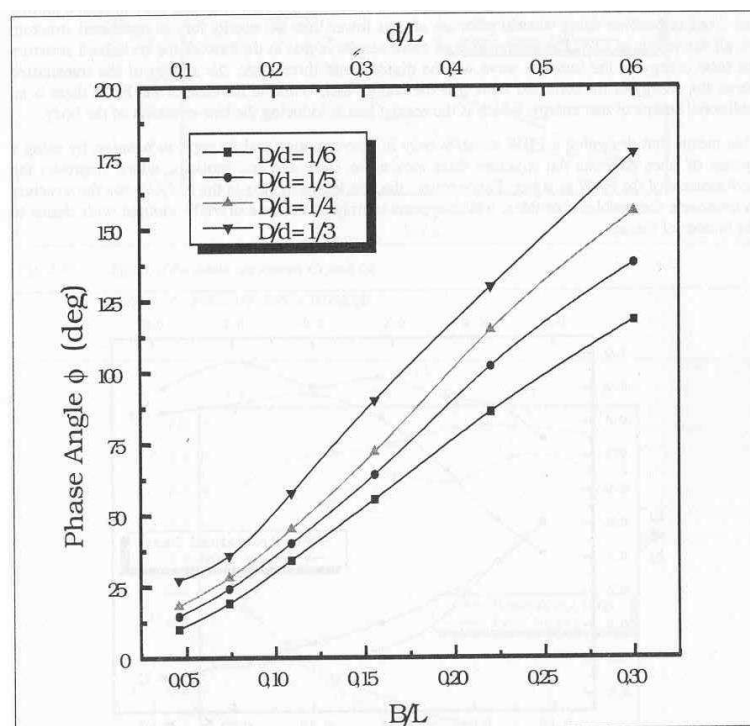


Figure A.3 Phase angle between the incoming waves and the heave motion [Tolba, 1999]

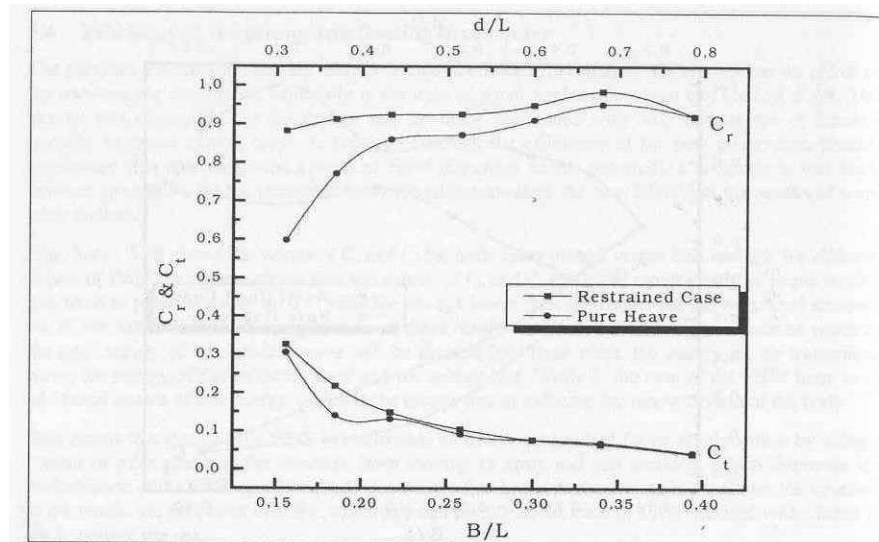


Figure A.4 Effect of the heave motion on the transmission coefficient [Tolba, 1999]

With the heave-only model it appeared that the wave-attenuating performance of such a structure is better than a restraint structure. It also showed that the performance depends on the structural dimensions of the breakwater. This can be explained due to the fact that the breakwater is moving out of phase with the incoming waves, creating an out-of-phase damping, see figure A.3 and figure A.4. The model, developed by [Fousert, 2006] can be tested with this data if the model is adapted to heave-only and only long waves are considered.

To prevent sway [Tolba, 1999] used poles to anchor the breakwater. The goal of this solution was to use the positive effects of the heave motion while the performance of the breakwater is not hampered by the negative influence of the sway motion. In his thesis [Silander, 1999] proved that this assessment was correct. Figure A.5 clearly shows that, for a swaying structure, the transmission coefficient is reduced when the spring constant is increased; increasing the spring constant leads to a reduction of the sway motion.

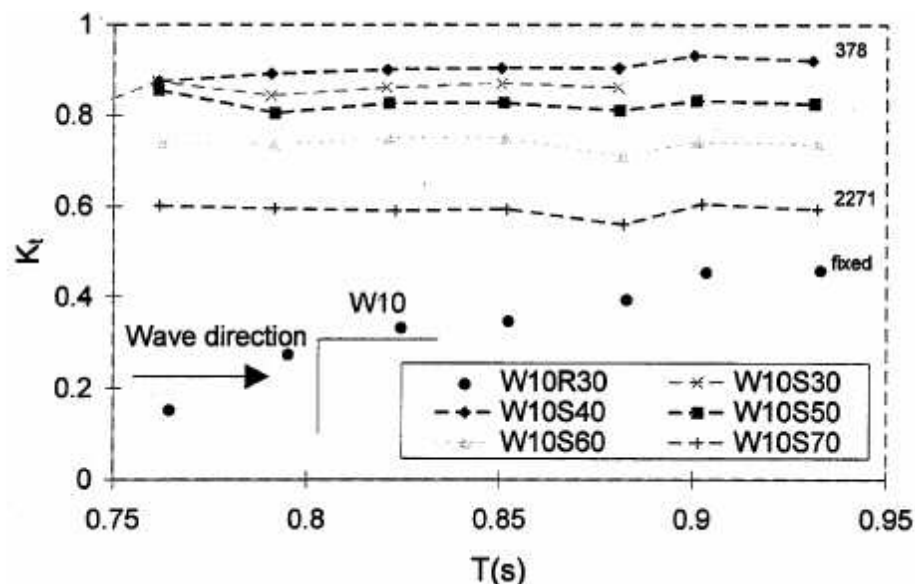


Figure A.5 Transmission through a single swaying barrier (**S,---) in the wave flume. (W10S**) stands for different spring systems. The structures had the same draft and no pipes as indicated in the figure. The spring constant ranged from 378 N/m for W10S40 to 2271 N/m for W10S70 [Silander, 1999]

A.1.2 Effects of the structural dimensions

Several structural dimensions can be changed to optimise the design of the floating breakwater, which include:

- The beam, or width, of the structure relative to the wave length;
- The draft of the element relative to the depth.

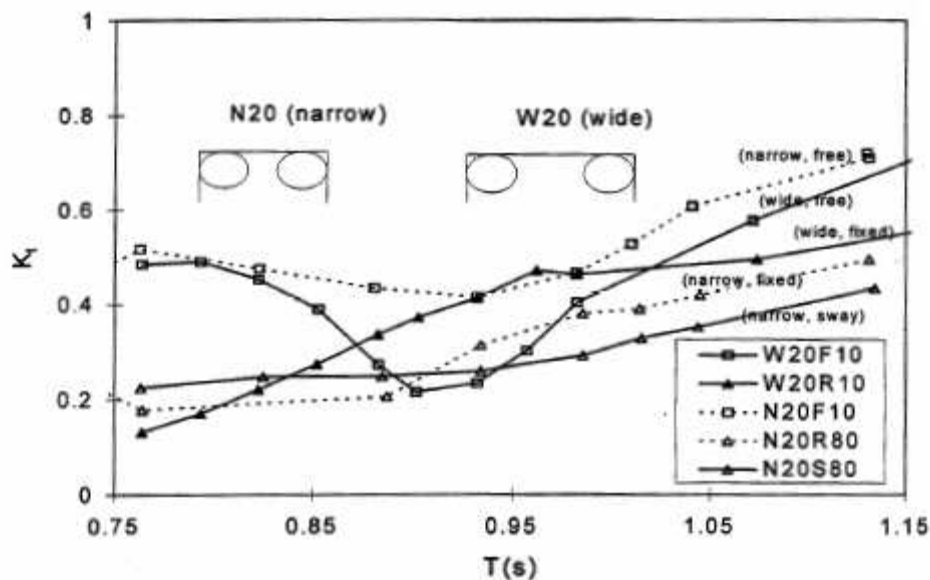


Figure A.6 Transmission through a normally anchored 0.4 meter wide (W20F) and a 0.3 meter wide (N20F) structures having the same draft. The (***)S stands for structures where sway is allowed, while (***)R stands for fixed structures. [Silander, 1999]

Structural dimensions do have an influence on the wave attenuating performance of floating breakwater, as proven by [Tolba, 1999] for a sway-limited breakwater and by [Silander, 1999] for fixed as well as freely floating breakwaters. In figure A.6 five types of structures are compared. It can be seen that a fixed floating breakwater, i.e. no sway motions allowed, can attenuate waves better than a freely floating, i.e. all motions are allowed, in a certain frequency range. Also figure A.6 shows that a wider structure does attenuate the waves better than a narrow structure does. This may be due to the fact that a narrow structure is not able to generate (positive) heave waves as high as a wide structure does. [Fousert, 2006]

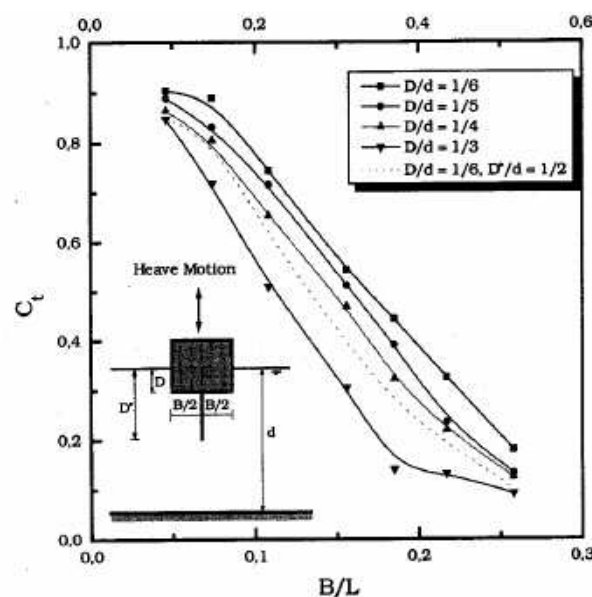


Figure A.7 The effect of the relative draft on the wave transmission coefficient [Tolba, 1999]

As a way to improve the breakwater [Tolba, 1999] did a test with a vertical rigid plate attached to the bottom of the floating body and extended downwards into the water. As shown in figure A.7, the immersed cross sectional area of the breakwater required to attenuate the waves to a certain value, can be minimised by using this plate. This leads to a more economic design.

A.1.3 Effects of oblique waves

The directional factor has received relative little attention and there is reason to expect that the generation of waves due to the sway motion of the breakwater will be reduced when the waves hit the structure under an angle. [Richey and Nece, 1974]

In his test [Silander, 1999] used wave angles of 90° , 75° and 60° to test the effect of oblique, irregular, waves on a floating breakwater. The results of this test are presented in figure A.8. This figure clearly indicates, that with an increasing wave angle the wave transmission coefficient is reduced. For seas there is a large directional spread of the wave spectrum, leading to a reduction of the transmission coefficient.

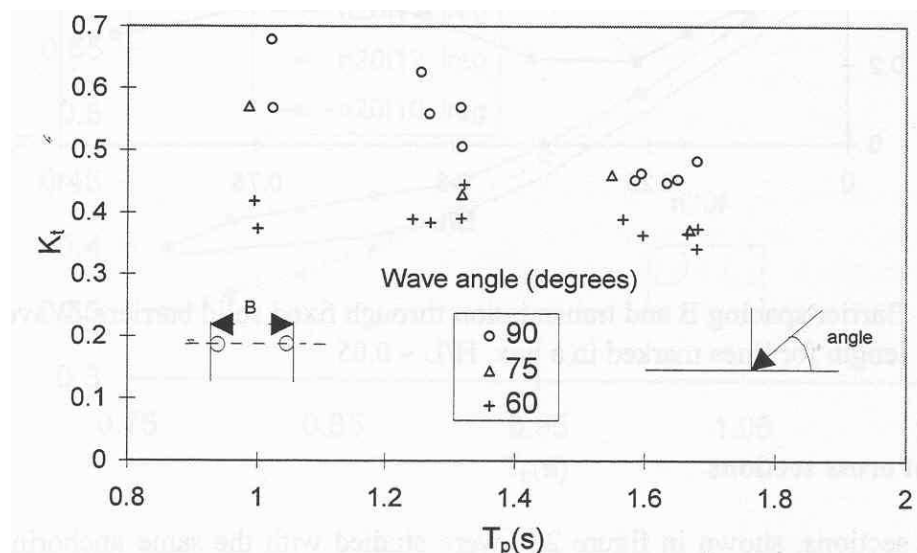


Figure A.8 The effect of the wave angle on the transmission coefficient for a 1.1 meter wide structure in a wave basin [Silander, 1999]

A.2 Proposed model test

In a earlier stage of this master thesis it was proposed to perform model tests to verify or falsify the result of the ReFBreak-model developed by [Fousert, 2006]. The wave basin of the TU Delft could be used to test the performance of a floating breakwater with both beam-on waves and oblique waves.

A.2.1 Wave basin

The wave basin that could be used for the model test is located at the faculty of Civil Engineering and Geosciences of the TU Delft. The dimensions of the basin are a width of 16.60 meter and a length of 28.60 meters. The maximum water-depth is 60 centimetres.

The waves are generated by three, piston type, hydraulically-driven wave generators. There is no compensation for reflected waves on the wave generators; also the sides of the basin are vertical, reflecting all incoming waves. The combination of these two facts leads to the necessity to seize any measurement as soon as the reflected waves hit the model or wave generator again.

A.2.2 Scale model

The floating breakwater will be modelled using a Froude-scale. This results in a different scale for length and time:

- Length : X ;
- Area : X^2 ;
- Volume : X^3 ;
- Force : X^3 ;
- Time : $\sqrt{X}$;
- Speed : $\sqrt{X}$;
- Acceleration : 1.

Because the ReFBreak-water assumes deep water, the maximum wave-length to water-depth ratio is 2.0. This results in a maximum wave-length of 80 centimetres when a water-depth of 40 centimetres is used. The wave-length is the most stringent criterion in determining the scale factor.

For wave periods up to 16 seconds a scale of 1:500 is needed in order not to exceed the maximum wave length of 80 centimetres. For wave periods of up to 11 seconds a scale of 1:250 is sufficient to fulfil the wave-length criterion.

To verify the ReFBreak-model two different scale-models can be used, one with the scale of 1:250 to test the dynamic behaviour in the wave period range of 6 to 11 seconds and one with the scale of 1:500 for wave periods of 10 seconds to 16 seconds. The overlap between the two scales is needed to check for different scale-effects.

By using two scales the accuracy of the model-test can be improved and it avoids the use of very small wave-lengths. The smallest wave period the wave-machine has to produce is 0.4 seconds.

A.2.3 Problem

A test-run has been made without a model to test the ability of the machine to produce waves with these small wave periods. From this test-run it could be concluded that the machine at the faculty was not able to produce these waves with a significant wave height and with a sinusoidal shape.

Wave basins that could possibly generate these waves are the Vinjé-basin at WL|Delft Hydraulics or the deep water basins at Marin.

It is concluded that for this master thesis it is not possible to do model tests, therefore a new subject within the field of floating breakwaters has been chosen.

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Appendix B Evaluation ReFBreak

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Appendix B Evaluation ReFBreak

The ReFBreak-model has been developed by [Fousert, 2006] in order to assess the influence of the structural dimension of the floating breakwater. This paragraph is a summary of the work that has been done.

The basis of the model is the flowchart that is depicted in figure B.1. Incoming waves are responsible for exciting forces and moments on the structure. These forces and moments will put the floating breakwater in an oscillatory motion, which in turn will cause waves radiating from both sides of the breakwater. The total transmitted wave towards the harbour-side of the floating breakwater is found by the summation of all radiated waves and the wave that passes underneath the breakwater.

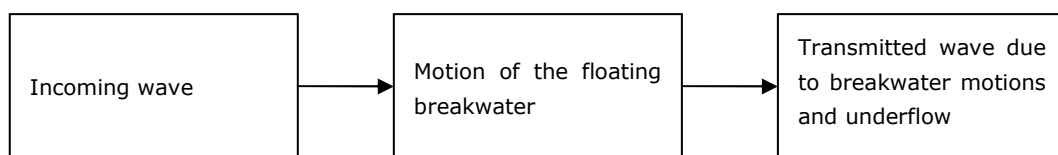


Figure B.1 Model flowchart

To assess the validity of the model, the dynamic behaviour of a FB will be investigated. The equations governing these motions will be treated and the hydrodynamic parameters that are of importance for these equations. The wave exciting forces and moments are required to find the steady-state solution; these are calculated with the Froude-Krilov assumption, i.e. the waves are assumed not to be distorted by the presence of the breakwater. The equations of motion are then solved for the sway, heave and roll motion. Finally the radiated waves are calculated and a summation is made to find the total transmitted wave.

B.1 Dynamic behaviour

A floating body in a three dimensional space has 6 degrees of freedom:

1. Surge
2. Sway
3. Heave
4. Roll
5. Pitch
6. Yaw

These motions are defined by the translation of the centre of gravity in the x-, y- and z-direction and the three rotations about those axes. This is shown in figure B.2, where the numbers are corresponding to the respective motion. [Journée and Massie, 2001]

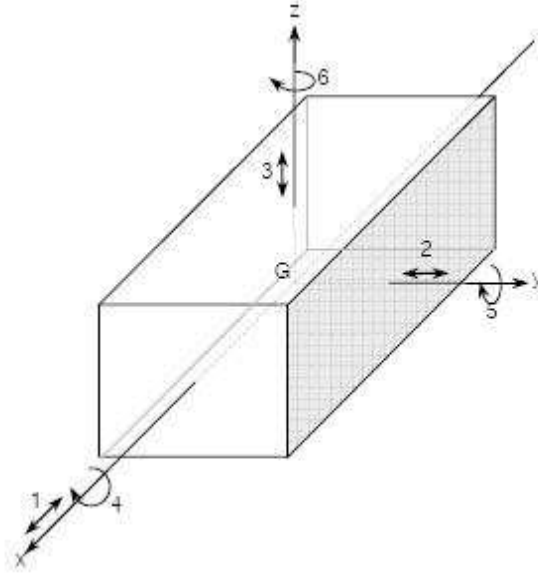


Figure B.2 Degrees of freedom for a floating body in a three-dimensional space [Fousert, 2006]

With respect to the axes in figure B.2 the formulas for the three translations and the three rotations are defined. With the motions of the centre of gravity (COG) known the motions in any point on the structure can be calculated by the use of superposition. The motions are:

$$\begin{aligned}
 x(t) &= x_a \cos(\omega t + \varepsilon_{x\zeta}) \\
 y(t) &= y_a \cos(\omega t + \varepsilon_{y\zeta}) \\
 z(t) &= z_a \cos(\omega t + \varepsilon_{z\zeta}) \\
 \phi(t) &= \phi_a \cos(\omega t + \varepsilon_{\phi\zeta}) \\
 \theta(t) &= \theta_a \cos(\omega t + \varepsilon_{\theta\zeta}) \\
 \psi(t) &= \psi_a \cos(\omega t + \varepsilon_{\psi\zeta})
 \end{aligned}
 \tag{B.1}$$

Where:

n_a	: Motion amplitude	[m], [rad]
$\varepsilon_{n\zeta}$	: Phase angle	[rad]
ω	: Circular wave frequency	[rad/s]

The motions are not independent in respect to each other. However, since only beam-on waves are considered, surge, pitch and yaw will not occur when a symmetric structure is assumed.

Lateral forces or moments can not be produced by the vertical motion of floating breakwater due to symmetry of the structure. Also, [Vugts, 1968] proved that the Φ - and y -components in the wave exciting forces are relatively small in respect to the z -components. Therefore heave can be considered as an uncoupled motion. [Fousert, 2006] considered heave, sway and roll as uncoupled motions.

The floating breakwater is loaded by waves. The wave elevation can be defined as:

$$\zeta(x, y, t) = \zeta_a \cos(\omega t - \sin(\mu)kx - \cos(\mu)ky)
 \tag{B.2}$$

Where

ζ_a	: Wave amplitude	[m]
ω	: Wave frequency	[rad/s]
k	: Wave number	[m ⁻¹]
μ	: Angle of attack	[rad]

For a 2 dimensional case with beam-on waves equation (B.2) can be simplified to:

$$\zeta(y,t) = \zeta_a \cos(\omega t - ky) \quad (\text{B.3})$$

The phase shifts of the previous mentioned motions are related to the harmonic wave elevation at the origin, i.e. the centre of gravity, even though no wave can be measured at this location. The wave elevation in the centre of gravity ($y = 0$):

$$\zeta_{COG}(t) = \zeta_a \cos(\omega t) \quad (\text{B.4})$$

B.2 Equation of motions

It is assumed that a floating breakwater will not generate waves by itself due to structural movements when floating in still water. The only hydro-mechanical reaction will occur when the structure is loaded by waves, causing wave exciting forces and moments. In other words the floating breakwater does not generate energy; it can only absorb energy of waves and re-transmit this energy. It is assumed that this relation is linear, i.e. doubling the input wave amplitude results in doubled output amplitude while possible phase changes between input and output will not change.

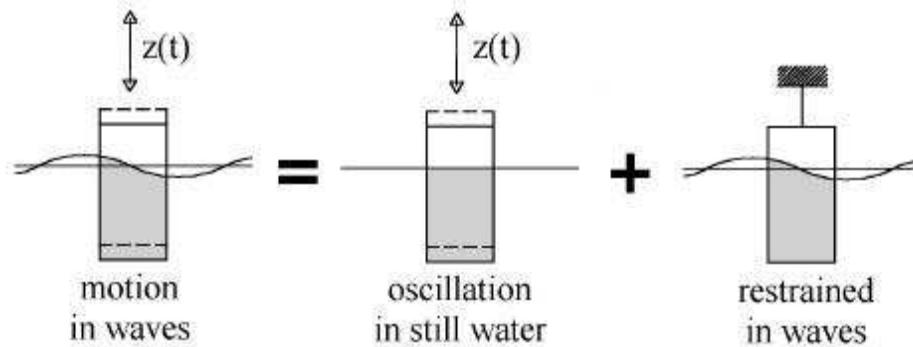


Figure B.3 Superposition of hydro-mechanical and wave loads [Journée and Massie, 2001]

Since the system is assumed to be linear, the resulting motion in waves can be seen as a superposition of the body in still water and the forces on a restraint body, as can be seen in the picture equation in figure B.3. This leads to two important assumptions [Journée and Massie, 2001]:

1. The hydro-mechanic forces and moments are induced by the harmonic oscillations of the rigid body in the undisturbed surface of the fluid;
2. The wave exciting forces and moments are produced by the waves coming in on the restraint body.

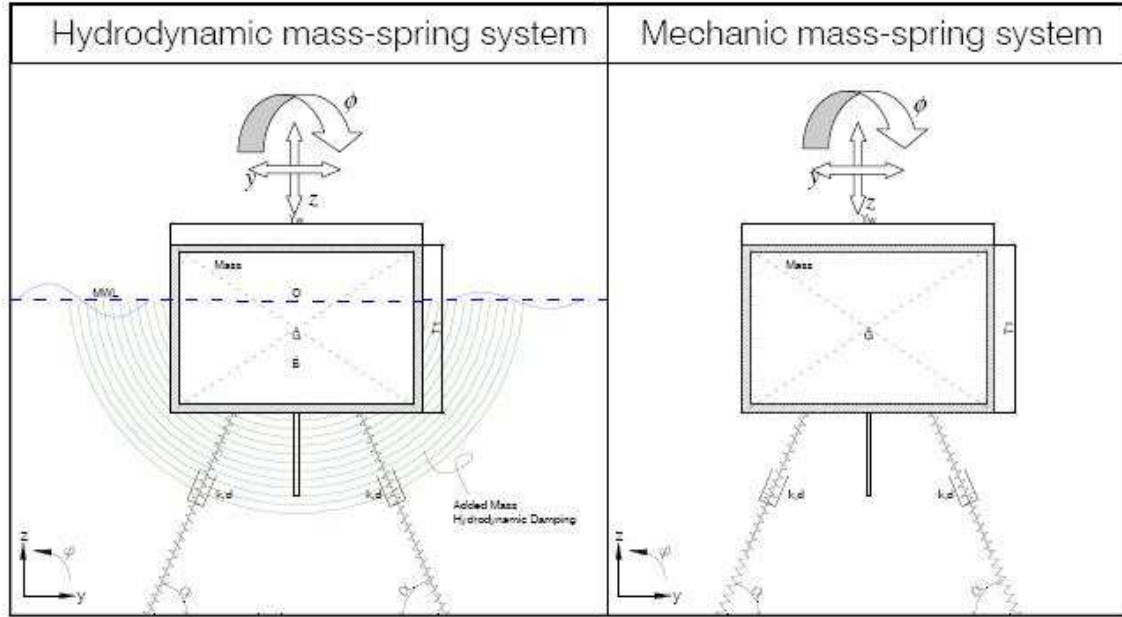


Figure B.4 A 2-dimensional hydrodynamic and mechanical mass-spring system [Fousert, 2006]

The floating breakwater can now be described with a hydrodynamic mass-spring system, as depicted in figure B.4, and the corresponding second-order linear differential equation [Fousert, 2006]:

$$\begin{aligned} (m + a_{22})\ddot{y} + (d_{cy} + b_{22})\dot{y} + (k_{cy} + c_{22})y &= F_{w;hor} \\ (m + a_{33})\ddot{z} + (d_{cz} + b_{33})\dot{z} + (k_{cz} + c_{33})z &= F_{w;ver} \\ (I_x + a_{44})\ddot{\phi} + (d_{c\phi} + b_{44})\dot{\phi} + (k_{c\phi} + c_{44})\phi &= M_{w;g\phi} \end{aligned} \quad (B.5)$$

Where:

- y, z, ϕ : Displacement in the y -, z -, and ϕ -direction
- a_{nn} : Mass or moment of inertia, including the hydrodynamic component;
- b_{nn} : Damping, including the hydrodynamic component
- c_{nn} : Spring constants
- F_w, M_w : Wave induced forces and moment

B.3 Hydrodynamic calculation parameters

In the ReFBreak-model the curves for the added mass and the hydrodynamic damping have been approached by the use of polynomials. Between the polynomial curves and outside the area that was investigated by [Vugts, 1968] the coefficients are interpolated and extrapolated respectively.

B.3.1 Added mass

The interaction of the oscillating (wetted) surface of the floating breakwater and the water particles accelerated by this oscillating surface is called the hydrodynamic mass or added mass. This hydrodynamic reaction force does not dissipate energy and manifests itself as a standing wave near the surface of the breakwater. Figure B.5 reflects the added mass coefficients for heave and sway for several breadth-to-draught ratios and their dependence on the dimension and wave frequency.

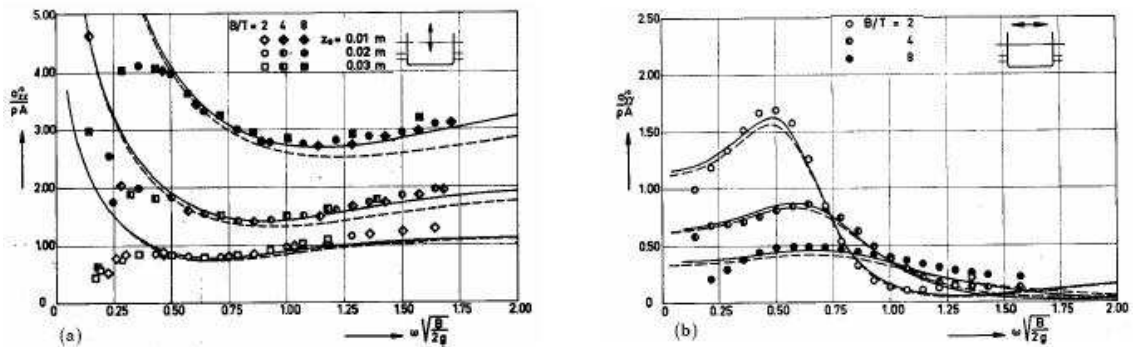


Figure B.5 Added mass coefficients for the heave motion (a) and the sway motion (b), valid for deep water [Vugts, 1968]

For roll [Vugts, 1968] performed numerous tests to determine the roll added mass, but these coefficients were not used in the ReFBreak-model. These tests were carried out with fixed roll amplitudes, while the roll amplitudes of the floating breakwater are not known beforehand. Instead of these measurements a three dimensional potential model, DELFRAC, has been used.

B.3.2 Hydrodynamic damping

The hydrodynamic damping is related to the amount of wave energy that is radiated by the floating breakwater to both sides of the structure. Because the radiated waves do dissipate energy from the breakwater's motions this is the reason the motion dies out. Since the magnitude of the hydrodynamic damping depends, just as for the added mass, on the interaction of the water particles with oscillating breakwater surface, the same factors will be involved. This is shown in figure B.6.

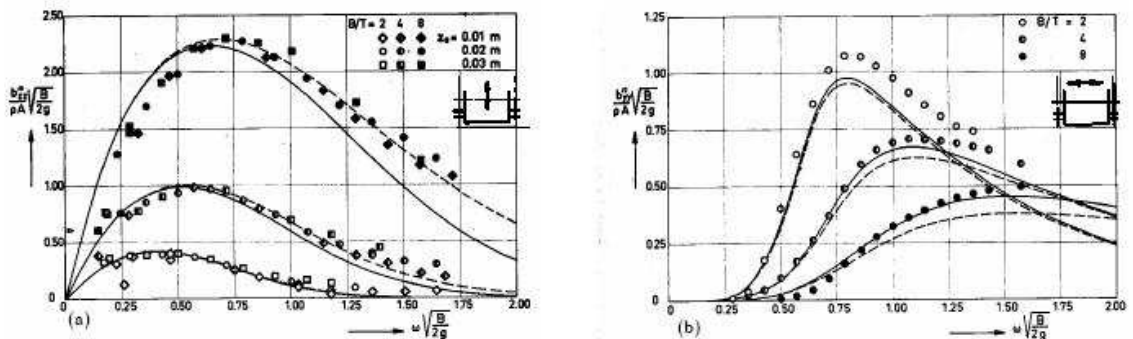


Figure B.6 Damping coefficients for the heave motion (a) and the sway motion (b), valid for deep water [Vugts, 1968]

For the same reason as with the roll added mass coefficient DELFRAC has been used for the determination of the roll hydro-dynamic damping.

B.3.3 Spring stiffness

The restoring force which pushes the floating breakwater back to its old position after an initial displacement is called the hydrodynamic spring coefficient. This force is produced by the change in water pressure underneath the floating breakwater when it is displaced from its initial position; this is based on the Archimedes principle, or the law of upthrust. According to this principle, the hydrodynamic spring stiffness is only possible in motions with vertical components.

The spring force can be calculated using the principle of Archimedes. As a spring stiffness is defined as:

$$F = k \cdot x \quad (B.6)$$

Where:

F	: Force due to the displacement;	[N]
k	: The spring stiffness;	[N/m]
x	: The displacement	[m]

The hydrodynamic spring coefficient can now be calculated as the force that is necessary for a unit displacement. According to Archimedes principle this force is equal to the displaced water volume.

The hydrodynamic spring coefficients for sway, heave and roll become:

$$\begin{aligned} c_{22} &= 0 \\ c_{33} &= \rho g Y_w L_w \\ c_{44} &= \rho g Y_w L_w T_1 \cdot \overline{GM} \end{aligned} \quad (B.7)$$

Where:

c_{xx}	: Spring stiffness, k in equation (B.6)	[N/m, Nm/rad]
ρ	: Water density, 1025 kg/m ³ for salt water	[kg/m ³]
Y_w	: Width of the floating breakwater element	[m]
L_w	: Length of the floating breakwater element	[m]
T_1	: Draught of the floating breakwater element	[m]
$\overline{GM}$	: Transverse metacentric height	[m]

B.4 Forces

When an object that is small compared to the wave length is considered, the Froude-Krilov force for the heave motion can be written as [Journée and Massie, 2001]:

$$F_{FK} = c_{33} e^{-kT_1} \zeta \quad (B.8)$$

Where:

k	: Wave number	[rad/m]
ζ	: Wave height	[m]

A floating breakwater can not be seen as an object that is small compared to the wave length. Therefore the Froude-Krilov force has to be corrected:

$$F_{FK} = C_3 c_{33} e^{-kT_1} \zeta \quad (B.9)$$

Where:

C_3	: Vertical wave distribution coefficient, $1 - \frac{Y_w}{k} e^{-kT_1}$
-------	---

This Froude-Krilov force has to be corrected because of diffraction. This results in the total wave exciting force for the heave motion:

$$F_{w;ver} = a_{33} \ddot{\zeta}_a^* + b_{33} \dot{\zeta}_a^* + C_3 c_{33} \zeta \quad (B.10)$$

The horizontal wave force can be written as the sum of a so-called inertia force and a drag force as has been proposed by [Grotmaack and Meylan, 2006] for small floating bodies:

$$F_{w;hor} = (a_{22}\ddot{\zeta}_a^* + b_{22}\dot{\zeta}_a^*) + (\rho A_z C_w |\ddot{\zeta}_a^* - \dot{y}| \cdot (\ddot{\zeta}_a^* - \dot{y})) \quad (B.11)$$

The drag force is not included in the ReFBreak model due to the influence of the mooring system. The mooring system does not influence the hydrodynamic behaviour in short term but it will hold the breakwater on its position in long term. The horizontal wave force will be therefore:

$$F_{w;hor} = (a_{22}\ddot{\zeta}_a^* + b_{22}\dot{\zeta}_a^*) \quad (B.12)$$

The wave exciting moment for roll can be determined by calculating the horizontal and vertical wave exciting forces and multiplying them respectively with the vertical and the horizontal lever arms, see figure B.7. The formula for the wave exciting moment is [Fousert, 2006]:

$$M_{w;\phi} = F_{w;hor} \cdot z_b + F_{w;ver} \cdot y_b \quad (B.13)$$

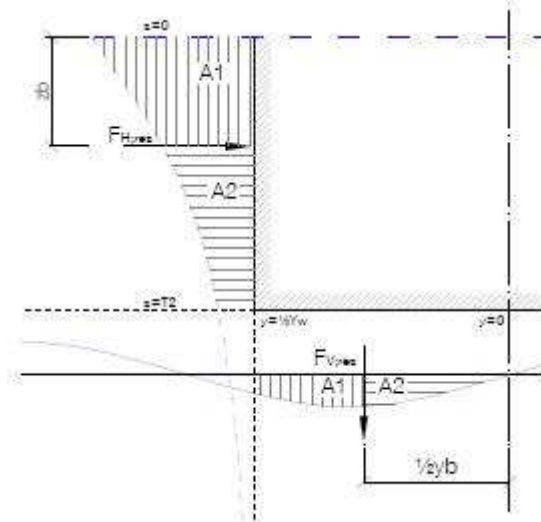


Figure B.7 Determination of z_b and y_b [Fousert, 2006]

The lever arm, relative to G, of the vertical wave force is:

$$y_b = 2 \frac{\arccos(\frac{1}{2} k Y_w)}{k} \quad (B.14)$$

The lever arm, relative to the mean water line, of the horizontal wave force is:

$$z_b = \frac{\ln\left(\frac{1+e^{-kT_2}}{2}\right)}{-k} \quad (B.15)$$

B.5 Model-setup

As every motion is considered to be uncoupled in respect to the other motions, the differential equations can be solved for each of the three motions separately. This will result in a transmitted wave-height towards the harbour-side of the floating breakwater for each motion with the corresponding phase change.

B.5.1 Heave

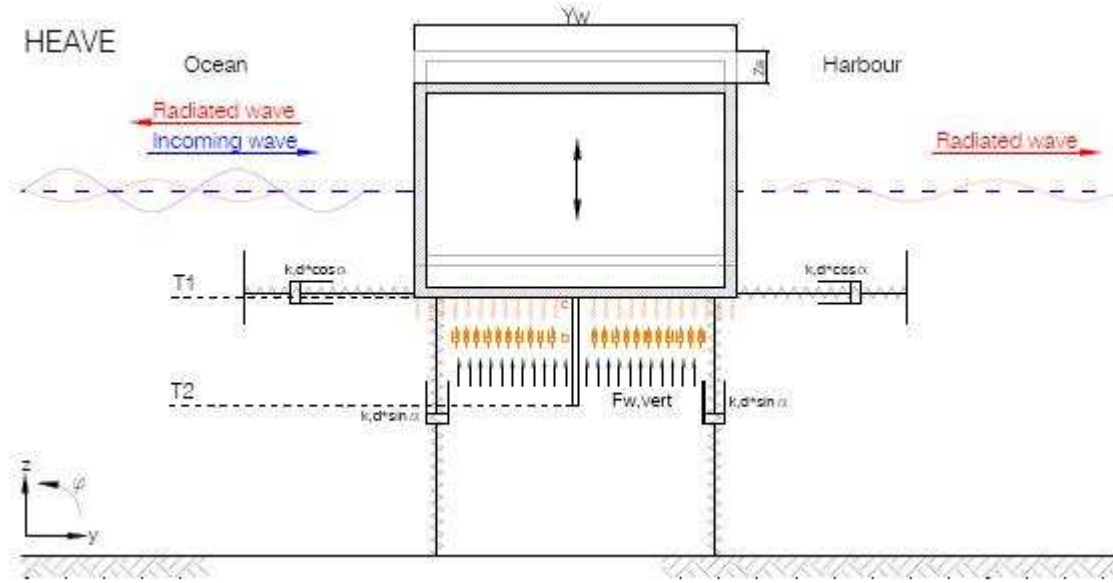


Figure B.8 Hydrodynamic system for heave [Fousert, 2006]

Figure B.8 shows the hydrodynamic system for heave. The second-order differential equation governing this system is given by:

$$(m + a_{33})\ddot{z} + (d_{cz} + b_{33})\dot{z} + (k_{cz} + c_{33})z = F_{w,ver} \quad (B.16)$$

When equation (B.16) is solved the heave motion is obtained:

$$z(t) = \zeta_a e^{-kT_1} \sqrt{\frac{(c_{33}C_3 - a_{33}\omega^2)^2 + (b_{33}\omega)^2}{((k_{cz} + c_{33}) - (m + a_{33})\omega^2)^2 + ((d_{cz} + b_{33})\omega)^2}} \cdot \sin(\omega t + e_{z\zeta}) \quad (B.17)$$

Where¹

$$e_{z\zeta} = \arctan \left(\frac{-mb_{33}\omega^3}{(c_{33}C_3 - a_{33}\omega)(k_{cz} + c_{33}) - (m + a_{33})\omega^2 + ((d_{cz} + b_{33})\omega)^2} \right) \quad (B.18)$$

The heave motion of the floating breakwater generates waves. These generated waves subtract energy from the oscillating motions. [Fousert, 2006] proved that the hydrodynamic damping is the cause of the oscillating motion dying out. The energy balance between the amplitude and the radiated wave-energy results in the amplitude of the radiated wave. The transmitted wave generated by heave becomes:

$$\zeta_{T;heave}(t) = \sqrt{\frac{b_{33}\omega^3 z_a^2}{\rho g^2}} \cos(\omega t + e_{z\zeta}) \quad (B.19)$$

¹ In the report of [Fousert, 2006] there is a mistake in parenthesis, the correct formula is shown here.

B.5.2 Sway

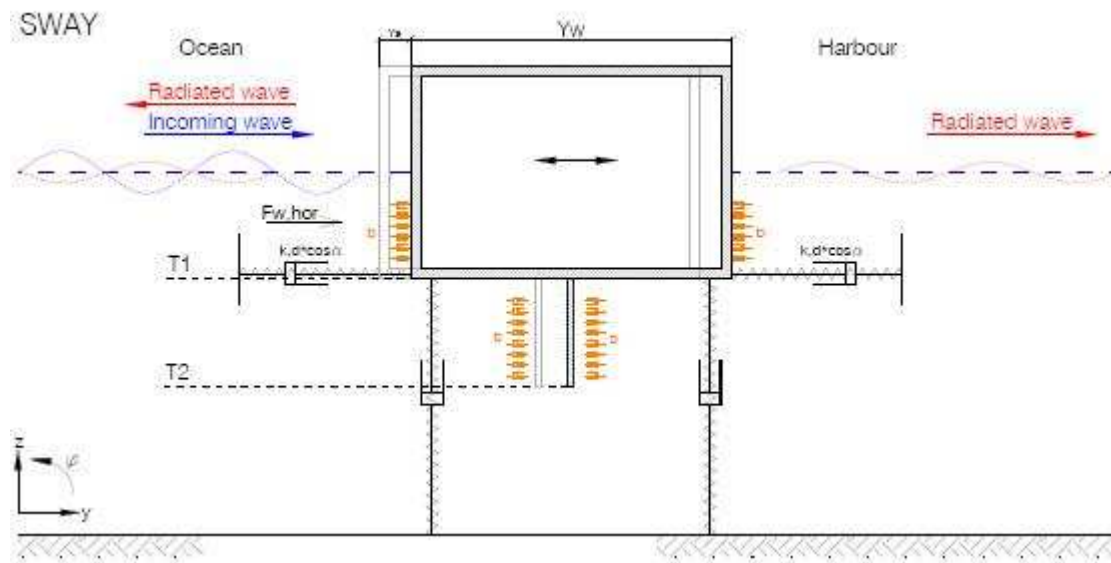


Figure B.9 Hydrodynamic system for sway [Fousert, 2006]

Figure B.9 shows the hydrodynamic system for sway. The second-order differential equation governing this system is given by:

$$(m+a_{22})\ddot{y}+(d_{cy}+b_{22})\dot{y}+(k_{cy})y=F_{w;hor} \quad (\text{B.20})$$

When equation (B.20) is solved the sway motion is obtained:

$$y(t) = \zeta_a e^{-kT_2} \sqrt{\frac{(a_{22}\omega^2)^2 + (b_{22}\omega)^2}{((k_{cz}) - (m + a_{22})\omega^2)^2 + ((d_{cy} + b_{22})\omega)^2}} \cdot \cos(\omega t + e_{y\zeta}) \quad (\text{B.21})$$

Where²

$$\mathcal{E}_{y_\zeta'} = \arctan \left(\frac{-mb_{22}\omega^3}{(a_{22}\omega^2)(-k_{cy} + (m + a_{22})\omega^2) + ((d_{cy} + b_{22})\omega)^2} \right) \quad (\text{B.22})$$

The sway motion of the floating breakwater generates waves. The same theory as was explained for the heave motion holds for the determination of the transmitted wave generated by the sway motion. The transmitted wave generated by sway becomes:

$$\zeta_{T; \text{sway}}(t) = \sqrt{\frac{b_{22} \omega^3 y_a^2}{\rho g^2}} \sin(\omega t + \varepsilon_{y_\zeta}) \quad (\text{B.23})$$

² In the report of [Fousert, 2006] there is a mistake in parenthesis, the correct formula is shown here.

B.5.3 Roll

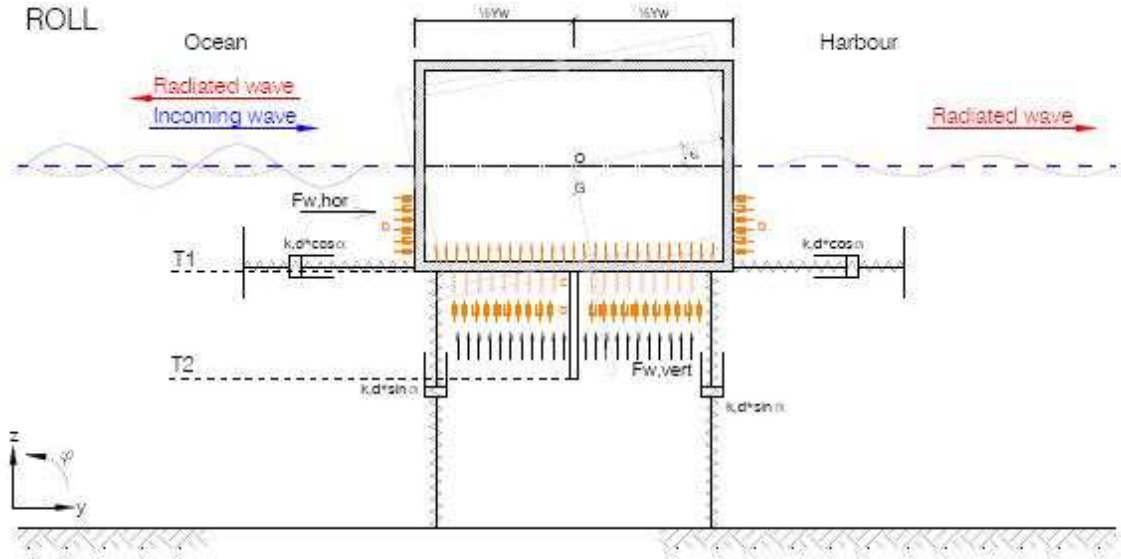


Figure B.10 Hydrodynamic system for roll [Fousert, 2006]

Figure B.10 shows the hydrodynamic system for roll. The second-order differential equation governing this system is given by:

$$(I_x + a_{44})\ddot{\phi} + \left(\frac{1}{2}Y_w^2 d_{cz} + 2T_1^2 d_{cy} + b_{44}\right)\dot{\phi} + \left(\frac{1}{2}Y_w^2 k_{cz} + 2T_1^2 k_{cy} + c_{44} + \overline{OG} \cdot mg\right)\phi = M_{w;\phi} \quad (B.24)$$

When equation (B.20) is solved the roll motion is obtained:

$$\phi(t) = \zeta_a \sqrt{\frac{C^2 + D^2}{A^2 + B^2}} \cdot \cos(\omega t + e_{\phi\zeta}) \quad (B.25)$$

Where

$$\varepsilon_{\phi\zeta} = \arctan\left(\frac{A \cdot D - C \cdot B}{C \cdot A - B \cdot D}\right) \quad (B.26)$$

And

$$\begin{aligned} A &= (I_x + a_{44})\omega^2 - \left(\frac{1}{2}Y_w^2 k_{cz} + 2T_1^2 k_{cy} + c_{44} + \overline{OG} \cdot mg\right) \\ B &= \left(\frac{1}{2}Y_w^2 d_{cz} + 2T_1^2 d_{cy} + b_{44}\right)\omega \\ C &= \left(e^{-kT_2} \cdot z_b \cdot a_{22}\omega^2\right) + \left(e^{-kT_1} \cdot y_b \cdot b_{33}\omega\right) \\ D &= \left(e^{-kT_2} \cdot z_b \cdot b_{22}\omega\right) + \left(e^{-kT_1} \cdot y_b \cdot a_{33}\omega^2\right) \end{aligned}$$

The roll motion of the floating breakwater generates waves. The same theory as was explained for the heave motion holds for the determination of the transmitted wave generated by the sway motion. The transmitted wave generated by sway becomes:

$$\zeta_{T;roll}(t) = \sqrt{\frac{b_{44}\omega^3\phi_a^2}{\rho g^2}} \sin(\omega t + \varepsilon_{\phi\zeta}) \quad (B.27)$$

B.5.4 Underflow

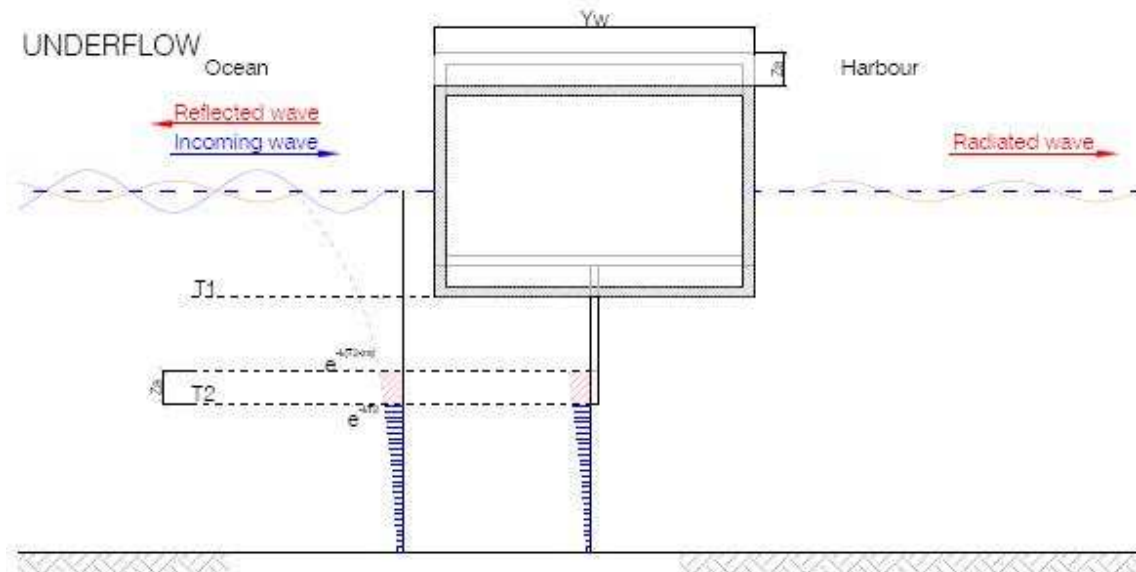


Figure B.11 The underflow phenomena [Fousert, 2006]

The part of the wave-energy which is not affected by the presence of the breakwater is called the underflow. It is assumed that all of the wave energy present beneath the draught point T_2 will radiate toward the harbour side of the breakwater, as is shown in figure B.11. With the energy balance between the energy that passes the breakwater undisturbed and the transmitted wave the transmitted wave can be calculated:

$$\zeta_{T;underflow}(t) = \sqrt{e^{-k(T_2 - z_a)}} \zeta_a^2 \quad (\text{B.28})$$

It is expected that the amplitude of the heave motion has an influence on the underflow when structures with a small draught are considered.

B.6 Transmission

The total transmitted wave amplitude consists of a summation of all the amplitudes that are radiated by the heave, sway and roll motions and the underflow. This total wave amplitude can be calculated by a combination of equations (B.19), (B.23), (B.27) and (B.28). From this combination follows the amplitude and the phase change relative to the incoming waves.

B.7 Conclusion

The ReFBreak-model makes it possible to analytically determine the dynamic reaction of a floating breakwater and the transmitted wave height towards the harbour side. With the aid of the model it can be quickly assessed which factors do influence the performance and to which extent. The factors that determine the performance of the floating breakwater are:

- The structural width;
- The structural draft;
- The screen draft;
- The structural mass;
- The mooring stiffness;
- The mooring damping.

The model can be used to produce a graph of the RAO, the so-called Response Amplitude Operator, as can be seen in figure B.12. This RAO can be used to quickly determine the most optimal floating breakwater for different wave frequencies.

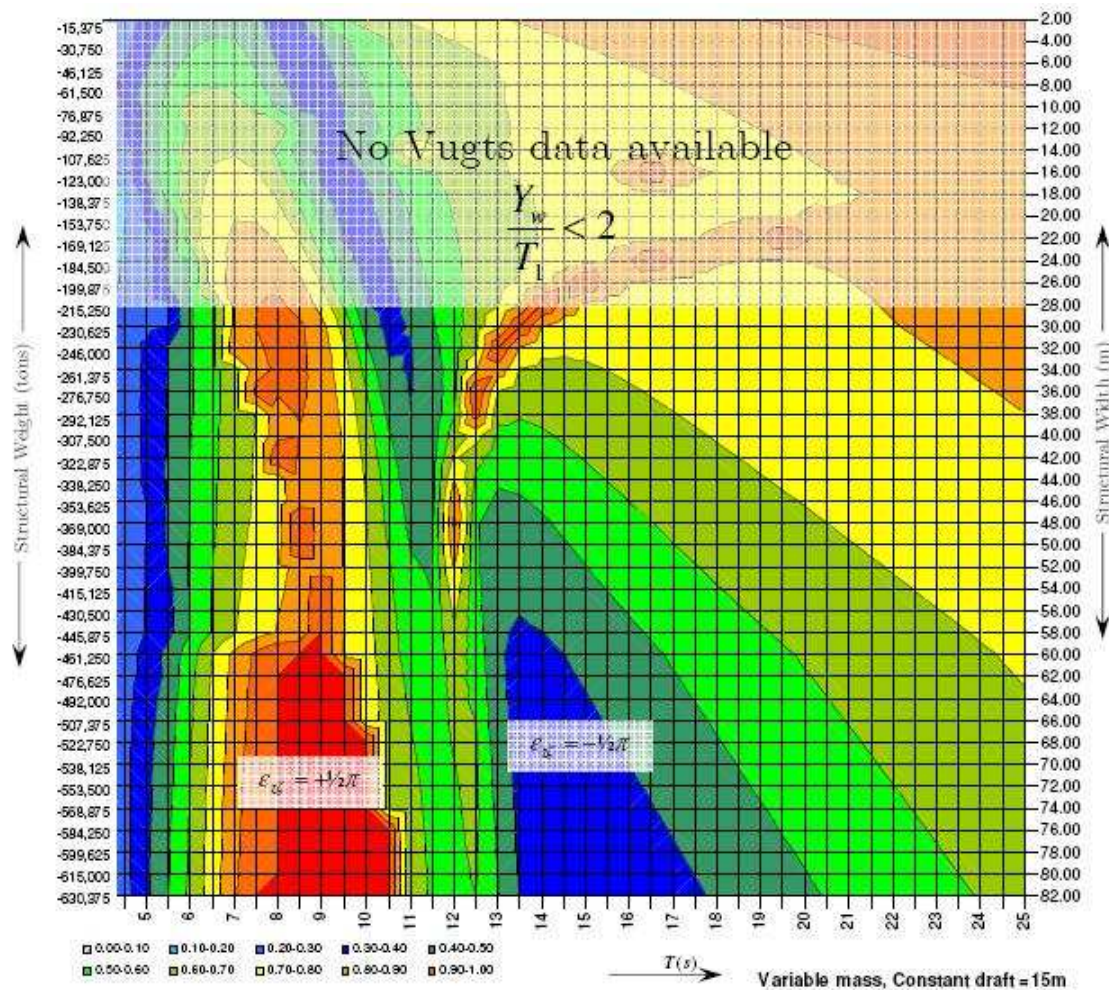


Figure B.12 Example of a RAO for the transmitted wave height [Fousert, 2006]

B.8 Model limitations and inaccuracies

While translating the ReFBreak-model from MS Excel to MATLAB some discrepancies between the model and the documentation were discovered. The model was deduced again on basis of [Fousert, 2006] and the differences between the new model and the old model are listed here.

The model has been setup and developed using the assumption that all motions are cosine shaped, each motion with a different phase change. [Fousert, 2006] deduced his model using both cosine and sine functions for the motions. This should not have any effect on the final result, as the cosine and the sine will evaluate to the same values with a phase change of $\frac{1}{2}\pi$.

B.8.1 Uncoupled motions

It was assumed that in the two dimensional case the heave, sway and roll motions are uncoupled. It is true that the heave motion is uncoupled from sway and roll motions due to symmetry, but the roll and sway motions however are not uncoupled.

When the centre of gravity is translated in horizontal direction, i.e. the sway motion, there is a force opposing this translation. If and only if the centre of gravity is exactly on the same height as the resulting force due to the translation there will be no moment causing roll. By applying a screen underneath the floater, as has been proposed to improve the efficiency, the height of the resultant force will change, but the centre of gravity is barely affected. From this it can be deduced that the motions are coupled.

In the three-dimensional case there are 6 degrees of motion and these are coupled in two groups [Journée and Massie, 2001]:

1. Surge – Heave – Pitch, the so-called symmetric motions
2. Sway – Roll – Yaw, the so-called asymmetric motion.

Note that this distinction between symmetric and anti-symmetric motions can disappear when the ship is anchored. Then, for instance, the pitch motions can generate roll motions via the anchor lines. This also holds for a floating breakwater element as it is held in its place by a kind of anchoring system.

Because of symmetry, the floating breakwater is a rectangular box with two axis of symmetry, it can be deduced that heave is also an uncoupled motion in a three dimensional situation when it is assumed that the anchoring system does not influence the coupling between the motions.

B.8.2 Hydrodynamic coefficients polynomials

The polynomials for the hydrodynamic coefficients are based on the curves of the research of [Vugts, 1968] and these are limited to a breadth-to-draught ratio of 2.0 and higher. In his research, [Fousert, 2006] indicated that the best performance of a floating breakwater is to be expected with breadth-to-draught ratios of 1.0 to 2.0. Although it can be questioned whether this is an incorrectness of the model, it is certainly a limitation.

To have a more accurate prediction on the performance of a floating breakwater more polynomials are required. As these cannot be based on the research of [Vugts, 1968], these have to be determined by the three dimensional potential model DELFRAC.

B.8.3 Wave exciting forces

In the ReFBreak model formula (B.9) was used to determine the vertical wave exciting force on the floating breakwater. This formula is an adaption from [Journée and Massie, 2001], where the formula is given for a small floating body in comparison with the wave length. A wave pressure coefficient, C_3 , is added to the formula to accommodate for the large structure.

In the hypothetical case of a wave-length equal to the width of the structure the Froude-Krilov force has to result in zero, i.e. if the diffraction was neglected the breakwater would not have moved in the heave direction. When this case is applied to the formula used in the ReFBreak-model it does not yield in the expected behaviour. Therefore it is expected that the formula used is not correct. Inconsistent units in the formula for the coefficient C_3 reinforce the presumption.

Formula (B.11) has been used to determine the horizontal wave-force. This formula is only valid for when small floating bodies are assumed, while a floating breakwater width is not small in comparison to the wave-length. The proposed width of the floating breakwater is up to 100 meters, while the wave lengths range from 100 meter up to 1000 meter.

In the ReFBreak-model the wave exciting moment is calculated by use of the wave exciting forces of the heave and the sway motions multiplied by the respective horizontal en vertical lever arms. As has been concluded above the horizontal and vertical wave forces were not correct in the model and thus the roll exciting moment is not correct either.

B.9 References

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Appendix C ReFBreak2D

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Appendix C ReFBreak2D

The ReFBreak-model has been developed by [Fousert, 2006] in order to assess the influence of the structural dimension of the floating breakwater. The ReFBreak2D model is based on the ReFBreak-model, but takes in account:

- The coupling of the sway and the roll motion;
- The effect of diffraction on the wave exciting forces and moments.

The basis of the model is the flowchart that is depicted in figure C.1. Incoming waves are responsible for exciting forces and moments on the structure. These forces and moments will put the floating breakwater in an oscillatory motion, which in turn will cause waves radiating from both sides of the breakwater. The total transmitted wave towards the harbour-side of the floating breakwater is found by the summation of all radiated waves and the wave that passes underneath the breakwater.

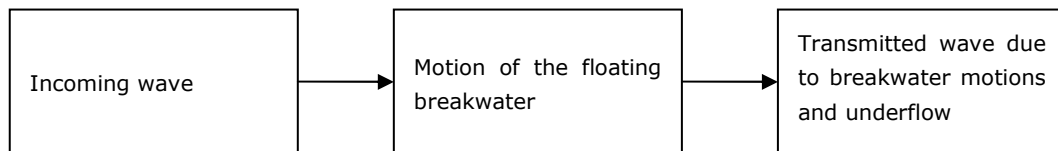


Figure C.1 Model flowchart

To assess the validity of the model, the dynamic behaviour of a FB will be investigated. The equations governing these motions will be treated and the hydrodynamic parameters that are of importance for these equations. The wave exciting forces and moments are required to find the steady-state solution; these are calculated with the method of Newman, i.e. there is a relation between the wave frequency, the hydrodynamic damping and the wave exciting force. The equations of motion are then solved for the system of motions. Finally the radiated waves are calculated and a summation is made to find the total transmitted wave.

C.1 Dynamic behaviour

A floating body in a three dimensional space has 6 degrees of freedom:

1. Surge
2. Sway
3. Heave
4. Roll
5. Pitch
6. Yaw

These motions are defined by the translation of the centre of gravity in the x-, y- and z-direction and the three rotations about those axes. This is shown in figure C.2, where the numbers are corresponding to the respective motion. [Journée and Massie, 2001]

With respect to the axes in figure C.2 the formulas for the three translations and the three rotations are defined. With the motions of the centre of gravity (COG) known the motions in any point on the structure can be calculated by the use of superposition.

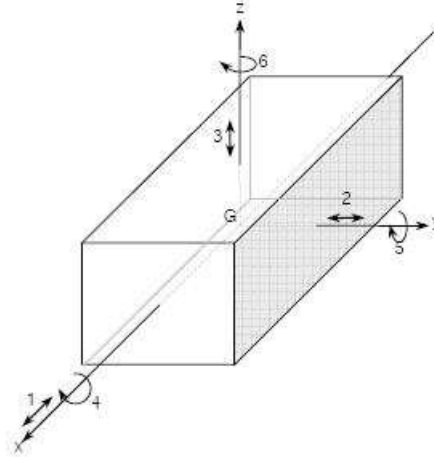


Figure C.2 Degrees of freedom for a floating body in a three-dimensional space [Fousert, 2006]

The motions are defined as:

$$\begin{aligned}
 x(t) &= x_a \cos(\omega t + \varepsilon_{x\zeta}) \\
 y(t) &= y_a \cos(\omega t + \varepsilon_{y\zeta}) \\
 z(t) &= z_a \cos(\omega t + \varepsilon_{z\zeta}) \\
 \phi(t) &= \phi_a \cos(\omega t + \varepsilon_{\phi\zeta}) \\
 \theta(t) &= \theta_a \cos(\omega t + \varepsilon_{\theta\zeta}) \\
 \psi(t) &= \psi_a \cos(\omega t + \varepsilon_{\psi\zeta})
 \end{aligned} \tag{C.1}$$

Where:

n_a	: Motion amplitude	[m], [rad]
$\varepsilon_{n\zeta}$	: Phase angle	[rad]
ω	: Circular wave frequency	[rad/s]

When the centre of gravity is translated the floating breakwater there is a force opposing this translation. If and only if the centre of gravity is exactly at the same height as the resulting force due to the translation there will be no moment causing roll. By applying a screen underneath the floater, as has been proposed to improve the efficiency, the height of the resultant force will change, but the centre of gravity is barely affected. From this it can be deduced that the motions are coupled.

In the three-dimensional case there are 6 degrees of motion and these are coupled into two groups [Journée and Massie, 2001]:

1. Surge – Heave – Pitch, the so-called symmetric motions
2. Sway – Roll – Yaw, the so-called asymmetric motion.

Note that this distinction between symmetric and anti-symmetric motions can disappear when the ship is anchored. Then, for instance, the pitch motions can generate roll motions via the anchor lines. This also holds for a floating breakwater element as it is held in its place by an anchoring system of some kind.

Because of symmetry, the floating breakwater is a rectangular box with two axis of symmetry, it can be deduced that heave is also an uncoupled motion in a three dimensional situation loaded with beam-on waves when it is assumed that the anchoring system does not influence the coupling between the motions.

The floating breakwater is loaded by waves. The wave elevation can be defined as:

$$\zeta(x, y, t) = \zeta_a \cos(\omega t - \sin(\mu)kx - \cos(\mu)ky) \quad (C.2)$$

Where:

ζ_a	: Wave amplitude	[m]
ω	: Wave frequency	[rad/s]
k	: Wave number	[m ⁻¹]
μ	: Angle of attack	[rad]

For a 2 dimensional case with beam-on waves equation (C.2) can be simplified to:

$$\zeta(y, t) = \zeta_a \cos(\omega t - ky) \quad (C.3)$$

The phase shifts of the previous mentioned motions are related to the harmonic wave elevation at the origin, i.e. the centre of gravity, even though no wave can be measured at this location. The wave elevation in the centre of gravity ($y = 0$):

$$\zeta_{COG}(t) = \zeta_a \cos(\omega t) \quad (C.4)$$

C.2 Equation of motions

It is assumed that a floating breakwater will not generate waves by itself due to structural movements when floating in still water. The only hydro-mechanical reaction will occur when the structure is loaded by waves, causing wave exciting forces and moments. In other words the floating breakwater does not generate energy; it can only absorb energy of waves and re-transmit this energy. It is assumed that this relation is linear, i.e. doubling the input wave amplitude results in doubled output amplitude, while possible phase changes between input and output will not change.

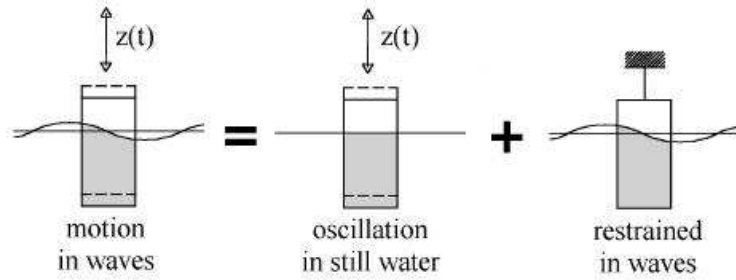


Figure C.3 Superposition of hydro-mechanical and wave loads [Journée and Massie, 2001]

Since the system is assumed to be linear, the resulting motion in waves can be seen as a superposition of the body in still water and the forces on a restraint body, as can be seen in the picture equation in figure C.3. This leads to two important assumptions [Journée and Massie, 2001]:

1. The hydro-mechanic forces and moments are induced by the harmonic oscillations of the rigid body in the undisturbed surface of the fluid;
2. The wave exciting forces and moments are produced by the waves coming in on the restraint body.

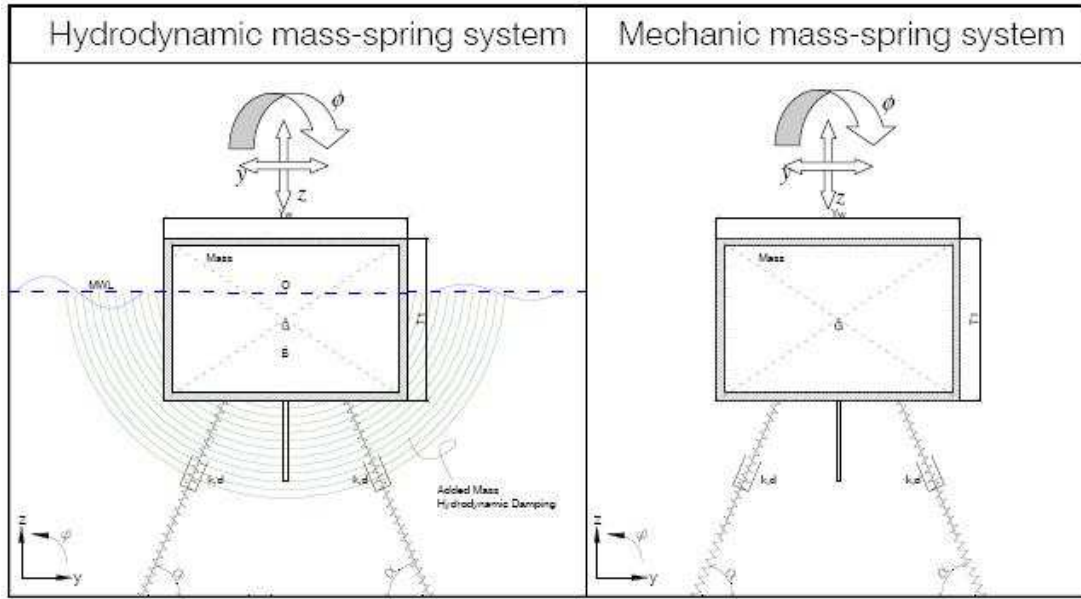


Figure C.4 A 2-dimensional hydrodynamic and mechanical mass-spring system [Fousert, 2006]

The coupled second order differential equation, governing all six degrees of motion is given by [Journée and Massie, 2001]:

$$M\ddot{\vec{X}} + N\dot{\vec{X}} + C\vec{X} = \vec{F} \quad (\text{C.5})$$

Where:

- $\vec{X}$: Translation and rotation vector;
- $\vec{F}$: Wave exciting forces and moments vector;
- C : Hydrodynamic spring stiffness matrix;

The translation and rotation vector

$$\vec{X} = \begin{bmatrix} x(t) \\ y(t) \\ z(t) \\ \phi(t) \\ \theta(t) \\ \psi(t) \end{bmatrix} \quad (\text{C.6})$$

The hydrodynamic mass matrix:

$$M = \begin{bmatrix} M_{11} & 0 & M_{31} & 0 & M_{51} & 0 \\ 0 & M_{22} & 0 & M_{24} & 0 & M_{26} \\ M_{31} & 0 & M_{33} & 0 & M_{53} & 0 \\ 0 & M_{42} & 0 & M_{44} & 0 & M_{46} \\ M_{51} & 0 & M_{53} & 0 & M_{55} & 0 \\ 0 & M_{62} & 0 & M_{64} & 0 & M_{66} \end{bmatrix} \quad (\text{C.7})$$

The hydrodynamic dampening matrix:

$$N = \begin{bmatrix} N_{11} & 0 & N_{31} & 0 & N_{51} & 0 \\ 0 & N_{22} & 0 & N_{24} & 0 & N_{26} \\ N_{31} & 0 & N_{33} & 0 & N_{53} & 0 \\ 0 & N_{42} & 0 & N_{44} & 0 & N_{46} \\ N_{51} & 0 & N_{53} & 0 & N_{55} & 0 \\ 0 & N_{62} & 0 & N_{64} & 0 & N_{66} \end{bmatrix} \quad (C.8)$$

From (C.7) and (C.8) it can be seen that the asymmetric motions are uncoupled from the symmetric motions, as has been described in paragraph C.1.

In the two dimensional case equation (C.5) reduces to:

$$\begin{bmatrix} M_{22} & 0 & M_{24} \\ 0 & M_{33} & 0 \\ M_{42} & 0 & M_{44} \end{bmatrix} \begin{bmatrix} \ddot{y} \\ \ddot{z} \\ \ddot{\phi} \end{bmatrix} + \begin{bmatrix} N_{22} & 0 & N_{24} \\ 0 & N_{33} & 0 \\ N_{42} & 0 & N_{44} \end{bmatrix} \begin{bmatrix} \dot{y} \\ \dot{z} \\ \dot{\phi} \end{bmatrix} + \begin{bmatrix} c_{22} & 0 & 0 \\ 0 & c_{33} & 0 \\ 0 & 0 & c_{44} \end{bmatrix} \begin{bmatrix} y \\ z \\ \phi \end{bmatrix} = \begin{bmatrix} F_{w;hor} \\ F_{w;ver} \\ M_{w;\phi} \end{bmatrix} \quad (C.9)$$

Where

- M_{nn} : The hydrodynamic mass;
- N_{nn} : The hydrodynamic dampening;
- c_{nn} : The hydrodynamic spring stiffness;
- x, z, ϕ : Translations and rotations.

It assumed that the floating breakwater does not have a forward speed in the water, i.e. there are no currents because the breakwater is stationary. In the case of a zero forward speed, the terms of mass-matrix and the damping-matrix will be symmetric about the diagonal, i.e. $M_{42} = M_{24}$ and $N_{42} = N_{24}$.

It can be seen from equation (C.9) that the heave motion is indeed uncoupled from the sway and roll motions. The hydrodynamic coefficients have to be determined by using DELFRAC. With the same program the wave induced forces and moments can also be determined.

C.3 DELFRAC

DELFRAC is a radiation/diffraction model program, based on a three-dimensional linear potential theory, including wave diffraction and drift forces acting on structures. A source distribution technique is used to solve the linearised velocity potential problem. The potential theory of DELFRAC will not be discussed in this thesis as it is out of the scope of the subject. Not all features of the program, such as the drift forces, will be used in this stadium, because they are not required to determine the coefficients for the ReFBreak2D-model.

C.3.1 Known limitations

In the research of [de Rooij, 2006], the floating container terminal in short waves could not be modelled due to the maximum of panels which can be used of in DELFRAC. This maximum was 10,000, but is in the current version of DELFRAC removed. Although this limit has been removed in the software, 10,000 panels can be still considered as the upper limit, due to the limitation on the computer hardware. Especially the memory usage can cause problems when excessive amounts of panels are used.

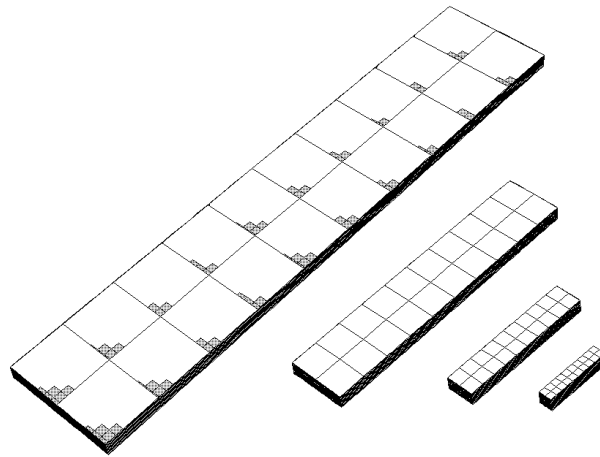


Figure C.5 The four DELFRAC panel-models, $W/D = 8.0, 4.0, 2.0$ and 1.0 respectively

C.3.2 Input parameters

The smallest panel size is determined by the shortest wave-length that has to be modelled. There should be at least five panels in one wave-length. When fewer panels are considered per wave-length the results will not be accurate. The coefficients are plotted against a dimension-less parameter x :

$$x = \omega \sqrt{\frac{Y_w}{2g}} \quad (\text{C.10})$$

Where

ω	: The radial frequency;	[1/s]
Y_w	: The beam of the element;	[m]
g	: The gravitational acceleration	[m/s ²]

This dimensionless parameter should be between 0.5 and 3; this is the frequency range of interest for a floating breakwater. From this parameter the wave frequency in relation to the beam can be calculated. In deep water the wave frequency is related to the wave-length:

$$L = 1.56T^2 \quad (\text{C.11})$$

Where

L	: The wave-length;	[m]
T	: The wave-period;	[s]

From this, the minimum wave-length can be calculated, leading to the maximum panel-dimensions for the DELFRAC simulation. This is presented in table C-1. DELFRAC is a three-dimensional refraction/diffraction program, while ReFBreak2D considers a two-dimensional situation. Therefore the length of the elements is chosen in such a way that the three-dimensional effects can be considered as minimal and therefore can be neglected. Figure C.5 shows the four panel-models which have been used for the calculations.

Table C-1 Number of panels for the different W/D -ratios per quarter of an element

W/D-ratio	1	2	4	8
Draught	5 m	5 m	5 m	5 m
Breadth	5 m	10 m	20 m	40 m
Length	25 m	50 m	100 m	200 m
Min. wave-length	1,75 m	3,50 m	7,00 m	14,00 m
Panel-length	0,35 m	0,70 m	1,40 m	2,80 m
Number of panels	859 [-]	859 [-]	859 [-]	859 [-]

In the DELFRAC program the waves are flowing through the body. This can cause problems, i.e. the so-called irregular frequencies. At these frequencies the simulation does not result in correct parameters. To avoid these problems, the box is closed with a lid. It is not necessary for these panels on top of the model to comply with maximum panel length. Figure C.6 shows the completed model for the beam-to-draught ratio (Y_w/T_1 -ratio) of 1.0, including the lid on top with a different panel-size.

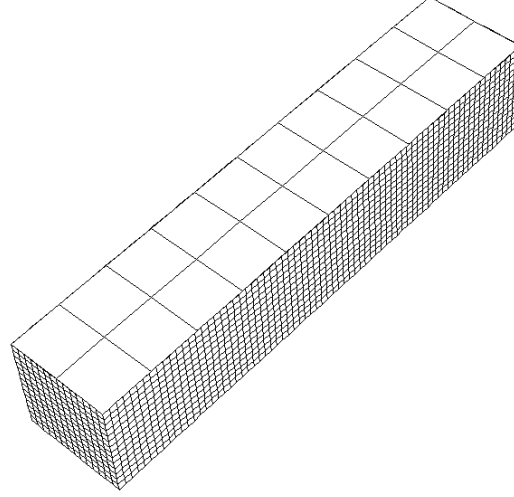


Figure C.6 Close-up of the $W/D = 1.0$ model, showing the different panel sizes

C.3.3 Influence of the COG

In the DELFRAC simulation the four floating breakwater elements are oscillated in roll around their centre of gravity, or COG. The added mass, a_{ij} , and hydrodynamic damping coefficients, b_{ij} , given by the DELFRAC simulation are defined for the centre of gravity. In all simulations the location centre of gravity is chosen to be the same as the location of the centre of buoyancy.

The COG will have an influence on the performance of a floating breakwater. A ship with a low COG will have a larger distance between the centre of gravity and its metacentre, resulting in a more stable breakwater. There are no extra DELFRAC calculations needed to calculate, because equation (C.9) can be rewritten for a Gyz-system with $\overline{BG} \neq 0$, but with a_{ij} , b_{ij} and c_{ij} defined for motion of the point O.

This is obtained by transforming [Vugts, 1968]

$$\begin{aligned}
 y_g &= y_o - \overline{BG} \cdot \phi_o \\
 z_g &= z_o \\
 \phi_g &= \phi_o \\
 F_{g,w,hor} &= F_{w,hor} \\
 F_{g,w,ver} &= F_{w,ver} \\
 M_{g,w,\phi} &= M_{w,\phi} + \overline{BG} \cdot F_{w,hor}
 \end{aligned} \tag{C.12}$$

Working this out and dropping the index G for motions and exciting forces results in:

$$(m + a_{22})\ddot{y} + b_{22}\dot{y} + (a_{24} + \overline{BG} \cdot a_{22})\ddot{\phi} + (b_{24} + \overline{BG} \cdot b_{22})\dot{\phi} = F_{w,hor} \tag{C.13}$$

$$(m + a_{33})\ddot{z} + b_{33}\dot{z} + c_{33}z = F_{w,ver} \tag{C.14}$$

$$\begin{aligned}
 & \left(I_{xx} + a_{44} + \overline{BG} \cdot a_{42} + \overline{BG}^2 \cdot a_{22} + \overline{BG} \cdot a_{24} \right) \ddot{\phi} + \\
 & \left(b_{44} + \overline{BG} \cdot b_{42} + \overline{BG}^2 \cdot b_{22} + \overline{BG} \cdot b_{24} \right) \dot{\phi} + \\
 & \left(mg \cdot \overline{GM} \right) \phi + \left(a_{42} + \overline{BG} \cdot a_{22} \right) \ddot{y} + \\
 & \left(b_{42} + \overline{BG} \cdot b_{22} \right) \dot{y} = M_{w,\phi} + \overline{BG} \cdot F_{w,hor}
 \end{aligned} \tag{C.15}$$

Thus the hydrodynamic mass matrix can be rewritten as:

$$M = \begin{bmatrix} (m + a_{22}) & 0 & (a_{24} + \overline{BG} \cdot a_{22}) \\ 0 & (m + a_{33}) & 0 \\ (a_{42} + \overline{BG} \cdot a_{22}) & 0 & (I_{xx} + a_{44} + \overline{BG} \cdot a_{42} + \overline{BG}^2 \cdot a_{22} + \overline{BG} \cdot a_{24}) \end{bmatrix} \tag{C.16}$$

And the hydrodynamic damping matrix as:

$$N = \begin{bmatrix} b_{22} & 0 & (b_{24} + \overline{BG} \cdot b_{22}) \\ 0 & b_{33} & 0 \\ (b_{42} + \overline{BG} \cdot b_{22}) & 0 & (b_{44} + \overline{BG} \cdot b_{42} + \overline{BG}^2 \cdot b_{22} + \overline{BG} \cdot b_{24}) \end{bmatrix} \tag{C.17}$$

It has to be noted with these equations that $\overline{BG}$ is positive when the G is above the centre of buoyancy, or B and negative when G is located below G. In the DELFRAC simulation for the ReFBreak2D model the centre of gravity was placed at the centre of buoyancy. This was done to circumvent problems with instability when the COG is place above the metacentre. The height of the metacentre, relative to the centre of buoyancy can be calculated with

$$BM = \frac{1}{12} \cdot \frac{Y_w^3}{Y_w T_1} \tag{C.18}$$

Especially the floating breakwater with an Y_w/T_1 ratio of 1.0 is not stable when the COG gravity is placed on the waterline. In the experiments of [Vugts, 1968], the centre of gravity of the models was placed at the waterline, resulting in different hydrodynamic parameters for roll and coupling coefficients.

C.4 Hydrodynamic calculation parameters

The polynomials for the hydrodynamic coefficients are based on the curves of the research of [Vugts, 1968], these are limited to a beam-to-draught ratio (Y_w/T_1 -ratio) of 2.0 and higher. In his research [Fousert, 2006] indicated that the best performance of a floating breakwater is to be expected with Y_w/T_1 -ratios of 1.0 to 2.0. To have a more accurate prediction on the performance of a floating breakwater more polynomials are required. As these cannot be based on the research of [Vugts, 1968], these have been determined by the three dimensional potential model DELFRAC.

All the polynomials are valid for the deep water boundary condition.

C.4.1 Added mass

The interaction of the oscillating (wetted) surface of the floating breakwater and the water particles accelerated by this oscillating surface is called the hydrodynamic

mass or added mass. This hydrodynamic reaction force does not dissipate energy and manifests itself as a standing wave near the surface of the breakwater.

The added mass coefficients for sway are shown in figure C.7 for Y_w/T_1 -ratios of 1.0, 2.0, 4.0 and 8.0. The curves used by [Fousert, 2006] are also plotted and are showing a good agreement with the values which were found by DELFRAC.

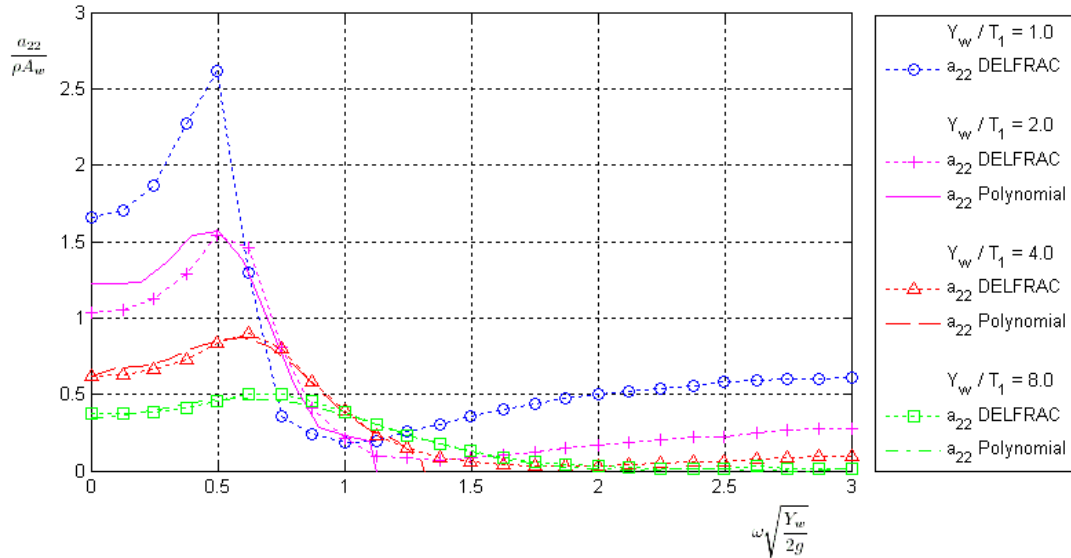


Figure C.7 Added mass coefficients for swaying from DELFRAC and [Fousert, 2006]

The added mass coefficients for heave are presented in figure C.8 for Y_w/T_1 -ratios of 1.0, 2.0, 4.0 and 8.0. The curves used by [Fousert, 2006] are also plotted and are showing a good agreement with the values which were found by DELFRAC for the Y_w/T_1 -ratios of 2.0 and 4.0 for the higher frequencies. A large deviation can be seen for the ratio of 8.0, which can be caused by three dimensional effects, which apparently do have a significant role in the results in this case.

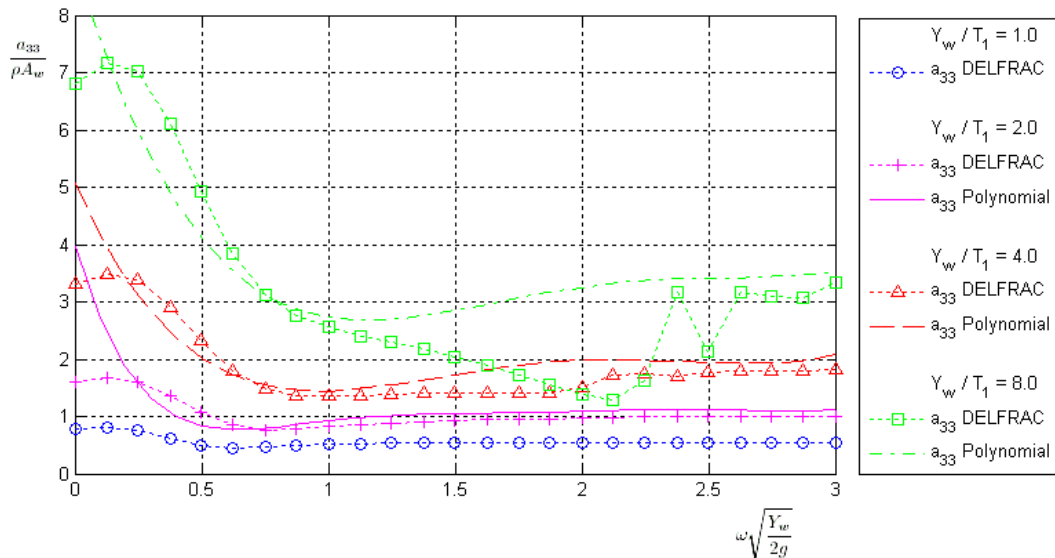


Figure C.8 Added mass coefficients for heaving from DELFRAC and [Fousert, 2006]

The deviation of the added mass coefficient for the heave motion is large in comparison with the deviation for the sway motion. This can be explained by the water mass that has to be accelerated. The sway accelerates the water mass almost solely in the direction of the y -axis like a wave-board and is not as much affected by three-dimensional effects as the heave motion which can be compared to a pebble that has been thrown in a pond.

The deviation between the results of [Vugts, 1968] and the DELFRAC calculation are diminishing when smaller ratios are considered. For the model the polynomial curves, based on the work of [Vugts, 1968], will be used for the Y_w/T_1 -ratios of 2.0, 4.0 and 8.0. The results of the DELFRAC calculation will be used for the ratio of 1.0.

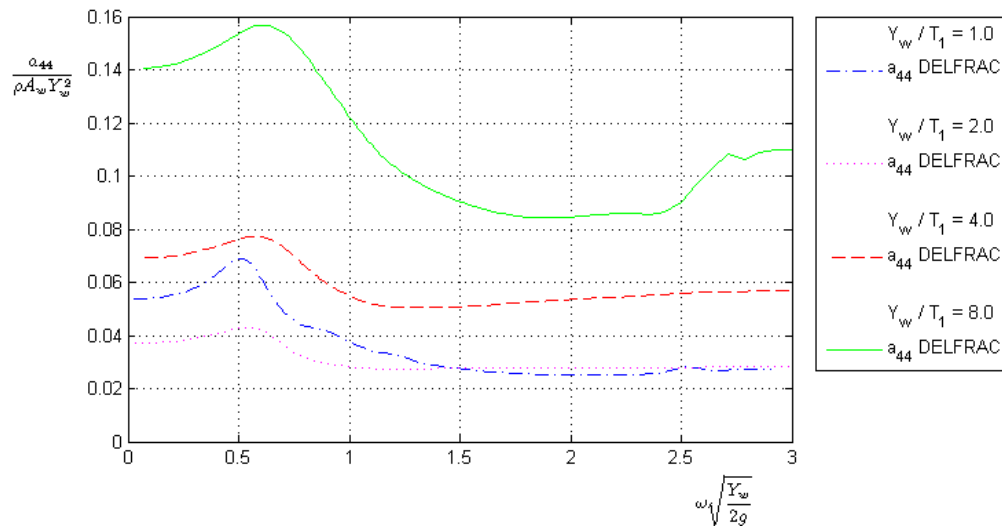


Figure C.9 Added mass coefficients for roll from DELFRAC

In his research [Fousert, 2006] did two calculations on the roll added mass, one for a Y_w/T_1 -ratio of 3.2 and one for a ratio of 10.2. For the intermediate values a linear interpolation was used. To get added mass coefficients which are more inline with the other two movements, the roll added mass has been determined for the four Y_w/T_1 ratios. The values found in the research of [Vugts, 1968] cannot be used because the axis were defined differently in the DELFRAC calculation

The mass coupling coefficients were omitted by [Fousert, 2006] because of the assumption that all motions were uncoupled. In the area of interest, e.g. around the heave natural frequency, the influence of these coupling coefficients cannot be neglected. As with roll, the mass coupling coefficient differ from the coefficients found in the work of [Vugts, 1968] due to the location of the origin of the axis.

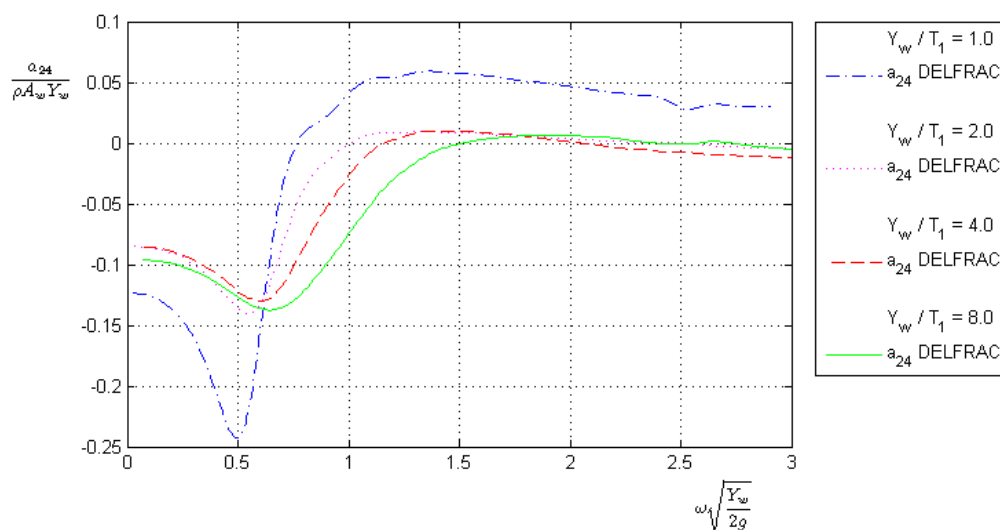


Figure C.10 Mass coupling coefficients for roll into sway from DELFRAC

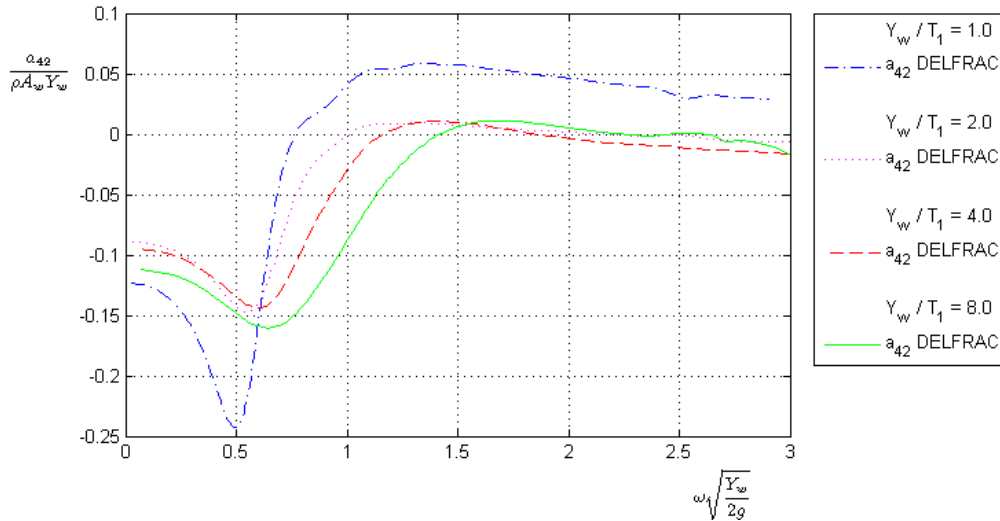


Figure C.11 Mass coupling coefficients for sway into roll from DELFRAC

The coefficients from roll into sway, see figure C.10, and from sway into roll, see figure C.11, should be equal to each other, however a very small deviation can be found. The extent of this deviation can be neglected and is of numerical origin.

C.4.2 Hydrodynamic damping

The hydrodynamic damping is related to the amount of wave energy that is radiated by the floating breakwater to both sides of the structure. Because the radiated waves do dissipate energy from the breakwater's motions this is the reason the motion dies out. Since the magnitude of the hydrodynamic damping depends, just as for the added mass, on the interaction of the water particles with oscillating breakwater surface, the same factors will be involved.

The hydrodynamic damping coefficients for sway are shown in figure C.12 for Y_w/T_1 -ratios of 1.0, 2.0, 4.0 and 8.0. The curves used by [Fousert, 2006] are also plotted and are showing a good agreement with the values which were found by DELFRAC.

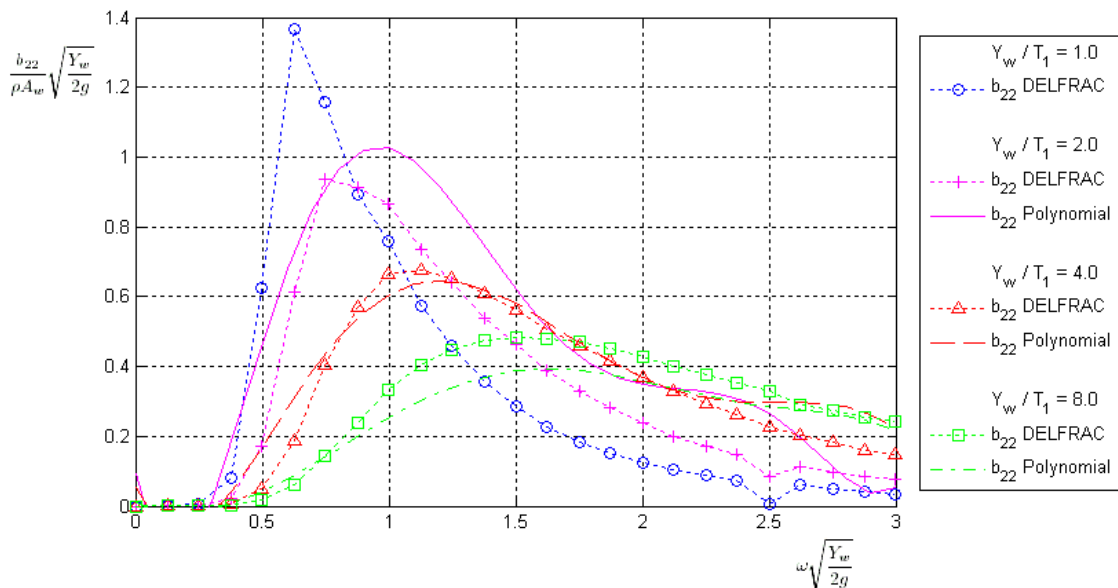


Figure C.12 Hydrodynamic damping coefficients for sway from DELFRAC and [Fousert, 2006]

The added mass coefficients for heave are presented in figure C.13 for Y_w/T_1 -ratios of 1.0, 2.0, 4.0 and 8.0. The curves used by [Fousert, 2006] are also plotted and are showing a good agreement with the values which were found by DELFRAC for the Y_w/T_1 -ratios of 2.0 and 4.0 for the higher frequencies.

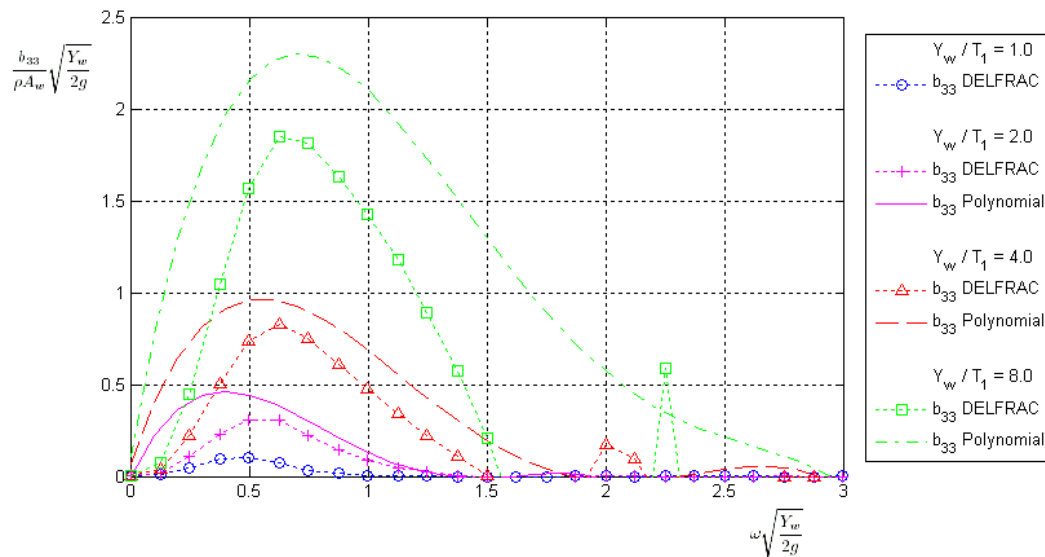


Figure C.13 Hydrodynamic damping coefficients for heaving from DELFRAC and [Fousert, 2006]

A large deviation can be seen for the ratio of 8.0. This can be caused by three dimensional effects of the DELFRAC program or a computational error due to the used panel sizes. A negative damping for heave can not occur because in this case it is not coupled with other motions. The results of [Vugts, 1968] will be used for the calculation of the heave hydrodynamic damping. The coefficients found in this way do not have to be corrected for the location of the COG.

The deviation between the results of [Vugts, 1968] and the DELFRAC calculation are diminishing when smaller ratios are considered. For the model the polynomial curves, based on the work of [Vugts, 1968], will be used for the Y_w/T_1 -ratios of 2.0, 4.0 and 8.0. The results of the DELFRAC calculation will be used for the ratio of 1.0.

In his research [Fousert, 2006] did two calculations on the roll hydrodynamic damping, one for a Y_w/T_1 -ratio of 3.2 and one for a ratio of 10.2. For the intermediate values a linear interpolation was used. To get damping coefficients which are more inline with the other two movements, the roll added mass has been determined for the four Y_w/T_1 ratios. The values found in the research of [Vugts, 1968] cannot be used because the axis were defined differently in the DELFRAC calculation.

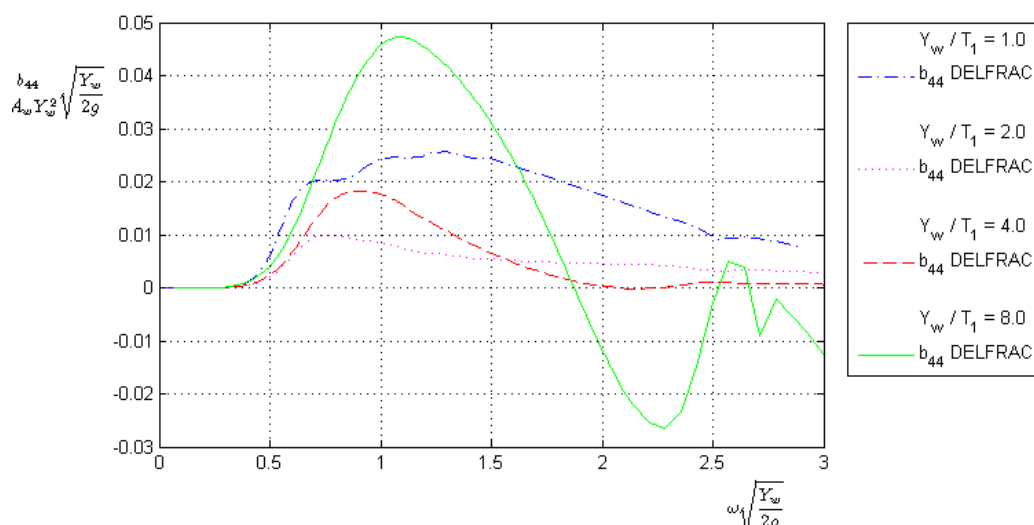


Figure C.14 Hydrodynamic damping coefficients for roll from DELFRAC

Figure C.14 shows values for the hydrodynamic damping coefficients for $Y_w/T_1=8.0$ which are negative. Negative damping means that energy is gained instead of dissipated from the system. This negative damping can come from the location of the centre of gravity, but other sources may be plausible as well. To avoid any problems due to this behaviour the scaled frequency should be limited to 1.5 for structures with an Y_w/T_1 ratio of 8.0.

Just as the mass coupling coefficients, the damping coupling coefficients were omitted by [Fousert, 2006]. The damping coupling coefficients used in ReFBreak2D are shown in figure C.15 and figure C.16. The coefficients should be equal. The small deviation that can be found between them is neglected as has been done with the mass coupling coefficients.

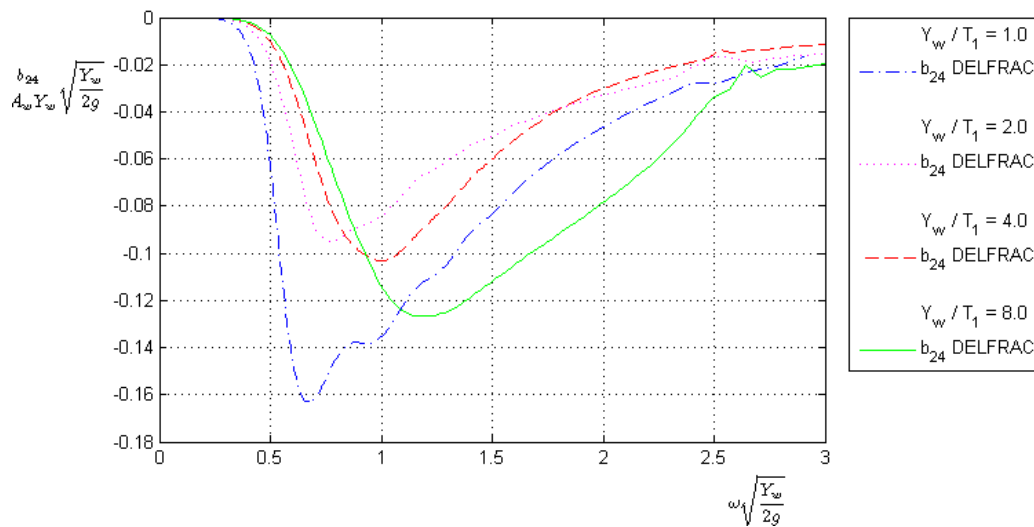


Figure C.15 Hydrodynamic damping for roll into sway from DELFRAC

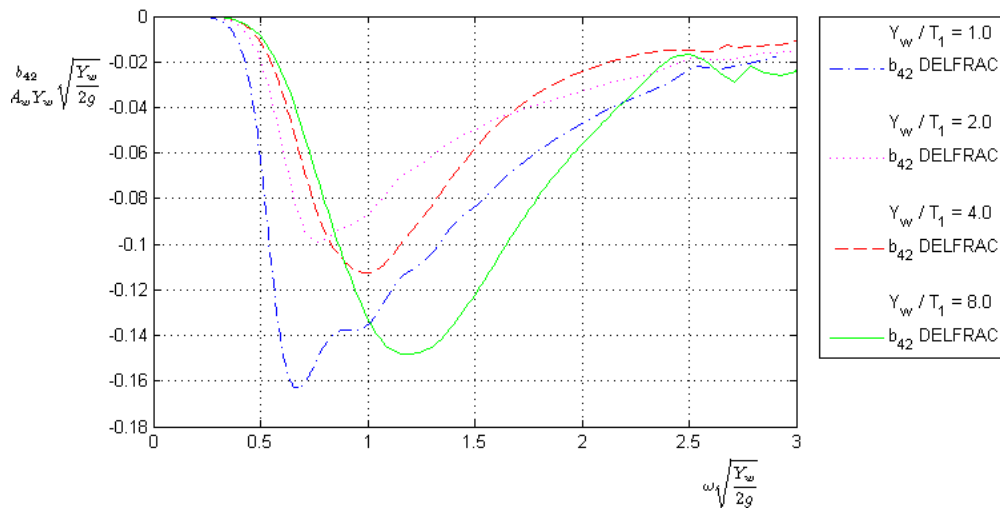


Figure C.16 Hydrodynamic damping for sway into roll from DELFRAC

C.4.3 Spring stiffness

The restoring force which pushes the floating breakwater back to its old position after an initial displacement is called the hydrodynamic spring coefficient. This force is produced by the change in water pressure underneath the floating breakwater when it is displaced from its initial position; this is based on the Archimedes principle, or the law of upthrust. According to this principle, the hydrodynamic spring stiffness is only possible in motions with vertical components.

The spring force can be calculated on the principle of Archimedes. As a spring stiffness is defined as:

$$F = k \cdot x \quad (C.19)$$

Where

F	: Force due the displacement;	[N]
k	: The spring stiffness;	[N/m]
x	: The displacement	[m]

The hydrodynamic spring coefficient can now be calculated as the force that is necessary for a unit displacement. According to Archimedes principle this force is equal to the displaced water volume.

The hydrodynamic spring coefficients for sway, heave and roll become:

$$\begin{aligned} c_{22} &= 0 \\ c_{33} &= \rho g Y_w L_w \\ c_{44} &= \rho g Y_w L_w T_1 \cdot \overline{GM} \end{aligned} \quad (C.20)$$

Where

c_{xx}	: Spring stiffness, k in equation (C.19)	[N/m, Nm/rad]
ρ	: Water density, 1025 kg/m ³ for salt water	[kg/m ³]
Y_w	: Width of the floating breakwater element	[m]
L_w	: Length of the floating breakwater element	[m]
T_1	: Draught of the floating breakwater element	[m]
$\overline{GM}$	: Distance between COG and metacentre	[m]

C.5 Wave exciting forces

The wave exciting forces, and moment in the case of roll, can be found by using one of the following methods:

- Froude-Krilov
This theory is based on the pressure distribution against the floating breakwater. It is valid for long waves in comparison to the structural width. A correction for diffraction is applied to extend the validity. The phase-change of the force can also be obtained by this theory.
- Newman
This theory is based on a relation between the wave frequency, the damping and the wave exciting force. The phase-change of this force cannot be found with this method.
- DELFRAC
The force can be calculated, together with the phase-change, with a radiation/diffraction program such as DELFRAC. For intermediate values interpolation is required.

These methods are compared with each other for each of the wave exciting forces and moments to find the optimal method. The method used by [Fousert, 2006] is also taken in consideration.

C.5.1 Horizontal wave exciting force

The Froude-Krilov wave force can be calculated by integrating the pressure-difference on the windward- and lee-side of the structure over the depth of the floater, as has been shown in figure C.17. It has to be noted that a screen underneath the structure does influence the Froude-Krilov force. The wave pressure on both sides is equal because the thickness of the screen is small compared to the wavelength.

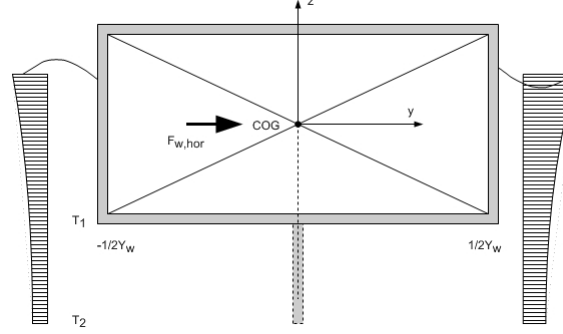


Figure C.17 Wave pressure on the sides of the floating breakwater

The integral that has to be solved is:

$$F_{FK} = \rho g \zeta_a L_w \left[\sin(\omega t - k \frac{1}{2} Y_w) - \sin(\omega t + k \frac{1}{2} Y_w) \right] \int_{-T_1}^0 e^{kz} dz \quad (C.21)$$

The solution of equation (C.21) is:

$$F_{FK} = -\frac{2}{k} \rho g L_w (e^{-kT_1} - 1) \cdot \sin\left(\frac{1}{2} k Y_w\right) \cdot \zeta_a \sin(\omega t) \quad (C.22)$$

Where:

k	: Wave number	[rad/m]
ω	: Circular wave frequency	[rad/s]
T_1	: Draught of the floater	[m]
Y_w	: Width of the floating breakwater	[m]
L_w	: Length of the floating breakwater	[m]

This Froude-Krilov wave force requires a correction because of diffraction, similar as has been done for the heave motion. Thus the total wave force can be described with:

$$F_{w,hor} = -a_{22} \cdot Q \cdot \zeta_a \sin(\omega t) - b_{22} \cdot Q \cdot \zeta_a \cos(\omega t) + C_2 \cdot Q \cdot \zeta_a \sin(\omega t) \quad (C.23)$$

Where:

a_{22}	: Added mass coefficient for the sway motion;
b_{22}	: Hydro-dynamic damping coefficient for the sway motion;
C_2	: Froude-Krilov force, $-\frac{2}{k} \rho g L_w (e^{-kT_1} - 1) \cdot \sin\left(\frac{1}{2} k Y_w\right)$;
Q	: An unknown reduced wave elevation height.

Similar to the wave force for the heave motion, there is a reduced wave-elevation used in equation (C.23). While it is easy to determine this reduced wave-elevation for the heave motion it is difficult to determine it for the sway motion. It is

expected that the reduced wave-elevation is depending on the draught of the structure and the wave-length. The exact relation for this reduction has to be found with the DELFRAC model.

The amplitude of the Froude-Krilov wave force can be found by adding the squares of the in-phase part and the out-of-phase part of (C.23):

$$F_{w,hor} = \sqrt{(b_{22}\omega)^2 + (C_2 - a_{22}\omega^2)^2} \quad (C.24)$$

The Froude-Krilov theory for the wave force is only valid for small structures compared to the wave length. Another formula for the wave force is given by [Newman, 1977] and is valid for the complete frequency range, as it is based on the rate of energy that is dissipated by the hydrodynamic damping. The formula is given as:

$$F_{w,hor} = \zeta_a \sqrt{\frac{\rho |b_{22}| g^2}{L_w \omega}} L_w \quad (C.25)$$

Where:

- b_{22} : Hydro-dynamic damping coefficient for the heave motion;
- ω : Circular wave frequency;
- L_w : Length of the element;
- ζ_a : Wave elevation height.

In his report, [Fousert, 2006] used an adapted form of the Froude-Krilov wave force, given by:

$$F_{w,hor} = -a_{22}\omega^2 \zeta_a^* \cos(\omega t) + b_{22}\omega \zeta_a^* \sin(\omega t) \quad (C.26)$$

Where:

- a_{22} : Added mass coefficient for the heave motion;
- b_{22} : Hydro-dynamic damping coefficient for the heave motion;
- ζ_a^* : Reduced wave elevation height, $\zeta_a e^{-kT_2}$.

The amplitude of the force according to [Fousert, 2006] can be found by adding the squares of the in-phase part and the out-of-phase part of (C.26):

$$F_{w,hor} = \sqrt{(b_{22}\omega)^2 + (a_{22}\omega^2)^2} \quad (C.27)$$

The wave forces are plotted in figure C.18 on the next page with the use of the added mass and hydrodynamic damping coefficients which were found by DELFRAC. The Froude-Krilov wave force, while [Fousert, 2006] underestimates the wave force for all frequencies. The method of [Newman, 1977] gives a good agreement over the whole frequency range and will be used to determine the wave exciting force for the sway movement.

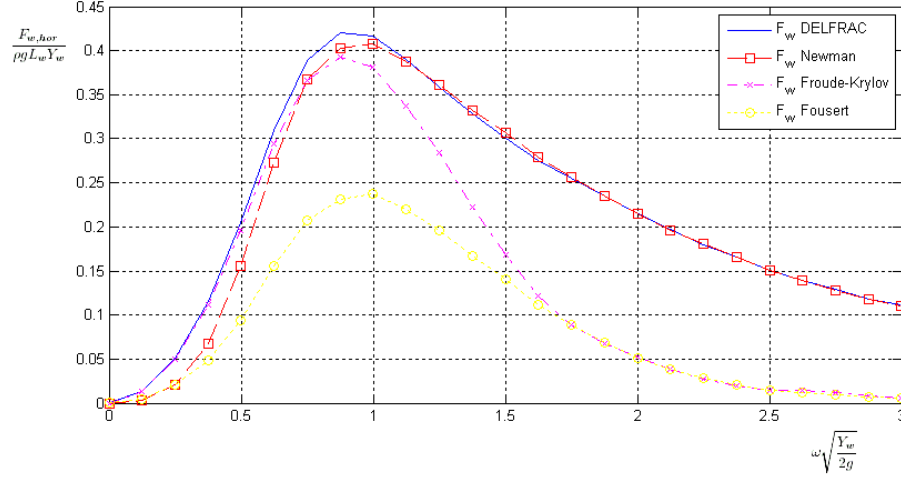


Figure C.18 Horizontal wave exciting forces for heaving by using the DELFRAC output

C.5.2 Horizontal wave exciting force, phase-change

The phase-change of the wave exciting force compared to the incoming waves can be determined with Froude-Krilov and [Fousert, 2006]. The method of Newman does not give a solution for the phase change, as this method uses the damping and the motion of the floating breakwater. As the motion, i.e. the phase change of the motion relative to the incoming waves, of the floating breakwater is not known yet the phase change of the force is not known either.

The phase-change according to Froude-Krilov is given by:

$$\varepsilon_{w,hor} = \arctan\left(\frac{b_{22}\omega}{C_2 - a_{22}\omega^2}\right) \quad (C.28)$$

Where:

$$\begin{aligned} a_{22} &: \text{Added mass coefficient for the sway motion;} \\ b_{22} &: \text{Hydro-dynamic damping coefficient for the sway motion;} \\ C_2 &: \text{Froude-Krilov force, } \frac{-\frac{2}{k}\rho g L_w (e^{-kT_1} - 1) \cdot \sin\left(\frac{1}{2}kY_w\right)}{Q}; \end{aligned}$$

The phase change according to [Fousert, 2006] is given by:

$$\varepsilon_{w,hor} = \arctan\left(\frac{b_{22}\omega}{-a_{22}\omega^2}\right) \quad (C.29)$$

Where:

$$\begin{aligned} a_{22} &: \text{Added mass coefficient for the heave motion;} \\ b_{22} &: \text{Hydro-dynamic damping coefficient for the heave motion;} \end{aligned}$$

The results from DELFRAC and the phase-change according to Froude-Krilov and [Fousert, 2006] are plotted in figure C.19. It can be seen that, as with the amplitude of the exciting force, that the Froude-Krilov and [Fousert, 2006] give a good result for long waves, but that there is a big deviance for the shorter wave lengths.

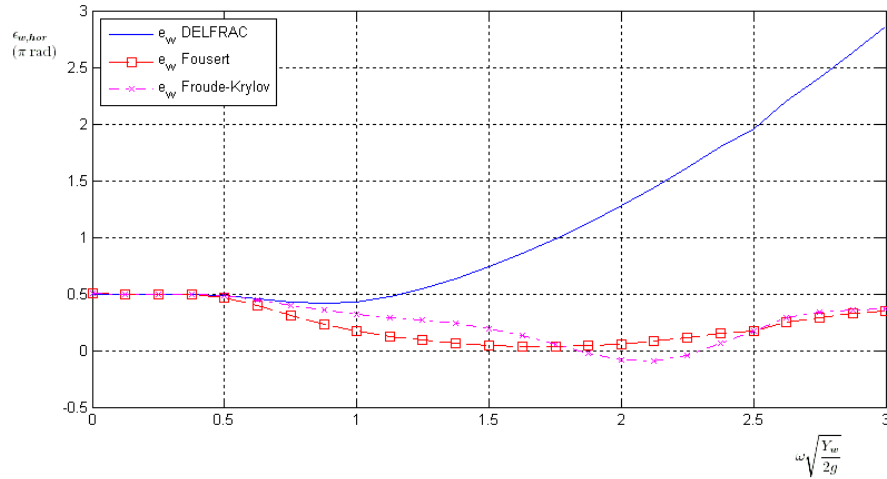


Figure C.19 Phase-change of the horizontal wave exciting force relative to the incoming wave

In figure C.20 the same figure is given again, but the data is plotted against the number of wave-lengths under the structure. It can be seen that there is a linear relationship between the phase-change and the number of wave-lengths underneath the structure for the shorter wave-lengths. This relation is given by:

$$\varepsilon_{w,hor} = n\pi \quad (C.30)$$

Where:

n : Number of wave-lengths underneath the structure;

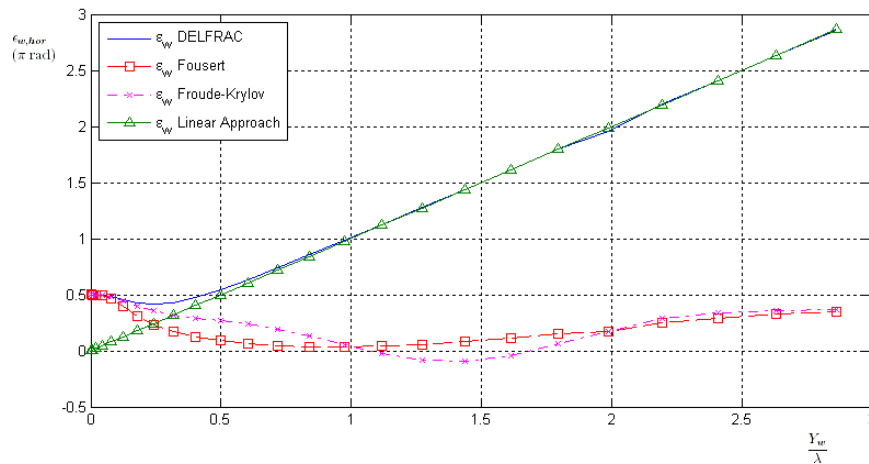


Figure C.20 Phase-change of the horizontal wave exciting force relative to the incoming wave against the number of wave-lengths underneath the construction

The phase-changes for different Y_w/T_1 -ratios are shown in figure C.21. In all cases the linear behaviour for longer wave lengths is shown for a structure that is at least as wide as a wave-length. For the longer waves there is a transition where the behaviour of the different ratios deviates from each other, giving room for optimising the performance of the floating breakwater.

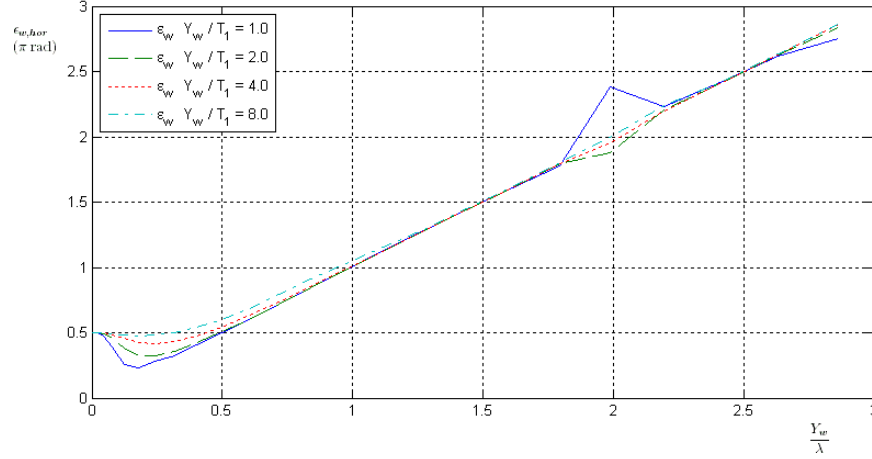


Figure C.21 Phase-change of the horizontal wave exciting force relative to the incoming wave for the different Y_w/T_1 -ratios

C.5.3 Vertical wave exciting force

The Froude-Krilov wave force can be calculated by integrating the pressure on the bottom of the structure over the width of the floater, as has been shown in figure C.22. It has to be noted that a screen underneath the structure does not have an influence on the Froude-Krilov force, because the thickness of the screen is small compared to the wavelength.

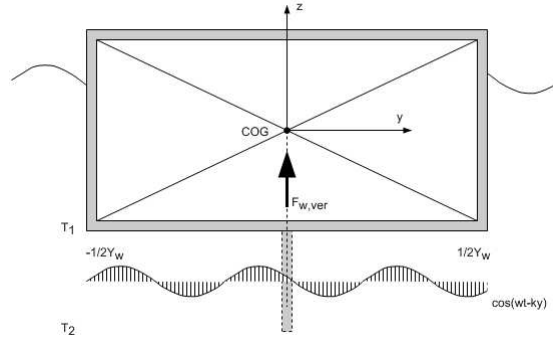


Figure C.22 Wave pressure underneath the floating breakwater

The integral that has to be solved is:

$$F_{FK} = \rho g \zeta_a e^{-kT_1} L_w \int_{-1/2 Y_w}^{1/2 Y_w} \cos(\omega t - ky) dy \quad (C.31)$$

The solution of equation (C.31) is:

$$F_{FK} = -\frac{2}{k} \rho g L_w \cdot \sin\left(\frac{1}{2} k Y_w\right) \cdot \zeta_a e^{-kT_1} \cos(\omega t) \quad (C.32)$$

Where:

k	: Wave number	[rad/m]
ω	: Circular wave frequency	[rad/s]
T_1	: Draught of the floater	[m]
Y_w	: Width of the floating breakwater	[m]
L_w	: Length of the floating breakwater	[m]

This Froude-Krilov wave force requires a correction because of diffraction as has been described in [Journée and Massie, 2001]. The total wave force can be described with:

$$F_{w,ver} = -a_{33}\omega^2\zeta_a^* \cos(\omega t) + b_{33}\omega\zeta_a^* \sin(\omega t) + C_3\zeta_a^* \cos(\omega t) \quad (C.33)$$

Where:

- a_{33} : Added mass coefficient for the heave motion;
- b_{33} : Hydro-dynamic damping coefficient for the heave motion;
- C_3 : Froude-Krilov force, $-\frac{2}{k}\rho g L_w \cdot \sin\left(\frac{1}{2}kY_w\right)$;
- ζ_a^* : Reduced wave elevation height, $\zeta_a e^{-kT_1}$.

The amplitude of the Froude-Krilov wave force can be found by adding the squares of the in-phase part and the out-of-phase part of (C.33):

$$F_{w,ver} = \sqrt{(b_{33}\omega)^2 + (C_3 - a_{33}\omega^2)^2} \quad (C.34)$$

The Froude-Krilov theory for the wave force is only valid for small structures compared to the wave length. Another formula for the wave force is given by [Newman, 1977] and is valid for the complete frequency range, as it is based on the rate of energy that is dissipated by the hydrodynamic damping. The formula is given as:

$$F_{w,ver} = \zeta_a \sqrt{\frac{\rho |b_{33}| g^2}{L_w \omega}} L_w \quad (C.35)$$

Where:

- b_{33} : Hydro-dynamic damping coefficient for the heave motion;
- ω : Circular wave frequency;
- L_w : Length of the element;
- ζ_a : Reduced wave elevation height.

In his report, [Fousert, 2006] used an adapted form of the Froude-Krilov wave force, given by:

$$F_{w,ver} = -a_{33}\omega^2\zeta_a^* \cos(\omega t) + b_{33}\omega\zeta_a^* \sin(\omega t) + C_3 c_{33}\zeta_a^* \cos(\omega t) \quad (C.36)$$

Where:

- a_{33} : Added mass coefficient for the heave motion;
- b_{33} : Hydro-dynamic damping coefficient for the heave motion;
- c_{33} : Restoring spring coefficient;
- C_3 : Froude-Krilov force, $1 - \frac{k}{Y_w} e^{-kT_1}$;
- ζ_a^* : Reduced wave elevation height, $\zeta_a e^{-kT_1}$.

The amplitude of the force according to [Fousert, 2006] can be found by adding the squares of the in-phase part and the out-of-phase part of (C.36):

$$F_{w,ver} = \sqrt{(b_{33}\omega)^2 + (C_3c_{33} - a_{33}\omega^2)^2} \quad (C.37)$$

The wave forces are plotted in figure C.23 with use of the added mass and hydrodynamic damping coefficients which were found by DELFRAC. The Froude-Krilov wave force and the wave force as has been used by [Fousert, 2006] show a good agreement for long waves, while [Newman, 1977] shows a big deviation in this range. For shorter waves, the formula of Newman has a good fit, while the Froude-Krilov and [Fousert, 2006] are giving a too high estimation.

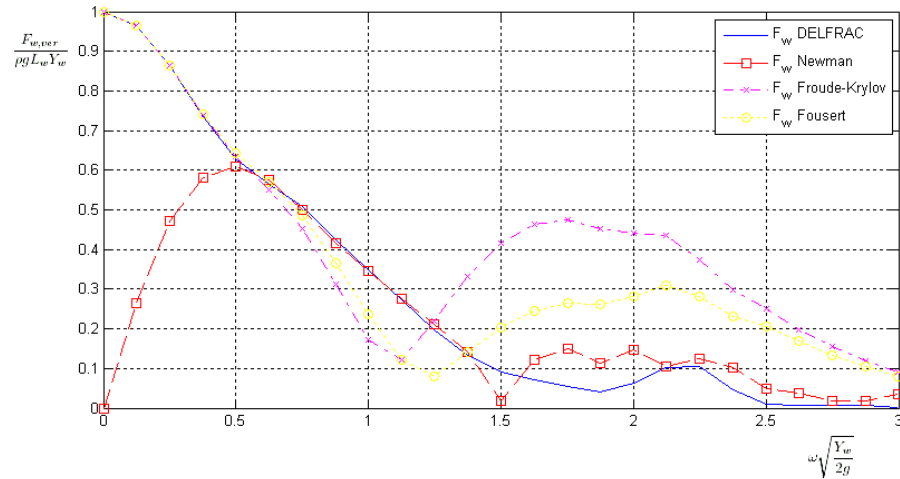


Figure C.23 Vertical wave exciting forces for heaving by using the DELFRAC output

When the curves for the added mass and the hydrodynamic damping are used, as has been discussed in paragraph C.4.2, the method of Newman gives a good agreement over the whole frequency range, while the other two methods are showing a good agreement for long waves only. The results of [Vugts, 1968] will be used for heave in the ReFBreak2D model, to avoid possible three-dimensional effects. Together with these results, the method of Newman gives a good result and will therefore be used in the model.

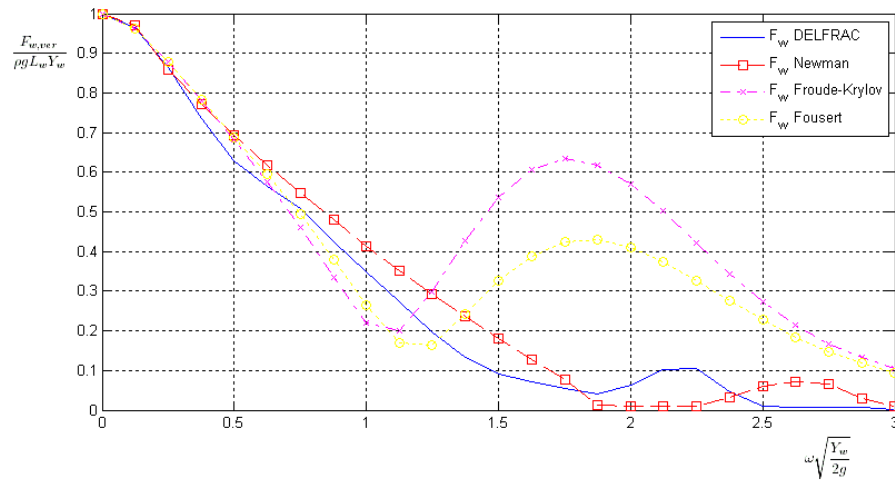


Figure C.24 Vertical wave exciting forces for heaving by using the polynomials for the coefficients

C.5.4 Vertical wave exciting force, phase-change

The phase-change of the wave exciting force compared to the incoming waves can be determined with Froude-Krilov and [Fousert, 2006]. The method of Newman does not give a solution for the phase change for the same reason as with the sway wave exciting force.

The phase-change according to Froude-Krilov is given by:

$$\varepsilon_{w,ver} = \arctan\left(\frac{b_{33}\omega}{C_3 - a_{33}\omega^2}\right) \quad (C.38)$$

Where:

- a_{33} : Added mass coefficient for the heave motion;
- b_{33} : Hydro-dynamic damping coefficient for the heave motion;
- C_3 : Froude-Krilov force, $-\frac{2}{k}\rho g L_w \cdot \sin\left(\frac{1}{2}kY_w\right)$;

The phase change according to [Fousert, 2006] is given by:

$$\varepsilon_{w,ver} = \arctan\left(\frac{b_{33}\omega}{C_3 c_{33} - a_{33}\omega^2}\right) \quad (C.39)$$

Where:

- a_{33} : Added mass coefficient for the heave motion;
- b_{33} : Hydro-dynamic damping coefficient for the heave motion;
- c_{33} : Restoring spring coefficient;
- C_3 : Froude-Krilov force, $1 - \frac{k}{Y_w} e^{-kT_1}$.

The results from DELFRAC and the phase-change according to Froude-Krilov and [Fousert, 2006] are plotted in figure C.25. It can be seen, as with the amplitude of the exciting force, that the Froude-Krilov and [Fousert, 2006] give a good result for long waves, but that there is a big deviance for the shorter wave lengths.

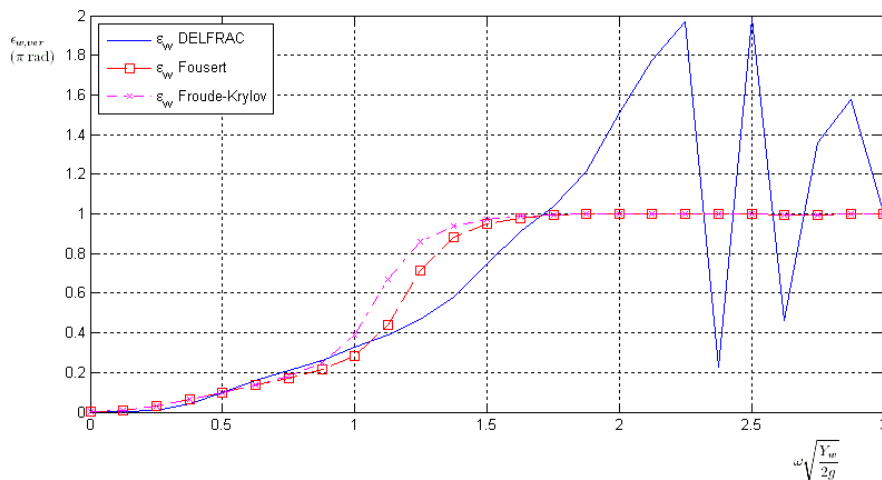


Figure C.25 Phase-change of the vertical wave exciting force relative to the incoming wave. The peaks are due to the fact that $\varepsilon_{w,ver} = k \bmod 2\pi$

In figure C.26 the same figure is given again, but the data is plotted against the number of wave-lengths under the structure. It can be seen that there is a linear relationship between the phase-change and the number of wave-lengths underneath the structure. This relation is given by:

$$\mathcal{E}_{w,ver} = n\pi \quad (C.40)$$

Where:

n : Number of wave-lengths underneath the structure;

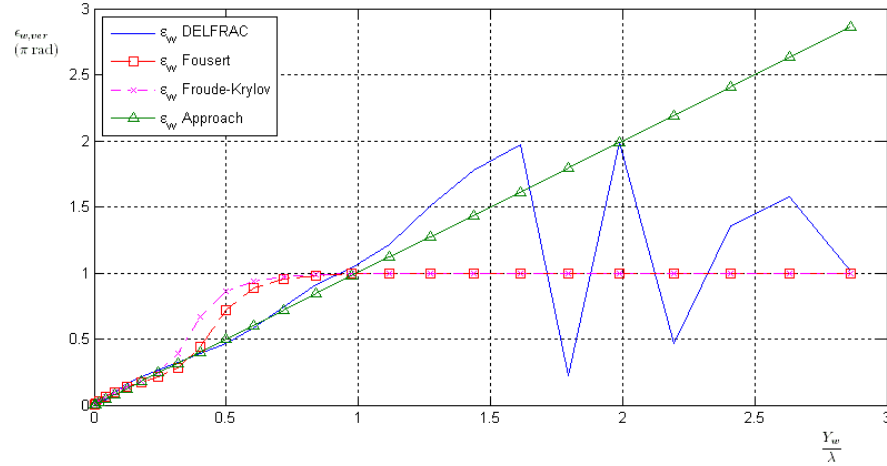


Figure C.26 Phase-change of the vertical wave exciting force relative to the incoming wave against the number of wave-lengths underneath the construction. The peaks are due to the fact that $\mathcal{E}_{w,ver} = k \bmod 2\pi$

The relation between the number of wave-lengths and the results from DELFRAC shows a good agreement until a ratio of 1.5. When the ratio becomes greater the DELFRAC solution becomes erratic. It is assumed that the relation is also valid with shorter wave-lengths. When looking at the phase-change of the sway wave exciting force the validity of this assumption is strengthened. The phase-change of the sway wave exciting force is linear for the whole range of short waves.

C.5.5 Wave exciting moment

The wave exciting moment can be determined by integrating the wave pressures multiplied by the lever arm relative to the centre of gravity, as is shown in figure C.27. This can be done separately for the vertical pressures and the horizontal pressures. The total wave exciting moment is the summation of these two.

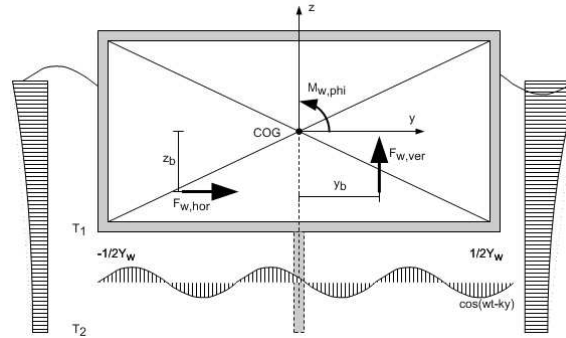


Figure C.27 Pressure causing the wave exciting moment.

The Froude-Krilov wave exciting moment caused by the vertical wave pressure can be obtained by solving the following integral:

$$M_{FK;ver} = \rho g \zeta_a e^{-kT_1} L_w \int_{-\frac{1}{2}Y_w}^{\frac{1}{2}Y_w} y \cos(\omega t - ky) dy \quad (C.41)$$

The solution to this integral is given by:

$$M_{FK;ver} = \rho g L_w \zeta_a e^{-kT_1} \left(\frac{2}{k^2} \sin\left(\frac{1}{2}kY_w\right) - \frac{Y_w}{k} \cos\left(\frac{1}{2}kY_w\right) \right) \sin(\omega t) \quad (C.42)$$

The Froude-Krilov wave exciting moment caused by the horizontal wave pressure, i.e. the difference in the wave pressure on the luff- and the leeside of the structure, can be obtained by solving the following integral:

$$M_{FK} = \rho g \zeta_a L_w \left[\sin(\omega t - k \frac{1}{2}Y_w) - \sin(\omega t + k \frac{1}{2}Y_w) \right] \int_{-T_1}^0 (-\overline{OG} - z) e^{kz} dz \quad (C.43)$$

The solution to this integral is given by:

$$M_{FK;hor} = \frac{2}{k^2} \rho g \zeta_a L_w \left(1 + (k \cdot \overline{OG} - kT_1) e^{-kT_1} - k \cdot \overline{OG} \right) \sin\left(\frac{1}{2}kY_w\right) \cdot \sin(\omega t) \quad (C.44)$$

The Froude-Krilov wave force requires a correction because of diffraction, similar as has been done for the heave motion. Thus the total wave force can be described with:

$$M_{w,\phi} = -a_{44} \cdot Q \cdot \zeta_a \sin(\omega t) - b_{44} \cdot Q \cdot \zeta_a \cos(\omega t) + C_4 \cdot Q \cdot \zeta_a \sin(\omega t) \quad (C.45)$$

Where:

- a_{44} : Added mass coefficient for the roll motion;
- b_{44} : Hydro-dynamic damping coefficient for the roll motion;
- C_4 : Froude-Krilov force, from equations (C.42) and (C.44);
- Q : An unknown reduced wave elevation height.

The amplitude of the Froude-Krilov wave moment can be found by adding the squares of the in-phase part and the out-of-phase part of (C.45):

$$M_{w,\phi} = \sqrt{(b_{44}\omega)^2 + (C_4 - a_{44}\omega^2)^2} \quad (C.46)$$

The Froude-Krilov theory for the wave moment is only valid for small structures compared to the wave length. Another formula for the wave moment is given by [Newman, 1977] and is valid for the complete frequency range, as it is based on the rate of energy that is dissipated by the hydrodynamic damping. The formula is given as:

$$M_{w,\phi} = \zeta_a \sqrt{\frac{\rho |b_{44}| g^2}{L_w \omega}} L_w \quad (C.47)$$

In his report, [Fousert, 2006] used an adapted form of the Froude-Krilov wave force, given by:

$$M_{w,\phi} = F_{w,hor} z_b + F_{w,ver} y_b \quad (C.48)$$

Where:

- $F_{w,hor}$: The horizontal wave exciting force;

- $F_{w,ver}$: The vertical wave exciting force;
 y_b : Horizontal lever arm, relative to the origin;
 z_b : Vertical lever arm, relative to the origin.

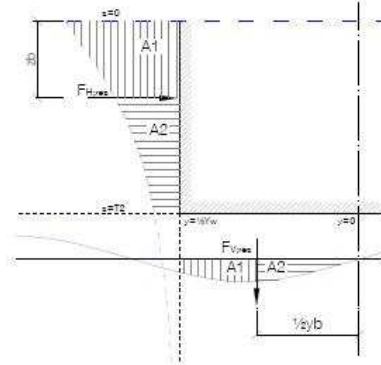


Figure C.28 Determination of y_b and z_b [Fousert, 2006]

The lever arms are determined according to figure C.28, where area A_1 and A_2 have to be equal to each other. This yields into the following formulas:

$$y_b = 2 \frac{\arccos\left(\frac{1 + \cos\left(\frac{1}{2}kY_w\right)}{2}\right)}{k} \quad (C.49)$$

$$z_b = \frac{\ln\left(\frac{1 + e^{-kT_2}}{2}\right)}{-k} \quad (C.50)$$

The amplitude of the force according to [Fousert, 2006] can be found by adding the squares of the in-phase part and the out-of-phase part of (C.48).

The wave moments are plotted in figure C.29 with the use of the added mass and hydrodynamic damping coefficients which were found by DELFRAC. The Froude-Krilov wave moment shows a good agreement for the longer waves, while [Fousert, 2006] is omitted in this graph because over the whole frequency range the moment is order of magnitude greater than the expected value. [Newman, 1977] gives a good agreement over the whole frequency range and will be used to determine the wave exciting moment for the roll movement.

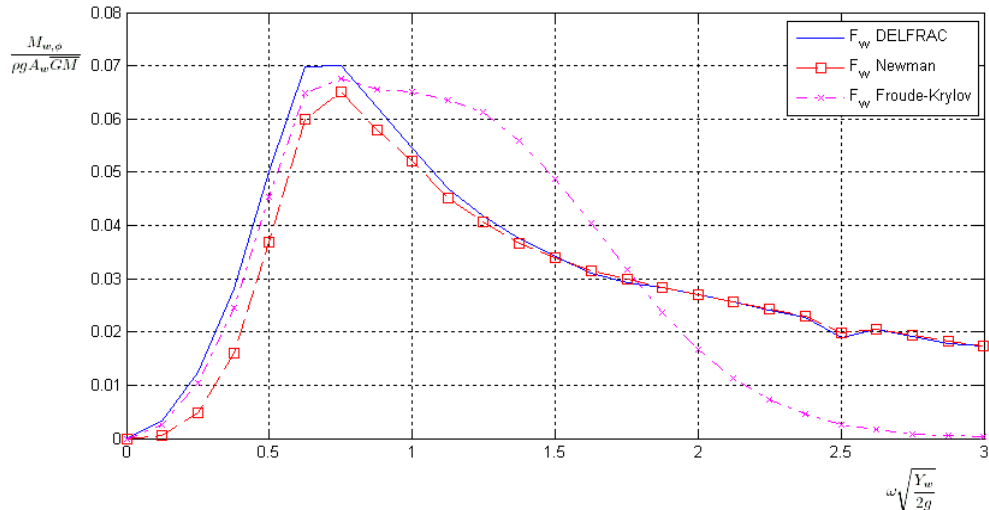


Figure C.29 Wave exciting moments for roll by using the DELFRAC output

C.5.6 Wave exciting moment, phase-change

The phase-change of the wave exciting moment compared to the incoming waves can be determined with Froude-Krilov. [Fousert, 2006] has been left out of consideration because of the poor results on predicting the wave exciting moments. The method of Newman does not give a solution for the phase change for the same reason as with the sway wave exciting force.

The phase-change according to Froude-Krilov is given by:

$$\varepsilon_{w,\phi} = \arctan\left(\frac{b_{44}\omega}{C_4 - a_{44}\omega^2}\right) \quad (C.51)$$

Where:

- a_{44} : Added mass coefficient for the roll motion;
- b_{44} : Hydro-dynamic damping coefficient for the heave motion;
- C_4 : Froude-Krilov force, from equations (C.42) and (C.44);

The results from DELFRAC and the phase-change according to Froude-Krilov are plotted in figure C.30. It can be seen that, as with the amplitude of the exciting force, that the Froude-Krilov gives a good result for long waves, but that there is a big deviance for the shorter wave lengths.

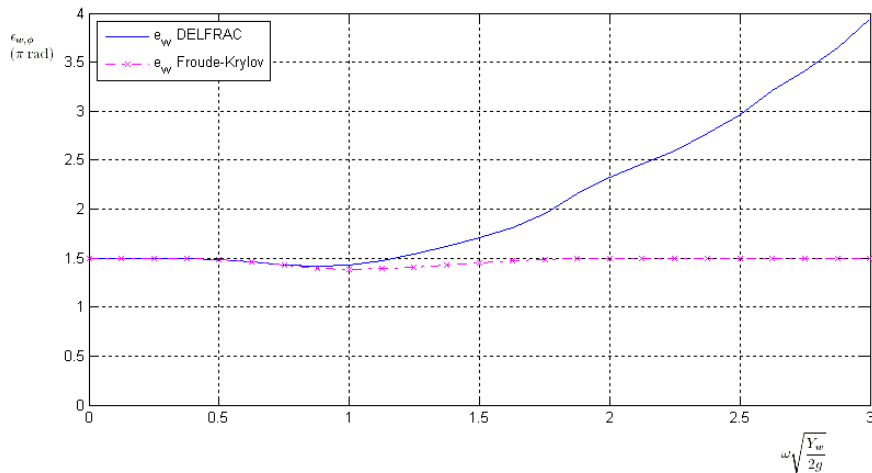


Figure C.30 Phase-change of the wave exciting moment relative to the incoming wave

In figure C.31 on the next page the same figure is given again, but the data is plotted against the number of wave-lengths under the structure. It can be seen that there is a linear relationship between the phase-change and the number of wave-lengths underneath the structure. This relation is given by:

$$\varepsilon_{w,\phi} = n\pi + \pi \quad (C.52)$$

Where:

- n : Number of wave-lengths underneath the structure;

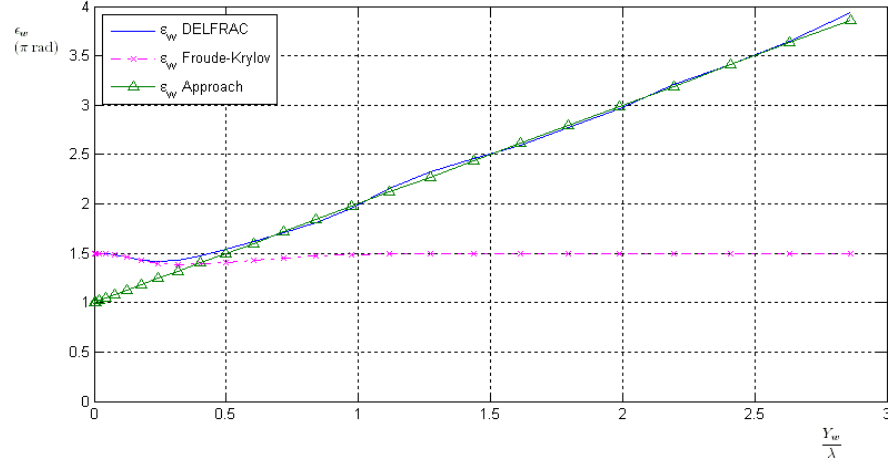


Figure C.31 Phase-change of the wave exciting moment relative to the incoming wave against the number of wave-lengths underneath the construction

C.6 Structural motions

The structural motions of the floating breakwater can be calculated now all the hydrodynamic coefficients are known. The equation that has to be solved has been given in (C.9):

$$M\ddot{\vec{X}} + N\dot{\vec{X}} + C\vec{X} = \vec{F} \quad (\text{C.53})$$

Where

- $\vec{X}$: Translation and rotation vector;
- $\vec{F}$: Wave exciting forces and moments vector;
- M : Hydrodynamic mass matrix;
- N : Hydrodynamic damping matrix;
- C : Hydrodynamic spring stiffness matrix.

The force in (C.53) is given as:

$$\vec{F} = \begin{bmatrix} F_{w,2} \cos(\omega t + \varepsilon_{w,2}) \\ F_{w,3} \cos(\omega t + \varepsilon_{w,3}) \\ F_{w,4} \cos(\omega t + \varepsilon_{w,4}) \end{bmatrix} \quad (\text{C.54})$$

Where:

- $F_{w,n}$: Amplitude of the wave exciting force or moment;
- ω : Circular wave frequency;
- $\varepsilon_{w,n}$: Phase-change of the wave exciting force relative to the COG.

This can be written in the complex plane as:

$$\vec{F} = \text{Re } F_0 e^{i\omega t} \quad (\text{C.55})$$

Where:

$$F_0 : \text{The complex amplitude of the force: } \begin{bmatrix} F_{w,2} \cdot e^{i\varepsilon_{w,2}} \\ F_{w,3} \cdot e^{i\varepsilon_{w,3}} \\ F_{w,4} \cdot e^{i\varepsilon_{w,4}} \end{bmatrix}.$$

It is assumed that the solution of the vector $\vec{X}$ is given as:

$$\vec{X} = \{X\}e^{i\omega t} \quad (C.56)$$

Now equation (C.53) can be rewritten, with the term $e^{i\omega t}$ omitted, as:

$$-M\omega^2 \{X\} + Ni\omega\{X\} + C\{X\} = F_0 \quad (C.57)$$

This is further elaborated on to:

$$Q\{X\} = F_0 \quad (C.58)$$

Where:

Q : Combined matrix, defined as $-M\omega^2 + Ni\omega + C$.

The solution of vector $\{X\}$ can be found by rewriting (C.58) to:

$$\{X\} = Q^{-1}F_0 \quad (C.59)$$

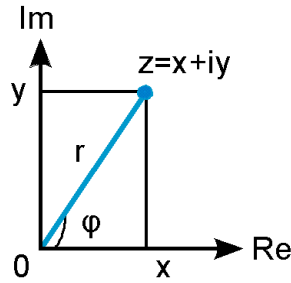


Figure C.32 A graphical representation of a vector in the complex plane

From the solution of (C.59) the amplitude of the structural motion and the phase-change can be obtained, as is shown in figure C.32 graphically. The amplitude and phase-change are respectively:

$$x_a = |\{X\}| \quad (C.60)$$

$$\varepsilon_\zeta = \arctan\left(\frac{\text{Im}\{X\}}{\text{Re}\{X\}}\right) \quad (C.61)$$

The final solution for the structural motion is:

$$\begin{aligned} y(t) &= x_{a,2} \cos(\omega t + \varepsilon_{\zeta,2}) \\ z(t) &= x_{a,3} \cos(\omega t + \varepsilon_{\zeta,3}) \\ \phi(t) &= x_{a,4} \cos(\omega t + \varepsilon_{\zeta,4}) \end{aligned} \quad (C.62)$$

C.7 Energy consideration

In the case of the floating breakwater it is assumed that the waves are linear. This means that the different waves radiated by the motions of the breakwater can be added with respect to their phase-changes. The total transmitted wave on the lee-side of the floating breakwater is the summation of all radiated waves and the wave-energy that is flowing underneath the floating breakwater.

C.7.1 Heave

To assess the amount of power that is dissipated towards the still water on the lee side of the floating breakwater, one has to determine the amount of work that is done by the mass, damping and the spring force per unit of time, averaged over one period of oscillation T .

The floating breakwater is assumed to carry out an oscillation motion which is defined by:

$$z(t) = z_a \cos(\omega t + \varepsilon_{z\zeta}) \quad (\text{C.63})$$

For the mass component the power that is dissipated is:

$$\begin{aligned} P_m &= \frac{1}{T} \int_0^T \{ (m_{33} + a_{33}) \cdot \dot{z} \} \cdot \{ \dot{z} \cdot dt \} \\ &= \frac{-z_a^3 (m_{33} + a_{33}) \omega^3}{T} \int_0^T \cos(\omega t + \varepsilon_{z\zeta}) \sin(\omega t + \varepsilon_{z\zeta}) dt \\ &= 0 \end{aligned} \quad (\text{C.64})$$

For the damping component the power that is dissipated becomes:

$$\begin{aligned} P_d &= \frac{1}{T} \int_0^T \{ b_{33} \cdot \dot{z} \} \cdot \{ \dot{z} \cdot dt \} \\ &= \frac{z_a^2 b_{33} \omega^2}{T} \int_0^T \sin^2(\omega t + \varepsilon_{z\zeta}) dt \\ &= \frac{1}{2} z_a^2 b_{33} \omega^2 \end{aligned} \quad (\text{C.65})$$

For the spring component the power that is dissipated is:

$$\begin{aligned} P_m &= \frac{1}{T} \int_0^T \{ c_{33} \cdot z \} \cdot \{ \dot{z} \cdot dt \} \\ &= \frac{-z_a c_{33} \omega}{T} \int_0^T \cos(\omega t + \varepsilon_{z\zeta}) \sin(\omega t + \varepsilon_{z\zeta}) dt \\ &= 0 \end{aligned} \quad (\text{C.66})$$

Only the damping force dissipates power, in other words: damping is the reason why the motion of the floating breakwater dies out. The power dissipated by the hydrodynamic damping is exactly the amount of power which is dissipated by the waves on both sides of the structure. The wave power that is radiated is defined by:

$$\begin{aligned} E_{r,n} &= \frac{1}{T} (2L_w)(E)(c_g T) \\ &= \frac{1}{T} (2L_w) \left(\frac{1}{2} \rho g \zeta_r^2 \right) \left(\frac{1}{2} \frac{g}{\omega} T \right) \\ &= \frac{L_w \rho g^2 \zeta_r^2}{2\omega} \end{aligned} \quad (\text{C.67})$$

Equalizing equation (C.65) and (C.67) gives the amplitude of the radiated wave due to the oscillatory motion of the floating breakwater:

$$\zeta_{r,a} = \zeta_i \sqrt{\frac{b_{33} \omega^3 z_a^2}{L_w \rho g^2}} \quad (\text{C.68})$$

In figure C.33 the power dissipated by the damping is plotted against the time for a fictive case. The velocity of the heave motion is plotted as well and it has to be noted that the positive velocity is defined to be upwards. An upward motion of the floating breakwater causes a negative wave, i.e. the water is moving towards the floating breakwater and causes a trough instead of a crest. The power of this wave is exactly the same as the power that is dissipated by the damping.

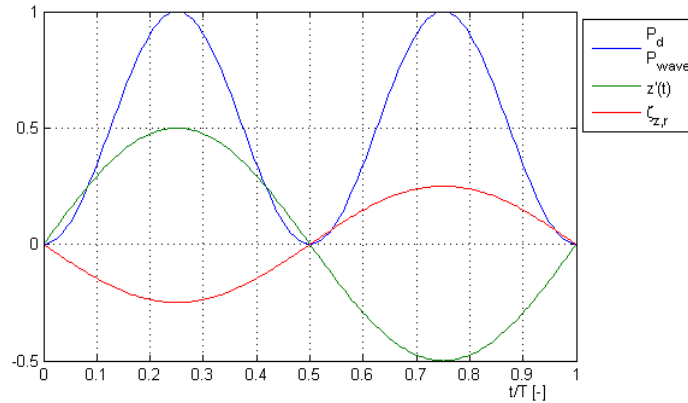


Figure C.33 Dissipated power, structural velocity and generated wave versus time for heave

Figure C.33 gives the phase-change of the radiated wave with respect to the incoming wave. The transmitted wave motion towards the lee side of the breakwater is now defined as:

$$\zeta_r(t) = \zeta_i \sqrt{\frac{b_{33} \omega^3 z_a^2}{L_w \rho g^2}} \sin(\omega t + \varepsilon_{z\zeta} - \pi) \quad (\text{C.69})$$

This can be rewritten to a cosine, making it easier to calculate the total transmitted wave:

$$\zeta_r(t) = \zeta_i \sqrt{\frac{b_{33} \omega^3 z_a^2}{L_w \rho g^2}} \cos\left(\omega t + \varepsilon_{z\zeta} - \frac{1}{2}\pi\right) \quad (\text{C.70})$$

C.7.2 Sway/Roll

To assess the amount of power that is dissipated towards the still water on the lee side of the floating breakwater by the coupled sway and roll motions, one has to determine the amount of work that is done by the mass, damping and the spring force per unit of time, averaged over one period of oscillation T . As for the case of the heave motion, the mass and the spring force do not dissipate power, only the damping force causes the oscillation motion of the floating breakwater to decay.

The floating breakwater is assumed to carry out a coupled oscillation motion which is defined by:

$$\begin{bmatrix} y(t) \\ \phi(t) \end{bmatrix} = \begin{bmatrix} y_a \cos(\omega t + \varepsilon_{y\zeta}) \\ \phi_a \cos(\omega t + \varepsilon_{\phi\zeta}) \end{bmatrix} \quad (\text{C.71})$$

The power that is dissipated by the damping force is defined as:

$$\begin{aligned}
 P_d &= \frac{1}{T} \int_0^T \left\{ N \cdot \begin{bmatrix} \dot{y} \\ \dot{\phi} \end{bmatrix} \right\} \cdot \left\{ \begin{bmatrix} \dot{y} \\ \dot{\phi} \end{bmatrix} \right\} dt \\
 &= \frac{\omega^2}{T} \int_0^T \left(\begin{aligned} &b_{22} y_a^2 \sin^2(\omega t + \varepsilon_{y\zeta}) \\ &+ b_{24} y_a \sin(\omega t + \varepsilon_{y\zeta}) \phi_a \sin(\omega t + \varepsilon_{\phi\zeta}) \\ &+ b_{42} y_a \phi_a \sin(\omega t + \varepsilon_{\phi\zeta}) \sin(\omega t + \varepsilon_{y\zeta}) \\ &+ b_{44} \phi_a^2 \sin^2(\omega t + \varepsilon_{\phi\zeta}) \end{aligned} \right) dt
 \end{aligned} \quad (C.72)$$

In equation (C.72) it has to be noted that only the hydrodynamic damping is taken in account. For the roll motion viscous damping is relevance, but this damping force does not generate waves, the power is dissipated in the form of turbulence. Also it can be seen that the power dissipation cannot uniquely assigned to one of the structural motions due to the coupling coefficients. This can be solved by transforming [Vugts, 1968]

$$\begin{aligned}
 y_o &= y_g - \overline{OG} \cdot \phi_g \\
 z_o &= z_g \\
 \phi_o &= \phi_g \\
 F_{o,w,hor} &= F_{w,hor} \\
 F_{o,w,ver} &= F_{w,ver} \\
 M_{o,w,\phi} &= M_{w,\phi} + \overline{OG} \cdot F_{w,hor}
 \end{aligned} \quad (C.73)$$

Working this out and dropping the index O for motions and exciting forces results in:

$$(m + a_{22}) \ddot{y} + b_{22} \dot{y} + (a_{24} + \overline{OG} \cdot a_{22}) \ddot{\phi} + (b_{24} + \overline{OG} \cdot b_{22} + \overline{OG} m_{22}) \dot{\phi} = F_{w,hor} \quad (C.74)$$

$$(m + a_{33}) \ddot{z} + b_{33} \dot{z} + c_{33} z = F_{w,ver} \quad (C.75)$$

$$\begin{aligned}
 &\left(I_{xx} + \overline{OG}^2 m_{22} + a_{44} + \overline{OG} \cdot a_{42} + \overline{OG}^2 \cdot a_{22} + \overline{OG} \cdot a_{24} \right) \ddot{\phi} + \\
 &\quad \left(b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24} \right) \dot{\phi} + \\
 &\quad \left(mg \cdot \overline{GM} \right) \phi + \left(a_{42} + \overline{BG} \cdot a_{22} + \overline{OG} m_{22} \right) \ddot{y} + \\
 &\quad \left(b_{42} + \overline{BG} \cdot b_{22} \right) \dot{y} = M_{w,\phi} + \overline{BG} \cdot F_{w,hor}
 \end{aligned} \quad (C.76)$$

Thus the hydrodynamic mass matrix can be rewritten as:

$$M = \begin{bmatrix} (m + a_{22}) & 0 & (a_{24} + \overline{OG} \cdot a_{22} + \overline{OG} m_{22}) \\ 0 & (m + a_{33}) & 0 \\ (a_{42} + \overline{OG} \cdot a_{22} + \overline{OG} m_{22}) & 0 & (I_{xx} + \overline{OG}^2 m_{22} + a_{44} + \overline{OG} \cdot a_{42} + \overline{OG}^2 \cdot a_{22} + \overline{OG} \cdot a_{24}) \end{bmatrix} \quad (C.77)$$

And the hydrodynamic damping matrix as:

$$N = \begin{bmatrix} b_{22} & 0 & (b_{24} + \overline{OG} \cdot b_{22}) \\ 0 & b_{33} & 0 \\ (b_{42} + \overline{OG} \cdot b_{22}) & 0 & (b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24}) \end{bmatrix} \quad (C.78)$$

Because of the symmetry of the floating breakwater and the zero forward speed the coefficients b_{24} and b_{42} are equal and $\overline{OG}$ can be chosen in such a way that the term $(b_{24} + \overline{OG} \cdot b_{22})$ is equal to zero. When this is substituted into equation (C.72) the power that is dissipated by the floating breakwater is:

$$P_d = \frac{\omega^2}{T} \int_0^T \left(b_{22} y_a^2 \sin^2(\omega t + \varepsilon_{y\zeta}) + (b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24}) \phi_a^2 \sin^2(\omega t + \varepsilon_{\phi\zeta}) \right) dt \quad (C.79)$$

$$= \frac{1}{2} \left(b_{22} y_a^2 + (b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24}) \phi_a^2 \right) \omega^2$$

Now the power that is dissipated can be assigned to the corresponding motion, where it has to be noted that roll dissipates far less power than sway. The power dissipated by the hydrodynamic damping is exactly the amount of power which is dissipated by the waves on both sides of the structure. The wave power that is radiated is defined by:

$$E_{r,n} = \frac{1}{T} (2L_w)(E)(c_g T)$$

$$= \frac{1}{T} (2L_w) \left(\frac{1}{2} \rho g \zeta_r^2 \right) \left(\frac{1}{2} \frac{g}{\omega} T \right) \quad (C.80)$$

$$= \frac{L_w \rho g^2 \zeta_r^2}{2\omega}$$

Equalizing (C.79) and (C.80) gives the amplitude of the radiated wave due to the oscillatory motion of the floating breakwater for sway:

$$\zeta_{r,a} = \sqrt{\frac{b_{22} \omega^3 y_a^2}{L_w \rho g^2}} \quad (C.81)$$

And for roll:

$$\zeta_{r,a} = \sqrt{\frac{(b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24}) \omega^3 \phi_a^2}{L_w \rho g^2}} \quad (C.82)$$

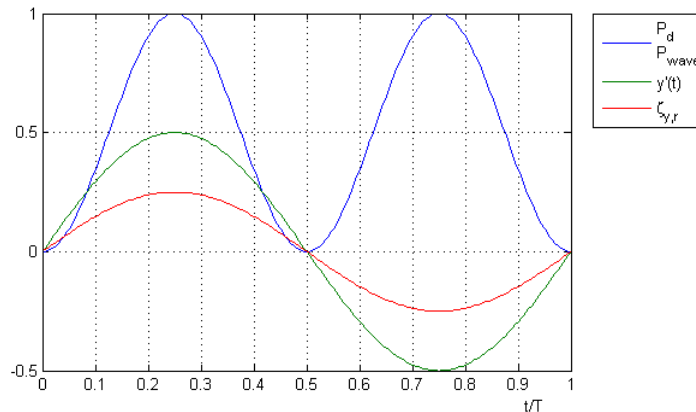


Figure C.34 Dissipated power, structural velocity and generated wave versus time for sway

Figure C.34 gives the phase-change of the radiated wave with respect to the incoming wave. The transmitted wave motion towards the lee side of the breakwater is now defined for sway as:

$$\zeta_r(t) = \sqrt{\frac{b_{22}\omega^3 z_a^2}{L_w \rho g^2}} \sin\left(\omega t + \varepsilon_{y\zeta} + \frac{1}{2}kY_w\right) \quad (C.83)$$

And for roll:

$$\zeta_r(t) = \sqrt{\frac{(b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24})\omega^3 \phi_a^2}{L_w \rho g^2}} \sin\left(\omega t + \varepsilon_{\phi\zeta} + \frac{1}{2}kY_w\right) \quad (C.84)$$

The phase-change of $\frac{1}{2}kY_w$ is due to the fact that the wave is pushed away in front of the floating breakwater wall, which location is not in the centre of gravity. Equations (C.83) and (C.84) can be rewritten to a cosine, making it easier to calculate the total transmitted wave for sway:

$$\zeta_r(t) = \sqrt{\frac{b_{22}\omega^3 z_a^2}{L_w \rho g^2}} \cos\left(\omega t + \varepsilon_{y\zeta} + \frac{1}{2}\pi + \frac{1}{2}kY_w\right) \quad (C.85)$$

And for roll:

$$\zeta_r(t) = \sqrt{\frac{(b_{44} + \overline{OG} \cdot b_{42} + \overline{OG}^2 \cdot b_{22} + \overline{OG} \cdot b_{24})\omega^3 \phi_a^2}{L_w \rho g^2}} \cos\left(\omega t + \varepsilon_{\phi\zeta} + \frac{1}{2}\pi + \frac{1}{2}kY_w\right) \quad (C.86)$$

C.7.3 Underflow

All energy underneath the floating breakwater is assumed to be transferred undisturbed to the lee side of the structure. The power of a wave is defined as:

$$\begin{aligned} P_{\text{wave}} &= \int_{-\infty}^z \{ \overline{p_+ u} \} \cdot dz \\ &= \int_{-\infty}^z \{ \rho g \zeta e^{kz} \omega a e^{kz} \} \cdot dz \\ &= \int_{-\infty}^z \{ \rho g \omega \zeta^2 e^{2kz} \} \cdot dz \end{aligned} \quad (C.87)$$

The power of the wave flowing undisturbed underneath of the structure is:

$$\begin{aligned} P_{\text{wave}} &= \int_{-\infty}^{-T_1} \{ \rho g \omega \zeta^2 e^{2kz} \} \cdot dz \\ &= \frac{1}{2} \rho g \omega \zeta^2 e^{-2kT_1} \end{aligned} \quad (C.88)$$

The power of the wave on the lee side of the floating breakwater, occupying the complete depth of the water is defined as:

$$P_{\text{wave}} = \frac{1}{2} \rho g \omega \zeta_{T,\text{under}}^2 \quad (C.89)$$

Equalizing equations (C.88) and (C.89) gives the wave height caused by the energy transmission underneath the structure, or underflow:

$$\zeta_{\text{under},a} = \zeta_i \sqrt{e^{-2kT_1}} \quad (C.90)$$

This transmitted wave has no phase-change relative to the incoming wave, thus the complete solution is:

$$\zeta_{under,a}(t) = \zeta_i \sqrt{e^{-2kT_1}} \cos(\omega t) \quad (C.91)$$

C.7.4 Total transmission

The total wave height on the lee side of the harbour can be obtained by adding equations (C.70), (C.85), (C.86) and (C.91) with respect to their relative phase-changes. The wave height is given by:

$$\zeta_{r,tot} = \zeta_{r,tot,a} \cos(\omega t + \varepsilon_{tot,\zeta}) \quad (C.92)$$

Where:

$$\begin{aligned} \zeta_{r,tot,a} &: \text{Amplitude of the transmitted wave: } \sqrt{x^2 + y^2}; \\ \varepsilon_{tot,\zeta} &: \text{Phase-change of the transmitted wave: } \arctan\left(\frac{y}{x}\right); \\ x &: \sum \zeta_{r,n} \cos(\varepsilon_{n,\zeta}); \\ y &: \sum \zeta_{r,n} \sin(\varepsilon_{n,\zeta}); \\ \zeta_{r,n} &: \text{Radiated wave height for the } n^{\text{th}} \text{ motion.} \\ \varepsilon_{n,\zeta} &: \text{Phase-change of the } n^{\text{th}} \text{ motion relative to the COG.} \end{aligned}$$

C.8 Validation

The ReFBreak2D will be validated with another DELFRAC simulation. With a parameter analysis the optimal dimensions of a FB element will be determined. These dimensions will be used as input for a new DELFRAC simulation in order to obtain the forces and pressures on the element. The panel model will be full scale, so no scaling effects will be present.

When the added mass, the hydrodynamic damping and the motions from the simulation is compared with the outcome of the ReFBreak2D-model, it can be assessed whether the formulas that have been derived in this appendix are correct.

This validation is done in Appendix E.6.

C.9 References

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- Newman, J. N. (1977), *"Marine Hydrodynamics"*, Massachusetts Institute of Technology,
- Vugts, J. H. (1968), *"The hydrodynamic coefficients for swaying, heaving and rolling cylinders in a free surface"*, Report No. 112 S, Netherlands ship research centre TNO, Delft

Appendix D Expansions on ReFBreak2D

APPENDIX D	EXPANSIONS ON REFBREAK2D.....	D-1
D.1	IRREGULAR WAVES.....	D-1
D.1.1	<i>Incoming wave spectrum</i>	D-1
D.1.2	<i>Motion spectrum</i>	D-2
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Appendix D Expansions on ReFBreak2D

With the ReFBreak2D-model the behaviour of one FB-element can be evaluated. Real ocean conditions can not be described with a single wave frequency; the surface can best be described with a spectrum of waves. Therefore an expansion on the ReFBreak2D-model has been developed to determine the performance of a FB element in irregular waves.

There is a possibility that one FB will not meet the requirements imposed on it by the harbour area. When a system of FBs is considered reflection between the two elements becomes of importance. An iterative way is described to determine the influence of reflection on the wave attenuating properties of the system.

D.1 Irregular waves

The original model is not able to determine the performance of the floating breakwater in an irregular wave field. Because the waves are assumed to be linear, the principle of superposition can be used to solve this. The original model was not able to determine the performance of the floating breakwater in an irregular wave field. Because the waves are assumed to be linear, the principle of superposition can be used to solve this. When a wave spectrum is known this can be translated in to a motion spectrum by combining information on the wave spectrum and the behaviour of the structure. This principle is explained in figure D.1.

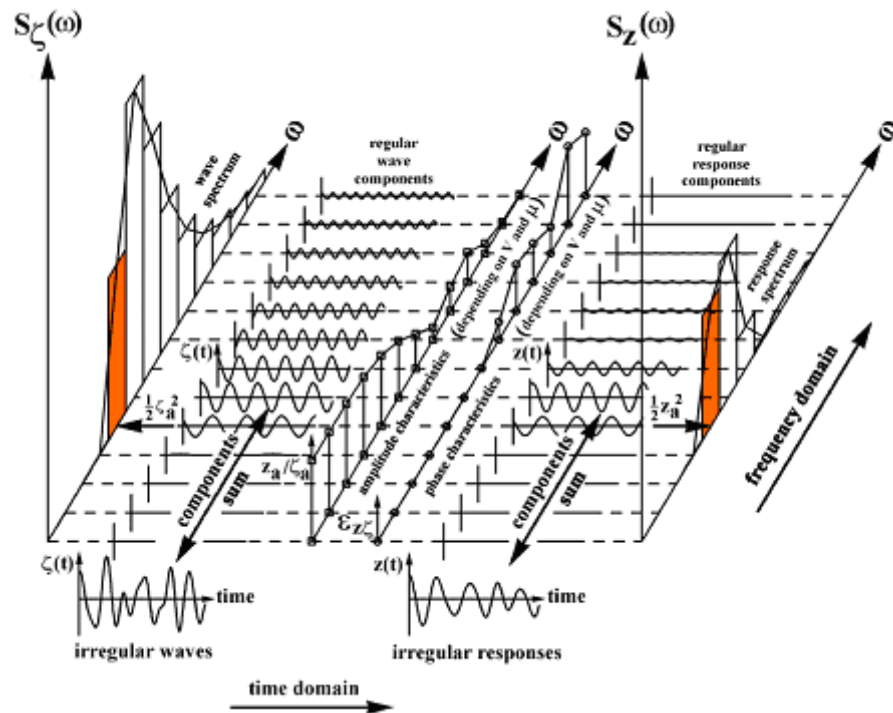


Figure D.1 The transfer of a wave spectrum into a response spectrum [Journée and Massie, 2001]

D.1.1 Incoming wave spectrum

Waves, especially wind generated waves, are very irregular, but can be seen as a superposition of many simple, regular harmonic waves, each with their own wave characteristics.

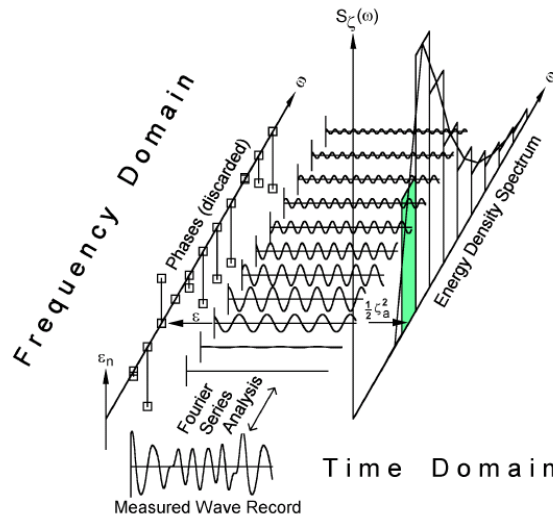


Figure D.2 Wave record analysis and an energy density spectrum [Journée and Massie, 2001]

As every wave component has its own specific energy an energy distribution spectrum can be made. An example of a wave energy spectrum and the way how it is derived can be seen in figure d.2. The wave energy spectrum is defined by [Journée and Massie, 2001]:

$$S_{\zeta}(\omega)d\omega = \frac{1}{2}\zeta_a^2(\omega) \quad (D.1)$$

Where:

ζ_a : Amplitude of the wave, dependent on the wave frequency.

D.1.2 Motion spectrum

Due to the loading by irregular waves the floating breakwater will have an irregular motion, or a motion spectrum. The motion spectrum, when considering only first-order effects, is a multiplication of the incoming wave spectrum and the amplification factor, or RAO for a particular motion.

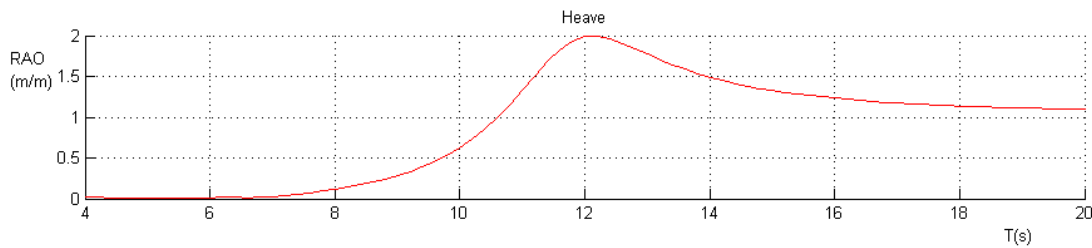


Figure D.3 The RAO for heave ($Y_w = 40$ meter, $T_1 = 20$ meter)

The ReFBreak2D gives as output amplification factors for the three motions that are considered, i.e. the sway, heave and roll motion. In figure D.3 the amplification factor for the heave of a floating breakwater motion is given. Analogous to the wave spectrum, as described in equation (D.1), the motion spectrum can be defined [Journée and Massie, 2001]:

$$\begin{aligned}
S_z(\omega) d\omega &= \frac{1}{2} x_{n,a}^2(\omega) \\
&= \left| \frac{x_{n,a}(\omega)}{\zeta_a} \right|^2 \frac{1}{2} \zeta_a^2(\omega) \\
&= \left| \frac{x_{n,a}(\omega)}{\zeta_a} \right|^2 S_\zeta(\omega) d\omega
\end{aligned} \tag{D.2}$$

Where:

$x_{n,a}(\omega)$: Motion amplitude, dependent on the wave frequency;
 S_ζ : The wave spectrum.

Thus, the heave response spectrum of a motion can be found by using the transfer function, or RAO, of the motion and the wave spectrum by:

$$S_z(\omega) = \left| \frac{x_{n,a}(\omega)}{\zeta_a} \right|^2 S_\zeta(\omega) \tag{D.3}$$

Where:

$\frac{x_{n,a}(\omega)}{\zeta_a}$: The transfer function of the motion;
 S_ζ : The wave spectrum.

From the amplification factor for heave, as shown in figure D.3, and a Bretschneider wave spectrum with a spectral peak period of 12 seconds a motion spectrum can be found now. This motion spectrum is shown in figure D.4.

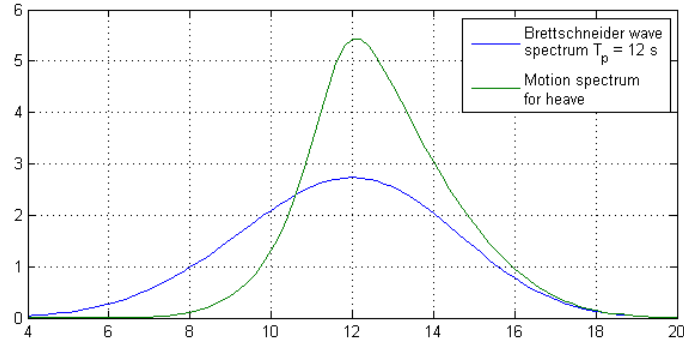


Figure D.4 Motion spectrum for heave ($Y_w = 40$ meter, $T_1 = 20$ meter, $H_{1/3} = 4$ meter)

D.1.3 Transmitted wave spectrum

Similar to the motion spectrum a transmitted wave spectrum can be obtained. The ReFBreak2D model gives a transfer function for the transmitted wave height towards the lee side of the floating breakwater. An example of such a transfer function is given in figure D.5.

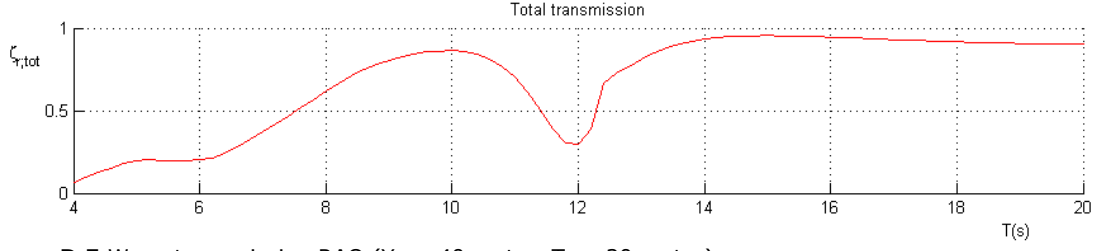


Figure D.5 Wave transmission RAO ($Y_w = 40$ meter, $T_1 = 20$ meter)

Analogous to the incoming wave spectrum and the motion wave spectrum, the transferred wave spectrum can be defined as well [Journée and Massie, 2001]:

$$\begin{aligned}
 S_{\zeta,T}(\omega) d\omega &= \frac{1}{2} \zeta_{T,a}^2(\omega) \\
 &= \left| \frac{\zeta_{T,a}}{\zeta_a}(\omega) \right|^2 \frac{1}{2} \zeta_a^2(\omega) \\
 &= \left| \frac{\zeta_{T,a}}{\zeta_a}(\omega) \right|^2 S_{\zeta}(\omega) d\omega
 \end{aligned} \tag{D.4}$$

Where:

$\zeta_{T,a}$: The amplitude of the transmitted wave;
 S_{ζ} : The wave spectrum.

Thus, the heave response spectrum of a motion can be found by using the transfer function, or RAO, of the motion and the wave spectrum by:

$$S_z(\omega) = \left| \frac{\zeta_{T,a}}{\zeta_a}(\omega) \right|^2 S_{\zeta}(\omega) \tag{D.5}$$

Where:

$\frac{\zeta_{T,a}}{\zeta_a}(\omega)$: The transfer function of the transmitted wave height;
 S_{ζ} : The wave spectrum.

From the amplification factor for the wave transmission, as shown in figure D.5, and a Bretschneider wave spectrum with a spectral peak period of 12 seconds a motion spectrum can be now found. This motion spectrum is shown in figure D.6.

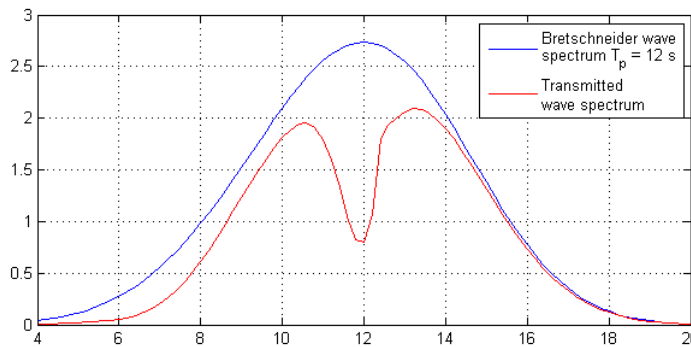


Figure D.6 Transmitted wave spectrum ($Y_w = 40$ meter, $T_1 = 20$ meter, $H_{1/3} = 4$ meter)

D.1.4 Conclusion

From figure D.6 it can be concluded that an effective floating breakwater has to fulfil the following requirements:

- The most effective frequency of the floating breakwater has to be tuned to coincide with spectral peak period;
- A floating breakwater should have a wide area of high efficiency, reducing the transmitted waves at other frequencies than spectral peak frequency.

D.2 Multiple breakwaters

When a single FB is unable to attenuate waves in the whole frequency range of the incoming wave spectrum a system of floating breakwaters can be considered. In a system not only the wave transmission will be important but the wave reflection as well. The first FB will react to the incoming waves but also on the waves reflected and generated by the other FB as well.

Here a simple hypothesis behind transmission and reflection will be discussed. It is assumed that the wave transmission calculated with this hypothesis is an upper limit, as no positive influence of the reflective waves is taken into account. In future research this hypothesis should be verified with a DELFRAC simulation, or a simulation with another model, where the hydrodynamic coupling of the two floating bodies is taken into account.

D.2.1 Process

When the incoming waves hit the first floating breakwater a part of the wave will be transmitted towards the lee side of the element and a part will be reflected back to the seaside. The processes that govern the transmission and the reflection are given in table D-1.

Table D-1 Processes governing the reflection and transmission of the waves

Reflection	Transmission
Waves generated by the sway/roll motion	Waves generated by the sway/roll motion
Waves generated by the heave motion	Waves generated by the heave motion
Reflected wave against hull	Underflow

The part of the wave that is transmitted towards the lee-side is the part of the wave that has to be attenuated by the second floating breakwater. The same processes as with the first FB will take place, but the waves will not be reflected towards the ocean but towards the first FB. This first FB will react on the waves that are reflected by the second floating breakwater.

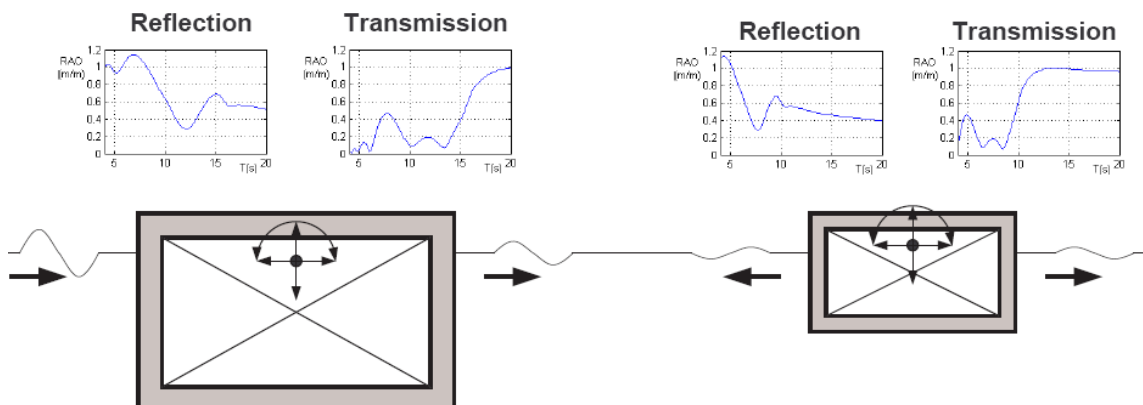


Figure D.7 Wave transmission and reflection, an iterative process

In its turn the first FB will transmit a part of the reflected waves towards the ocean, this part of wave energy leaves the system. Another part however is reflected back towards the second FB again, which will react again and so on. This iterative effect is shown in figure D.7. The amount of energy that is contained in one wave which is reflected back and forth is reduced, so a limited number of iterations are required to obtain the total transmitted wave towards the harbour area, see figure D.8.

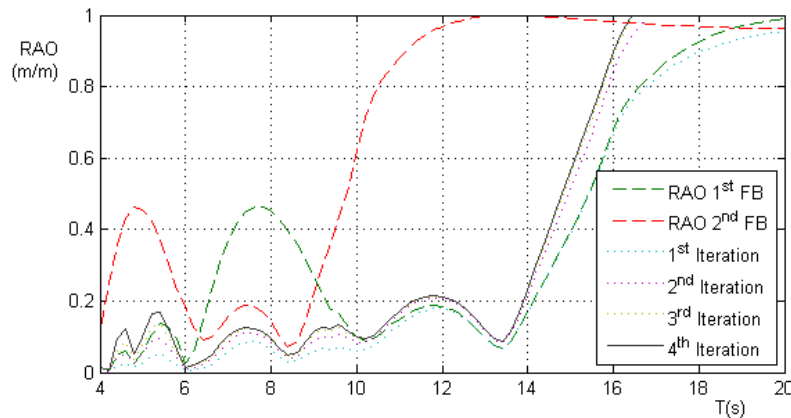


Figure D.8 An example of the wave transmission of two FBs with the converging iterations

The distance between the floating breakwaters is of importance, as the reflected wave can have a positive or negative effect on the motions and thus on the waves that are being produced. The reflected wave of the second breakwater will cause the first breakwater to move and generate waves. These motions and generated waves have to be added, on the basis of superposition, to the motions and generated waves due to the incoming waves from the ocean.

The addition of the two generated waves, i.e. the generated wave due to the incoming wave and due to the reflected wave, can lead to a lower transmitted wave height towards the harbour area when they are in anti-phase. When the two generated waves are in phase with each other the transmitted wave is amplified instead of damped.

The wave lengths where the positive effect on the wave attenuating performance of the FB system is present differ with each wave period. Therefore it can not be relied on in a wave spectrum, consisting of several wave periods. Also due to low frequency motions, caused by the drift forces, it is virtually impossible to maintain the desired distance between the two FB elements.

Therefore it is assumed that reflection causes only a negative effect on the wave-attenuating performance. The transmission coefficients obtained in this way are assumed to be maximum coefficients; depending on the wave period and actual distance between the two FBs a lower transmission coefficient may be obtained.

D.3 References

Journée, J. M. J. and Massie, W. W. (2001), *"Offshore Hydromechanics, First Edition"*, Delft University of Technology, Delft

Appendix E Parameter research

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Appendix E Parameter research

There are basically four parameters that can be varied:

- The location of the centre of gravity relative to the centre of buoyancy;
- The beam of the structure;
- The draught of the structure;
- The viscous roll damping.

The mass of the structure cannot be changed because it is inherent to the volume that is displaced by the floating breakwater, which in its turn depends on the dimensions, i.e. the beam and draught.

The influence of the location of the centre of gravity has been assessed with structures S1, S2, S3 and S4, where the location was varied from 10 meters below the centre of buoyancy, to 5 meters above. Higher than 5 meters is not possible, the floating breakwater with this Y_w/T_1 -ratio would not be stable.

The effect of the beam and the draught is studied in three ways:

- Constant draught, variable beam; structures S5, S6, S7 and S8;
- Variable draught, constant beam; structures S9, S10, S11 and S12;
- Constant mass, variable Y_w/T_1 -ratio; structures S13, S14, S15 and S16.

For structures S5 to S16 all calculations are done twice, once with a low centre of gravity and once with a high centre of gravity. This is denoted in table E-1 with the letters L/H in combination with the structure number.

The effect of turbulence, or viscous roll damping, has been calculated for structures S17, S18, S19 and S20. The value of this damping ranged from 0% to 15% of the critical damping. A practical and realistic value is 5%.

All structures upon which calculations were performed are listed in table E-1 below.

Table E-1 Structures as they were calculated

Structure number	Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
	L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
	[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S1	250	40	20	2,0	205.000	40.180	0,0	-10
S2	250	40	20	2,0	205.000	40.180	0,0	-5
S3	250	40	20	2,0	205.000	40.180	0,0	0
S4	250	40	20	2,0	205.000	40.180	0,0	5
S5 L/H	250	10	10	1,0	25.625	314	0,0	-2,5 / 2,5
S6 L/H	250	20	10	2,0	51.250	2.511	0,0	-2,5 / 2,5
S7 L/H	250	40	10	4,0	102.500	20.090	0,0	-2,5 / 2,5
S8 L/H	250	80	10	8,0	205.000	160.720	0,0	-2,5 / 2,5
S9 L/H	250	40	5	8,0	51.250	10.045	0,0	-1,3 / 1,3
S10 L/H	250	40	10	4,0	102.500	20.090	0,0	-2,5 / 2,5
S11 L/H	250	40	20	2,0	205.000	40.180	0,0	-5,0 / 5,0
S12 L/H	250	40	40	1,0	410.000	80.360	0,0	-10,0 / 10,0
S13 L/H	250	28,3	28,3	1,0	205.000	20.090	0,0	-7,1 / 7,1
S14 L/H	250	40	20	2,0	205.000	40.180	0,0	-5,0 / 5,0
S15 L/H	250	56,6	14,1	4,0	205.000	80.360	0,0	-3,5 / 3,5
S16 L/H	250	80	10	8,0	205.000	160.720	0,0	-2,5 / 2,5
S17	250	40	20	2,0	205.000	40.180	0,0	0
S18	250	40	20	2,0	205.000	40.180	5,0	0
S19	250	40	20	2,0	205.000	40.180	10,0	0
S20	250	40	20	2,0	205.000	40.180	15,0	0

E.1 Reading guide

In figure E.1 and figure E.2 the layout of the parameter analysis is given as an example of how the model output can be used to assess the wave attenuating performance of the design or designs considered.

Table E-3 Structural dimension for structures S5L, S6L, S7L and S8L

Structure number	Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
	L	B	T	L/T	M	I	γ	z_G
	[m]	[m]	[m]	[-]	[10^6 kg]	[10^8 kg m ²]	[%]	[m]
S5 L	250	10	10	1,0	25.625	314	0,0	-2,5
S6 L	250	20	10	2,0	51.250	2.511	0,0	-2,5
S7 L	250	40	10	4,0	102.500	20.090	0,0	-2,5
S8 L	250	90	10	8,0	205.000	160.720	0,0	-2,5

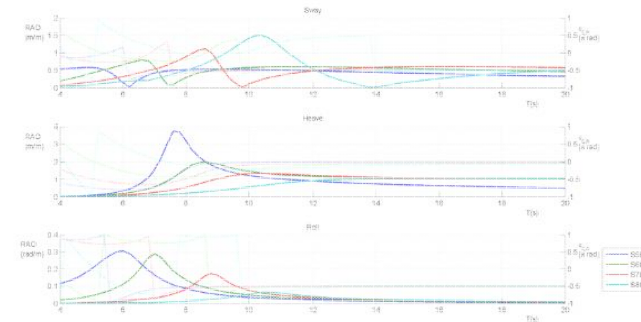


Figure E.5 The amplitude factors for sway, heave and roll for structures S5L, S6L, S7L and S8L

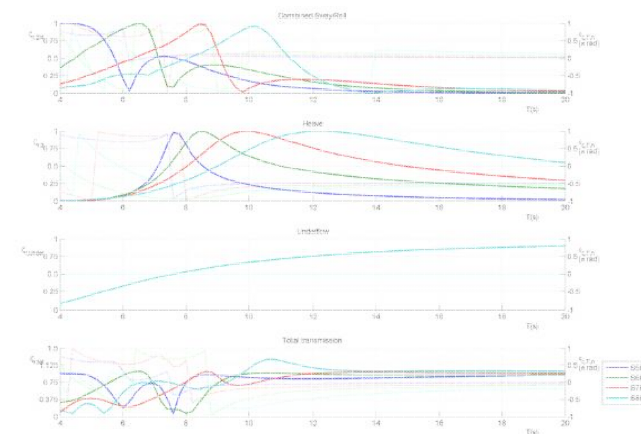


Figure E.6 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Figure E.1 Layout of the parameter analysis, page 1

The model output that is presented on the first page is:

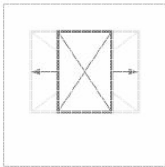
1. Recapitulation of Table E.1.
2. Motion amplification factor (RAO) of the modes:
 - a. Sway
 - b. Heave
 - c. Roll

The amplitude is given as a solid line; the phase change relative to the incoming wave height is given as a dashed line.

3. Transmitted wave RAO due to the modes:
 - a. Sway/Roll
 - b. Heave
 - c. Underflow
 - d. Total transmitted wave, a summation of the contributions (a-c) with respect of their relative phase-change.

The amplitude is again given as a solid line; the phase change relative to the incoming wave height is given as a dashed line.

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	10
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	[10^4 kg]	Variable
Viscous roll damping ratio	ζ	[%]	0.0
Centre of gravity	O/G	[m]	-0.25 T_1



1. Recapitulation of the design variables being considered

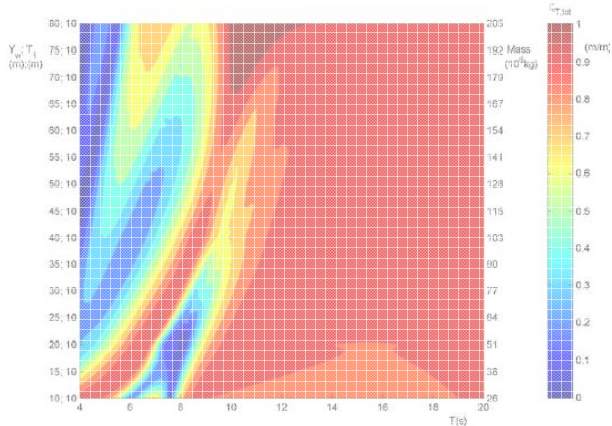


Figure E.7 RAO for a structure with a fixed draught of 10 meters and a variable beam

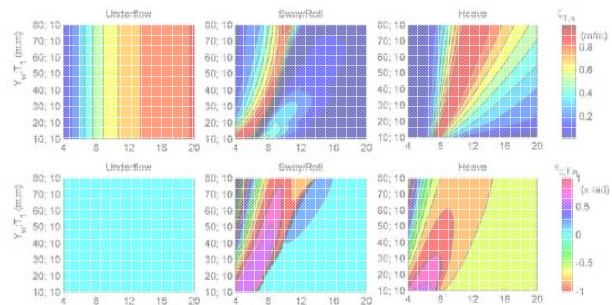


Figure E.8 Components of the transmitted wave RAO for the structure

Figure E.2 Layout of the parameter analysis, page 2

The model output that is presented on the second page is:

1. Recapitulation of the design variables being considered.
2. Wave amplification factor (RAO) of the transmitted wave towards the harbour area. A RAO of 0.1 means a wave transmission of 10%, i.e. a 1 metre high wave is attenuated to 0.1 metre. A lower RAO leads to a more effective design at that wave frequency.
This figure is a summation of the three RAOs, presented in (3), with respect to their phase-changes, presented in (4). See also Appendix C.7.4.
3. Transmitted wave RAO due to the modes:
 - e. Underflow
 - f. Sway/Roll
 - g. Heave
4. Phase change relative to the incoming waves for the considered modes

E.2 Influence of the centre of gravity

Table E-2 Structural dimension for structures S17, S18, S19 and S20

Structure number	Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
	L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
	[m]	[m]	[m]	[-]	$[10^3 \text{ kg}]$	$[10^6 \text{ kg m}^2]$	[%]	[m]
S1	250	40	20	2,0	205.000	40.180	0,0	-10
S2	250	40	20	2,0	205.000	40.180	0,0	-5
S3	250	40	20	2,0	205.000	40.180	0,0	0
S4	250	40	20	2,0	205.000	40.180	0,0	5

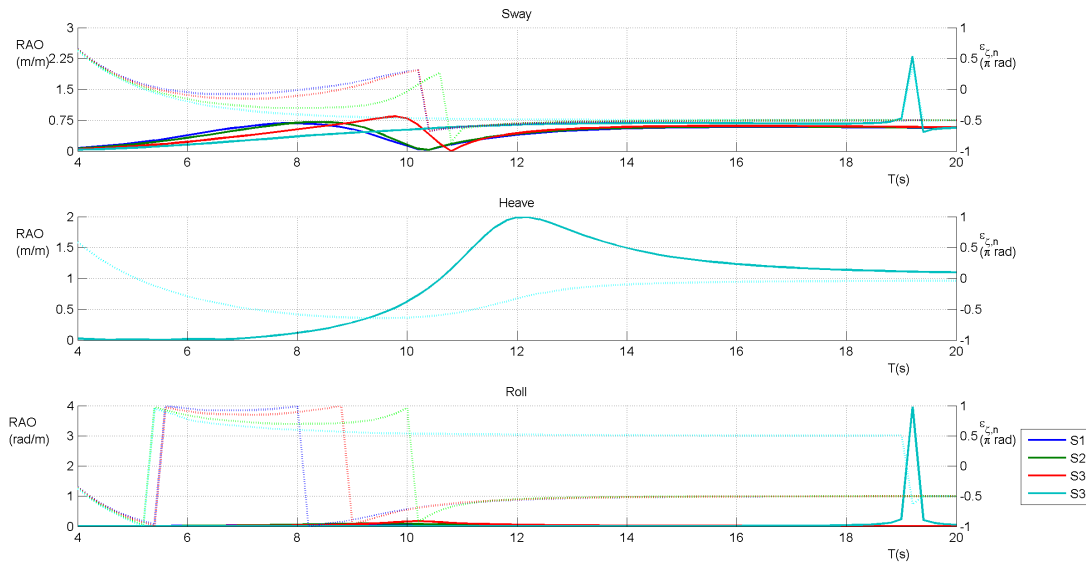


Figure E.3 The amplitude factors for sway, heave and roll for structures S1, S12, S3, S4

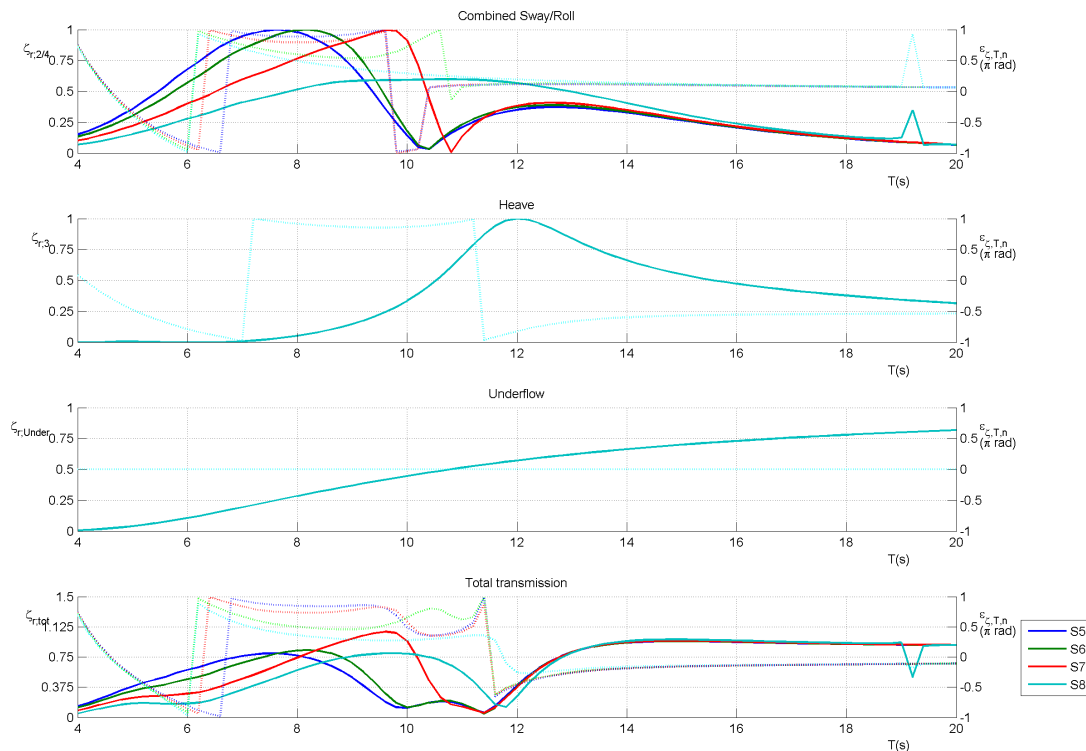


Figure E.4 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	40
Draught	T_1	[m]	20
Width to draught ratio	Y_w/T_1	[-]	2,0
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	BG	[m]	Variable

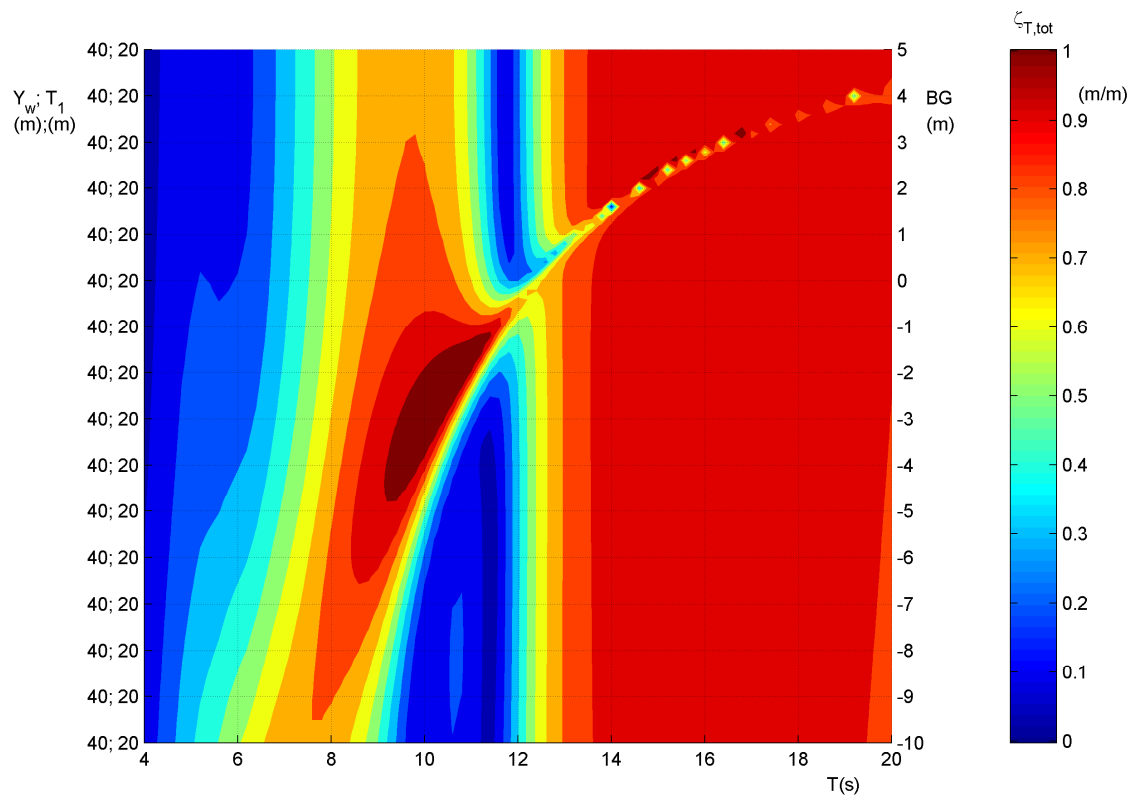
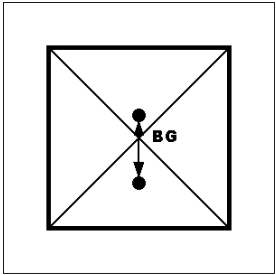


Figure E.5 Transmitted wave RAO for a structure with a fixed breadth and draught, $\overline{BG}$ is variable

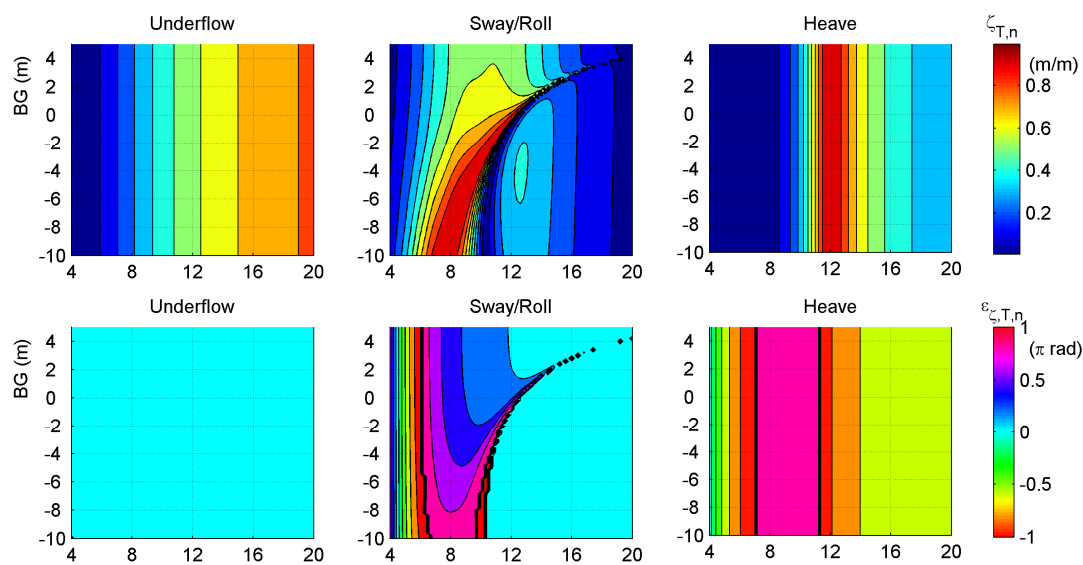


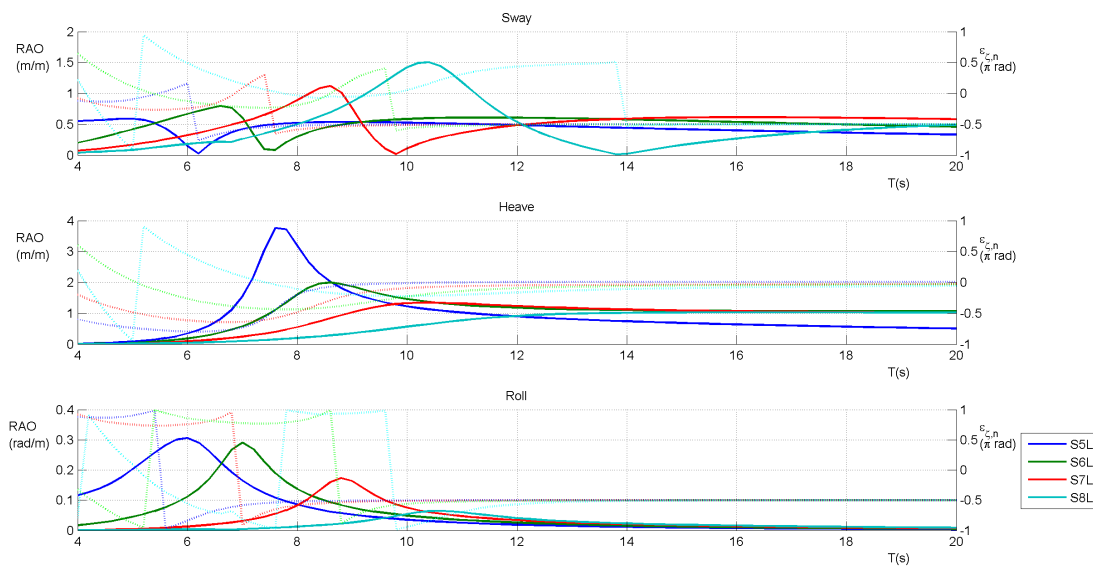
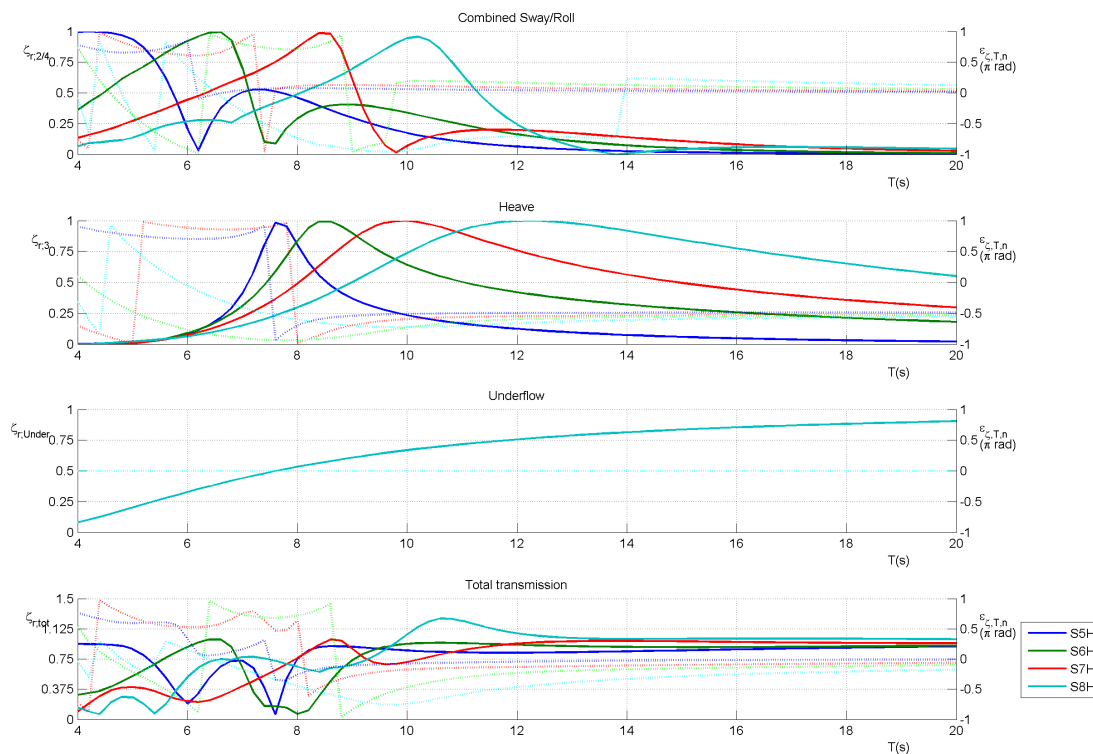
Figure E.6 Components of the transmitted wave RAO for the structure

E.3 Influence of the breadth to draught ratio

E.3.1 Constant draught, low centre of gravity

Table E-3 Structural dimension for structures S5L, S6L, S7L and S8L

Structure number		Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
		L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
		[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S5	L	250	10	10	1,0	25.625	314	0,0	-2,5
S6	L	250	20	10	2,0	51.250	2.511	0,0	-2,5
S7	L	250	40	10	4,0	102.500	20.090	0,0	-2,5
S8	L	250	80	10	8,0	205.000	160.720	0,0	-2,5


Figure E.7 The amplitude factors for sway, heave and roll for structures S5L, S6L, S7L and S8L

Figure E.8 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	10
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	Variable
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	-0,25 T_1

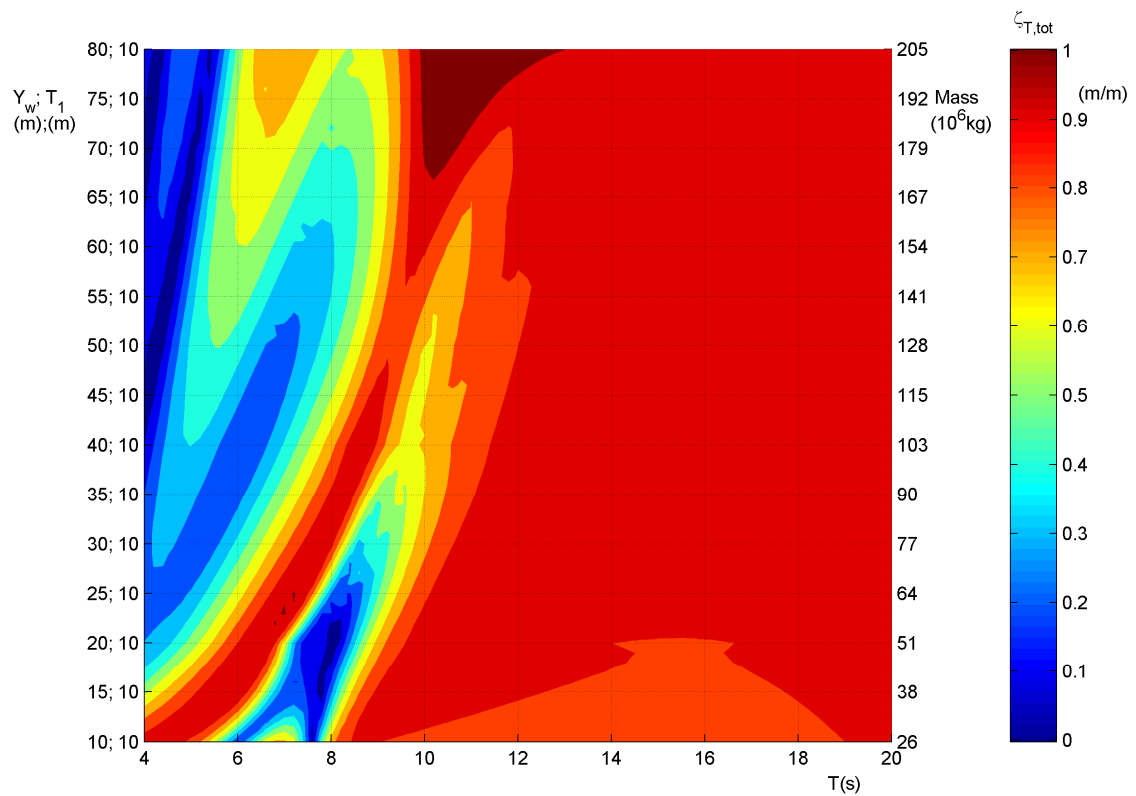
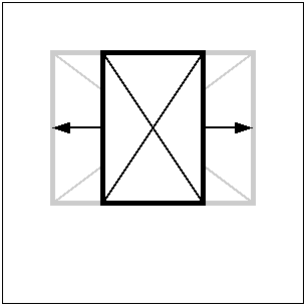


Figure E.9 RAO for a structure with a fixed draught of 10 meters and a variable beam

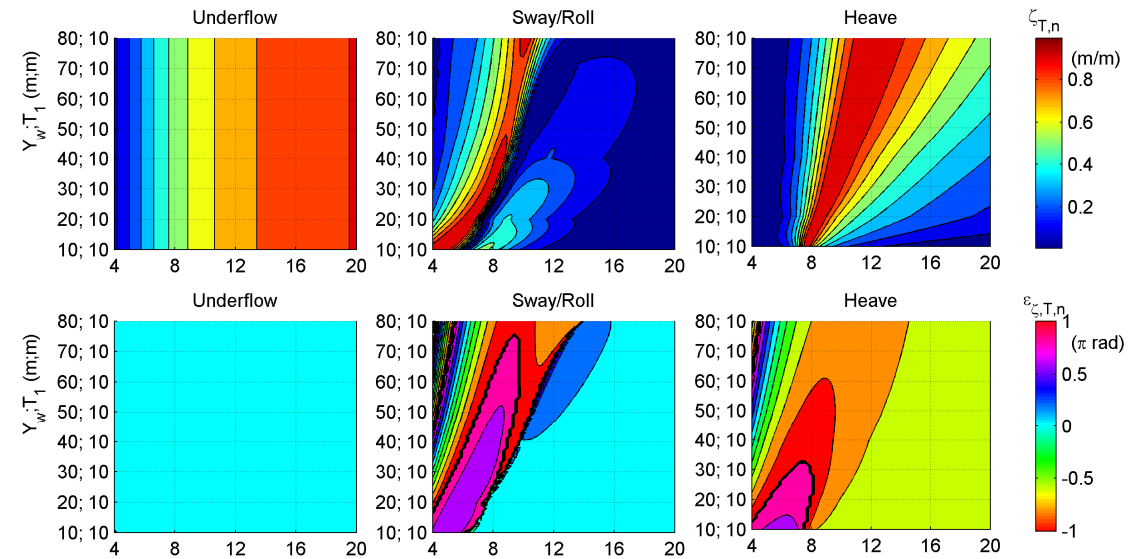


Figure E.10 Components of the transmitted wave RAO for the structure

E.3.2 Constant draught, high centre of gravity

Table E-4 Structural dimension for structures S5H, S6H, S7H and S8H

Structure number		Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
		L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
		[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S5	H	250	10	10	1,0	25.625	314	0,0	2,5
S6	H	250	20	10	2,0	51.250	2.511	0,0	2,5
S7	H	250	40	10	4,0	102.500	20.090	0,0	2,5
S8	H	250	80	10	8,0	205.000	160.720	0,0	2,5

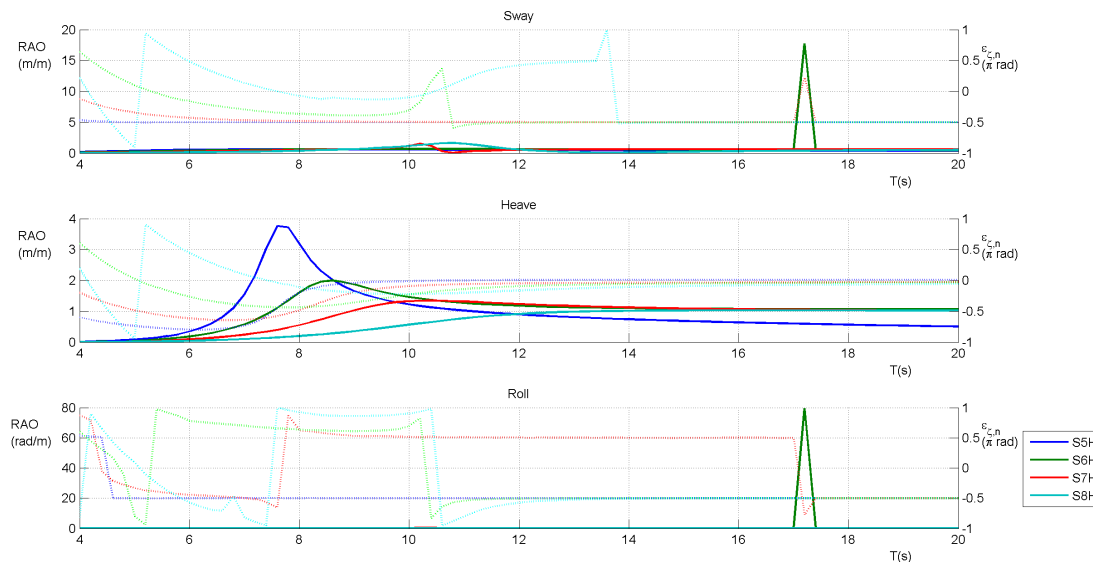


Figure E.11 The amplitude factors for sway, heave and roll for structures S5H, S6H, S7H and S8H

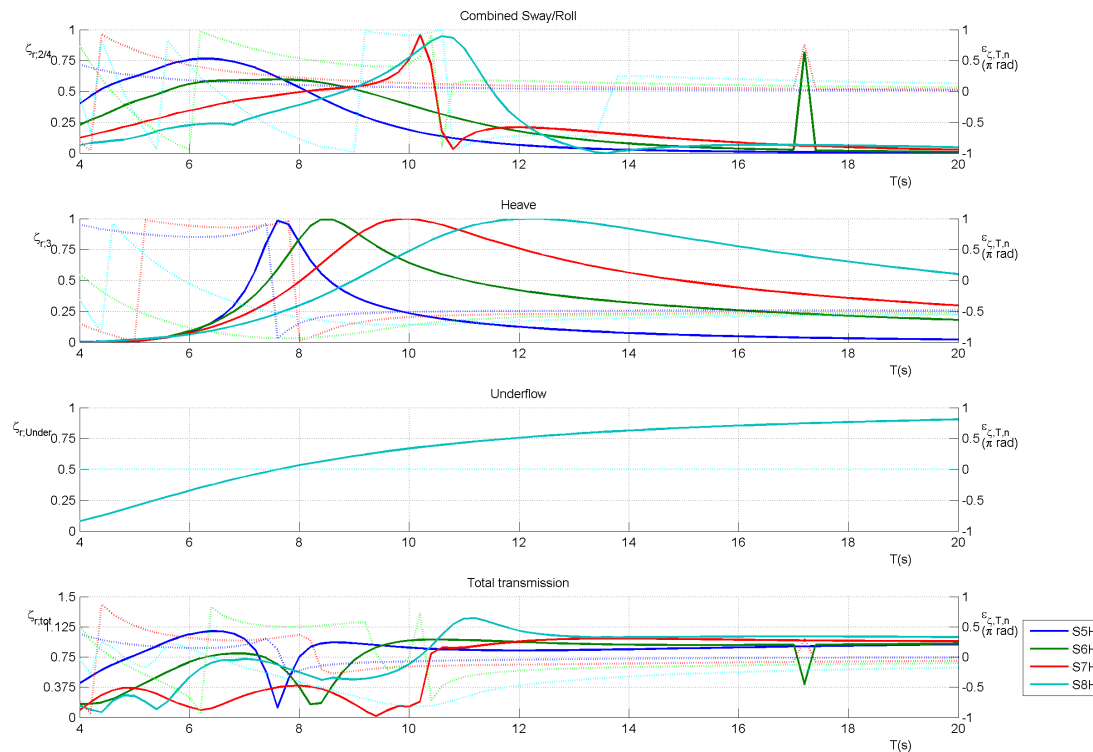


Figure E.12 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	10
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	Variable
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	+0,25 T_1

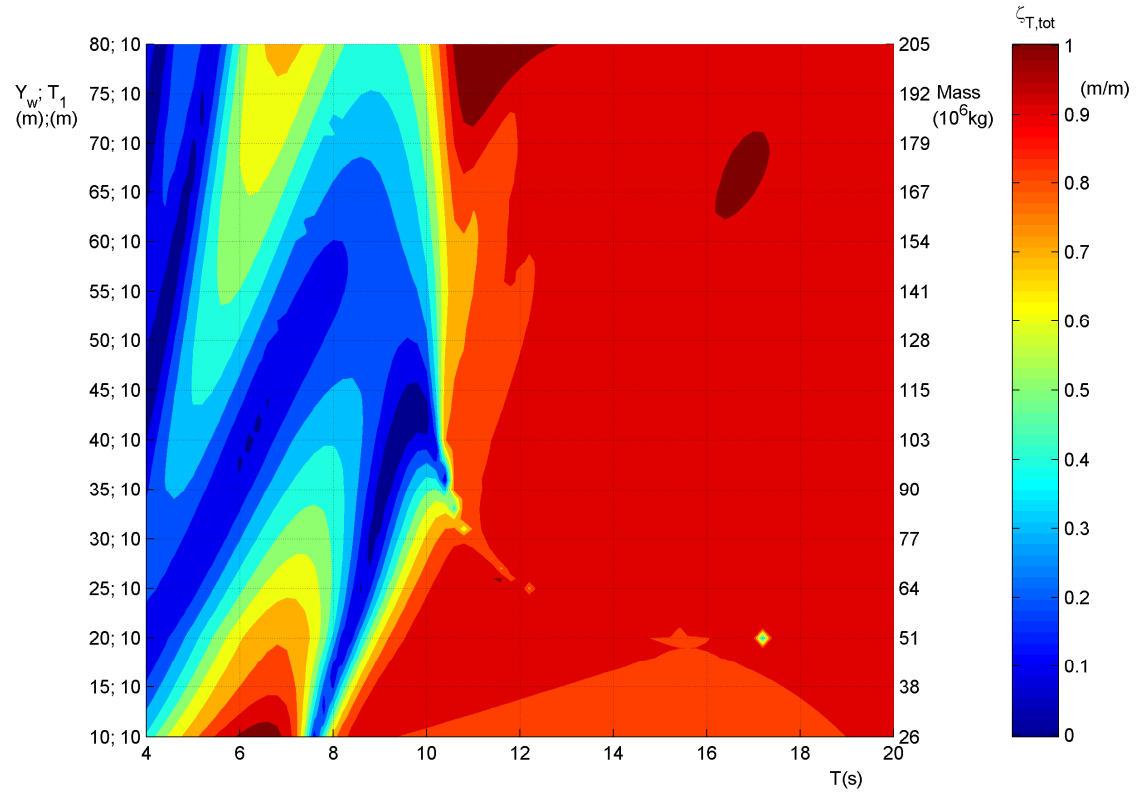
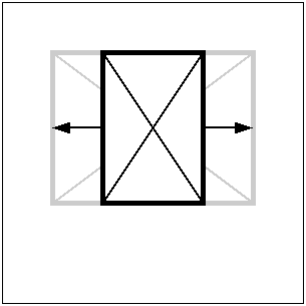


Figure E.13 RAO for a structure with a fixed draught of 10 meters and a variable beam

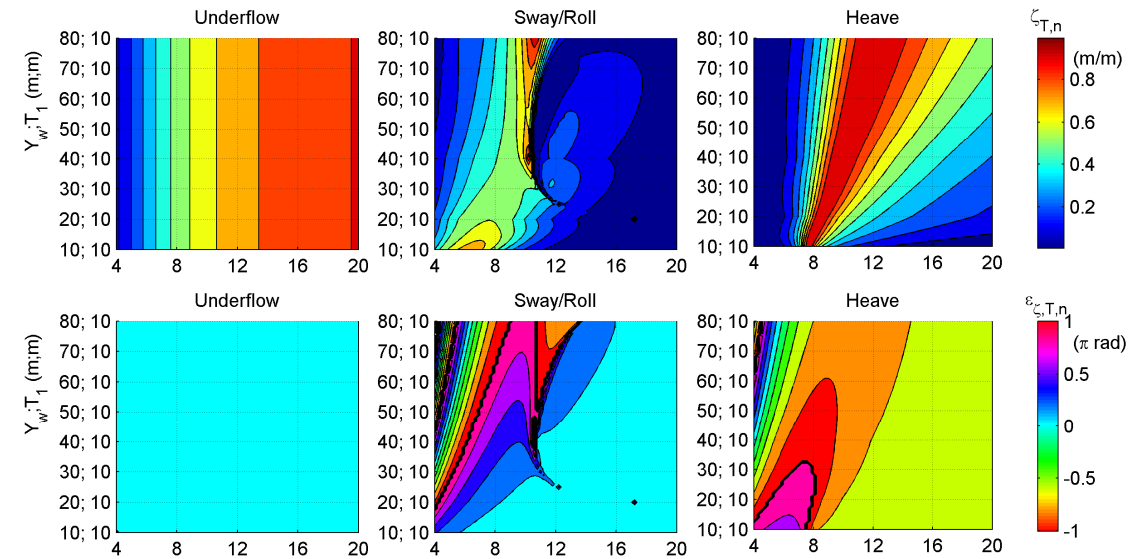


Figure E.14 Components of the transmitted wave RAO for the structure

E.3.3 Constant beam, low centre of gravity

Table E-5 Structural dimension for structures S9L, S10L, S11L and S12L

Structure number	Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
	L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
	[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S9 L	250	40	5	8,0	51.250	10.045	0,0	1,25
S10 L	250	40	10	4,0	102.500	20.090	0,0	2,5
S11 L	250	40	20	2,0	205.000	40.180	0,0	5
S12 L	250	40	40	1,0	410.000	80.360	0,0	10

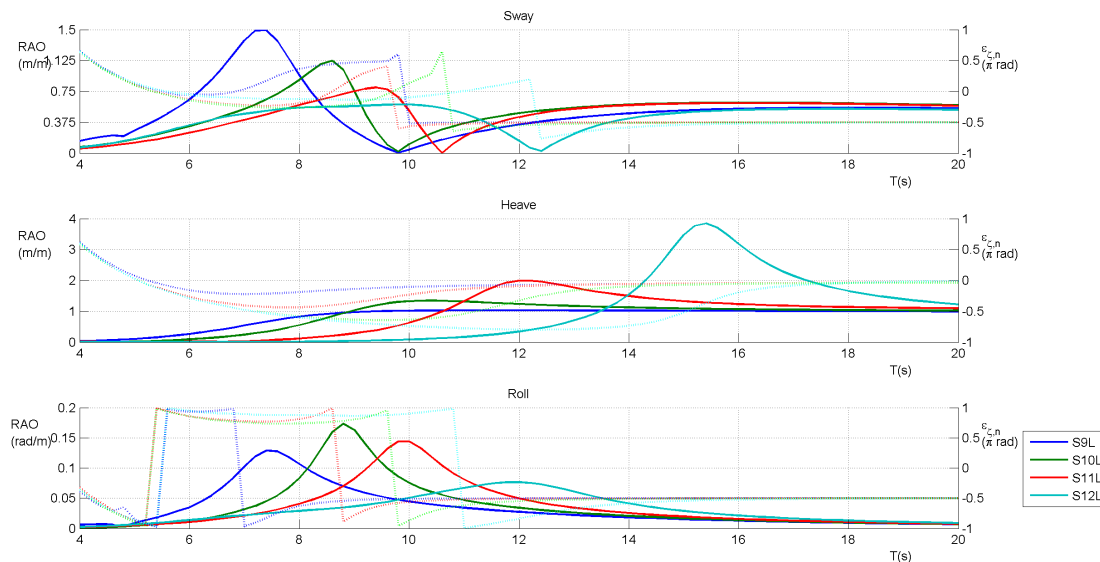


Figure E.15 The amplitude factors for sway, heave and roll for structures S9L, S10L, S11L and S12L

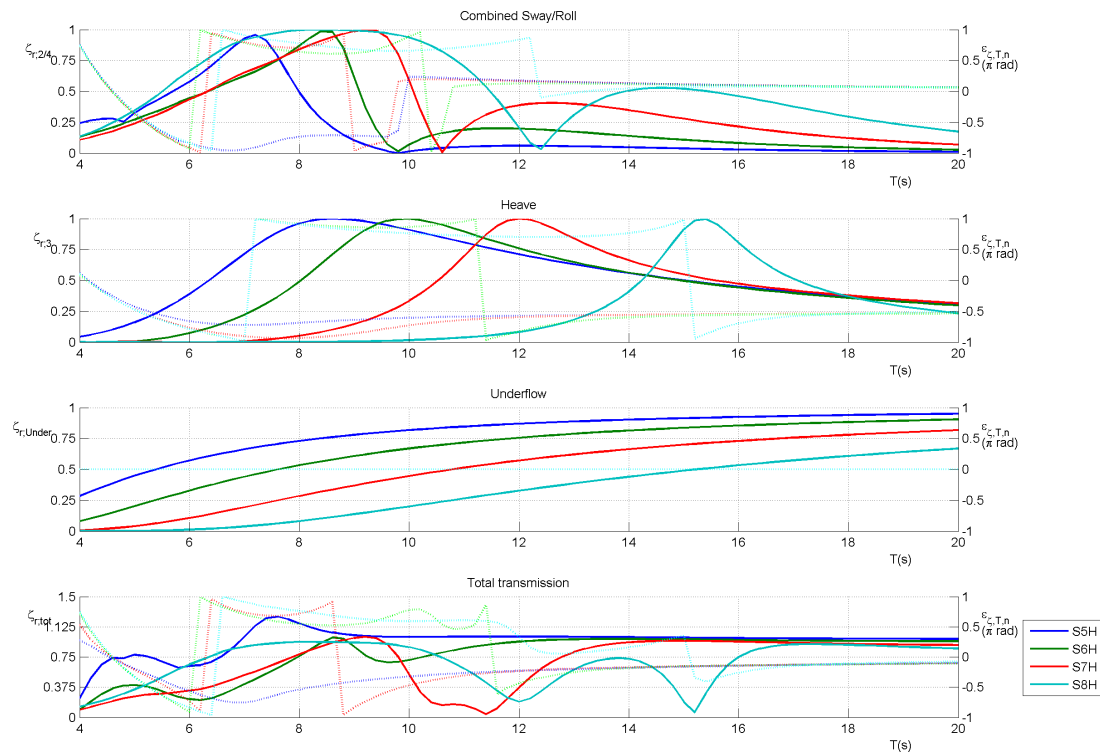


Figure E.16 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	40
Draught	T_1	[m]	Variable
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	Variable
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	-0,25 T_1

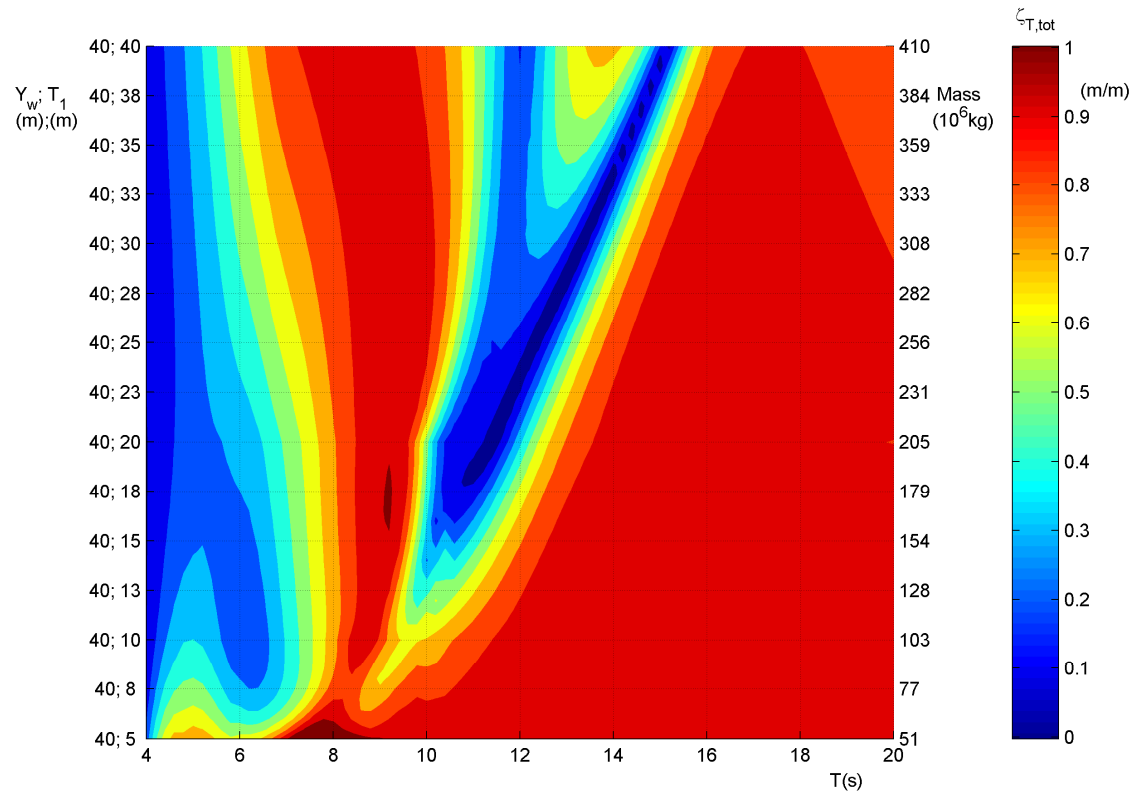
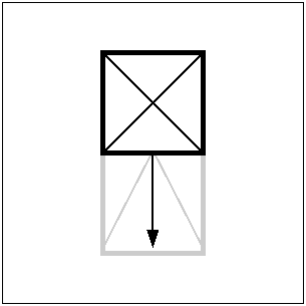


Figure E.17 RAO for a structure with a fixed beam of 40 meters and a variable draught

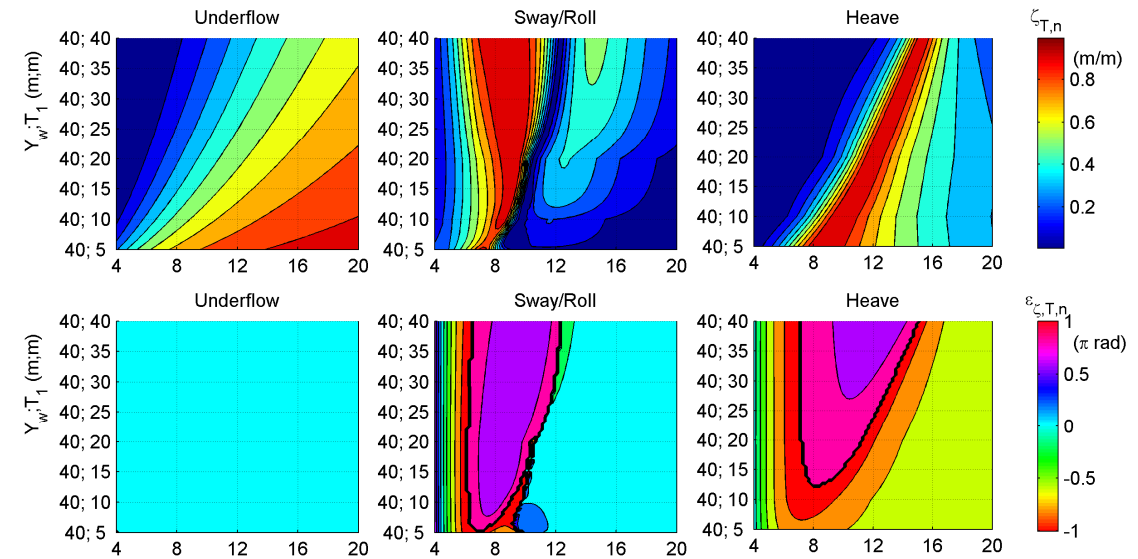


Figure E.18 Components of the transmitted wave RAO for the structure

E.3.4 Constant beam, high centre of gravity

Table E-6 Structural dimension for structures S9H, S10H, S11H and S12H

Structure number		Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
		L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
		[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S9	H	250	40	5	8,0	51.250	10.045	0,0	1,25
S10	H	250	40	10	4,0	102.500	20.090	0,0	2,5
S11	H	250	40	20	2,0	205.000	40.180	0,0	5
S12	H	250	40	40	1,0	410.000	80.360	0,0	10

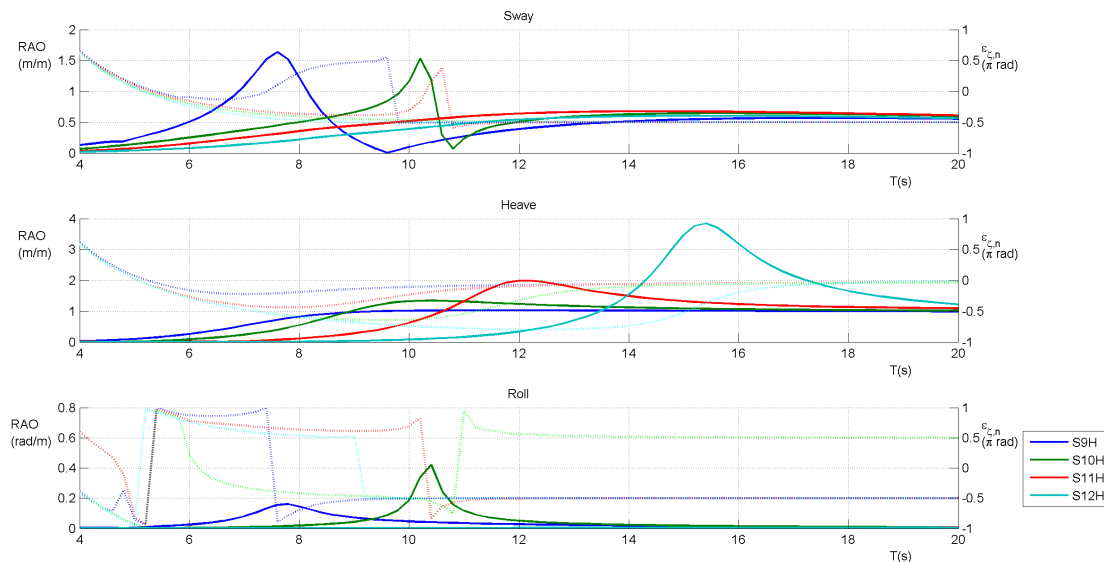


Figure E.19 The amplitude factors for sway, heave and roll for structures S9H, S10H, S11H and S12H

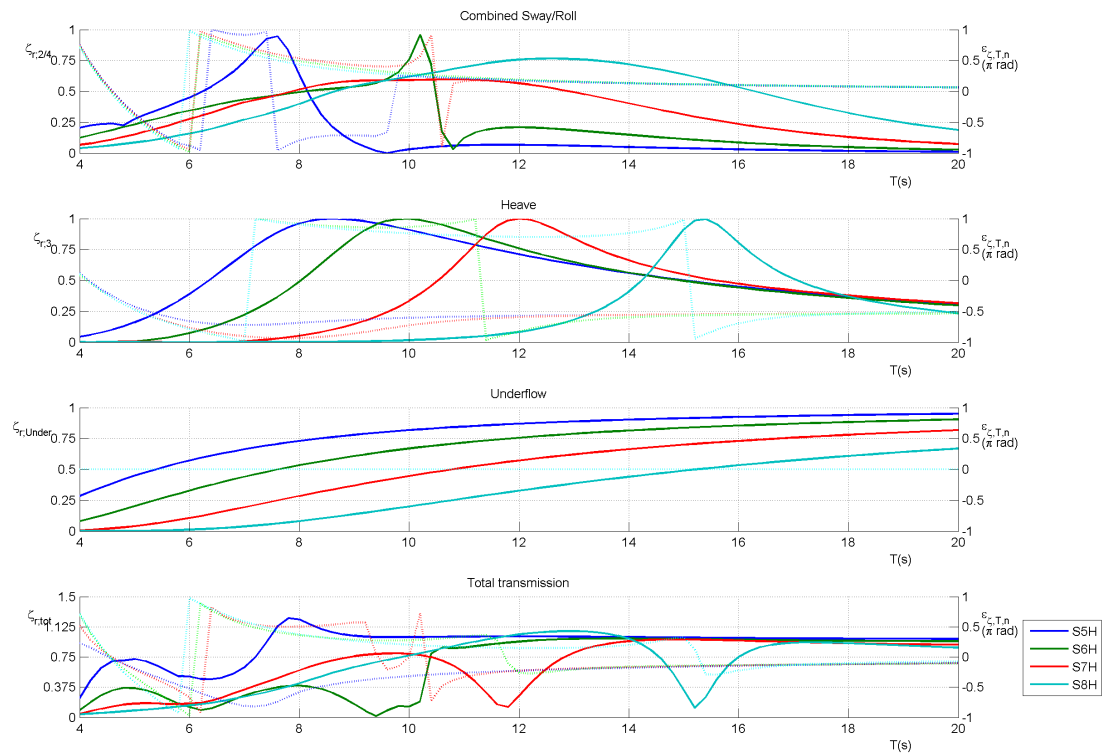


Figure E.20 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	40
Draught	T_1	[m]	Variable
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	Variable
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	+0,25 T_1

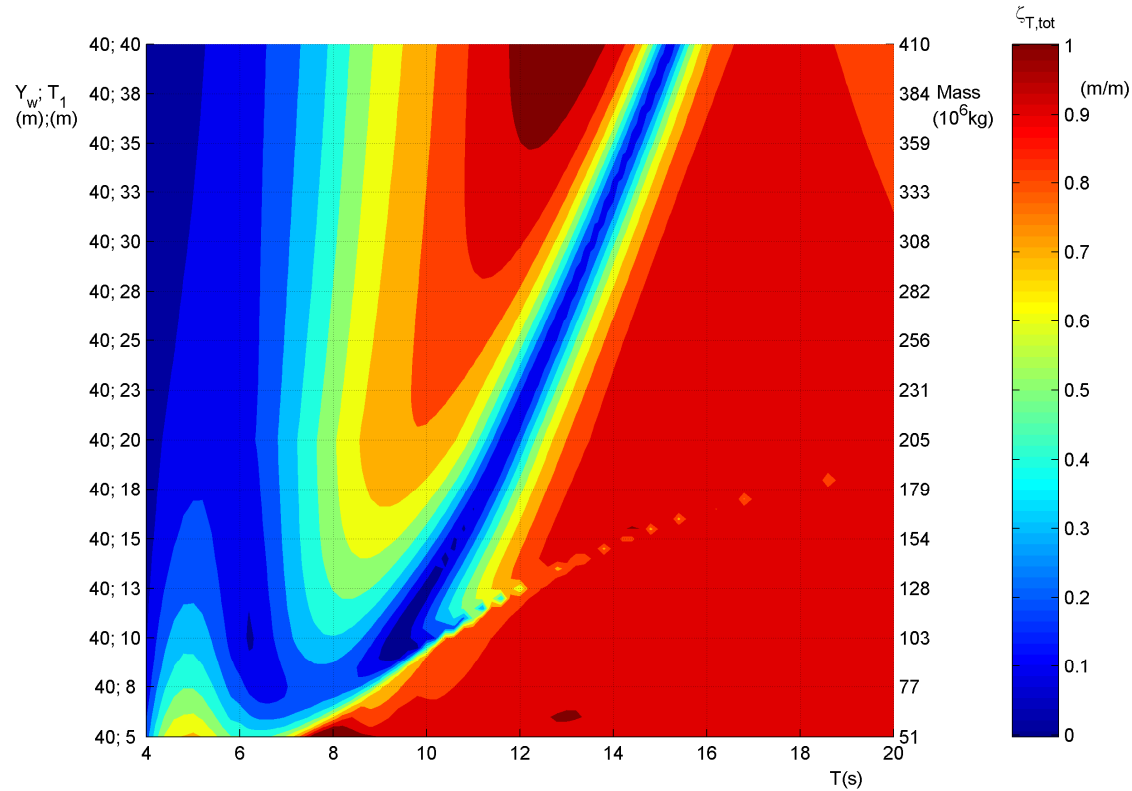
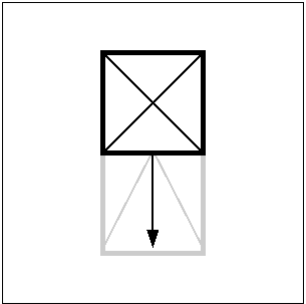


Figure E.21 RAO for a structure with a fixed beam of 40 meters and a variable draught

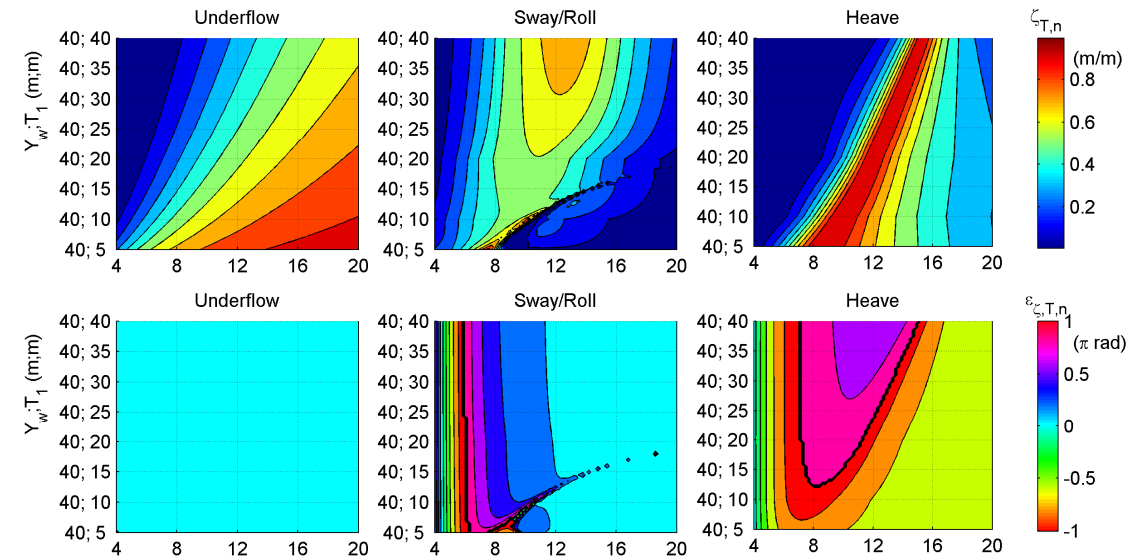


Figure E.22 Components of the transmitted wave RAO for the structure

E.3.5 Constant mass, low centre of gravity

Table E-7 Structural dimension for structures S13L, S14L, S15L and S16L

Structure number		Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
		L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
		[m]	[m]	[m]	[-]	$[10^3 \text{ kg}]$	$[10^6 \text{ kg m}^2]$	[%]	[m]
S13	L	250	28,3	28,3	1,0	205.000	20.090	0,0	7,1
S14	L	250	40	20	2,0	205.000	40.180	0,0	5,0
S15	L	250	56,6	14,1	4,0	205.000	80.360	0,0	3,5
S16	L	250	80	10	8,0	205.000	160.720	0,0	2,5

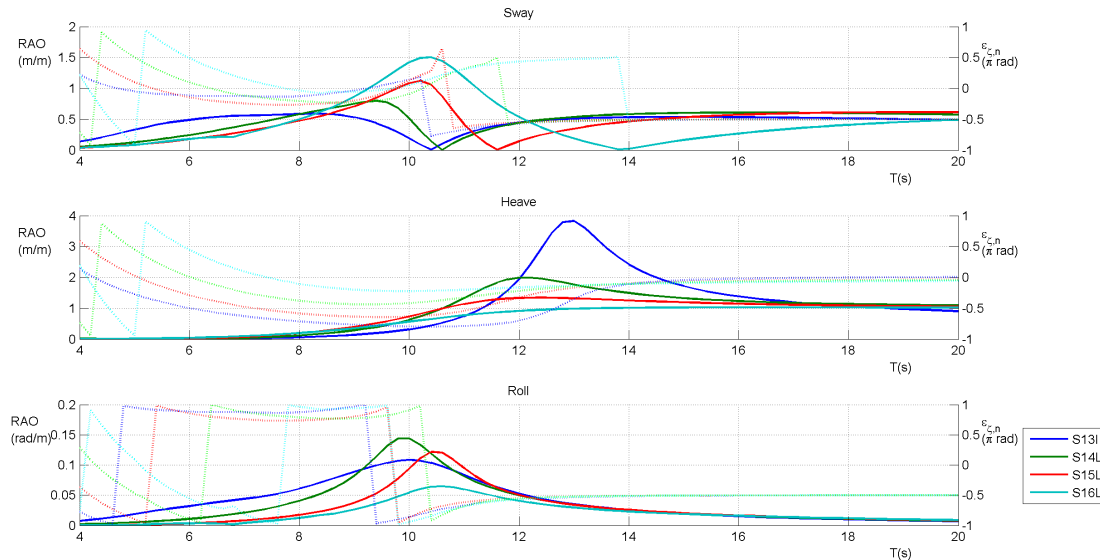


Figure E.23 The amplitude factors for sway, heave and roll for structures S13L, S14L, S15L and S16L

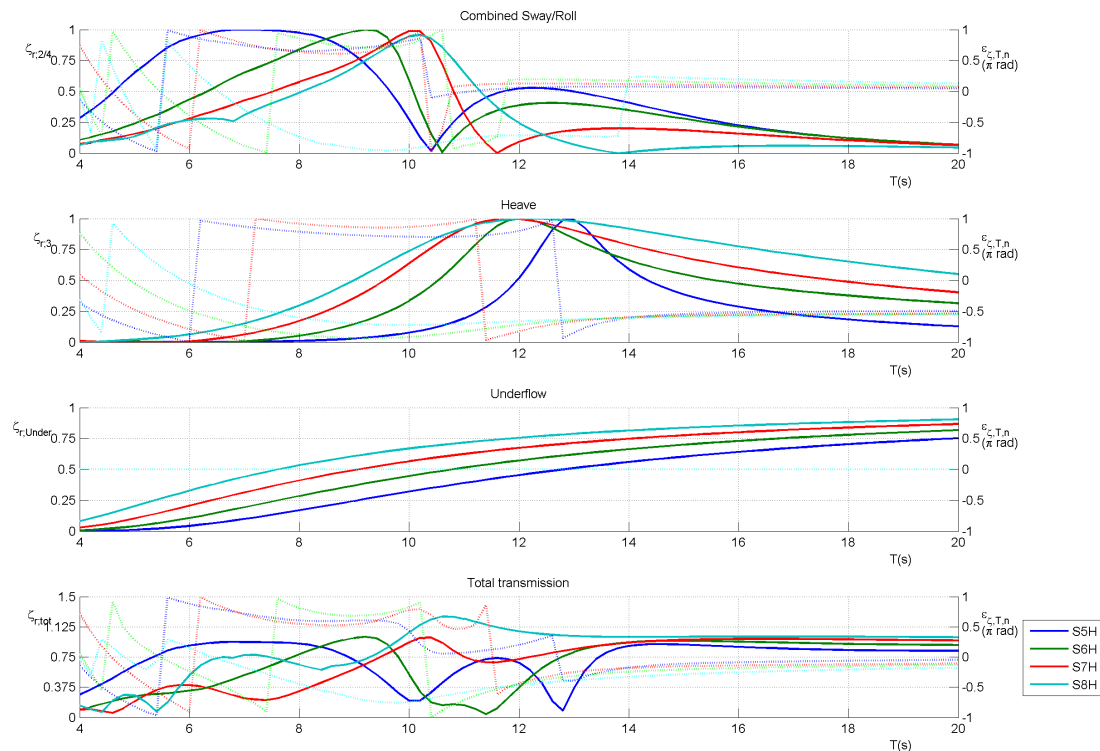


Figure E.24 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	Variable
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	820
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	-0,25 T_1

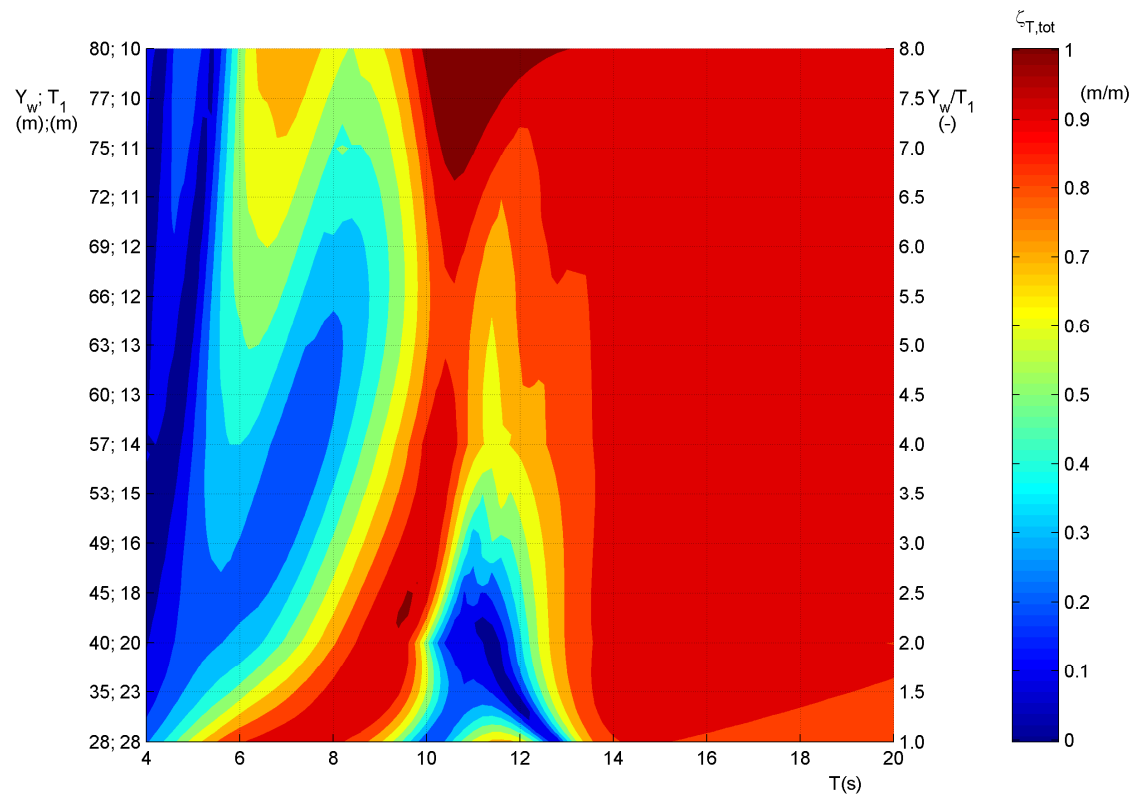


Figure E.25 RAO for a structure with a fixed mass of 820 tons and a variable draught and beam

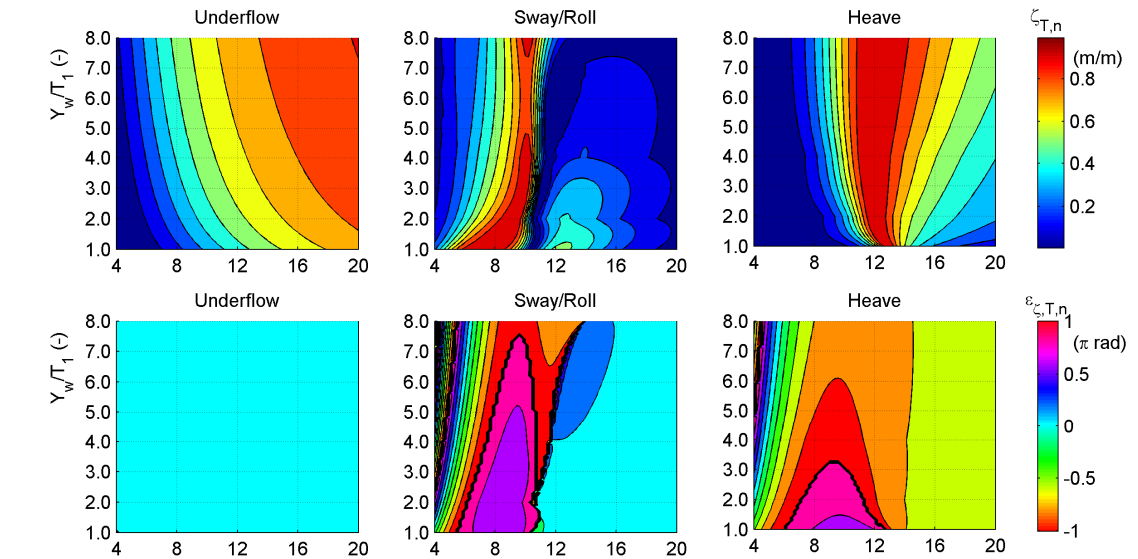


Figure E.26 Components of the transmitted wave RAO for the structure

E.3.6 Constant mass, high centre of gravity

Table E-8 Structural dimension for structures S13H, S14H, S15H and S16H

Structure number		Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
		L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
		[m]	[m]	[m]	[-]	[10^3 kg]	[10^6 kg m ²]	[%]	[m]
S13	H	250	28,3	28,3	1,0	205.000	20.090	0,0	7,1
S14	H	250	40	20	2,0	205.000	40.180	0,0	5,0
S15	H	250	56,6	14,1	4,0	205.000	80.360	0,0	3,5
S16	H	250	80	10	8,0	205.000	160.720	0,0	2,5

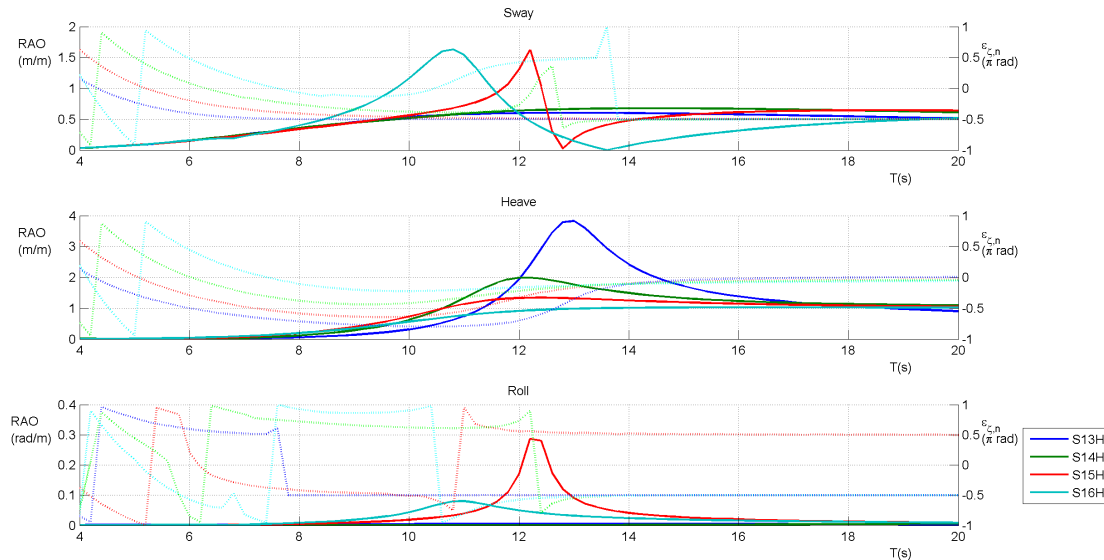


Figure E.27 The amplitude factors for sway, heave and roll for structures S13H, S14H, S15H and S16H

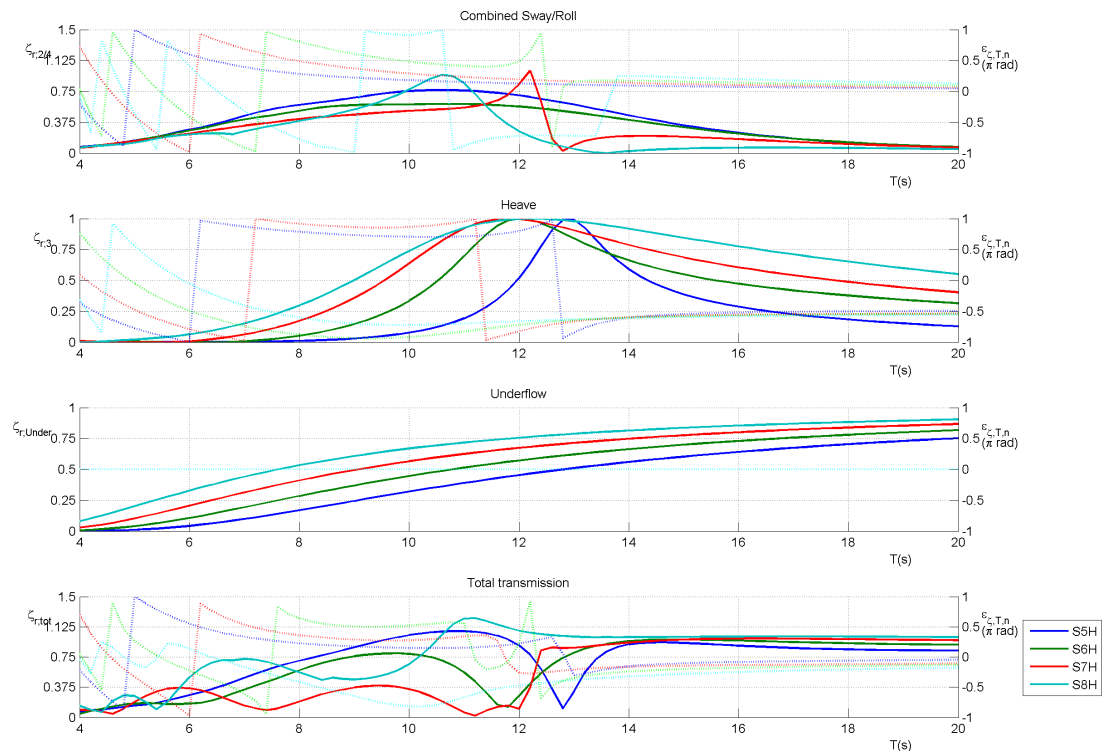


Figure E.28 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	Variable
Width to draught ratio	Y_w/T_1	[-]	Variable
Mass	m	$[10^3 \text{ kg}]$	820
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	OG	[m]	+0,25 T_1

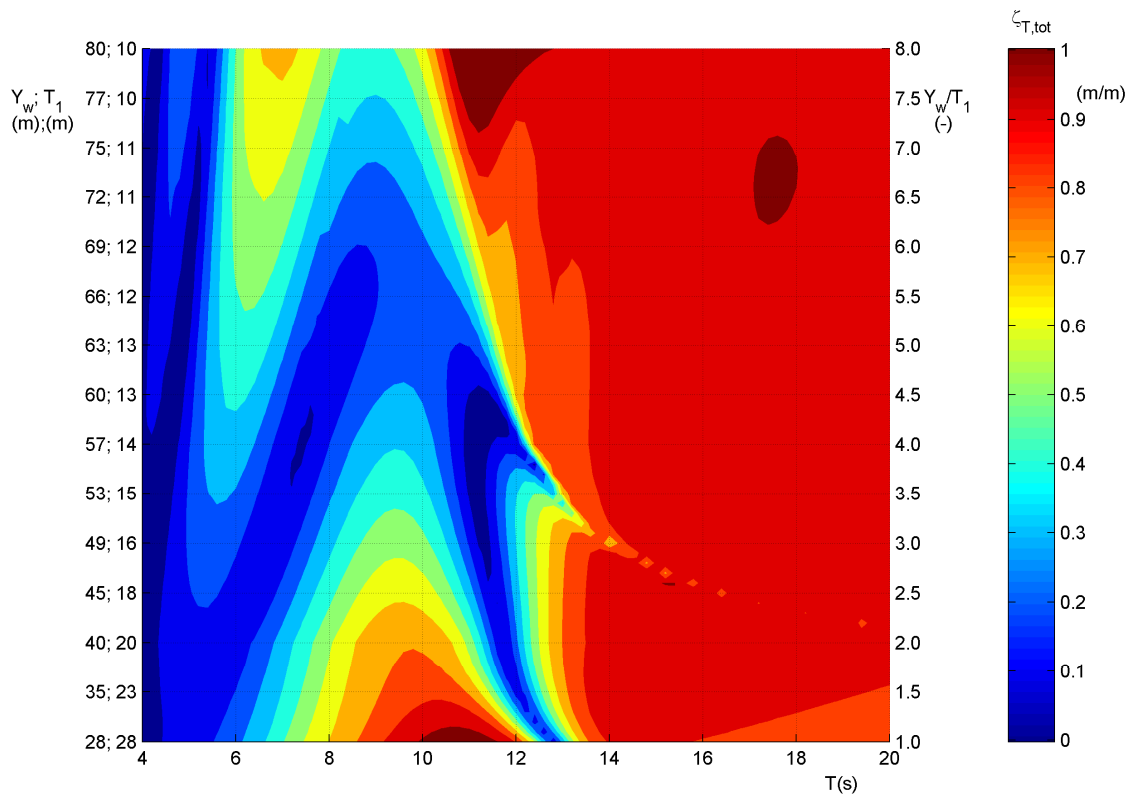


Figure E.29 RAO for a structure with a fixed mass of 820 tons and a variable draught and beam

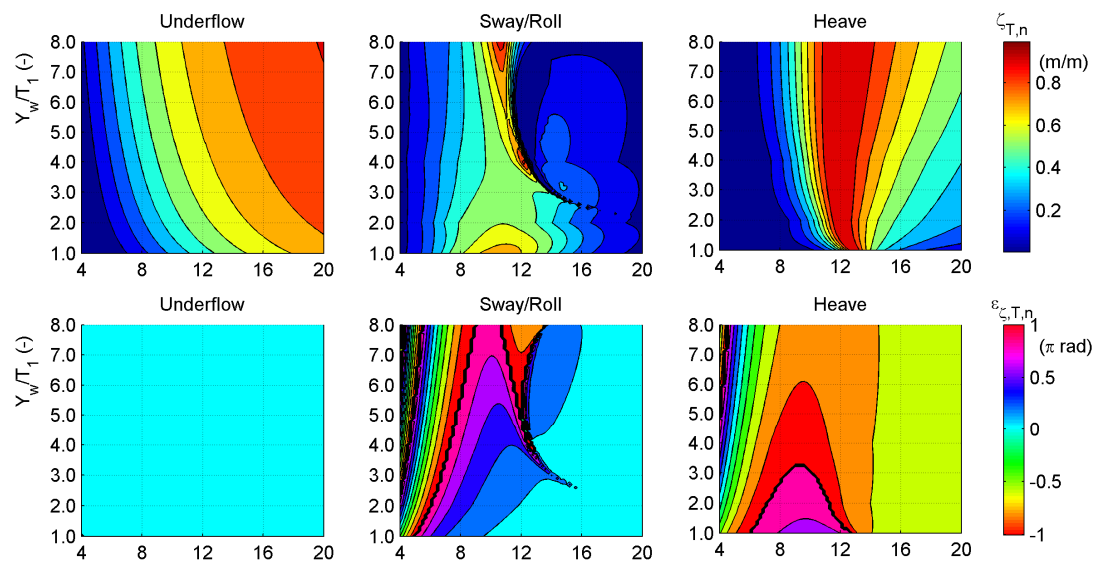


Figure E.30 Components of the transmitted wave RAO for the structure

E.4 Influence of the viscous roll damping

Table E-9 Structural dimension for structures S21, S22, S23 and S24

Structure number	Length	Width	Draught	Width to draught ratio	Mass	Moment of inertia	Viscous roll damping ratio	Centre of gravity
	L_w	Y_w	T_1	Y_w/T_1	m	I_{xx}	ζ	BG
	[m]	[m]	[m]	[-]	$[10^3 \text{ kg}]$	$[10^6 \text{ kg m}^2]$	[%]	[m]
S17	250	40	20	2,0	205.000	40.180	0,0	0
S18	250	40	20	2,0	205.000	40.180	5,0	0
S19	250	40	20	2,0	205.000	40.180	10,0	0
S20	250	40	20	2,0	205.000	40.180	15,0	0

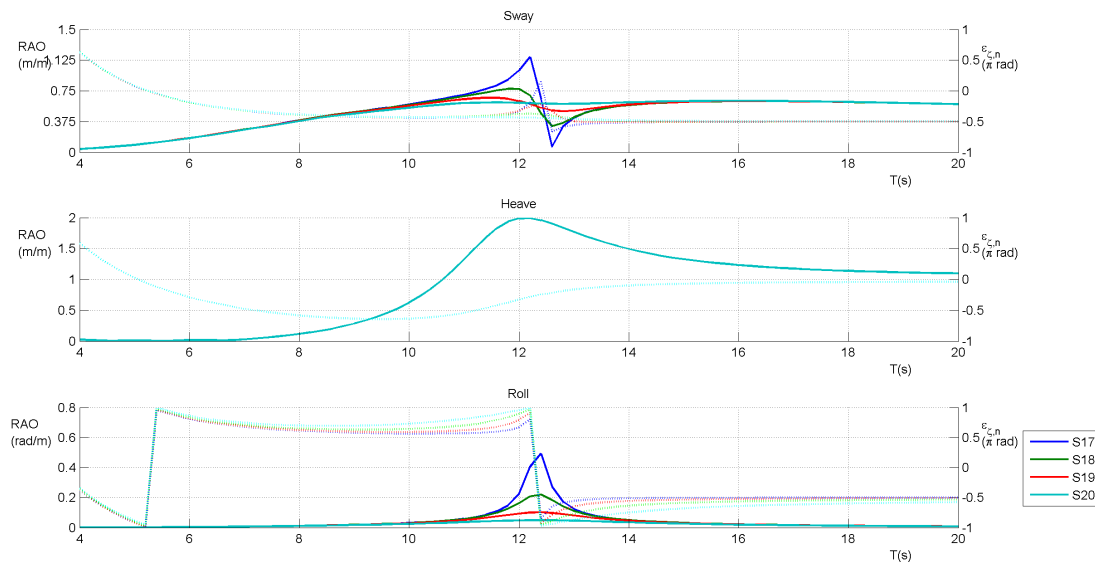


Figure E.31 The amplitude factors for sway, heave and roll for structures S17, S18, S19, S20

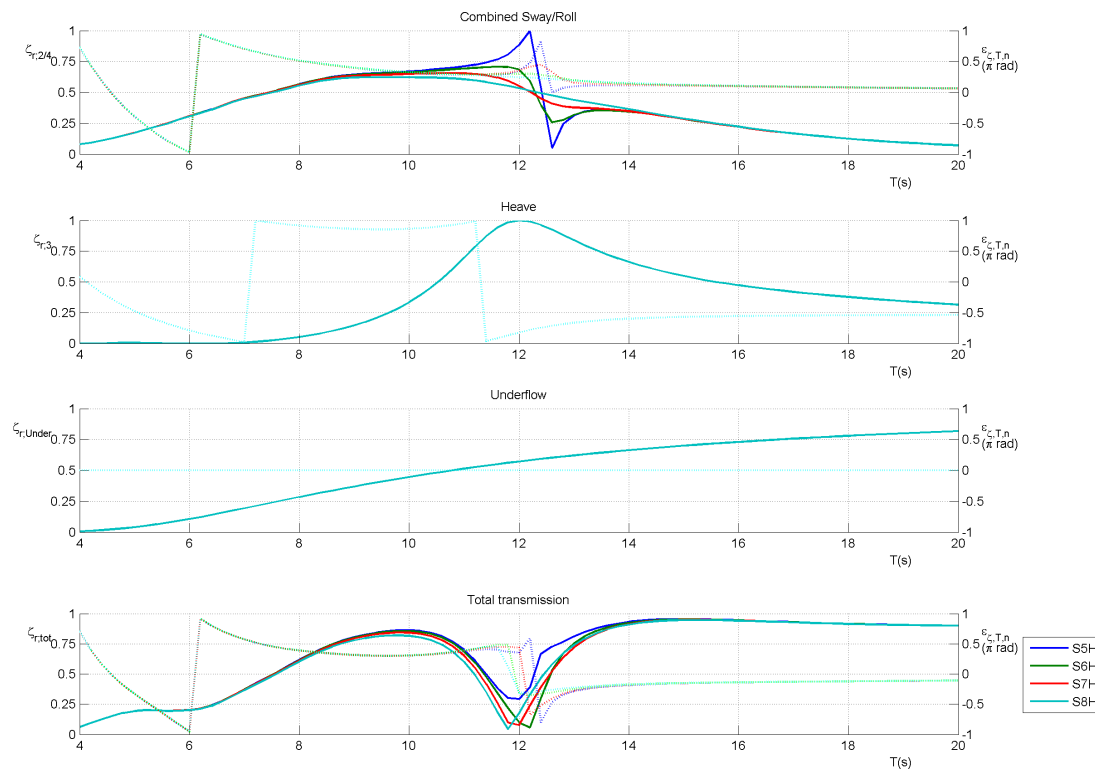


Figure E.32 Radiated wave due to combined sway/roll, heave and underflow and the transmitted wave height towards the lee side of the floating breakwater

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	40
Draught	T_1	[m]	20
Width to draught ratio	Y_w/T_1	[-]	2,0
Mass	m	$[10^3 \text{ kg}]$	820
Viscous roll damping ratio	ζ	[%]	Variable
Centre of gravity	OG	[m]	0,0

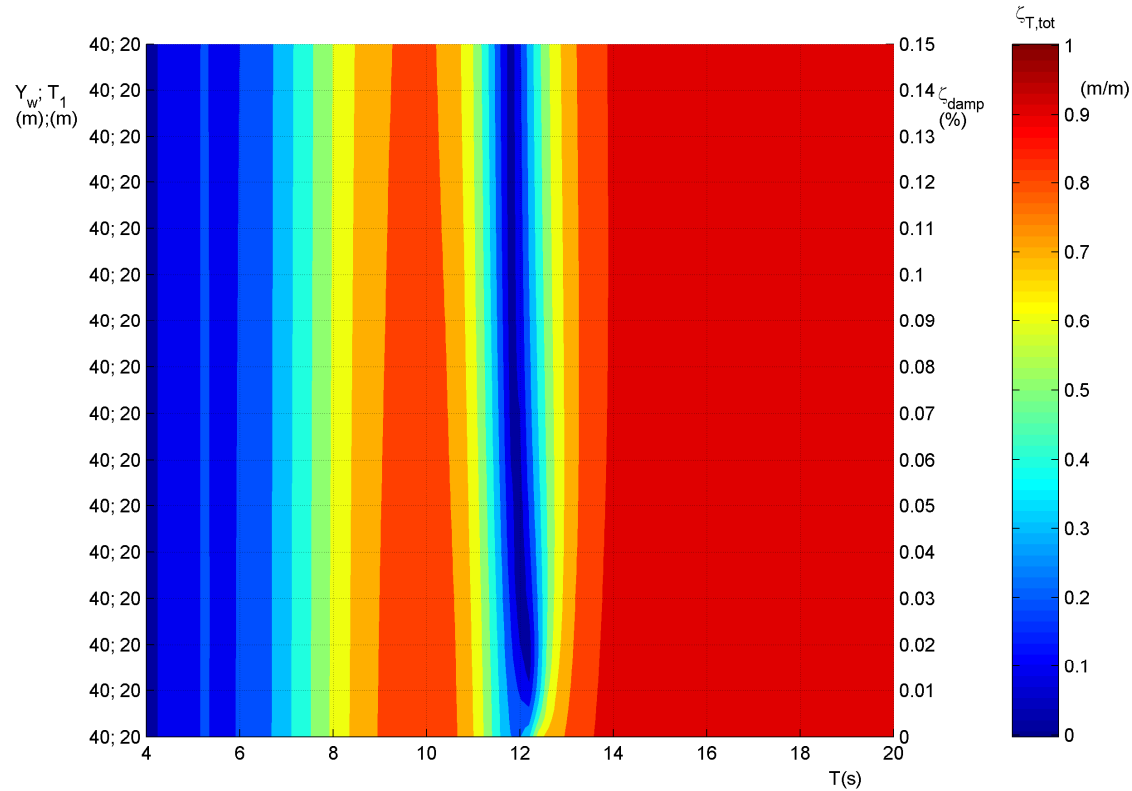
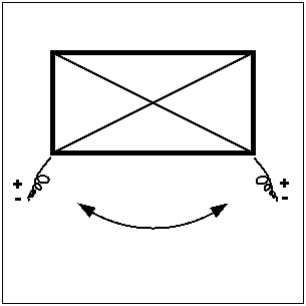


Figure E.33 Transmitted wave RAO for a structure with a fixed breadth and draught with a variable viscous roll damping

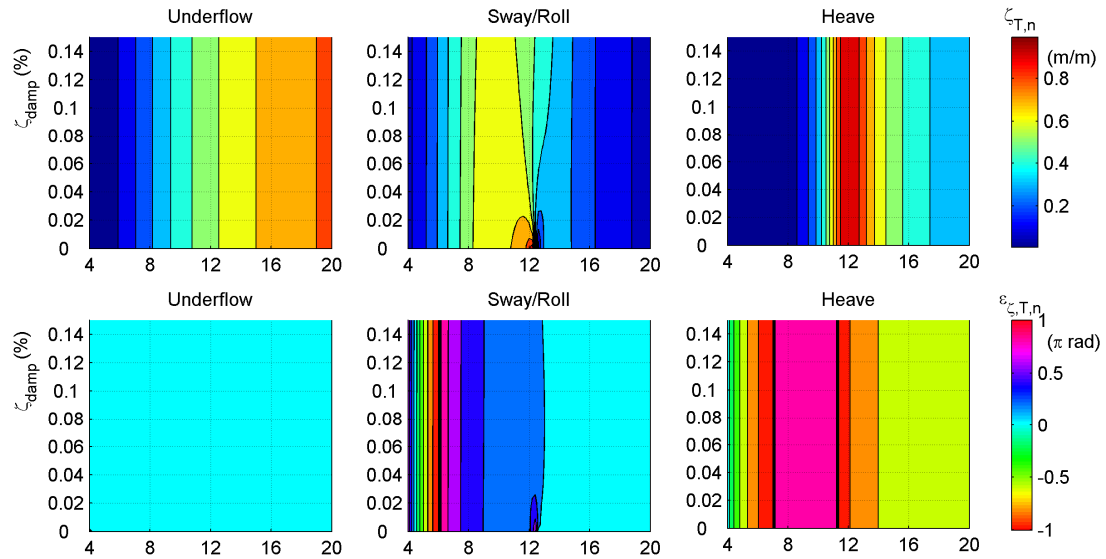


Figure E.34 Components of the transmitted wave RAO for the structure

E.5 Conclusion

The centre of gravity does have a big influence on the performance of the floating breakwater. Shifting the centre of gravity also shifts the sway/roll natural frequency, possibly solving the problem with the coinciding heave natural frequency and the sway/roll natural frequency. A low centre of gravity gives a stable structure and gives a better performance for a wider frequency range for a structure with an Y_w/T_1 -ratio of 2.0. A high centre of gravity shifts the roll natural period to a lower frequency, giving better results than a coinciding roll and heave natural frequency. Apparently the centre of gravity of the DELFRAC calculation was chosen on a, for a floating breakwater, bad position with a low performance. This can be seen from:

- Figure E.5 and figure E.6, $\overline{BG} = -10..5$.

When only the beam of the structure is varied, i.e. the draught I constant, it can be concluded that it is impossible to tune the breakwater. This is due to the invariability of the heave natural frequency with the beam. When the centre of gravity is located low, thus creating a stable pontoon, the roll natural frequency coincides with the heave natural frequency. The coinciding natural frequencies do have a negative impact on the wave attenuating character. A Y_w/T_1 -ratio of 5.0 is the most effective to attenuate waves in a broad band of frequencies when the COG is located high. This can be seen from:

- Figure E.9 and figure E.10, constant beam, variable draught and low COG.
- Figure E.13 and figure e.14, same as above, but with a high COG.

When the draught is varied it is possible to tune the breakwater. When the draught increases, the heave natural frequency increases as well. As has been described above, a low COG is not effective, while a high COG is. Also it can be seen again that an Y_w/T_1 -ratio is the most effective to attenuate waves in a broad band of frequencies. This can be seen from:

- Figure E.17 and figure E.18, variable beam, constant draught and low COG.
- Figure E.21 and figure E.22, same as above, but with a high COG.

The mass of the structure can be seen as an economical parameter in this research. When more mass is required one can assume that the initial costs of the structure will be higher. With a constant mass and a variable Y_w/T_1 -ratio the optimal ratio can be determined. An optimal performance of the floating breakwater is the capability of attenuating waves in a broad band of frequencies, which is in this case obtained by applying a breakwater with a Y_w/T_1 -ratio of 5.0. This can be seen from:

- Figure E.25 and figure E.26, variable Y_w/T_1 -ratio, constant mass, low COG.
- Figure E.29 and figure E.30, same as above, but with a high COG.

The viscous roll damping does have some influence on when its value is changed from 0% of the critical damping to 2%, a change from 2% to 5% or even higher does not have an apparent effect on the wave height on the lee side of the floating breakwater. A practical and realistic value of this damping is 5% of the critical damping; it is assumed that this value can be used in further research. This can be seen from:

- Figure E.33 and figure E.34

The most optimal floating breakwater element which can attenuate (irregular) waves in a broad band of frequencies should have an Y_w/T_1 -ratio of 5.0 and a viscous roll damping of 5%. The RAO for such a structure is given in figure E.35 and figure E.36 on the next page.

E.6 Optimal design

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	250
Width	Y_w	[m]	Variable
Draught	T_1	[m]	Variable
Width to draught ratio	Y_w/T_1	[-]	5,0
Mass	m	$[10^3 \text{ kg}]$	Variable
Viscous roll damping ratio	ζ	[%]	5,0
Centre of gravity	OG	[m]	0,0

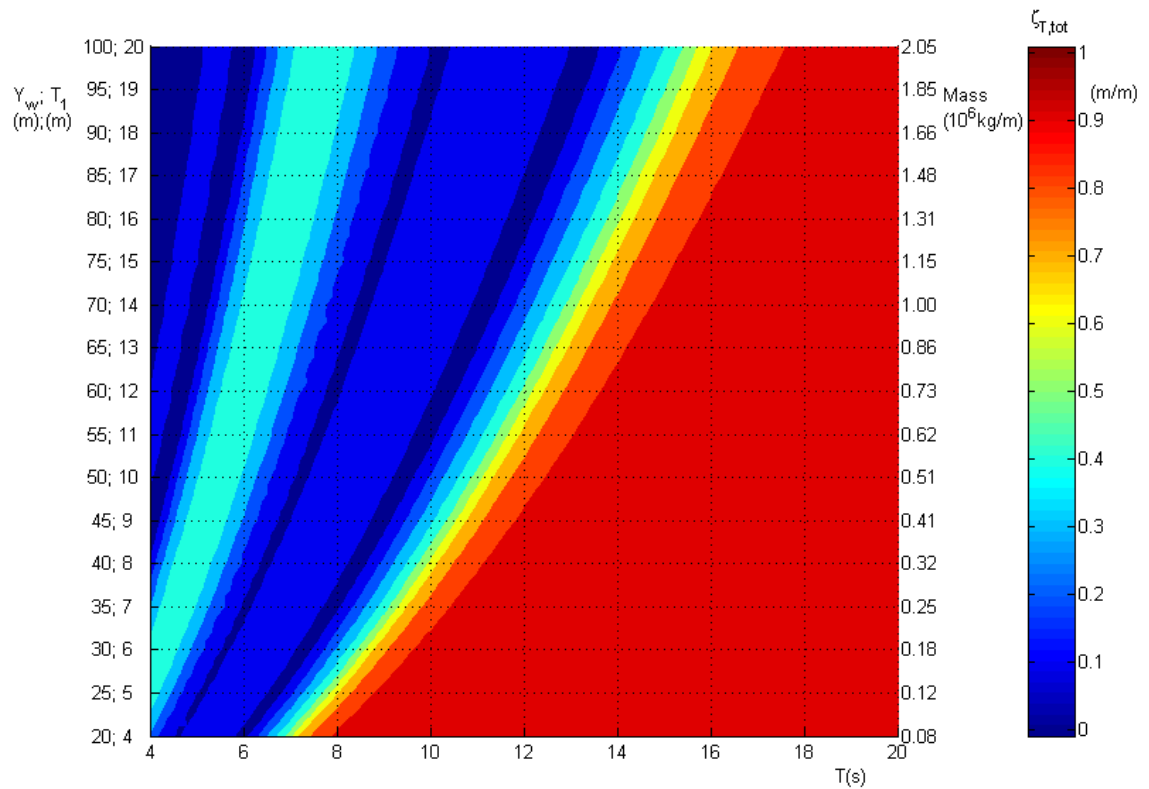


Figure E.35 Transmitted wave RAO for a structure with an Y_w/T_1 -ratio of 5.0 and a viscous roll damping of 5%.

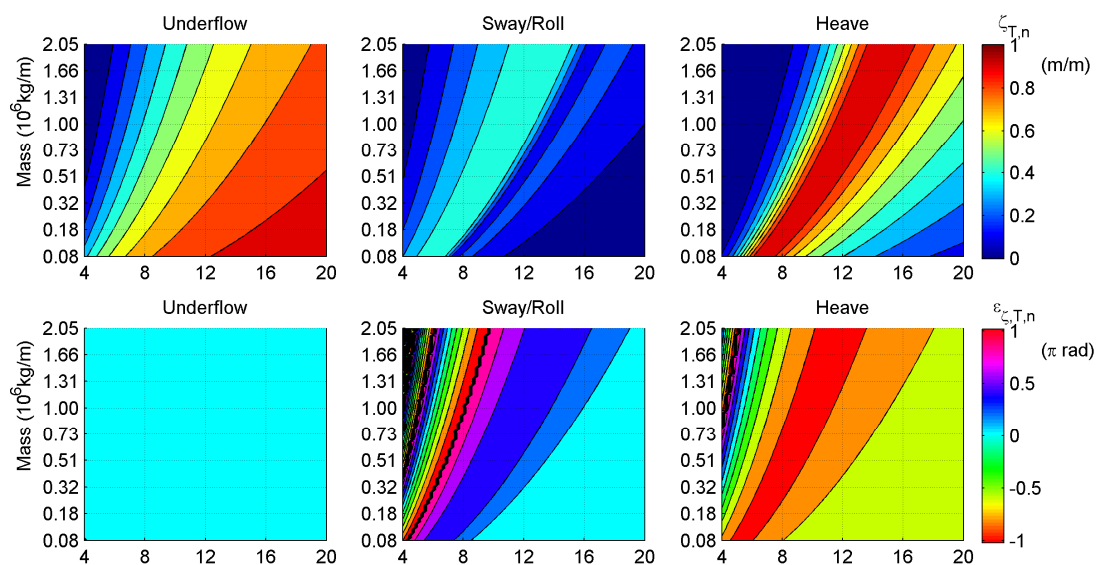


Figure E.36 Components of the transmitted wave RAO for the structure

E.7 Check with DELFRAC

To come to the design graph as has been presented on the previous page many operations have been done to scale the values of the added mass and hydrodynamic damping to the required size. In order to verify those operations a DELFRAC simulation has been done on a FB element with a Y_w/T_1 -ratio of 5.0, see table E-10.

Table E-10 Structure in DELFRAC simulation

Parameter	Symbol	Unit	Magnitude
Length	L_w	[m]	500
Width	Y_w	[m]	100
Draught	T_1	[m]	20
Width to draught ratio	Y_w/T_1	[-]	5
Mass	m	[10^3 kg]	2050
Viscous roll damping ratio	ζ	[%]	0,0
Centre of gravity	BG	[m]	5,0

In figure E.37 and figure E.38 below the hydrodynamic parameters are given.

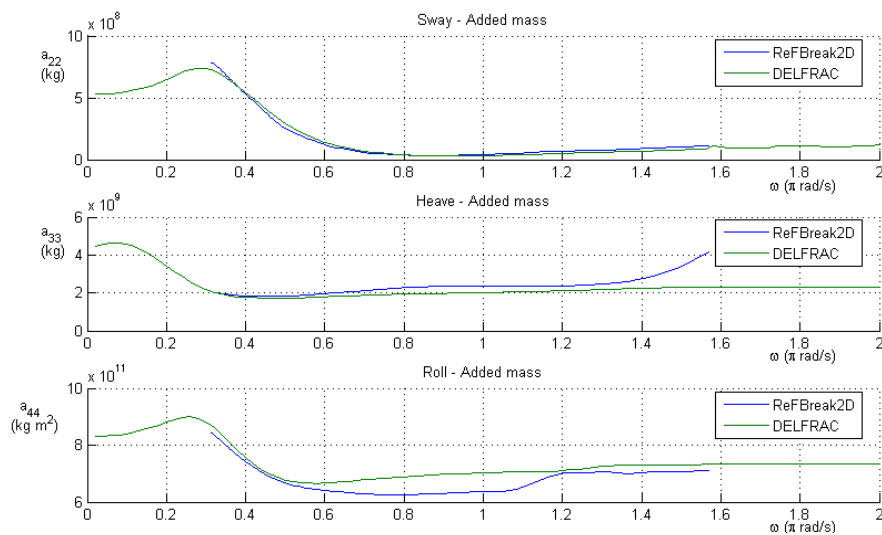


Figure E.37 Added mass coefficients from DELFRAC and ReFBreak2D

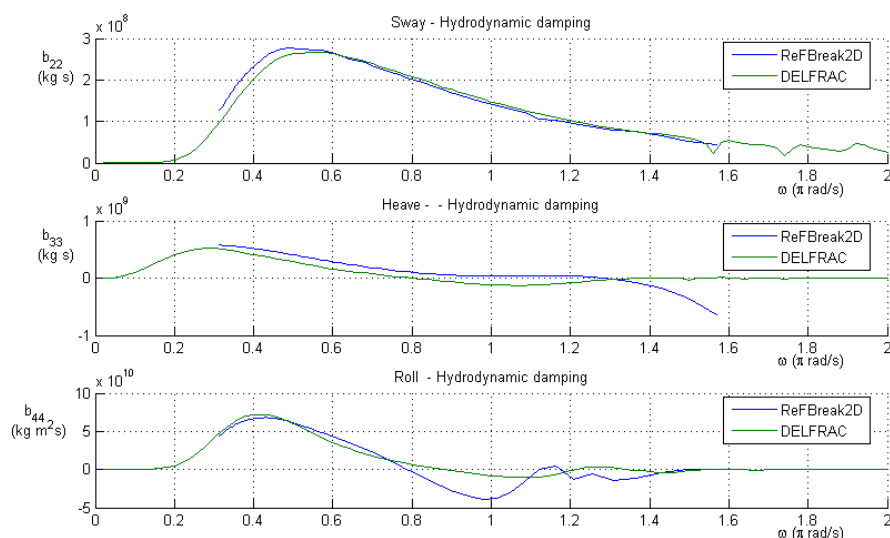


Figure E.38 Hydrodynamic damping coefficients from DELFRAC and ReFBreak2D

From figure E.37 and figure E.38 it can be seen that the hydrodynamic coefficients that are calculated in ReFBreak2D on basis of interpolation and scaling rules give a good results when compared to the DELFRAC simulation. When the frequency gets very high, i.e. 1.4 radians per second or higher, the hydrodynamic parameters of heave deviate from the expected value of DELFRAC. This is due to calculation method for these parameters. For the calculation of these parameters the polynomials are used based on the work of [Fousert, 2006]. The polynomials are only accurate for a limited scale and frequency, the deviation that can be seen is due to passing this limit. When larger structures do have to be calculated higher order polynomials will be required.

The hydrodynamic parameters, especially damping, do have a direct influence on the wave attenuating performance of the floating breakwater in combination with the motions. Therefore the parameters from DELFRAC have been used to calculate the first order motions to compare these with the ReFBreak2D-model, see figure e.39.

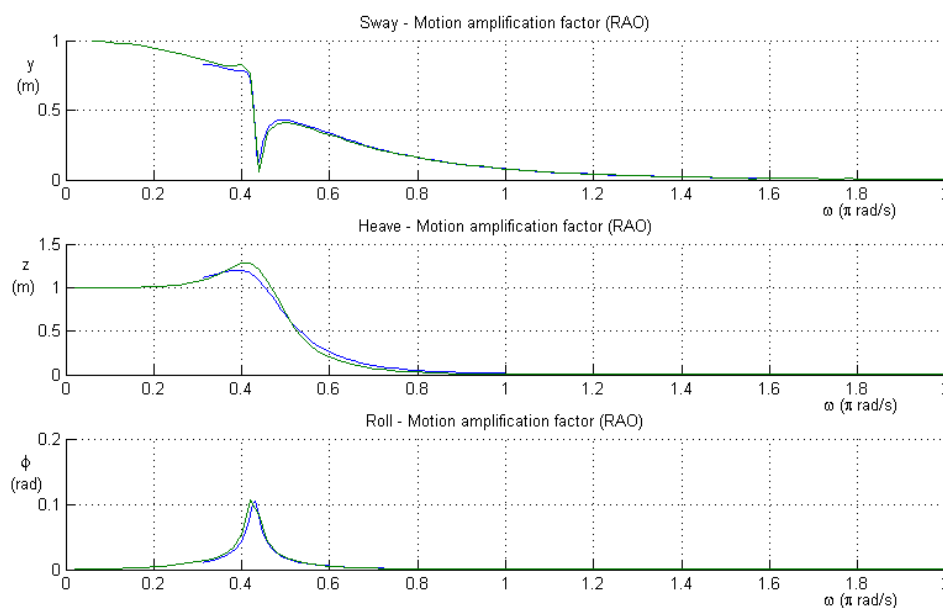


Figure E.39 Amplification factor for the first order wave motions from DELFRAC and ReFBreak2D

From figure E.39 it can be seen that all motions calculated with the developed model have the same values as the DELFRAC results. Therefore it can be concluded that the formulas used in the model to interpolate the hydrodynamic coefficients, the formulas used to up scale these coefficient and the formulas used to accommodate the shift of the centre of gravity are correct.

E.8 References

Fousert, M. W., (2006), "*Floating Breakwater, Theoretical study of a dynamic wave attenuating system*", MSc. Thesis, TU Delft, Delft

Appendix F Hydrodynamic loads

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Appendix F Hydrodynamic loads

There are three sources of loads on the structure, these are:

- The permanent load due to the own weight;
- The hydrostatic load caused by the pressure of the still water;
- The hydrodynamic load consisting of the pressures and forces caused by the wave field.

The permanent load due to the own weight and the hydrostatic loads are considered to be static and are relative easy to determine. These loads will be determined for each construction phase during the design process. The dynamic loads are dependent on the interaction of the structure and the wave field and can be divided in two categories:

- Dynamic pressures caused by the wave field;
- Drag forces exerted by the waves on the structure.

The dynamic pressure on the hull of the structure has to be added to the hydrodynamic load and will be dealt with in the first section. The drag forces are not as important to the strength of the hull as the dynamic pressures but will determine the minimum dimension of the anchoring system. These drag forces are dealt with in the second section.

F.1 Dynamic pressures

The dynamic pressures, i.e. the pressures caused by the waves, are calculated with the help of a DELFRAC-simulation. A probabilistic approach is used to determine the design hydrodynamic pressures. The hull of the FB element has to withstand both the hydrodynamic pressure and the hydrostatic pressure.

F.1.1 DELFRAC

For the purpose of determining the pressures on the hull of the FB element a DELFRAC calculation is made. The required dimensions of the floating breakwater have been determined with the ReFBreak2D model, which is based on earlier DELFRAC calculations. The input of the new calculation is given in table F-1.

Table F-1 Comparison between the old DELFRAC model used for ReFBreak2D and the new calculation

W/D-ratio	1 (OLD)	2 (OLD)	4 (OLD)	8 (OLD)	5 (NEW)
Draught	5 m	5 m	5 m	5 m	20 m
Breadth	5 m	10 m	20 m	40 m	100 m
Length	25 m	50 m	100 m	200 m	500 m
Min. wave-length	1,75 m	3,50 m	7,00 m	14,00 m	25,00 m
Panel-length	0,35 m	0,70 m	1,40 m	2,80 m	5,00 m
Number of panels	859 [-]	859 [-]	859 [-]	859 [-]	760 [-]

The model is not scaled as the input of the ReFBreak2D model was. The reason for this is twofold:

1. It is not necessary to scale the model as DELFRAC can calculate a full-scale model and the required dimensions are known at this moment. When a full-scale model is used, there is no need for scaling the pressures, eliminating a possible source of errors.
2. The output of DELFRAC, especially the coefficients for the added mass and the hydrodynamic damping, can be used to validate the results of the ReFBreak2D-model.

An image of the modelled quarter of the floating breakwater is given in figure F.1.

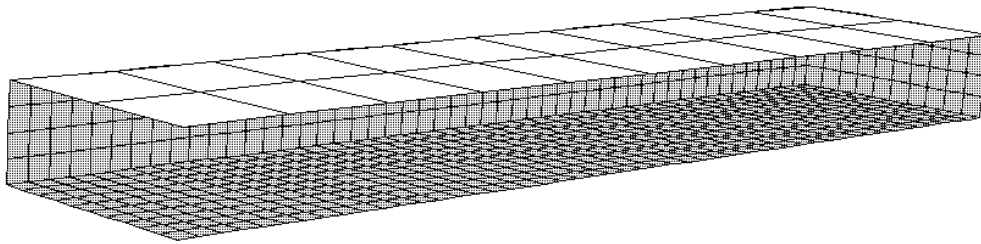


Figure F.1 The modelled quarter of the hull of the floating breakwater

The DELFRAC model gives output in a number of ASCII-files of which two are of importance for the determination of the hydrodynamic pressure on the floating breakwater. These files are "PANDATVA.OUT", which gives information on the location of the panels, and "PRESSAA.OUT", which gives information on the pressure on each panel at each frequency. Examples of these files are given below.

```

50 49 4
DATE : 02/28/08
TIME : 18:03:42.06

WD 5 including drift forces

DATA ON PANEL DISTRIBUTION

VOLUMES AND C.O.B.
X-VOLUME : 1.000E+06
Y-VOLUME : 1.000E+06
Z-VOLUME : 1.000E+06
SELECTED VOLUME : 1.000E+06
X-C.O.B. : 0.000E+00
Y-C.O.B. : 0.000E+00
Z-C.O.B. : -1.000E+01

WATERPLANE AREA AND METACENTRIC HEIGHTS
WATERPLANE AREA : 5.000E+04
VERTICAL C.OF G.: -5.000E+00
TRANS.METC.HGHT.ABOVE C.of B : 4.167E+01
LONG.METAC.HGHT.ABOVE C.of B : 1.042E+03
TRANS.METC.HGHT.: 3.667E+01
LONG.METAC.HGHT.: 1.037E+03

PAN.  N1    N2    N3    XP      YP      ZP      AREA
NO.   --    --    --    M      M      M      M*M
  1  0.000E+00  1.000E+00  0.000E+00  2.500E+00  5.000E+01 -2.500E+00  2.500E+01
  2  0.000E+00  1.000E+00  0.000E+00  2.500E+00  5.000E+01 -7.500E+00  2.500E+01
  3  0.000E+00  1.000E+00  0.000E+00  2.500E+00  5.000E+01 -1.250E+01  2.500E+01
...
3038 0.000E+00  0.000E+00 -1.000E+00  2.475E+02 -1.250E+01 -2.000E+01  2.500E+01
3039 0.000E+00  0.000E+00 -1.000E+00  2.475E+02 -7.500E+00 -2.000E+01  2.500E+01
3040 0.000E+00  0.000E+00 -1.000E+00  2.475E+02 -2.500E+00 -2.000E+01  2.500E+01

```

Code PANDATVA.OUT Location of the panels

```

WAVE DIRECTION IN DEGR : 0.000
TOTAL PRESSURES AT PANEL CENTER
PANEL  PRESSURE AMP  PHASE ANGLE
NUMBER  IN KN/M*M    IN DEGREES
WAVE FREQUENCY IN R/S : 0.020
  1      0.9986      270.0000
  2      0.0000      179.4999
  3      1.0000      360.0000
...
3038      0.0504      324.3792
3039      0.0485      326.9791
3040      0.0466      337.0210
...
WAVE FREQUENCY IN R/S : 2.000
  1      0.0011      257.7396
  2      0.0000      121.3914
  3      0.0000      35.3758
...
3038      0.0794      193.3590
3039      0.0821      193.1624
3040      0.0812      192.5244

```

Code PRESSAA.OUT Pressures on the panels at every frequency

With a MATLAB-routine these two files have been imported and visualisation of the pressure on the hull of the floating breakwater at various frequencies has been made possible.

F.1.2 Statistics

The structure is loaded by wind generated waves. The individual wave height of these irregular waves can be described by a Rayleigh distribution. The pressures on the sides and bottom of the floating breakwater are directly related to the incoming waves and are therefore assumed describable with a Rayleigh distribution as well. A typical Rayleigh probability density spectrum is given in figure F.2.

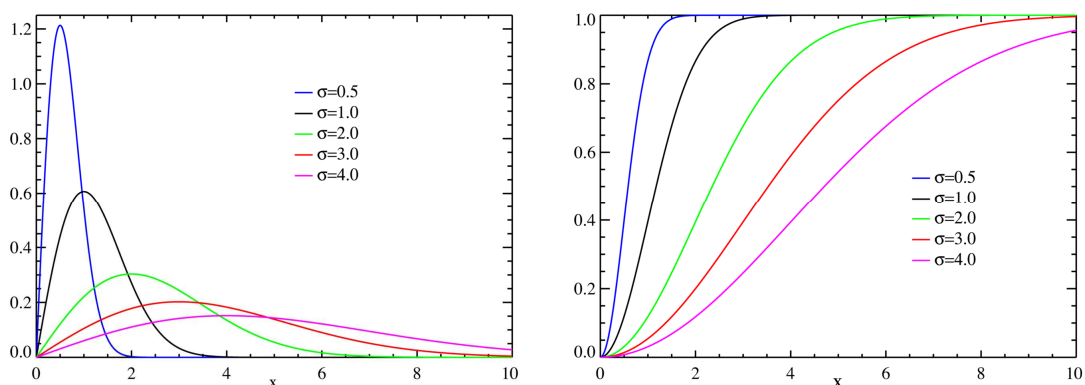


Figure F.2 Rayleigh probability density function (left) and cumulative distribution function (right)¹

For this engineering problem, as with many when waves are considered, it is important to understand the statistical characteristic of extremes. Normally the maximum surface elevation, called $\eta_{\max}$, within a certain duration D (for example in a storm) is of interest [Holthuijsen, 2007], but in this case the maximum pressures

¹ http://en.wikipedia.org/wiki/Rayleigh_distribution

on the sides of the FB element, called $\underline{p}_{\max}$, is the value of interest. First it is described how the probability of $\underline{p}_{\max}$ is obtained. Secondly, the requirements are stated on the failure probability and the duration of the design storm.

The probability of the pressure exceeding a certain level, p , is denoted as:

$$\Pr\{\underline{p}_{crest} > p\} = Q_{crest} = e^{-\frac{p^2}{2m_0}} \quad (F.1)$$

Where:

- $\Pr\{\underline{p}_{crest} > p\}$: Probability of exceeding the value p ;
- Q_{crest} : Probability distribution function, a Rayleigh distribution;
- m_0 : Zeroth moment of the variance density spectrum.

Equation (F.1) gives the probability that the pressure on a point of the FB is exceeded due to a single wave. The probability of this arbitrarily chosen pressure is not exceeded is then:

$$\Pr\{\underline{p}_{crest} \leq p\} = 1 - Q_{crest} \quad (F.2)$$

The probability that two arbitrarily chosen pressures do not exceed the level p is:

$$\Pr\{\underline{p}_{crest,1} \leq p \cap \underline{p}_{crest,2} \leq p\} = (1 - Q_{crest})^2 \quad (F.3)$$

This can be expanded to the probability that all pressures in duration D do not exceed the level p . This probability is given by:

$$\Pr\left\{\bigcap_{n=1}^N \underline{p}_{crest,n} \leq p\right\} = (1 - Q_{crest})^N \quad (F.4)$$

Where:

- N : The number of waves during D .

When the number of waves is sufficiently large, as in case of a storm, and the probability of exceedance Q_{crest} is small enough, then:

$$(1 - Q_{crest})^N \approx e^{-NQ_{crest}} \quad (F.5)$$

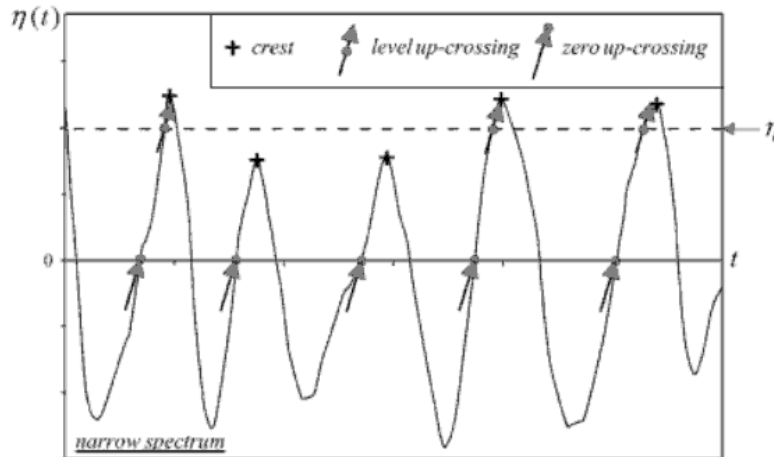


Figure F.3 Number of waves relative to the number of zero-crossings [Holthuijsen, 2007]

All that remains is to compute the value of (F.4) is the number of waves N during duration D . This can be done by noting that, that for a narrow spectrum, the number of wave is equal to the number of upward or downward zero-crossings [Holthuijsen, 2007], see figure F.3. This can be determined by the mean zero-crossing frequency and the duration:

$$N = \bar{f}_0 D = \frac{\bar{\omega}_0}{2\pi} D = \frac{\sqrt{m_2/m_0}}{2\pi} D \quad (\text{F.6})$$

Where:

- $\bar{f}_0$: Mean zero-crossing frequency;
- m_n : The n^{th} moment of the variance density spectrum;
- D : Duration of the storm.

F.1.3 Requirements

Before calculating the design pressures on the sides and bottom of the FB element the wave spectra and the duration of the storm have to be known. There are two conditions that are considered in this preliminary design:

- SLS-condition

The FB is loaded with beam-on waves. The waves are described with a Bretschneider spectrum corresponding to a storm with a magnitude of 7 on the scale of Beaufort.

- ULS-condition

The FB is loaded with head-on waves. The waves are described with a Bretschneider spectrum corresponding to a storm with a magnitude of 12 on the scale of Beaufort.

The wave spectra of both design storms are given in figure F.4.

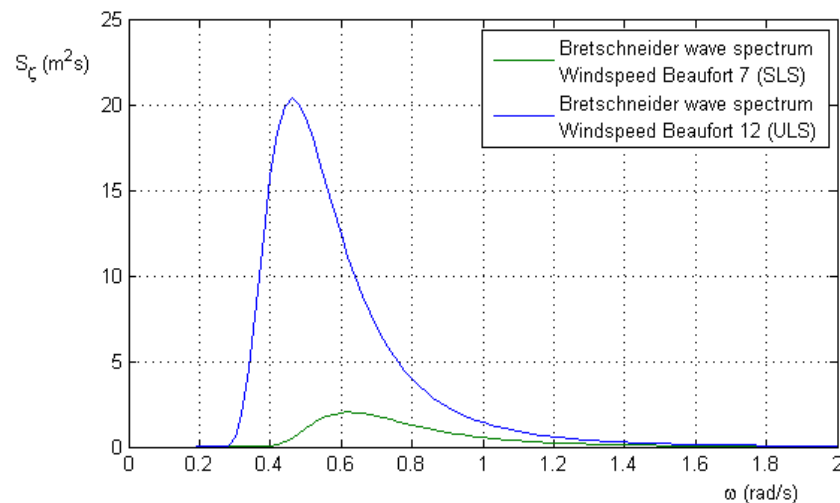


Figure F.4 Bretschneider wave spectra for SLS-condition and ULS-condition

As no storm or wave-data is known at this moment, as there is no location known, the return-period and the duration of the design storms are to be estimated. Heavy storms, i.e. with winds of up to Beaufort 12, will have a lower return-period and a smaller duration than a smaller storm with a magnitude of Beaufort 7.

The following is assumed:

- SLS-condition

The average return-period of this storm is once every year. The duration of the storm is estimated to be 12 hours.

- ULS-condition

The average return-period of this storm is once every 100 years. The duration of the storm is estimated to be 6 hours.

The duration of the design storm does dictate the number of waves the structure is loaded with. An error in the duration of the storm will lead to an under-estimation or over-estimation of the number of waves. This is not problematic as the maximum wave elevation and thus the wave pressure on the structure is rather insensitive to the number of waves when a large number of waves are considered [Holthuijsen, 2007].

When a FB element fails due to the wave loading there is damage. The direct damage is the capital costs for the elements which are lost. The indirect damage is the inability to operate the floating harbour that is supposed to be protected by the FB. When the direct and indirect costs are known the most economical solution can be chosen, i.e. the probability of failure is chosen so that the combined value of the risk and the required investment are minimal, see also figure F.5.

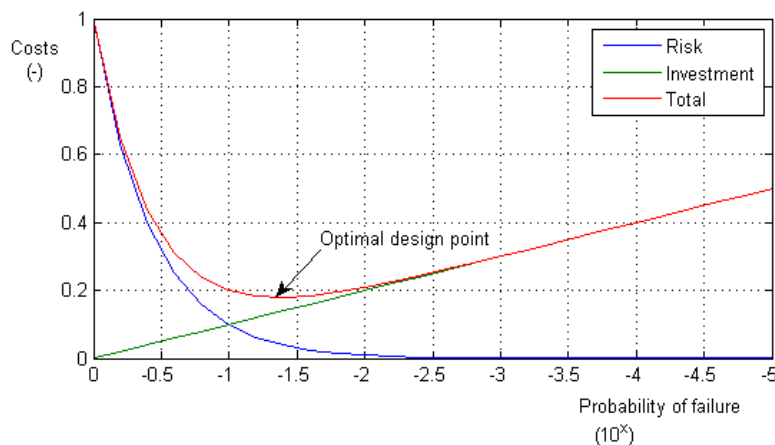


Figure F.5 The optimal design probability of failure derived from the risk and the investment, a simplified case

The analysis of the optimal probability of failure is considered to be out of the scope of this master thesis. The maximum probability of failure is chosen to be equal to the probability of failure of the Dutch sea-defence. This value is 10^{-4} per year or in words, once every 10.000 years.

All data on the design storm, number of waves and the required non-exceedance probability is given in table F-2.

Table F-2 Summary on wave statistics

Parameter	SLS	ULS
Wave direction	Beam-on	Head-on
Spectrum	Bretschneider	Bretschneider
Magnitude	Beaufort 7	Beaufort 12
Duration	12 [hrs]	6 [hrs]
Number of waves	5680 [-]	2165 [-]
Return-period of design storm	1 [yr]	100 [yr]
Probability of storm during life-time	1 [-]	0,64 [-]
Required non-exceedance probability	0,0001 [-]	0,00015 [-]

F.1.4 Calculated pressures; beam-on waves

Analogous to the determination of the wave spectrum that is transmitted towards the harbour area, as has been described in Appendix D.1 the pressure spectrum on the sides of the floating breakwater can be found:

$$S_{p,T}(\omega)d\omega = \left| \frac{p_{\zeta}}{\zeta_a}(\omega) \right|^2 S_{\zeta}(\omega)d\omega \quad (\text{F.7})$$

Where:

$S_{p,T}(\omega)d\omega$: The pressure spectrum;

$\left| \frac{p_{\zeta}}{\zeta_a}(\omega) \right|$: The transmission coefficient for the pressure;

$S_{\zeta}(\omega)d\omega$: The Bretschneider wave spectrum;

An example of the pressure spectrum for a panel located at the exposed side of the floating breakwater can be seen in figure F.6.

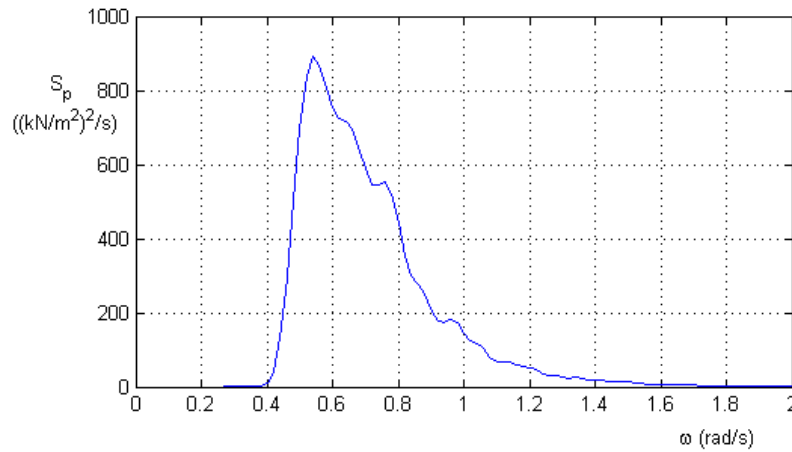


Figure F.6 Pressure spectrum on a panel for the Beaufort 7 design conditions

The spectrum, as shown in figure F.6 is obtained for all panels of the floating breakwater element. For each of these panels the design pressure is obtained by using the theory that is explained in F.1.2. The results of this calculation can be seen in figure F.7 for an element folded open.

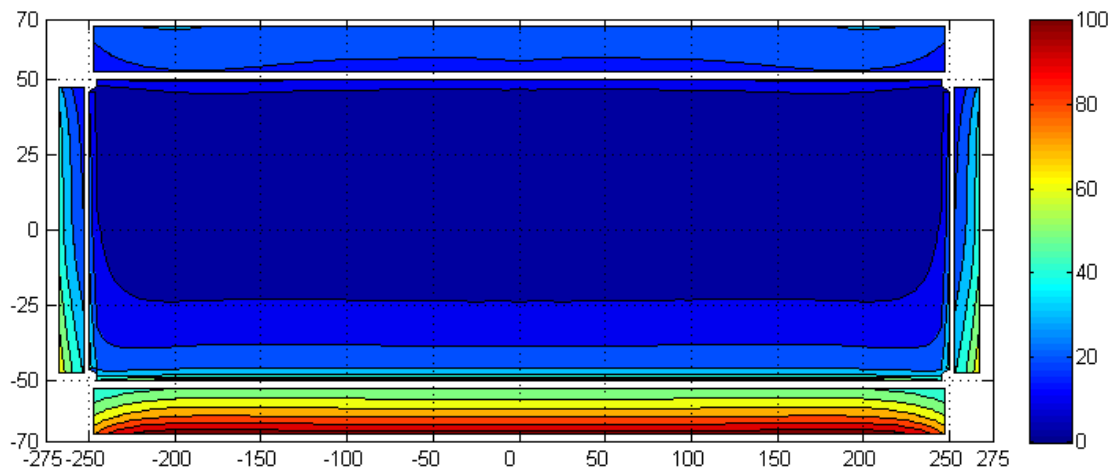


Figure F.7 Design wave pressure on a FB element loaded by beam-on waves in kN/m^2

Figure F.7 clearly shows that the highest design pressures are to be expected on the exposed side of the element. On this side pressures up to 100 kN/m^2 have to be taken into account during the design. For the other sides of the element lower pressures are found. They are between 10 and 20 kN/m^2 .

F.1.5 Calculated pressures; head-on waves

Similar to the way how the design wave pressures on the element loaded with beam-on waves were determined, the design pressure when an element is loaded with head on waves is calculated. For the same panel as in F.1.4 the pressure spectrum is given in figure F.8 for the Beaufort 12 design storm.

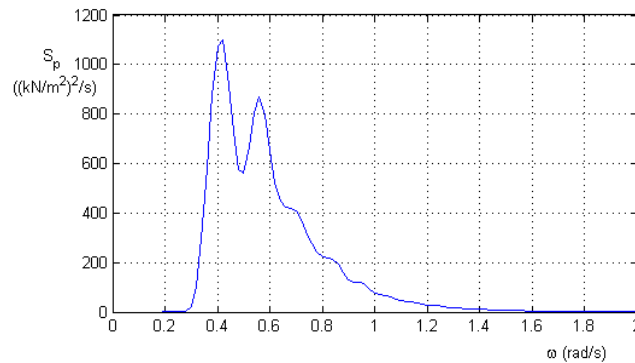


Figure F.8 Pressure spectrum on a panel for the Beaufort 12 design conditions

From the spectrum shown in figure F.8 the design pressure on the structure is determined as is described in F.1.2. The results of this calculation are shown in figure F.9.

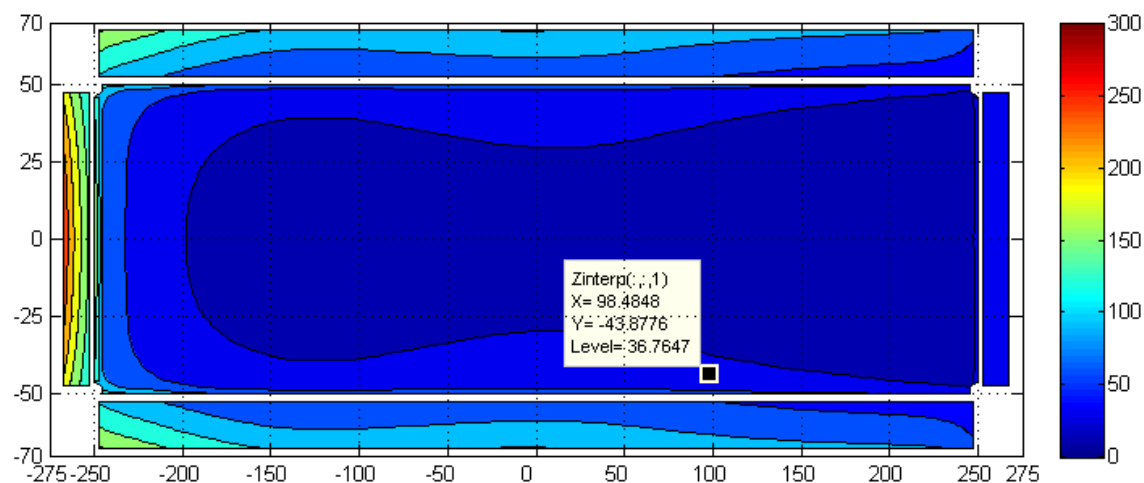


Figure F.9 Design wave pressure on a FB element loaded by head-on waves in kN/m^2

F.1.6 Conclusion

For the bow and the stern part of the floating breakwater the maximum pressures are determined by the Beaufort 12 storm condition. However for the sides of the element the beam-on waves of a storm with a magnitude of 7 on the scale of Beaufort is the determining case.

Besides the dynamic pressures, caused by the waves, the hydrodynamic pressure has to be taken into account also. In figure F.10 on the next page the total pressure, consisting of hydrodynamic pressures and hydrostatic pressures, is given.

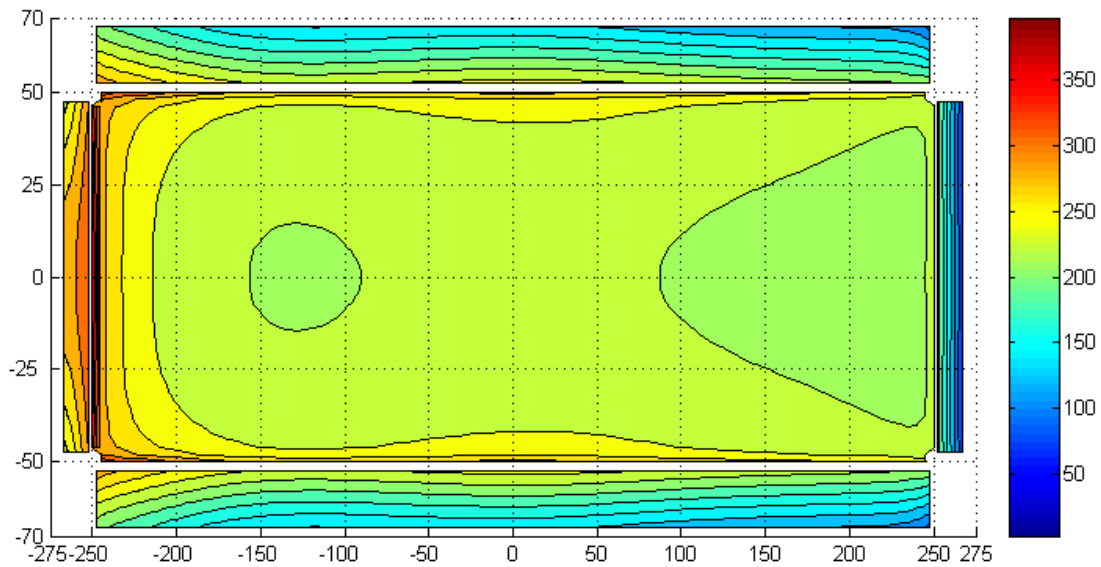


Figure F.10 Design total pressure on a FB element loaded by head-on waves in kN/m^2

When the FB element is build, it will be built in sections due to its enormous dimensions. From an economical point of view the number of different section types should be minimized. In principle it is sufficient to have three different types of sections:

- Bow and stern section

The bow and stern section are severely loaded when the element is rotated to receive the waves head-on. Extra measurements have to be taken to accommodate the pressures on the bow plane of the element.

Furthermore the bottom of the section as well as the sides of the sections have to be stronger then its neighbouring elements because the prevailing hydrodynamic pressures are higher.

In this element the thruster assembly may be located as well to avoid an extra type of element that is required.

- Turret mooring section

This section is similar to the normal section, except that it houses the turret mooring system.

- Normal section

The sides of this section are loaded when the FB element is facing the waves beam-on. The pressure on the bottom of this element is determined by the pressure caused by head-on waves, but is less then the pressures on the bow and stern sections.

The sections will be further elaborated on in Appendix G.

F.2 Drift forces

The ReFBreak2D model and the pressures which have been described earlier do consider the first order motions of and pressures on a structure that is attacked by waves. Beside these first order effects, there are also second order effects, especially when an irregular wave-field is considered. In [Journée and Massie, 2001] it is stated that:

"The effects of second order wave force are most apparent in the behaviour of moored floating structures. [...] these are horizontally restrained by some form of mooring system."

The floating breakwater system consists of several moored elements, so the second order wave forces will be apparent in the behaviour of a FB. The analysis of the forces and the motions of a FB element include three important components [Journée and Massie, 2001]:

1. An oscillating displacement of the structure with a frequency corresponding to the incoming waves.

These are the motions due to the first order wave loads. The time-averaged wave load on the structure and the resulting average displacement are zero. The ReFBreak2D model deals with the first order behaviour of the FB. This will not be further discussed in this section.

2. A mean displacement of the structure, resulting from a constant load.

Clear sources of a constant load are the wind and (tidal) currents. In addition to these loads there is also a so-called mean wave drift force caused by non-linear (second-order) wave potential effects. This load will, influenced by the mooring system determine, the new equilibrium around which it will oscillate with its first order displacements.

3. An oscillating displacement of the structure with a frequency much lower than those of the incoming irregular waves.

These motions are caused by non-linear elements in the wave load, the low frequency wave forces. The low frequency wave forces are caused by the differential frequencies that exist in the components of an irregular wave field. Generally a ship, and thus a FB-system, will have a low natural frequency in the horizontal directions as well as a very little damping at such frequencies. These low-frequency wave forces can result in very large motion amplitudes and corresponding loads in the mooring system.

The theory of the mean drift force is given in [Journée and Massie, 2001] and will not be discussed in this section. It will be used to determine the loads and displacements due to the mean wave drift force and the low-frequency wave drift force. The coefficients needed for the calculation are determined by using DELFRAC.

F.2.1 DELFRAC

In order to calculate the wave drift forces the same DELFRAC simulation is used as for the determination of the pressure on the hull. In order to calculate the drift forces, the waterline of the floating breakwater element has to be defined, see also figure F.11. An example of the definition of these waterline elements is given on the next page.

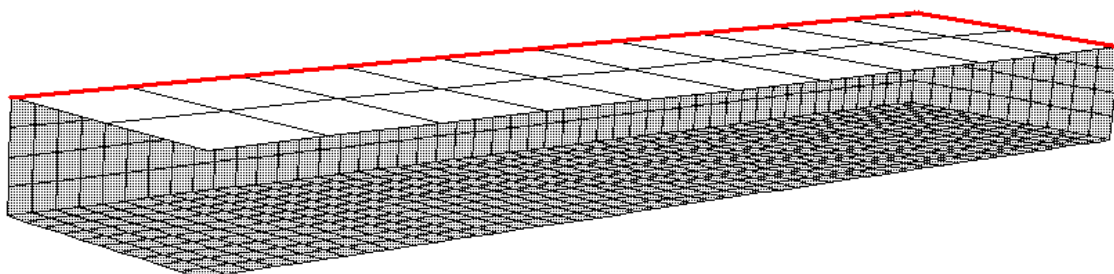


Figure F.11 The waterline elements (in red) required by DELFRAC in order to calculate the drift forces

61	60	4	#nr of waterline vertices, nr of waterline elements, panel integration	
1	250.000	0.000	#no. of waterline vertice, x-co-ordinate, y-co-ordinate (anti-clockwise)	
2	250.000	5.000		
...				
49	5.000	50.000		
50	0.000	50.000		
1	1	2	#no. of waterline element, first w.l. point, second w.l. point	
2	2	3		
...				
49	59	60		
50	60	61		

Code WD5.DAT Definition of the waterline elements

The output of DELFRAC for the wave drift forces consist of six files. The files "AQAA_1_1.OUT" to "AQAB_6_6.OUT" give the quadratic transfer function for each of the processes that are governing the wave drift forces (the first number) for every type of motion (the second number) and for each wave direction (A and B). The files "DRIFTFAA.OUT" and "DRIFTFAB.OUT" give a summation quadratic transfer function and will be used to determine the wave drift force.

F.2.2 Mean wave drift force

The mean wave drift force is a constant load on a floating structure and is caused by non-linear, or second-order, wave potential effects. The mean wave drift force will determine the equilibrium position for the harmonic oscillation of FB. The theory behind the mean wave drift force is extensively discussed in [Journée and Massie, 2001] and will not be further treated in this report.

The mean wave drift force is defined as:

$$F_{1_{mean}}^{(2)} = 2 \int_0^{\infty} S_{\zeta}(\omega) P(\omega, \omega) d\omega \quad (F.8)$$

Where:

$$\begin{aligned} S_{\zeta}(\omega) &: \text{Wave spectrum} & [\text{m}^2\text{s}] \\ P(\omega, \omega) &: \text{Mean drift force coefficient in regular waves} & [\text{Ns}/\text{m}^2] \end{aligned}$$

Two limiting cases can be defined:

1. The FB is in service, protecting the floating harbour area (SLS)

The FB is loaded by beam-on waves. The wave spectrum is defined as a Bretschneider wave spectrum with an average wave period $T_1 = 7.8$ seconds and a significant wave height $H_{1/3} = 3.75$ metres.

2. The FB is in survival condition (ULS)

All harbour action has stopped and the FB is rotated to face the waves head-on. The wave spectrum is defined as a Bretschneider wave spectrum with an average wave period of $T_1 = 10.5$ seconds and a significant wave height $H_{1/3} = 10.25$ metres.

The two wave spectra are shown in figure F.12 on the next page.

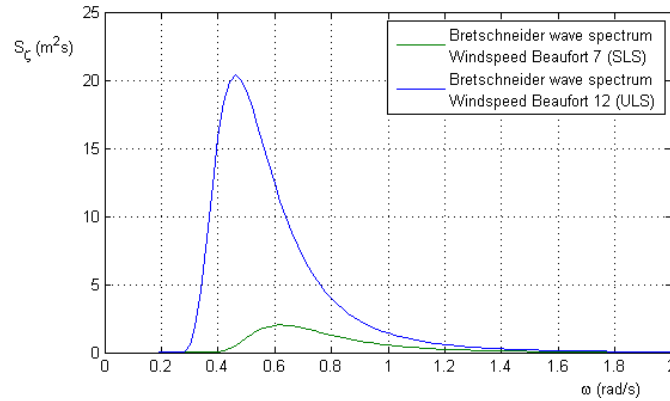


Figure F.12 Bretschneider wave spectra for SLS-condition and ULS-condition

The mean drift force coefficient is not a continuous function of the wave frequency, but is determined with DELFRAC for a number of frequencies. To estimate the mean wave drift force, equation (F.8) is rewritten into a summation:

$$F_{1_{mean}}^{(2)} = 2 \sum_{1}^n S_{\zeta}(\omega_n) P(\omega_n, \omega_n) d\omega \quad (F.9)$$

Where:

ω_n	: Wave frequency that has been simulated	[rad/s]
$d\omega$	: Step between to simulated frequencies	[rad/s]
$S_{\zeta}(\omega_n)$	: Wave spectrum	[m ² s]
$P(\omega_n, \omega_n)$	: Mean drift force coefficient in regular waves	[Ns/m ²]

The program MATLAB is used to calculate this summation, the code that has been used can be found below.

```

omega      = 0.02:0.02:2.0;
delta_omega = 0.02;

%% Wave spectrum
H13 = 10.25;
T1  = 10.50;
Sw  = zeros(1, length(omega));
for i = 1:length(omega)
    Sw(i) = 173 * (H13^2 / T1^4) * (omega(i))^(-5) ...
            * exp((-692/T1^4) * (omega(i))^(-4));
end

%% Mean wave drift forces summation
F_drift_mean = 0
for i = 1:length(omega)
    F_drift_mean = F_drift_mean + 2 * Sw(i) * P(i) * Delta_omega;
end
    
```

Code MATLAB Calculation of the mean wave drift force

The total mean wave drift force that is acting on a FB element is:

- Head-on position, wind speed Beaufort 12: 4700kN ;
- Beam-on position, wind speed Beaufort 7: 3800kN .

The relation between the wind speed, i.e. the wave spectrum, and the mean wave drift force is shown in figure F.13 on the next page.

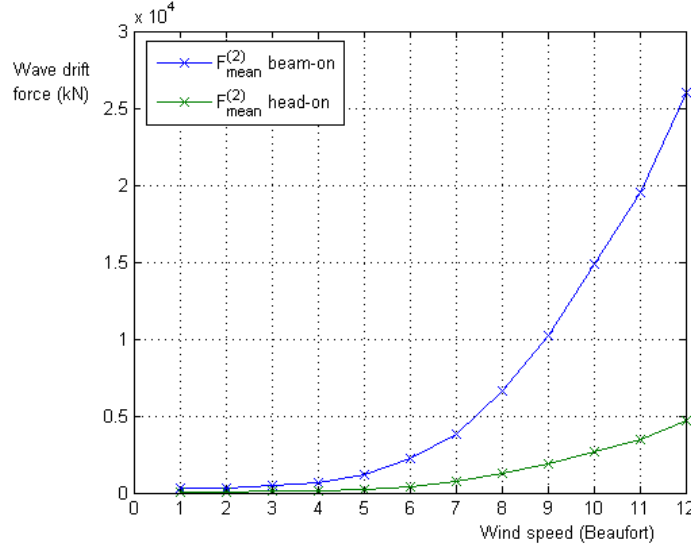


Figure F.13 Mean wave drift forces on the element in head-on and beam-on position

F.2.3 Low-frequency wave drift forces

The low frequency wave forces are caused by the differential frequencies that exist in the components of an irregular wave field and are oscillatory of nature. As with the mean wave drift force, the theory behind the low-frequency wave drift force is extensively discussed in [Journée and Massie, 2001] and will not be further treated in this report.

The low-frequency wave spectrum is defined as:

$$S_F(\mu) = 8 \int_0^{\infty} S_{\zeta}(\omega_1) S_{\zeta}(\omega_2) |T(\omega_1, \omega_2)|^2 d\omega \quad (\text{F.10})$$

Where:

$S_{\zeta}(\omega)$	: Wave spectrum	[m ² s]
$T(\omega_1, \omega_2)$	: Amplitude of the quadratic transfer function	[N/m ²]
μ	: Differential frequency, defined as $\omega_1 - \omega_2$	[rad/s]

With use of the expression for μ equation (F.10) can be rewritten to:

$$S_F(\mu) = 8 \int_0^{\infty} S_{\zeta}(\omega + \mu) S_{\zeta}(\omega) |T(\omega + \mu, \omega)|^2 d\omega \quad (\text{F.11})$$

As with the mean wave drift force two limiting cases can be defined:

1. The FB is in service, protecting the floating harbour area (SLS)
2. The FB is in survival condition (ULS)

For the calculation of the low-frequency wave drift forces, the same wave-spectrum will be used as has been used for the calculation of the mean wave drift force. A graphical representation of the two wave spectra can be found in figure F.12.

The amplitude of the quadratic transfer function has only been determined for a number of frequencies. Therefore equation (F.11) is rewritten to a summation:

$$S_F(\mu_m) = 8 \sum_{n=1}^n S_{\zeta}(\omega_n + \mu_m) S_{\zeta}(\omega_n) |T(\omega_n + \mu_m, \omega_n)|^2 d\omega \quad (\text{F.12})$$

This summation can be solved by using MATLAB; the code that has been used is given below.

```

omega      = 0.02:0.02:2.0;
mu         = 0.02:0.02:1.8;
delta_omega = 0.02;

%% Wave spectrum
H13 = 10.25;
T1  = 10.50;

Sw = zeros(1,length(omega));
for i = 1:length(omega)
    Sw(i) = 173 * (H13^2 / T1^4) * (omega(i))^5 ...
            * exp((-692/T1^4) * (omega(i))^4);
end

%% Low-frequency wave drift forces
for i=1:length(mu)
    Sf(i) = 0;
    for j = 1:(length(omega)-i)
        Sf(i) = Sf(i) + 8 * Sw(j) * Sw(j+i) ...
                * (abs(QTF(j+i,j)))^2 * delta_omega;
    end
end
end

```

Code MATLAB Calculation of the mean wave drift force

The spectrum that has been calculated is shown in figure F.14 for the SLS as well as for the ULS.

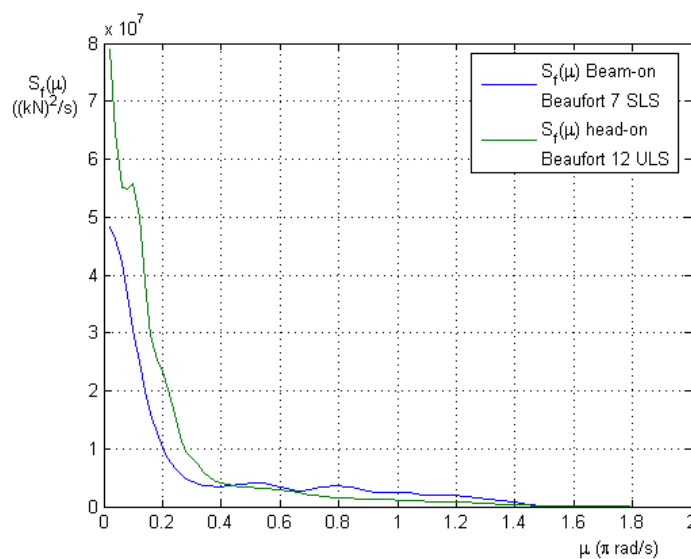


Figure F.14 Low frequency wave drift forces on the element in head-on and beam-on position

When on the spectrum shown figure F.14, the probabilistic approach is used the following design values for the low frequency drift force is obtained:

- Head-on position, wind speed Beaufort 12: 20750kN ;
- Beam-on position, wind speed Beaufort 7: 17650kN .

These values are an overestimation of the drift force because the effect of the mooring system is neglected. It is assumed static with no deflection possible from its equilibrium position. The mooring system can be modelled as a spring system with the FB element as a lumped mass. The spring characteristics and total system

is shown in figure F.15. In this model the FB is able to deviate from its equilibrium position, limiting the forces that have to be resisted.

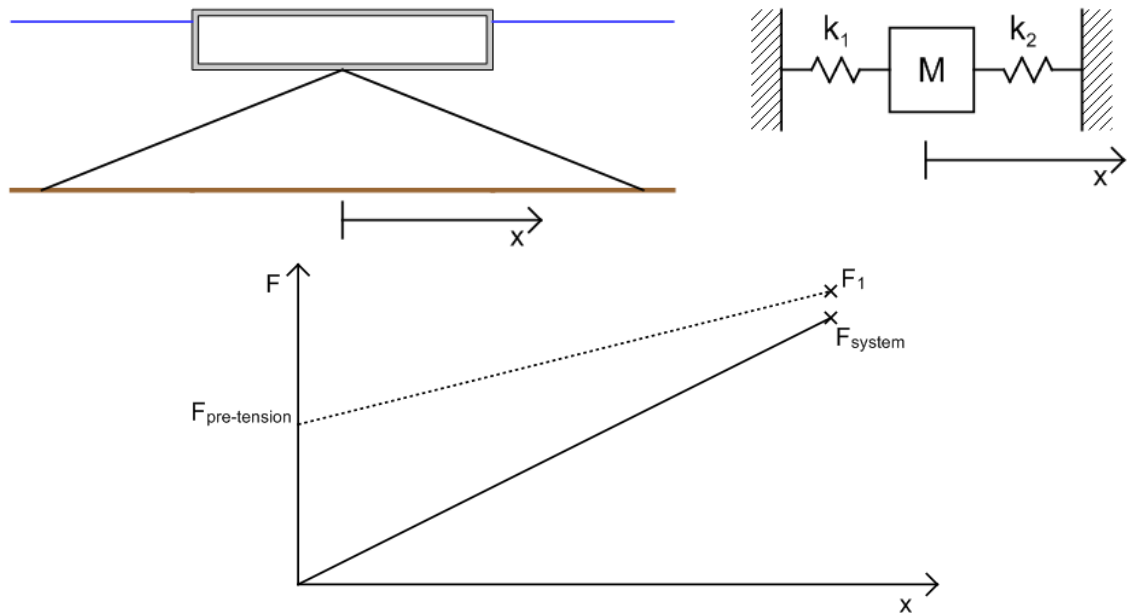


Figure F.15 The FB system and its spring characteristics

Besides the low frequency wave drift force the FB will be loaded by the following "static" forces:

- Mean wave drift force;
- (Tidal) current;
- Wind;

These forces will result in an offset from its original equilibrium position, $x = 0$, to a new equilibrium position around which the FB will oscillate, see figure F.16. Both the first order motions and the second order (low frequency drift) motions will have this new position as their equilibrium position.

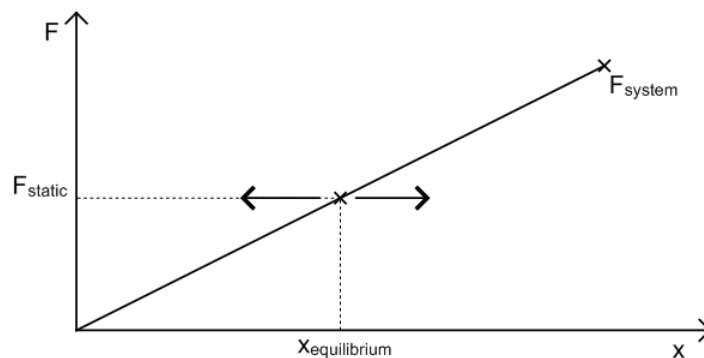


Figure F.16 The FB system and its spring characteristics

The first order motions will be in the order of the wave-height, which will be in the order of metres. This means that the first order motions hardly influence the mooring force, while the first order wave exciting force is significantly higher than the low frequency drift force.

In order to resist the low frequency drift force the mooring system should be flexible enough to accommodate the large amplitude of the low frequency motions. This means a low stiffness of the mooring system is required and thus a low natural frequency. This can lead to resonance which will aggravate the motions and thus the forces in the mooring lines.

It can be concluded that the low frequency mooring force depends on the response of the FB on the low frequency drift forces, which in their turn are depending on the mooring system. The determination of the mooring force is very complex and is regarded as out of scope for this preliminary design study.

To deal with the low frequency wave force it is assumed that the low frequency oscillatory force which acts on the mooring system is equal to halve the sum of the steady forces [Ali, 2005].

F.2.4 Current and wind drag

The forces due to current and wind drag can be calculated with:

$$F_{drag} = \frac{1}{2} \rho C_s A v^2 \quad (F.13)$$

Where:

- ρ : The density of the medium;
- C_s : Block coefficient, $C_s = 1$ for rectangular barges;
- A : The area of the FB element;
- v : The speed of the medium.

The current is assumed to be 1 m/s for SLS-conditions and 2 m/s for ULS conditions. The wind speed comes from the Beaufort scale. In this case it is 17 m/s for Beaufort 7 and 32 m/s for Beaufort 12. The parameters of the current and wind drag are summarised in table F-3. The resulting forces are also presented in this table.

Table F-3 Parameters governing current and wind drag forces and the resultant forces

	SLS	ULS
Currents		
ρ	1025 [kg/m ³]	1025 [kg/m ³]
A	10000 [m ²]	2000 [m ²]
Speed	1 [m/s]	2 [m/s]
Drag force	5125 [kN]	4100 [kN]
Wind		
ρ	1,25 [kg/m ³]	1,25 [kg/m ³]
A	5000 [m ²]	1000 [m ²]
Speed	17 [m/s]	32 [m/s]
Drag force	903 [kN]	640 [kN]
Total	6028 [kN]	4740 [kN]

F.2.5 Conclusion

When no current is present and the waves are assumed to be linear, the FB element will oscillate around an equilibrium position which does not move. In reality the equilibrium position will shift to a new position when no mooring system is present. This is due to:

- the mean drift forces;
- the low frequency drift forces;
- the (tidal) currents;
- and the wind drag.

These forces have been calculated in the previous sections and are presented in table F-4. It can be noted that the drift and drag forces in SLS and ULS do have the same order of magnitude due to the ability of the FB element to weathervane in storm conditions.

Table F-4 Drift and drag forces working on the FB element

	SLS	ULS
Wave direction	Beam-on	Head-on
Spectrum	Bretschneider	Bretschneider
Magnitude	Beaufort 7	Beaufort 12
Drift forces		
Mean wave drift force	4800 [kN]	4700 [kN]
Low frequency drift force	5414 [kN]	4720 [kN]
Currents		
Speed	1 [m/s]	2 [m/s]
Drag force	5125 [kN]	4100 [kN]
Wind		
Speed	17 [m/s]	32 [m/s]
Drag force	903 [kN]	640 [kN]
Total	16242 [kN]	14160 [kN]

F.3 Internal moments

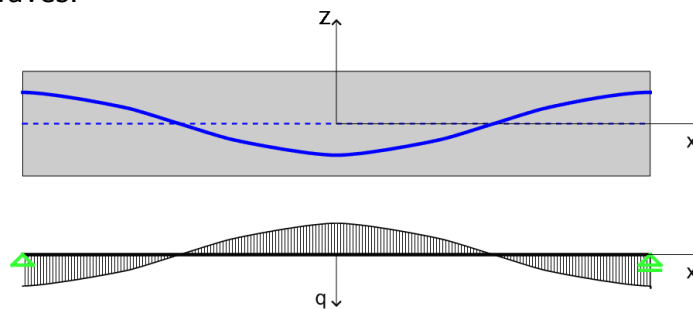
Waves are causing pressures but will also cause bending moments in the floating structure. These bending moments are working on the complete structure, and not only on a certain member, distinguishing these bending moments from the local bending moments induced by own weight, pressures on panels, etc. The internal moments can be calculated in three ways:

- Froude-Krilov;
- Froude-Krilov with reduced wave-elevations;
- DELFRAC design pressures.

The first method is the simplest and will be explained first. Later the shortcomings of this method are discussed and the other two methods are compared and proposed to overcome the possible problems.

F.3.1 Froude-Krilov

In figure F.17 the mechanic schematisation of the FB loaded with a sinusoidal wave is shown. This sinusoidal load will cause a moment in the FB that is here represented as a beam that has no draught and is assumed to be small with respect to the waves.

**Figure F.17** Mechanical schematisation of a beam loaded by sinusoidal waves.

The moment in a beam which is loaded with a distributed load can be determined with the following formula:

$$\frac{d^2 M}{dx^2} + q = 0 \quad (\text{F.14})$$

Where:

- M : The moment in the beam;
- q : The distributed load, defined as $q = q_0 \cos(kx)$, in which q_0 denotes the distributed load amplitude.

When (F.14) is integrated twice the formula for the bending moment is obtained:

$$M = \frac{1}{k^2} q_0 \cos(kx) + Ax + B \quad (\text{F.15})$$

Where:

A, B : Integration constants;

The integration constant can be obtained from the boundary conditions:

$$\begin{aligned} x = -\frac{1}{2}L_w &\rightarrow M = 0 \\ x = \frac{1}{2}L_w &\rightarrow M = 0 \end{aligned} \quad (\text{F.16})$$

When the boundary conditions of (F.16) are applied to equation (F.15) the moment in the beam can be calculated for any wave length:

$$M = \frac{1}{k^2} q_0 \cos(kx) - \frac{1}{k^2} q_0 \cos\left(k \frac{1}{2}L_w\right) \quad (\text{F.17})$$

Where:

q_0 : The amplitude of the distributed load, $q_0 = \rho g \zeta_a$;

k : The wave number, $k = \frac{2\pi}{L} = \frac{\omega^2}{g}$;

L_w : The length of the FB element.

The maximum moment at $x=0$ will occur when the wave length is twice the FB element length. In this case (F.17) will reduce to:

$$\begin{aligned} M_{\max; x=0} &= \frac{1}{k^2} q_0 \cos(0) - \frac{1}{k^2} q_0 \cos\left(\frac{1}{2}\pi\right) \\ &= \frac{1}{k^2} q_0 - 0 \\ &= \frac{L_w^2 q_0}{\pi^2} \end{aligned} \quad (\text{F.18})$$

The maximum value for the bending moment as described in (F.18) can be found in various literature sources and formula (F.17) is assumed to be correct. With this equation a transfer function can be obtained which can be used to translate waves into global bending moments:

$$\begin{aligned} \left| \frac{M_\zeta}{\zeta_a}(\omega) \right| &= \frac{M_{\zeta, \max}(\omega)}{\zeta_a} \\ &= \frac{\frac{1}{k^2} q_0 - \frac{1}{k^2} q_0 \cos\left(k \frac{1}{2}L_w\right)}{\zeta_a} \end{aligned} \quad (\text{F.19})$$

$$\left| \frac{M_{\zeta}}{\zeta_a}(\omega) \right| = \frac{\frac{g}{\omega^2} \rho g \zeta_a - \frac{g}{\omega^2} \rho g \zeta_a \cos\left(\frac{\omega^2}{g} \frac{1}{2} L_w\right)}{\zeta_a} \quad \text{continued (F.19)}$$

$$= \rho \frac{g^2}{\omega^2} \left(1 - \cos\left(\frac{\omega^2}{g} \frac{1}{2} L_w\right) \right)$$

Where:

$\left \frac{M_{\zeta}}{\zeta_a}(\omega) \right $	: The transfer coefficient for the maximum bending moment;
ρ	: The density of water, 1025 kg/m ³ ;
g	: Gravitational acceleration, 9.81m/s ²
ω	: The circular wave frequency;
L_w	: The length of the FB element.

The transfer coefficient for the global bending moment is shown in figure F.18.

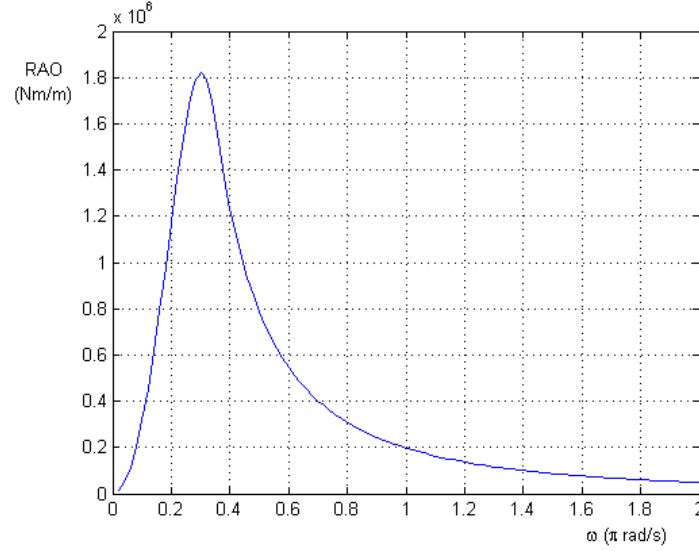


Figure F.18 The transfer coefficient for the global bending moment in Nm/m

F.3.2 Moment spectrum and design moment

Analogous to the determination of a pressure-spectrum the moment spectrum can be obtained by combining the wave-spectrum and the moment transfer function. Analogous to (F.7):

$$S_{M,T}(\omega) d\omega = \left| \frac{M_{\zeta}}{\zeta_a}(\omega) \right|^2 S_{\zeta}(\omega) d\omega \quad (\text{F.20})$$

Where:

$S_{M,T}(\omega) d\omega$	: The moment spectrum;
$\left \frac{M_{\zeta}}{\zeta_a}(\omega) \right $	: The transmission coefficient for the bending moment;
$S_{\zeta}(\omega) d\omega$	: The Bretschneider wave spectrum;

The moment spectrum for an FB element loaded with head-on waves during a storm with a magnitude of 12 on the scale of Beaufort (Bretschneider wave-spectrum) is given in figure F.19.

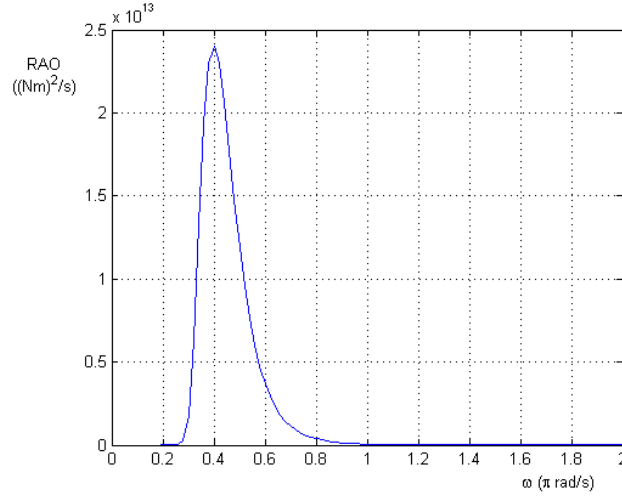


Figure F.19 The transfer coefficient for the global bending moment in Nm/m

From this moment spectrum the design moment can be calculated, analogous to the method used to calculate the design pressure on the hull. This method has been explained in F.1.2.

This design moment that has to be taken up by the connections of the floating breakwater is $12.1 \cdot 10^6 \text{ Nm}$ or 12.1 MNm per metre width of the FB element. It is estimated a pre-stressing force of 900 kN per metre width is required to avoid tensile stresses in the concrete due to this moment.

F.3.3 Froude-Krilov with wave reduction

The design moment calculated in the previous section will be an overestimation because the draught of the element was neglected. The draught of the element is 20 metres and is a dimension that ultimately cannot be neglected. The wave pressures are decaying with the depth, resulting in reduced wave pressures and thus in a reduced bending moment in the elements hull. The method of reducing the wave pressures is explained in Appendix B.5.1 where it was used in the ReFBreak-model to calculate the heave-motion.

The reduced wave pressure on the bottom can be found by:

$$q_{0;FK} = e^{-kT_1} \rho g \zeta_a \quad (\text{F.21})$$

Where:

k	: Wave number	[rad/m]
T_1	: Draught	[m]
e^{-kT_1}	: Reduced wave elevation	[-]
ζ_a	: Wave height	[m]

The internal moment can now be found by using equation (F.18), but now the reduced wave elevation is used:

$$M_{\max;x=0} = \frac{L_w^2 q_{0;FK}}{\pi^2} \quad (\text{F.22})$$

The moment spectrum can be found with equation (F.19):

$$\left| \frac{M_{\zeta}}{\zeta_a}(\omega) \right| = \rho \frac{g^2}{\omega^2} \left(1 - \cos \left(\frac{\omega^2}{g} \frac{1}{2} L_w \right) \right) \quad (\text{F.23})$$

The effect of the draught is shown in figure F.20, where the moment spectra of the original and the reduced pressure method are compared.

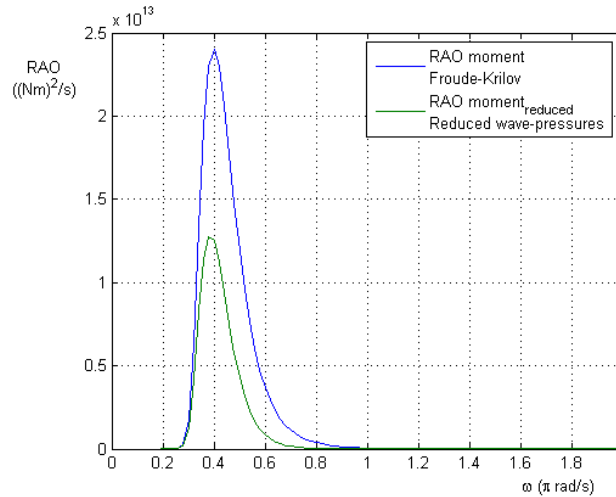


Figure F.20 The transfer coefficient for the global bending moment in Nm/m when reduced wave pressures are used

F.3.4 Design moment using DELFRAC

The design moments calculated with the Froude-Krilov assumption are not accurate because of the assumptions that have been used:

- The element length is assumed to be small compared to the wave-length.

The element length was assumed to be small, while it was concluded that the most severe bending moment acts on the structure when the wave-length is of the same order as the element length. The Froude-Krilov assumption will not be valid as no diffraction effects have been taken in account.

- The motions of the element have been neglected.

A lot of load components contribute to dynamic forces and moments in a hull; the motion components as well as the direct hydro-mechanical and wave exciting forces and moments contribute directly and sometimes in more than one way.

The dynamics can be investigated using the principle of dynamic equilibrium, using Newton's second law. This method will be explained in this section and it should be used in future research. For the preliminary design this method has not been used.

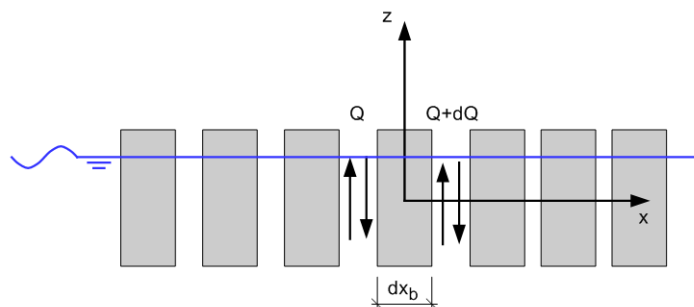


Figure F.21 The considered section of element.

Consider the vertical forces acting on a section of the ship with a length dx_b , see also figure F.21. According to Newton's second law it holds:

$$F = m \cdot a \quad (F.24)$$

Where:

F	: Force acting on the section	[rad/m]
m	: Mass of the considered section	[rad/m]
a	: Local acceleration	[m]

The forces that act on the section are the hydro-mechanical and wave forces [Journée and Massie, 2001]. The hydro-mechanic forces and moments are induced by the harmonic oscillations of the rigid body in the undisturbed surface of the fluid. The wave exciting forces and moments are produced by the waves coming in on the restraint body. This is explained in Appendix C.2 where the basis of the ReFBreak2D-model is discussed.

The acceleration of the section is only considered in the z-direction and consists of two contributions. The first contribution is the acceleration due to the heave motion; the second contribution is the acceleration due to the pitch motion times the lever arm. The total rusting force on a section due to the acceleration, with respect to the governing axes convention, is:

$$\begin{aligned} F(x_b) &= -m'(x_b) \{ \ddot{z} - x_b \ddot{\theta} \} \\ &= \omega^2 \cdot m'(x_b) \{ z \cos(\omega t + \varepsilon_{\zeta,3}) - x_b \theta \cos(\omega t + \varepsilon_{\zeta,5}) \} \end{aligned} \quad (F.25)$$

Where:

$m'(x_b)$	: Mass of the considered section	[kg]
z, θ	: Amplitudes of the motions	[m, rad]
$\varepsilon_{\zeta,3}, \varepsilon_{\zeta,5}$	: Phase-change of the motions	[rad]
ω	: Wave frequency	[rad/s]

When the length of the considered section, dx_b , is taken infinitely small the forces action on a section of the element can be translated into a distributed load:

$$q_z(x_b) = X'_{hz}(x_b) + X'_{wz}(x_b) - m'(x_b) \{ \ddot{z} - x_b \ddot{\theta} \} \quad (F.26)$$

Where:

$q_z(x_b)$	: The distributed load	[N/m]
$X'_{hz}(x_b)$	: The hydro-mechanical force	[N/m]
$X'_{wz}(x_b)$	: The wave force	[N/m]
$m'(x_b)$	: Mass of the considered location	[kg/m]
$\ddot{z}, \ddot{\theta}$	: The acceleration of the element	[m/s ² , rad/s ²]

The contributions of distributed load due to the hydro-mechanical force and the wave force can be found by using the pressures on the panels given by DELFRAC. The pressures on the panels are given for each wave frequency in the file "PRESSAA.OUT". Also the phase changes relative to the oncoming waves at the COG are given in this file.

The acceleration can not be obtained directly from the DELFRAC output. The amplitudes of the motions can be used to calculate the corresponding acceleration.

In order to obtain the time dependent distributed load equation (F.26) can be rewritten to:

$$q_z(x_b) = X'_{tot}(x_b) \cos(\omega t + \varepsilon_{\zeta, X}(x_b)) + \omega^2 m'(x_b) \{ z_a \cos(\omega t + \varepsilon_{\zeta, 3}) - x_b \theta_a \cos(\omega t + \varepsilon_{\zeta, 5}) \} \quad (F.27)$$

Where:

$q_z(x_b)$	: The distributed load	[N/m]
$X'_{tot}(x_b)$	: The pressures from the output of DELFRAC	[N/m]
$\varepsilon_{\zeta, X}(x_b)$	: The phase changes of the pressures at x_b	[N/m]
ω	: The wave frequency	[rad/s]
$m'(x_b)$	: Mass of the considered location	[kg/m]
z_a, θ_a	: The amplitude of the motions	[m, rad]

From the distributed load the shear force in elements hull can be found with [Journée and Massie, 2001]:

$$Q_z(x_1) = Q_{za} \cos(\omega t + \varepsilon_{\zeta Q_z}) = - \int_{stern}^{x_1} q_z(x_b) dx_b \quad (F.28)$$

Where:

Q_{za}	: The amplitude of the shear force	[N/m]
$\varepsilon_{\zeta Q_z}$	: The phase change of the shear force	[N/m]

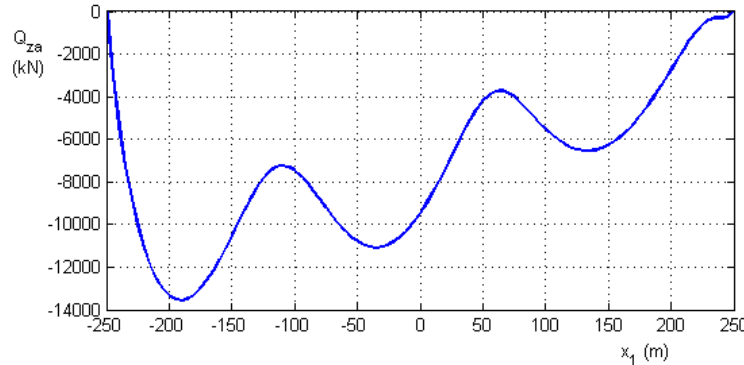


Figure F.22 The amplitude of the shear force, Q_{za} , over the length of the element. The wave period is 10.4 seconds, the wave length is 170 metres.

The pressure in the x-direction can result in a bending moment. Therefore the distributed load in the x-direction has to be known as well. Similar to the distributed load in the z-direction the load in the x-direction can be found [Journée and Massie, 2001]:

$$q_x(x_b) = +X'_{hx}(x_b) + X'_{wx}(x_b) - m'(x_{xz}) \{ \ddot{x} - \overline{bG}(x_b) \ddot{\theta} \} \quad (F.29)$$

Where:

$q_x(x_b)$	: The distributed load in the x-direction	[N/m]
$X'_{hx}(x_b)$	: The hydro-mechanical force	[N/m]

$X'_{wx}(x_b)$	: The wave force	[N/m]
$m'(x_b)$	: Mass of the considered location	[kg/m]
$\overline{bG}(x_b)$	: The distance of the centroid of the cross section to the x_b -axis.	[m]
$\ddot{z}, \ddot{\theta}$	: The acceleration of the element	[m/s ² , rad/s ²]

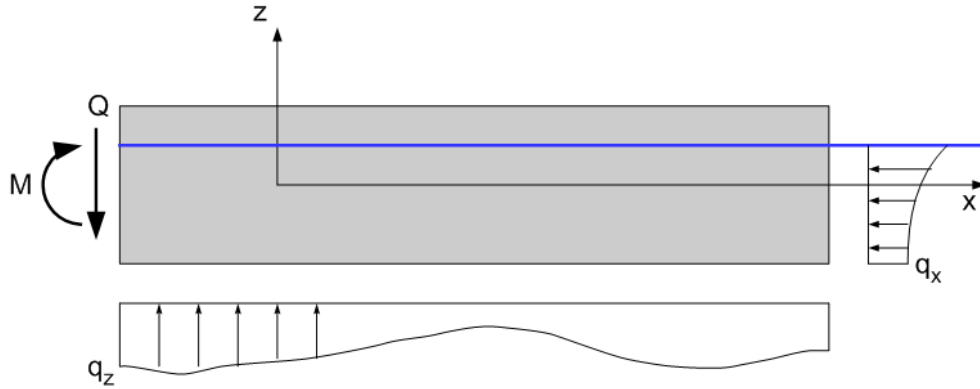


Figure F.23 Equilibrium of the beam

The moments in the element can be calculated by considering the equilibrium of a cross-section, see figure F.23. The total bending moment is defined as [Journée and Massie, 2001]:

$$\begin{aligned}
 M_y(x_1) &= M_{ya} \cos(\omega t + \varepsilon_{\zeta M_y}) \\
 &= + \int_{stern}^{x_1} q_x \cdot \overline{bG}(x_b) dx_b + \int_{stern}^{x_1} q_z \cdot x_b \cdot (x_b) dx_b + Q_z(x_1) \cdot x_1
 \end{aligned}
 \quad (F.30)$$

Where:

M_{ya}	: The amplitude of the shear force	[N/m]
$\varepsilon_{\zeta M_y}$	: The phase change of the shear force	[N/m]

A MATLAB routine has been written to calculate the amplitude of the bending moment. Unfortunately the results from the routine do not fulfil the boundary conditions, i.e. the routine is not correct and the results are unreliable. The moment at the bow and stern of the FB should be zero, but the calculation does not yield in this expected behaviour. Therefore this method is not used in this report, but it should be included in future research.

F.3.5 Conclusion

Three methods have been used to estimate the design global bending moments:

- Froude-Krilov;
- Froude-Krilov with reduced wave-elevations;
- DELFRAC design pressures.

The results of these calculations are summarised in table F-5 below.

Table F-5 Design moments according to various methods

Method	Design moment per metre width	Total design moment
Froude-Krilov	12,1 [MNm/m]	1210 [MNm]
Froude-Krilov with reduced wave-elevation	8,1 [MNm/m]	810 [MNm]
DELFRAC	N/A [MNm/m]	N/A [MNm]

The dynamic moments can be investigated using the principle of dynamic equilibrium, using Newton's second law. This method is explained in F.3.4 and it should be used in future research. For the preliminary design this method has not been used.

The Froude-Krilov also overestimates the bending moments as the presence of the structure is neglected. The wave pressures are exponentially decaying with the depth. When this is taken into account the design global bending moment is $8.1 \cdot 10^6 \text{ Nm}$ or 8.1 MNm per metre width of the FB element. This value will be used for elaborating on the design of the elements.

F.4 References

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Appendix G Structural design

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Appendix G Structural design

G.1 Introduction

It is virtually impossible to build the FB element as one piece; therefore it is divided into sections. The number of different sections should be minimized to get a better repetition factor. A high repetition factor will lead to shorter construction time, reusable formwork and reduced material consumption, and thus to a more economic design of the floating breakwater.

The structural design presented in this appendix is a preliminary design and has the purpose to assess the main dimensions of the details of the FB element, such as, but not limited to:

- the subsequent loading of the hull during the construction process;
- the required thickness of the hull at various places;
- the required amount of pre-tensioning to prevent cracking of the concrete.

First a short list of the types of elements will be given, each with their relative place in the FB element. Then for each of the sections the construction phases will be listed with the possible loads that will occur during that construction phase. Finally a short description of the possible failure mechanisms will be given together with an evaluation to point out the decisive mechanism.

G.1.1 Types of sections

The floating breakwater will be divided into seven sections, each 70 metres long, 100 metres wide and 30 metres high, see figure G.1. The height of the sections is the design draught of the FB element together with the required freeboard. Exceptions to these dimensions are the bow and the stern sections, which are each 75 metres long. The extra 5 metres is available for the structure that has to cope with the extreme wave pressures during the Beaufort 12 storm condition.

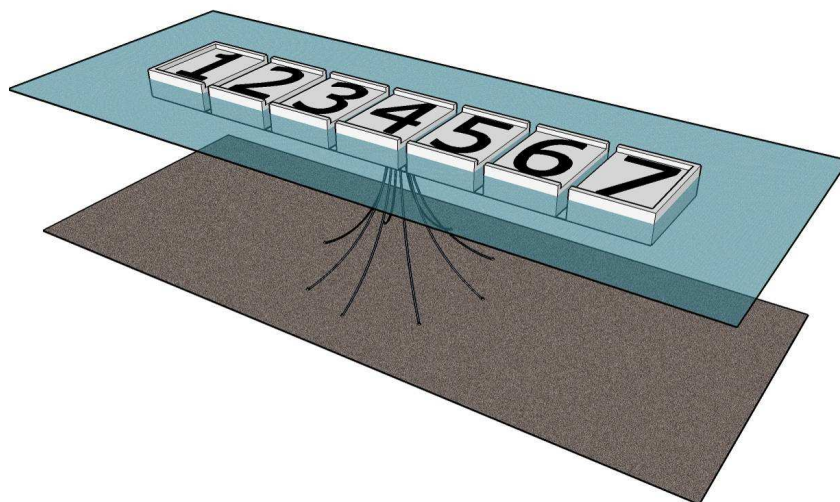


Figure G.1 The different types of sections

The types of elements that can be distinguished are:

- Normal elements

The normal sections are the sections numbered 2, 3, 5 and 6 in figure G.1. These elements are always coupled to two other elements, removing the need to design the bulkheads to withstand the extreme wave loads.

- Turret mooring section

The turret mooring section is the section numbered 4 in figure G.1 and is located in the middle of the FB section. The design of this section is similar to the normal section, except for the space that is required for the turret mooring system.

- Bow and stern section

The bow/stern sections are numbered 1 and 7 in figure G.1. The section is only coupled on one side to another section. The other side of the section is exposed to wave-attack when the FB element is attacked by head-on waves and has to be dimensioned for the occurring pressures. In Appendix F it was also shown that this element needs a stronger bottom and sides than a normal section due to the higher wave induced pressures.

The whole element, and thus every section, is divided into separate compartments. Because of the function and the location of the structure the likelihood is high of the structure having to withstand a collision with a ship. Although concrete is very resilient, one or more compartments may be damaged in a collision, resulting in the flooding of that compartment. When a compartment of the FB element is damaged it must not lead to the loss of the function or to a complete loss of the FB element.

The element has a double hull, similar to modern oil carriers. The function of the double hull on oil carriers is to limit the oil spill in the unfortunate case of a collision or when the ship is run aground. The function of the double hull on the FB element is different from that one on an oil carrier. The double hull has to retain water in the outer compartments when these are flooded due to an unfortunate event. The small size of the outer compartments leads to a reduced impact of the flooding of a compartment on the stability and location of the COG and thus to a reduced impact on the function of the FB element.

G.1.2 Construction phases

Although there are three different sections, the design of these sections will be similar as well as the construction method. The construction phases are:

- Construction of the segments;

The bottom part of the floating breakwater will be constructed in a dry-dock. The displacement of the bottom part of the FB section has to be large enough to support the weight, i.e. it has to float.

During this construction phase the bottom slab will be pre-stressed to avoid tensile stresses in the concrete during use. The pre-stress has to be designed that no tensile stresses will occur when the element is not loaded with the hydrostatic pressure as is the case in the dry-dock.

The dry-dock will be flooded, or the finished bottom part will be launched from a slipway. It has to be towed to a quay-wall where the sides of the sections will be casted. For the transport of the unfinished element it is assumed that the quay-wall is located in the same harbour at a sheltered area. When the bottom part has to be transported over sea to another finishing location it has to be checked whether the pre-stress is sufficient.

The element will be finished along a quay-wall to make room in the dry-dock for the following section, making a faster construction scheme possible. When the sides are poured the weight of the element will not be divided over its surface equally, leading to a bending moment in the bottom slab.

- Joining the segments to an element;

The sections are mated in a sheltered area. To guarantee a perfect alignment between the elements a guidance structure must be used. A rubber sealing, similar to the Gina-profile as used in the Dutch immersed tunnels, will provide a watertight connection. The joints will be pre-stressed, ensuring that no tensile stresses will occur during the use of the element.

- Transport and installation

The complete element is towed to its installation site where it is ballasted and installed. The mooring system may be pre-installed, making the installation of the element easier.

Some of construction phases need extra attention. These phases are:

- Finishing along the quay-wall, due to the bending moments in the bottom slab;
- Mating the sections. The bulkheads between the sections have to withstand the water pressure during the use when the seal is leaking. Furthermore the sealing and connection itself have to be detailed;
- Installation and ballasting, while applying ballast the position of the COG has to be ensured. Furthermore, longitudinal bending moments have to be expected when the ballast is not evenly distributed over the length of the element.

G.1.3 Failure mechanism

There are five sources of forces acting on the structure; these were all discussed in Appendix F. Each of them can induce a failure mechanism:

- Construction;

During construction bending moments may exist in the bottom slab causing cracking during construction.

- Waves during service combined with a collision;

In SLS with waves beam-on and an increased draught the pressures may cause tensile stresses in the hull shell.

- Extreme waves;

In ULS with waves head-on the pressures may cause tensile stresses and thus cracks in the hull.

- Drift and drag forces;

The drift and drag forces may cause the mooring system to fail and set the FB element a drift, with possible damage to surrounding structures and ships.

- Global moments.

Global moments may break the element or may cause the connections between separate sections to fail. The separate sections may sink or cause damage in the harbour area.

In table G-1 the decisive load cases are given for the different components of the FB element. In the further design of the element the loads that have been discussed will be used to determine the required cross-sections and strength.

Table G-1 Load cases on the different structural components

Component	Location (longitudinal)	Decisive load case	Check needed
Bottom	-180 metre to 180 metre	Extreme waves	Construction
Bottom bow/stern	-250 metre to -180 metre and 180 metre to 250 metre	Extreme waves	Construction
Side	-180 metre to 180 metre	Waves during service combined with a collision	
Side bow/stern	-250 metre to -180 metre and 180 metre to 250 metre	Extreme waves	
Bow/stern	At -250 metre		
Connections		Global wave induced moments	
Turret mooring	At 0 metre	Drift forces	

G.2 Pre-stressing

When pre-stress is applied to a section three requirements need to be fulfilled:

- During full load no tensile stresses may be present in the section;
- During full load the compressive strength of the concrete may not be exceeded;
- During construction, or when the structure is minimal loaded, no tensile stresses may occur in the section.

The first requirement dictates the minimum required amount of pre-stress, while the second requirement dictates the maximum amount. When the moment capacity of the structure is not sufficient, or the minimum required amount of pre-stress is higher than the maximum, the dimensions have to be adapted. The third requirement does not determine a maximum or minimum pre-stress force; it only puts limitation on the placement of the pre-stressing system.

It should be noted that tensile force are denoted with a plus-sign in this section, while compressive forces are denoted with a minus-sign.

G.2.1 Minimum pre-stressing force

Moments have to be absorbed in the concrete without tensile stresses forming cracks. When the maximum tensile stress in concrete is assumed to be zero, which is a safe assumption as concrete will have some tensile capacity, the moment that can be absorbed, when pre-stressing force is applied, can be found with:

$$\frac{\frac{1}{2}h \cdot M}{I_x} + \frac{N}{A} \leq 0 \quad (G.1)$$

Where:

- M : The occurring moment;
 I_x : The section area moment of inertia;
 N : The occurring normal force;
 A : The section area.

The normal force in the concrete area may come from pre-stress and from the hydrostatic pressures. The maximum moment in the member, without forming cracks, can be found by rewriting equation (G.1) :

$$M \leq -\frac{\frac{N}{A} I_x}{\frac{1}{2}h} \quad (G.2)$$

An increase in the compressive force in the member leads to a higher bending moment that can be absorbed, showing the relevance of pre-stressing. Equation (G.2) is only valid when the compressive force is centred in the neutral line of the section. When the compressive force is not acting along the neutral line there will be an additional internal bending moment in the member, see figure G.2.

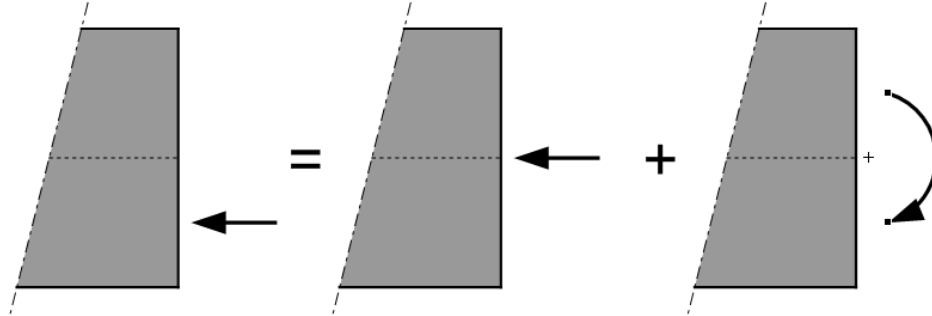


Figure G.2 An eccentric normal force will lead to an bending moment

When this additional moment is taken into consideration, the maximum moment, as has been described with equation (G.2) becomes:

$$M \leq -\frac{N}{\frac{1}{2}h} \frac{I_x}{A} + Ne_p \quad (\text{G.3})$$

Where:

e_p : The eccentricity of the normal force;

When the pre-stress system is eccentrically placed in the section an additional moment can be absorbed, see (G.3). Equation (G.3) can be rewritten to obtain the minimal required pre-stress directly:

$$N_{\min} \leq \frac{-\frac{I_x}{\frac{1}{2}h \cdot A} + e_p}{M} \quad (\text{G.4})$$

G.2.2 Maximum pre-stressing force

Analogously to the previous section, the maximum pre-stressing force in the concrete can be determined. The maximum compressive force in the concrete depends on the chosen quality and is here denoted as f_b . The maximum tension in the section can be found with:

$$\frac{\frac{1}{2}h \cdot M}{I_x} + \frac{N}{A} = \sigma \geq f_b \quad (\text{G.5})$$

Where:

M : The occurring moment;
 I_x : The section area moment of inertia;
 N : The occurring normal force;
 A : The section area.

When it is assumed that the normal force in the section is eccentric an additional moment has to be added:

$$-\frac{\frac{1}{2}h \cdot M}{I_x} + \frac{N}{A} + \frac{\frac{1}{2}h \cdot N e_p}{I_x} = \sigma \geq f_b \quad (G.6)$$

Where:

e_p : The eccentricity of the normal force;

The maximum pre-stressing force now becomes:

$$\begin{aligned} -\frac{\frac{1}{2}h \cdot M}{I_x} + \frac{N}{A} + \frac{\frac{1}{2}h \cdot N e_p}{I_x} &= \sigma \geq f \\ -\frac{\frac{1}{2}h \cdot M}{I_x} + \left(\frac{1}{A} + \frac{\frac{1}{2}h \cdot e_p}{I_x} \right) N &= f \\ N &= \frac{f_b}{A} + \frac{\frac{1}{2}h \cdot e_p \cdot f_b}{I_x} + \frac{\frac{1}{2}h \cdot M}{I_x} \end{aligned} \quad (G.7)$$

G.2.3 Location of the pre-stressing force

When the section is not loaded no tensile stresses may occur due to the pre-stressing force. When the pre-stress is not eccentric in the section there will not be any tensile forces and therefore no problem. However, in the previous sections it has been shown that an eccentric pre-stress tendon is beneficial to the moment capacity of the member. The requirement that has to be fulfilled is:

$$\frac{\frac{1}{2}h \cdot N e_p}{I_x} + \frac{N}{A} \leq 0 \quad (G.8)$$

Where:

e_p : The eccentricity of the normal force;

I_x : The section area moment of inertia;

N : The occurring normal force;

A : The section area.

Equation (G.8) can be rewritten to:

$$e_p \leq -\frac{1}{A} \cdot \frac{I_x}{\frac{1}{2}h} \quad (G.9)$$

This shows that in the case of no hydrodynamic or hydrostatic pressures, i.e. when the structure is not loaded, not the extent of the pre-stressing force is of any importance, but the location. When the section is assumed to be rectangular, which plates and slabs generally are, the area moment and the area are defined as:

$$A = bh \quad (G.10)$$

$$I_x = \frac{1}{12}bh^3 \quad (G.11)$$

When equations (G.10) and (G.11) are substituted in (G.9) the outer limits of the location of the normal force in the section are obtained:

$$e_p \leq \frac{1}{6}h \quad (G.12)$$

The area in which a compressive force may act without causing tensile forces in the member is called the "centre of the section". See figure G.3 for the centre of the section for a rectangular beam.

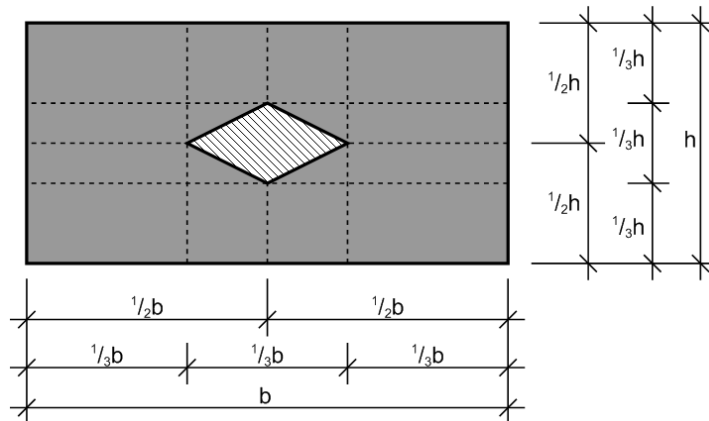


Figure G.3 Centre of the section for a rectangular beam

G.2.4 Conclusion

The design of the concrete members of the section can be obtained by applying the formulas derived in the previous sections, see also table G-2. These formulas will be used in the spreadsheets used to calculate the dimensions of the sections in the following sections.

Table G-2 Formulas to dimension pre-stressing properties

Requirement	Variable	Formula
Full load, no tension	$F_{p,min}$	$F_{p,max} = \frac{f_b}{A} + \frac{\frac{1}{2}h \cdot e_p \cdot f_b}{I_x} + \frac{\frac{1}{2}h \cdot M}{I_x}$
Full load, no exceedance of compressive strength	$F_{p,max}$	$F_{p,min} = \frac{-\frac{I_x}{\frac{1}{2}h \cdot A} + e_p}{M}$
No load, no tension	e_p	$e_p \leq \frac{1}{6}h$

The found relations between minimum pre-tension force, maximum pre-tension force and the location can also be obtained graphically, which has been done in figure G.4.

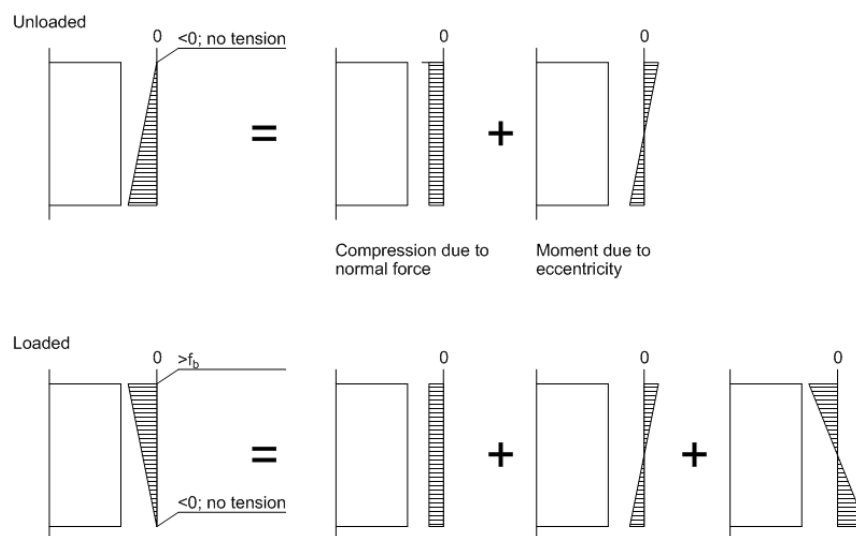


Figure G.4 Graphical representation of the principle of pre-stressing members

G.3 Normal sections

Normal sections will be completely elaborated with paying attention to the various load cases and critical construction phases. It is this section, which is the basis of the design of the turret mooring section and the bow/stern section. The formulas used to calculate the bending moments and corresponding stresses are explained in this section and will not be repeated in the calculation of the other sections.

G.3.1 Bottom

The extreme conditions for the bottom slab of a section are the extreme wave induced pressures from a storm with an average return period of 100 year. This storm is assumed to have a magnitude of 12 on the scale of Beaufort and the waves will be received head-on, see also Appendix F.1.5. Besides, the bottom slab is also loaded by the hydrostatic pressures and its own weight, although the last one is a positive contribution. These distributed loads on the structure are summarised in table G-3.

Table G-3 The distributed loads on the bottom slab of the floating breakwater

Load - Distributed loads	p	γ	p_d
Hydrostatic load	201 kN/m ²	1	201 kN/m ²
Waterdepth 20 meter			
Rho 1025 kg/m ³			
Wave load	30 kN/m ²	1	30 kN/m ²
Own weight	14,4 kN/m ²	0,9	13 kN/m ²
Thickness 0,6 meter			
Rho 2400 kg/m ³			
Total			218 kN/m ²

The bottom slab is divided into compartments to reduce the moments. There are two alternative designs, see table G-4 and figure G.5.

Table G-4 The two designs that will be elaborated

Dimensions	Design 1	Design 2
Width (y-direction)	14 m	35 m
Length (x-direction)	14 m	14 m
L_y/L_x	1 [-]	2,5 [-]
Concrete quality B75	45 N/mm ²	45 N/mm ²
Minimum cover	150 mm	150 mm

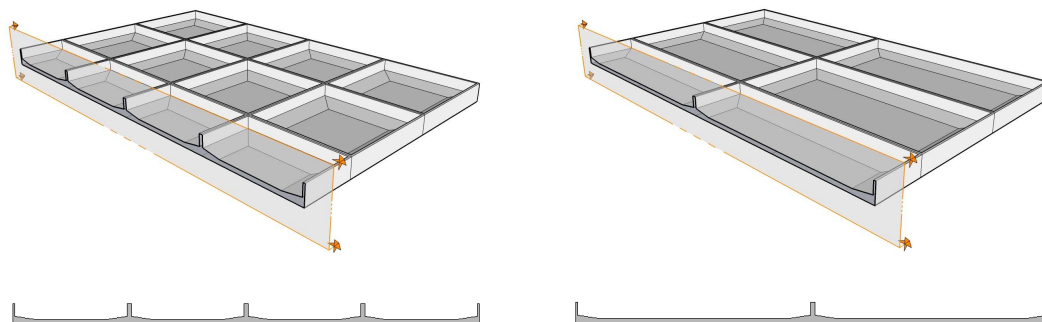


Figure G.5 Design 1 with 14x14 metres cells (left) and design 2 with 14x35 metres cells (right)

The moment in both directions, i.e. x- and y-direction can be found by using the coefficients found in figure G.6, adapted from the Dutch NEN6720 code for concrete structures.

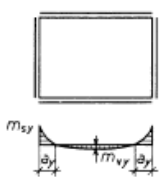
l_y/l_x		1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0
II									
	$m_{vx} = 0,001 p_d l_x^2$	18	26	32	36	39	41	42	43
	$m_{vy} = 0,001 p_d l_y^2$	18	16	12	10	10	10	10	10
	$m_{sx} = -0,001 p_d l_x^2$	51	63	72	78	81	82	83	83
	$m_{sy} = -0,001 p_d l_y^2$	51	54	55	54	54	53	51	49
	$a_x / l_x =$ $a_y / l_y =$	0,18 0,18	0,19 0,16	0,19 0,15	0,19 0,13	0,19 0,11	0,19 0,10	0,19 0,07	0,19 0,06

Figure G.6 Coefficients to determine the field and support bending moment according to NEN6720

Table G-5 lists the moments that are found. The bending moments may not cause tensile stresses in the concrete as has been mentioned before. Therefore the complete structure will be pre-tensioned to withstand these moments.

Table G-5 The field and support bending moments for design 1 and 2

Load - Moments	Design 1		Design 2	
	Factor	Moment	Factor	Moment
Field moment				
M_{fx}	18 [-]	769,6 kNm/m	42 [-]	1795,8 kNm/m
M_{fy}	18 [-]	769,6 kNm/m	10 [-]	427,6 kNm/m
Support moment				
M_{sx}	51 [-]	2180,6 kNm/m	83 [-]	3548,8 kNm/m
M_{sy}	51 [-]	2180,6 kNm/m	51 [-]	2180,6 kNm/m

From table G-6 and table G-7 it can be seen that a smaller cell-size leads to a smaller required pre-stressing force and also to a smaller cross-section of the bottom slab. On the other hand there are more bulkheads and formwork is required when the cells are smaller.

It has been chosen to have a pre-stressing force that is constant over the width of the cell, to avoid anchors in the cells. Anchors are relative difficult details to fabricate, requiring lot of rebar to withstand the splitting forces caused by the introduction of the high normal forces. A constant normal force will lead to an increase of the thickness of the bottom slab in the field section; else the maximum pre-stress force should have been exceeded.

Table G-6 The required pre-stressing for design 1

Design 1	Field moments		Support moments	
	M_{fx}	M_{fy}	M_{sx}	M_{sy}
Resistance				
Thickness	550 mm	550 mm	800 mm	800 mm
$F_{p,max}$	8662 kN	8662 kN	12600 kN	12600 kN
Eccentricity	92 mm	92 mm	133 mm	133 mm
M_{ecc}	794 kNm	794 kNm	1680 kNm	1680 kNm
M_{conc}	794 kNm	794 kNm	1680 kNm	1680 kNm
M_{total}	1588 kNm	1588 kNm	3360 kNm	3360 kNm
Sollicitation				
M_d	770 kNm	770 kNm	2181 kNm	2181 kNm
Design				
Thickness	550 mm	550 mm	800 mm	800 mm
F_p	8500 kN	8500 kN	8500 kN	8500 kN
Eccentricity	92 mm	92 mm	133 mm	133 mm
$M_{total,d}$	1558 kNm	1558 kNm	2267 kNm	2267 kNm

Table G-7 The required pre-stressing for design 2

Design 2	Field moments		Support moments	
	M_{fx}	M_{fy}	M_{sx}	M_{sy}
Resistance				
Thickness	700 mm	700 mm	1000 mm	1000 mm
$F_{p,max}$	11025 kN	11025 kN	15750 kN	15750 kN
Eccentricity	117 mm	117 mm	167 mm	167 mm
M_{ecc}	1286 kNm	1286 kNm	2625 kNm	2625 kNm
M_{conc}	1286 kNm	1286 kNm	2625 kNm	2625 kNm
M_{total}	2572 kNm	2572 kNm	5250 kNm	5250 kNm
Solicitation				
M_d	1796 kNm	428 kNm	3549 kNm	2181 kNm
Design				
Thickness	700 mm	700 mm	1000 mm	1000 mm
F_p	10750 kN	6750 kN	10750 kN	6750 kN
Eccentricity	117 mm	117 mm	167 mm	167 mm
$M_{total,d}$	2508 kNm	1575 kNm	3583 kNm	2250 kNm

It is chosen to evaluate design 1 because the savings on the material are significant compared with design 2.

The cells that have been calculated above are the cells that are located in the middle of the FB. As can be seen in figure G.7 the cells on the outside of an element are loaded with higher hydrodynamic pressures. While the cells in the middle do have hydrodynamic pressures of up to 30 kN/m² the cells on the outside do have pressures up to 60 kN/m², see also table G-8.

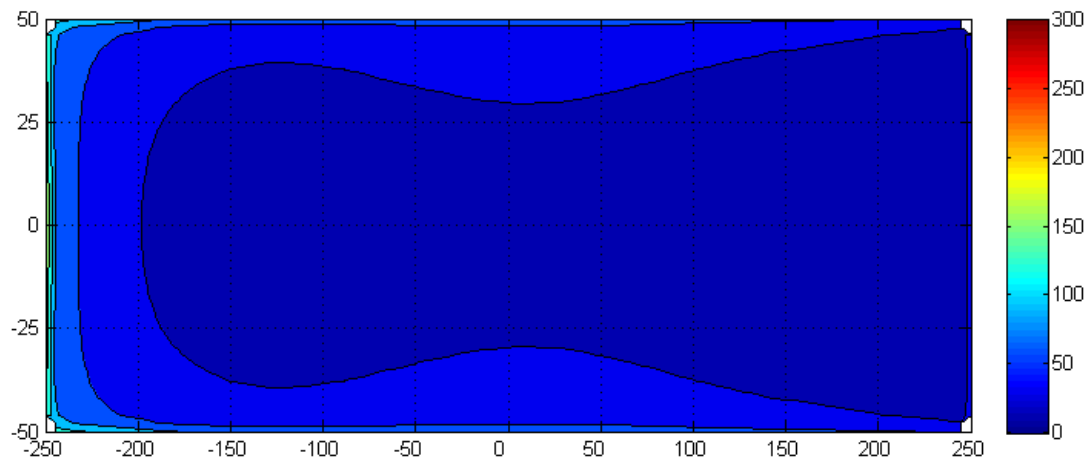


Figure G.7 Design wave pressure on the bottom of a FB element loaded by head-on waves in kN/m²

Table G-8 The distributed loads on the bottom slab located on the outside of the floating breakwater

Load - Distributed loads		p	γ	p_d
Hydrostatic load		201 kN/m ²	1	201 kN/m ²
Waterdepth	20 meter			
Rho	1025 kg/m ³			
Wave load		50 kN/m ²	1	50 kN/m ²
Own weight		14,4 kN/m ²	0,9	13 kN/m ²
Thickness	0,6 meter			
Rho	2400 kg/m ³			
Total				238 kN/m ²

The same cell-size has been used to determine the required pre-stressing force and the thickness, see table G-9.

Table G-9 The required pre-stressing for design 1 for a cell located on the outside of the FB

Design 1	Field moments		Support moments	
	M_{fx}	M_{fy}	M_{sx}	M_{sy}
Resistance				
Thickness	600 mm	600 mm	800 mm	800 mm
$F_{p,max}$	9450 kN	9450 kN	12600 kN	12600 kN
Eccentricity	100 mm	100 mm	133 mm	133 mm
M_{ecc}	945 kNm	945 kNm	1680 kNm	1680 kNm
M_{conc}	945 kNm	945 kNm	1680 kNm	1680 kNm
M_{total}	1890 kNm	1890 kNm	3360 kNm	3360 kNm
Sollicitation				
M_d	840 kNm	840 kNm	2380 kNm	2380 kNm
Design				
Thickness	600 mm	600 mm	800 mm	800 mm
F_p	9000 kN	9000 kN	9000 kN	9000 kN
Eccentricity	100 mm	100 mm	133 mm	133 mm
$M_{total,d}$	1800 kNm	1800 kNm	2400 kNm	2400 kNm

When comparing table G-9 and table G-6 it can be noticed the thickness in the field is only slightly increased, from 550 millimetres to 600 millimetres.

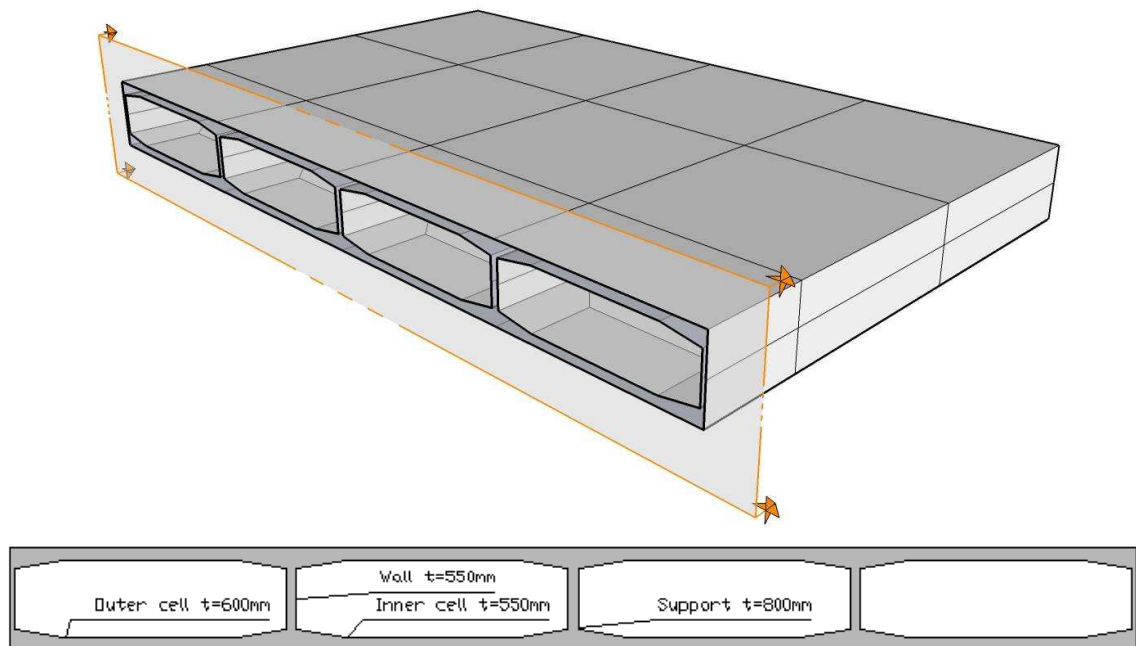


Figure G.8 Cross-section of the inner and outer shell of the bottom part of the section

The bottom part of the hull does consist of a double shell, as described in paragraph G.1.1, see also figure G.8. The inner hull does have to withstand the load of the outer hull when one of the compartments is flooded. Therefore the inner shell does have the same dimensions and strength as the outer shell.

The required distance between the inner and outer shell forming the hull depends on:

- The bottom part of the FB section has to be floated out of a dry-dock and towed to a jetty to be finished. It will have to have a displacement that is large enough to float with a high enough freeboard for transport if necessary. This determines the minimum height of the cells.

- It has to withstand the bending moments due to the not equal distributed loads from the walls and deck. This also determines the minimum height of the cells.
- When one-compartment is flooded it may not lead to stability problems or sinking. This determines the maximum height of the cells.

The second point is no governing criterion; this will be explained when the required ballast of the floating breakwater is calculated. For the determination of the bending moments and the required constructional heights the weight of the superstructure has to be known. This calculation will be done after dimensioning the sides of the section are dimensioned.

The first point will give a minimum distance between the two slabs, i.e. it will give the minimum required height of the pontoon to be able to float. The weight of the pontoon, per running meter, can be estimated with:

$$mass = 2(Y_w t_{avg} \rho_{conc}) + 0.03(Y_w h \rho_{conc}) \quad (G.13)$$

Where:

- Y_w : The width of the FB element;
- t_{avg} : The average thickness of the bottom slab;
- h : The height of the bottom section;
- ρ_{conc} : The density of concrete in kN/m³.

The displacement of the pontoon is:

$$V = Y_w h \rho_w \quad (G.14)$$

Where:

- ρ_w : The density of water in kN/m³.

From equations (G.13) and (G.14) the minimum required height can be found:

$$h = \frac{2(t_{avg} \rho_{conc})}{(\rho_w - 0.03 \rho_{conc})} \quad (G.15)$$

The required height of the element is 3.25 metres. During construction formwork and equipment will be placed on the floating structure, so the required height will be larger. A structure with a height of 5 metres is assumed to have a sufficient displacement to carry its own weight plus the formwork and equipment placed on it during construction.

G.3.2 Sides

The sides of the floating breakwater are loaded with a hydrostatic pressure that is increasing with the depth and the hydrodynamic pressures that are caused by the waves. The hydrodynamic pressures are oscillatory of nature. When the pressures are used which were found by DELFRAC, suction may occur around the water line, see figure G.9a. In reality the pressure distribution on the breakwater will look more like depicted in figure G.9b.

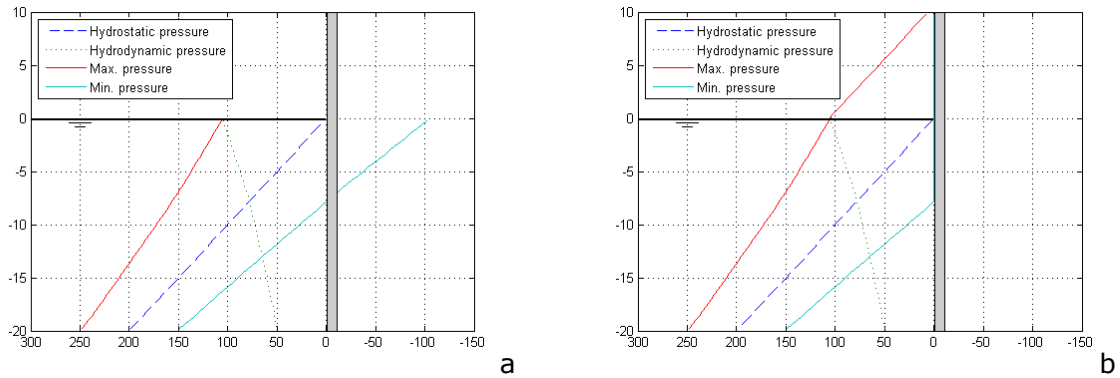


Figure G.9 Pressures according to DELFRAC (a) and idealised pressures on a wall (b)

The sides of the section consist of two parts, a smaller section near the bottom which is heavily loaded and a larger section in the upper part. A larger section will lead to an increase in the moments, but this is partly compensated by the lower occurring pressures. See for the dimensions of the panels table G-10 and figure G.10.

Table G-10 Dimensions of the panels

Dimensions		Upper panel	Lower panel
Height (y-direction)		15 m	5 m
Width (x-direction)		14 m	14 m
L_y/L_x		1,1 [-]	0,357 [-]
Concrete quality	B75	45 N/mm ²	45 N/mm ²
Minimum cover		150 mm	150 mm

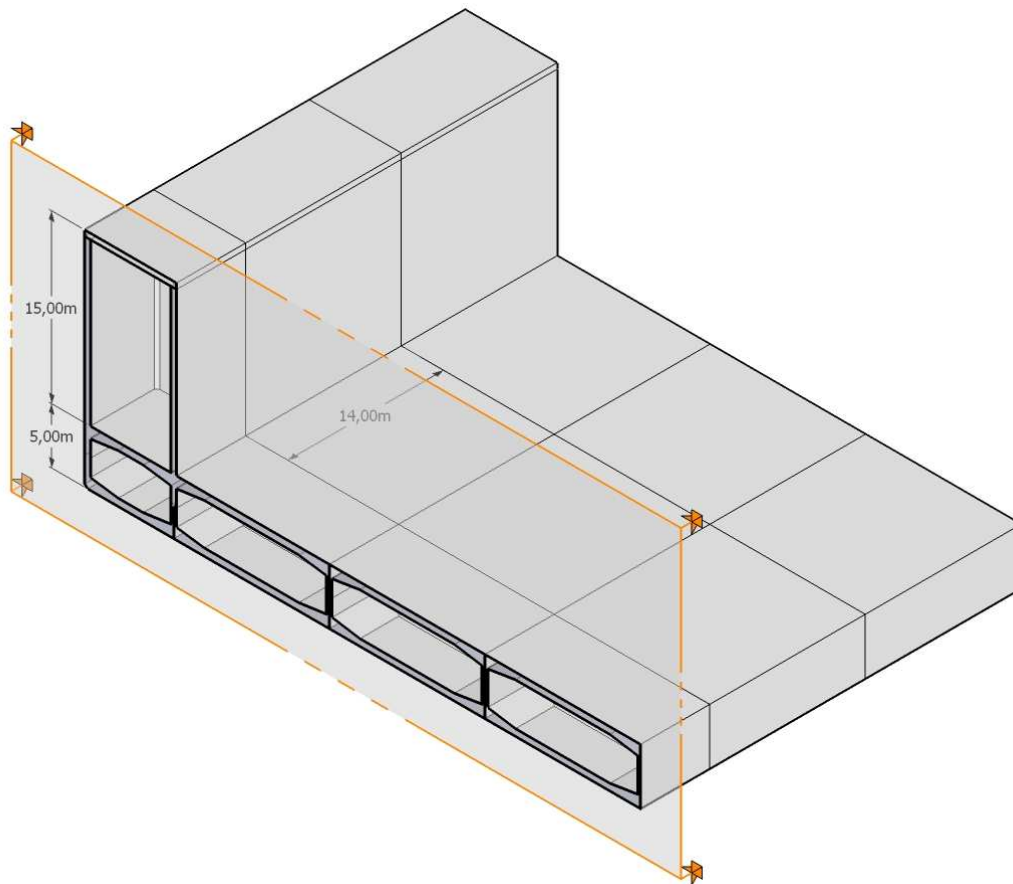


Figure G.10 A cross-section of the floating breakwater hull, showing the outer hull and inner hull.

The upper panel will be elaborated first and with the same procedure the required dimensions of the lower panel will be determined afterwards.

Table G-11 Distributed load on the upper panel

Load - Distributed loads	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 0 meter Rho 1025 kg/m ³	0 kN/m ²	1	0 kN/m ²			
Hydrostatic load Waterdepth 15 meter Rho 1025 kg/m ³				150 kN/m ²	1	150 kN/m ²
Wave load	100 kN/m ²	1	100 kN/m ²	60 kN/m ²	1	60 kN/m ²
Total			100 kN/m ²			210 kN/m ²

The maximum pressure on the panel, as has been shown in figure G.9a, can be approximated by a trapezoidal shaped distributed load. This load can be split into two contributions. The first contribution is an equally distributed load and the second contribution is a triangular distributed load, see figure G.10. The approximated loads on the structure are given in table G-11.

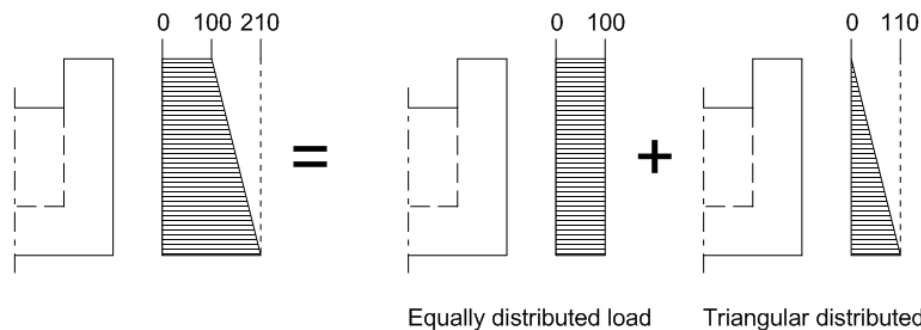


Figure G.11 A trapezoidal shaped distributed load split in an equally and a triangular distributed load

For the calculation of the moments caused by the equally distributed loads the table from Dutch standard NEN6720 can be used as it was used for calculation of the bottom. The table from NEN6720 is given in figure G.6. The moments caused by the triangular distributed loads are given in figure G.12.

Although the extreme moments caused by the equally and the triangular distributed loads are not located on the same place, it is a safe assumption to estimate the total moment by adding the extremes. This will lead to a slight overestimation of the moment. The moments found by this method are listed in table G-12.

Table G-12 The field and support bending moments for the upper panel

Load - Moments	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	22 [-]	431,2 kNm/m	10,9 [-]	235,1 kNm/m	- [-]	666,3 kNm/m
M_{fy}	17 [-]	333,2 kNm/m	10,2 [-]	220,0 kNm/m	- [-]	553,2 kNm/m
Support moment						
M_{sx}	57 [-]	1117,2 kNm/m	30,2 [-]	651,4 kNm/m	- [-]	1768,6 kNm/m
$M_{sy,min}$	52 [-]	1019,2 kNm/m	18 [-]	388,5 kNm/m	- [-]	1407,7 kNm/m
$M_{sy,max}$	52 [-]	1019,2 kNm/m	36,4 [-]	784,0 kNm/m	- [-]	1803,2 kNm/m

$l_y : l_x$	1,0	1,1	1,2	1,3	1,4	1,5	2,0
$m_{x_{ermin}} = -$	36,9	33,1	29,8	27,5	25,6	23,9	20,2
$m_{x_m} =$	113,6	91,7	78,7	69,9	63,7	59,2	50,0
$m_{y_{erm}} (y=0) = -$	30,0	27,5	26,1	25,0	24,1	23,8	21,9
$m_{y_{erm}} (y=l_y) = -$	56,2	55,5	57,2	59,5	62,9	67,1	92,5
$m_{y_{max}} =$	98,0	98,0	98,0	99,0	99,0	100,0	100,0
$q_{x_{ermax}} (x=0 \text{ u. } l_x) = \pm$	4,14	3,86	3,59	3,39	3,25	3,13	2,83
$q_{y_{erm}} (y=0) =$	3,07	2,90	2,80	2,74	2,70	2,64	2,49
$q_{y_{erm}} (y=l_y) = -$	8,25	8,40	8,77	9,40	9,99	10,70	14,29
$f_m = \frac{p_1 \cdot l_x^4}{E \cdot d^3}$	0,0076	0,0091	0,0104	0,0115	0,0124	0,0132	0,0152

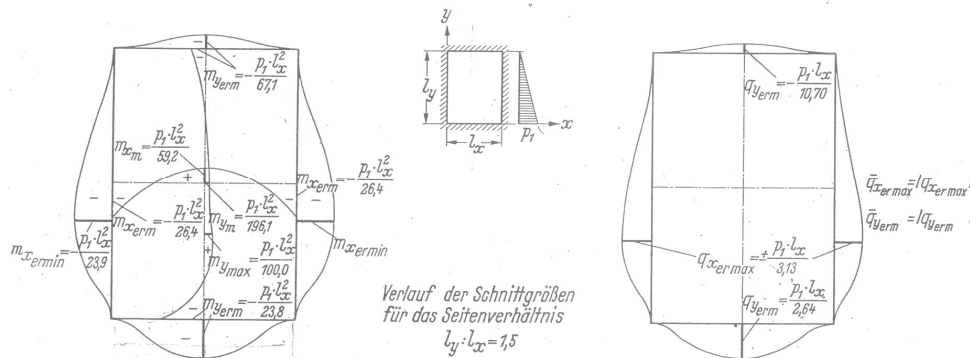


Figure G.12 Coefficients to determine the field and support bending moment for a triangular load¹

The required dimensions of the upper panel are calculated in case of no allowed tension in the concrete. The results of the calculation can be found in table G-13. In this table it can be seen that for the upper support moment there is no contribution of the eccentricity of the pre-stressing force to the moment bearing capacity of the upper support. Because of the oscillatory nature the moments can be positive or negative, leading to an optimal design where the pre-stressing will be located in the neutral line.

Table G-13 The required pre-stressing and dimensions for the upper panel

	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$	11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}	1477 kNm	0 kNm	1477 kNm	0 kNm	1477 kNm
M_{conc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}	2953 kNm	1477 kNm	2953 kNm	1477 kNm	2953 kNm
Solicitation					
M_d	666 kNm	553 kNm	1769 kNm	1408 kNm	1803 kNm
Design					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
F_p	7500 kN	11500 kN	7500 kN	11500 kN	11500 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$	1875 kNm	1438 kNm	1875 kNm	1438 kNm	2875 kNm

¹ It has to be noted that the coefficients from figure G.6 and figure G.12 are not derived in the same way. The coefficients in figure G.12 are the reciprocal of the coefficients of figure G.6 and the coefficients that are used in the calculation

Similar to the calculations for obtaining the dimensions of the upper panels the lower panel can be calculated. The pressures on this panel are listed in table G-14. The resulting bending moments, found by using figure G.6 and figure G.13 are listed in table G-15. Finally the required dimensions and the design is given in

Table G-14 Distributed load on the lower panel

Load - Distributed loads	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load	150 kN/m ²	1	150 kN/m ²			
Waterdepth 15 meter						
Rho 1025 kg/m ³						
Hydrostatic load				200 kN/m ²	1	200 kN/m ²
Waterdepth 20 meter						
Rho 1025 kg/m ³						
Wave load	60 kN/m ²	1	60 kN/m ²	40 kN/m ²	1	40 kN/m ²
Total			210 kN/m ²			240 kN/m ²

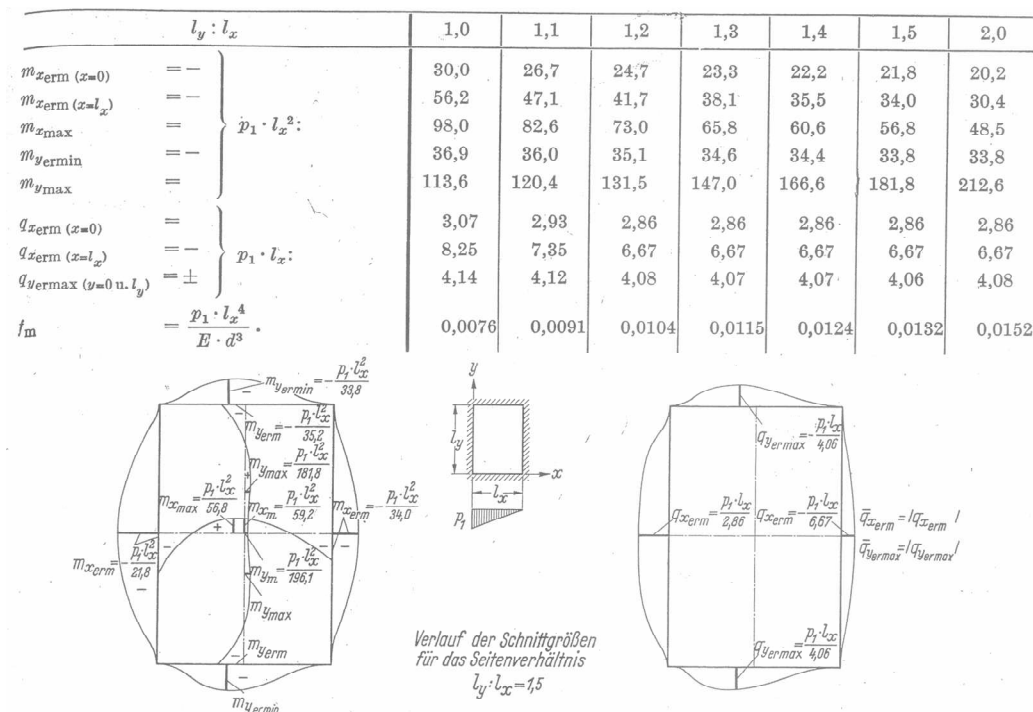


Figure G.13 Coefficients to determine the field and support bending moment for a triangular load²

Table G-15 The field and support bending moments for the lower panel

Load - Moments	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	10 [-]	52,5 kNm/m	4,7 [-]	3,5 kNm/m	- [-]	56,0 kNm/m
M_{fy}	43 [-]	225,8 kNm/m	25 [-]	18,8 kNm/m	- [-]	244,5 kNm/m
Support moment						
M_{sx}	49 [-]	257,3 kNm/m	29,6 [-]	22,2 kNm/m	- [-]	279,4 kNm/m
$M_{sy,min}$	83 [-]	435,8 kNm/m	38,5 [-]	28,8 kNm/m	- [-]	464,6 kNm/m
$M_{sy,max}$	83 [-]	435,8 kNm/m	53,8 [-]	40,3 kNm/m	- [-]	476,1 kNm/m

² It has to be noted that the coefficients from figure G.6 and figure G.13 are not derived in the same way. The coefficients in figure G.13 are the reciprocal of the coefficients of figure G.6 and the coefficients that are used in the calculation

Table G-16 The required pre-stressing and dimensions for the lower panel

	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$	11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{conc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}	2953 kNm	2953 kNm	2953 kNm	2953 kNm	2953 kNm
Sollicitation					
M_d	56 kNm	245 kNm	279 kNm	465 kNm	476 kNm
Design					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
F_p	2000 kN	2000 kN	2000 kN	2000 kN	2000 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$	500 kNm	500 kNm	500 kNm	500 kNm	500 kNm

G.3.3 Global moments

The slabs that are forming the outer hull of the FB section are now designed, but as with the bottom slabs, this outer hull works together with the inner hull, see also Figure G.10. This inner hull provides strength to the outer hull and protects the FB element against sinking by dividing the element in compartments.

The inner and outer hulls are together forming a beam which has to withstand the global moments. These moments are introduced in the structure due to the building process and the hydrostatic forces and can be translated in to a mechanics schematisation, see also figure G.14.

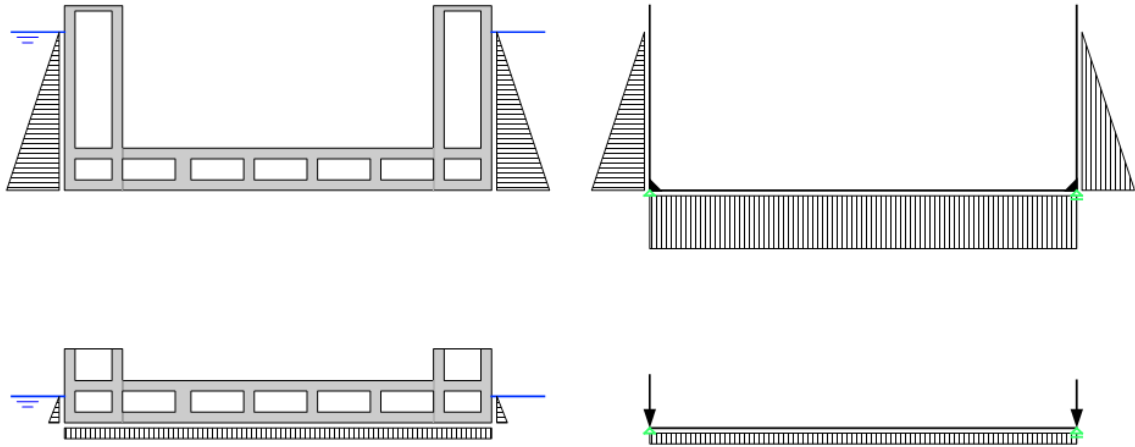


Figure G.14 Global moments due to hydrostatic pressures (above) and the construction scheme (below) and their corresponding mechanics schematisation

Due to the pressures on the bottom and sides of the floating breakwater, it is not likely the bottom can withstand the occurring moment. The ultimate moment can be estimated with the following rule of thumb:

- Free-free support

Field moment:

$$M = \frac{1}{8} q L^2 = \frac{1}{8} \cdot 200 \left[\frac{kN}{m^2} \right] \cdot (100[m])^2 = 250 MNm \quad (G.16)$$

Support moment:

$$M = 0 \quad (G.17)$$

- Fixed-fixed support

Field moment:

$$M = \frac{1}{24} qL^2 = \frac{1}{24} \cdot 200 \left[\frac{kN}{m^2} \right] \cdot (100[m])^2 = 83 MNm \quad (G.18)$$

Support moment:

$$M = \frac{1}{12} qL^2 = \frac{1}{12} \cdot 200 \left[\frac{kN}{m^2} \right] \cdot (100[m])^2 = 167 MNm \quad (G.19)$$

When the bottom height is known or assumed the stresses in the concrete can be determined. In paragraph G.3.1 a minimum height of 3.25 metres was determined to be able to float the bottom part out of a dry-dock. When a height of 5 metres is assumed the stresses in the concrete will become:

$$\sigma = \frac{M}{h \cdot A}$$

$$\sigma = \frac{250 \cdot 10^9 [Nmm]}{6000 [mm] \cdot 600 \cdot 10^3 [mm^2]} = 69.4 \left[\frac{N}{mm^2} \right] \quad (G.20)$$

The calculated stress by equation (G.20) is too high to be taken up by the concrete. To have the bottom spanning the complete beam of the FB would be unrealistic. Therefore walls inside the FB section are required, see also figure G.15. These walls also have the function to support the deck of the FB section.

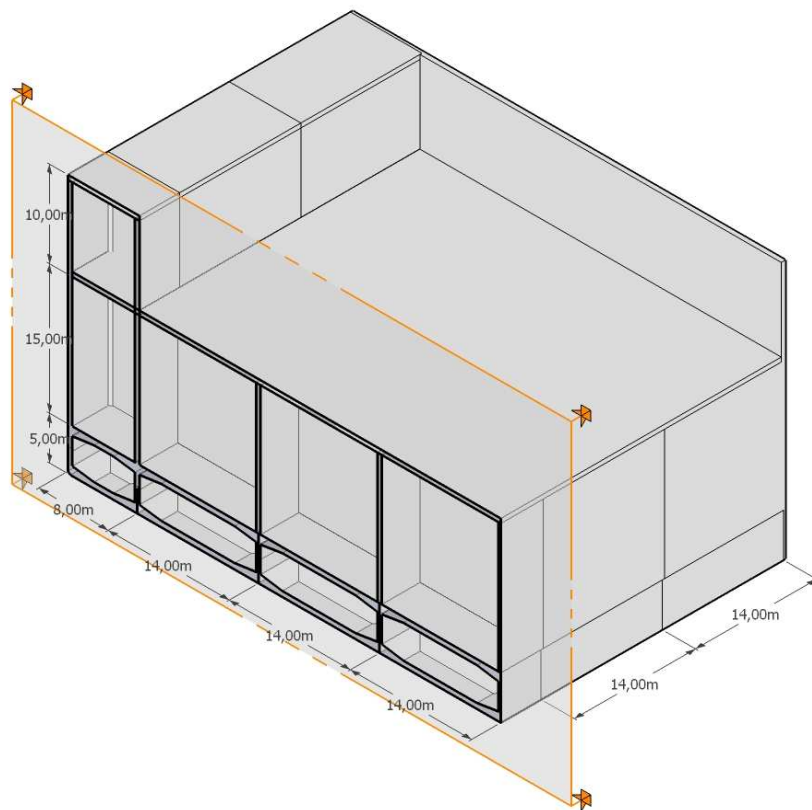


Figure G.15 Inner walls to compensate for global moments

During construction however there is the possibility these walls are not present yet, while the sides are already being built. Due to the mass of the sides there is an uneven distribution of weight over the beam of the FB element, which causes moments, see also figure G.14. The mass of one side per running metres length is:

$$\begin{aligned} F_{mass} &= 2h_{wall}t_{wall}\rho_{conc} \\ F_{mass} &= 2 \cdot 20 \cdot 0.75 \cdot 24 = 720 [kN] \end{aligned} \quad (G.21)$$

Where:

- F_{mass} : The mass of the side of the hull;
- h_{wall} : The height of the side;
- t_{wall} : The thickness of the side;
- ρ_{conc} : The density of concrete, 24 kN/m³.

The factor 2 in equation (G.21) comes from the fact that there is an inner and an outer hull. Due to the fact that weight is added to the structure the draught of the structure will change, leading to an additional hydrostatic pressure:

$$p_{side} = \frac{F_{mass}}{Y_w L} = 7.2 \left[\frac{kN}{m^2} \right] \quad (G.22)$$

Where:

- p_{side} : The increased pressure on the hull due to the added weight;
- Y_w : The beam of the FB element;
- L : The considered length, in this case 1 meter.

The moment per meter length is:

$$M = \frac{1}{8} q L^2 = \frac{1}{8} \cdot 7.2 \left[\frac{kN}{m^2} \right] \cdot (100[m])^2 = 9000 \left[\frac{kNm}{m} \right] \quad (G.23)$$

And the occurring stresses due this moment are:

$$\begin{aligned} \sigma &= \frac{M}{h \cdot A} \\ \sigma &= \frac{9000 \cdot 10^6 [Nmm]}{6000 [mm] \cdot 600 \cdot 10^3 [mm^2]} = 2.5 \left[\frac{N}{mm^2} \right] \end{aligned} \quad (G.24)$$

This stress can be taken up by the concrete and the pre-stress in the concrete.

G.3.4 Ballast

The section has to be ballasted for two purposes:

- To obtain the required draught

The mass of the concrete hull is not enough to obtain the design draught of 20 metres.

- To position the centre of gravity above the centre of buoyancy

The mass of the hull is concentrate near the heavy bottom, giving it a centre of gravity which is located lower than the centre of buoyancy. This may be beneficial to the stability of the FB element but it has a negative impact on

the wave attenuating properties. The combined own weight and ballast should have a COG located 5 metres above the COB.

In figure G.16 a cross-section of the floating breakwater is given with all voids where ballast can be placed. It can be seen that the weight of the structure is located near the bottom while the centre of gravity has to be located above the centre of buoyancy ultimately. The draught without any ballast is 9.64 metres while the COG is located 9.38 metres above the keel, which should be at least 15 metres, see also table G-17.

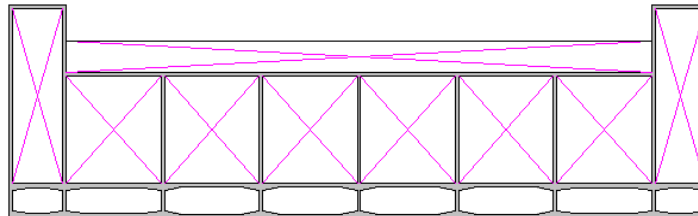


Figure G.16 The empty compartments which can be used for ballast

Water can be used as ballast material and it has the advantage that it is the cheapest material that is available. Also it can be placed during installation on site, resulting in lower draught, and thus resistance, during transport.

A free water surface does have a negative impact on the stability. When the breakwater rolls the water will flow to the lowest side, increasing the roll exciting moment. To avoid this sloshing of water inside the breakwater the water has to be stored in the compartments, see figure G.17.

Due to the low density of water there is a large volume required to obtain the required draught. When the ballast is equally distributed over the FB element there is an average layer of water in the compartments of 11 metres. The combined COG of the structure and ballast will thus be too low, see also table G-17.

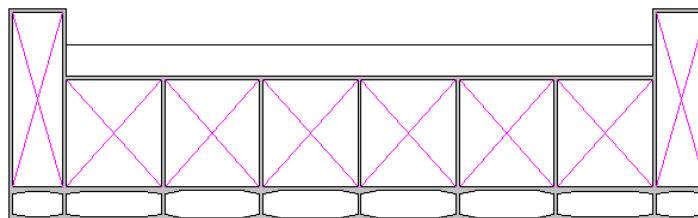


Figure G.17 The empty compartments which have to be used for ballast when water is used

Sand or rubble can also be used as ballast, which has the disadvantage over water that there will be costs attached to the use of it. It is also more difficult to place the ballast on site, possibly leading to the requirement that the FB element is ballasted in the construction harbour. This will result in higher transport costs.

Sand or rubble does not suffer from sloshing as it is a solid material. It does not have to be placed in compartments and it has less volume that is required. The reduced volume makes it possible to store the sand or rubble on the deck of the floating breakwater, see figure G.18.

When a density of 20 kN/m^3 is assumed, which is the a proper value for wet sand, a layer of 5.5 metres over the complete surface of the deck is necessary to get the required draught of 20 metres. The centre of gravity will be positioned at 16.47 metres above the keel, which is sufficiently high.

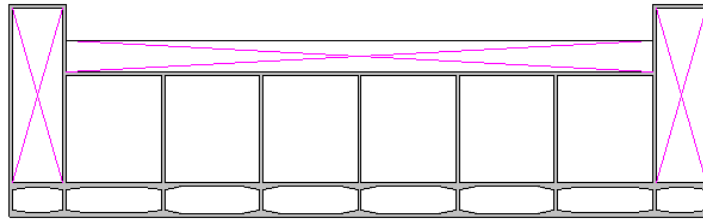


Figure G.18 The empty compartments which have to be used for ballast when sand and water is used

An economical solution can be found by combining water and sand/rubble ballast. The sand/rubble ballast is used to obtain a COG which is positioned high enough, while water is used for the ballast required to obtain the design draught. The water is placed in the outside compartments only to have a high water level in this void and thus a high COG for this ballast, see figure G.18.

During transport the water ballast will be absent, resulting in a smaller draught, and thus a reduced resistance. This will lead to lower transport costs and ads to a more economical solution.

All alternatives are summarised in table G-17.

Table G-17 Variants on ballasting the section

	No ballast	Water ballast	Sand ballast	Optimal
Mass concrete	896400 kN	896400 kN	896400 kN	896400 kN
Mass ballast water	0 kN	503600 kN	0 kN	132904 kN
Mass ballast sand	0 kN	0 kN	503600 kN	370696 kN
Total mass	896400 kN	1400000 kN	1400000 kN	1400000 kN
Draught	12,81 m	20 m	20 m	20 m
KG	11,69 m	11,44 m	15,45 m	15,00 m
Draught at float out	- m	12,81 m	20 m	18,10 m
KG at float	- m	11,69 m	15,45 m	14,58 m

G.3.5 Inner walls and decks

The inner walls do have two functions. Firstly they have to stiffen the bottom part to prevent large moments and secondly they have to support the deck on which the sand ballast is placed. The walls, except for the compartment walls in the lower compartments, do not have to withstand any hydrostatic pressures. If a compartment would be flooded it would be one of the compartments between the inner and outer hull. It is estimated that the required thickness of the inner wall should be around 500 mm.

The first deck is the structure on which the sand ballast is placed has to be designed to carry the load of the ballast. The acceleration, up to 20% of the gravitational acceleration, has to be taken into account because of the motions and accelerations of the element. As with the inner walls the thickness is estimated to be 500 mm.

The second deck is the deck which shelters the ballast from water ingress due to wave overtopping. The same reasoning holds for the second deck as for the first, leading to an estimated thickness of 500 mm.

For all inner walls and decks the design should be further elaborated on in follow-up studies.

G.3.6 Conclusion

In this section the normal section has been designed. All important dimensions are found in the cross-sections shown in figure G.19 and figure G.20.

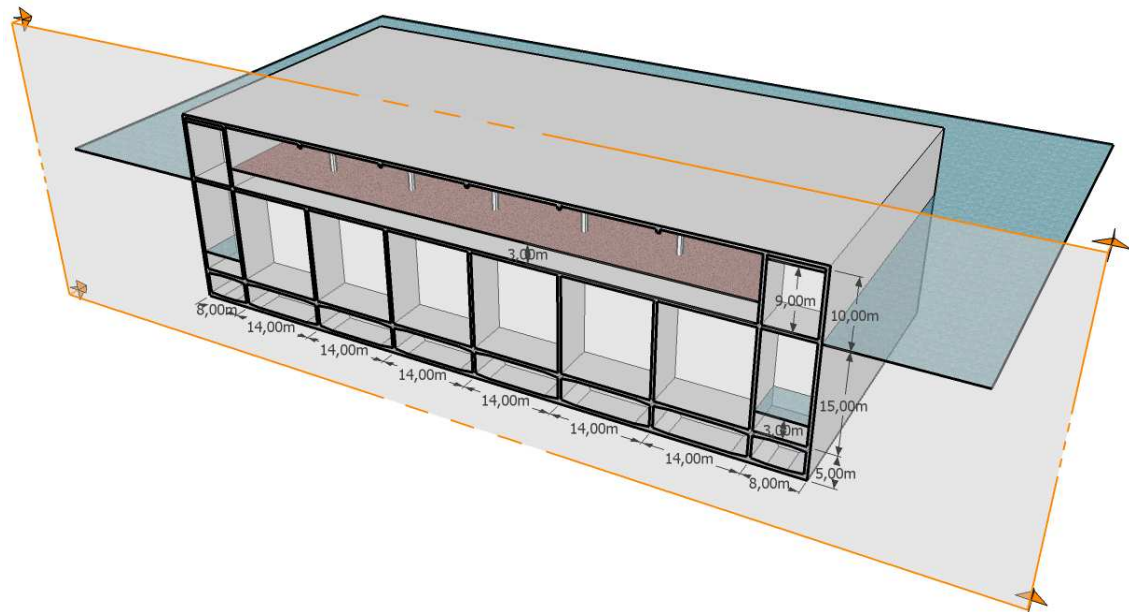


Figure G.19 Transverse cross-section of the normal section

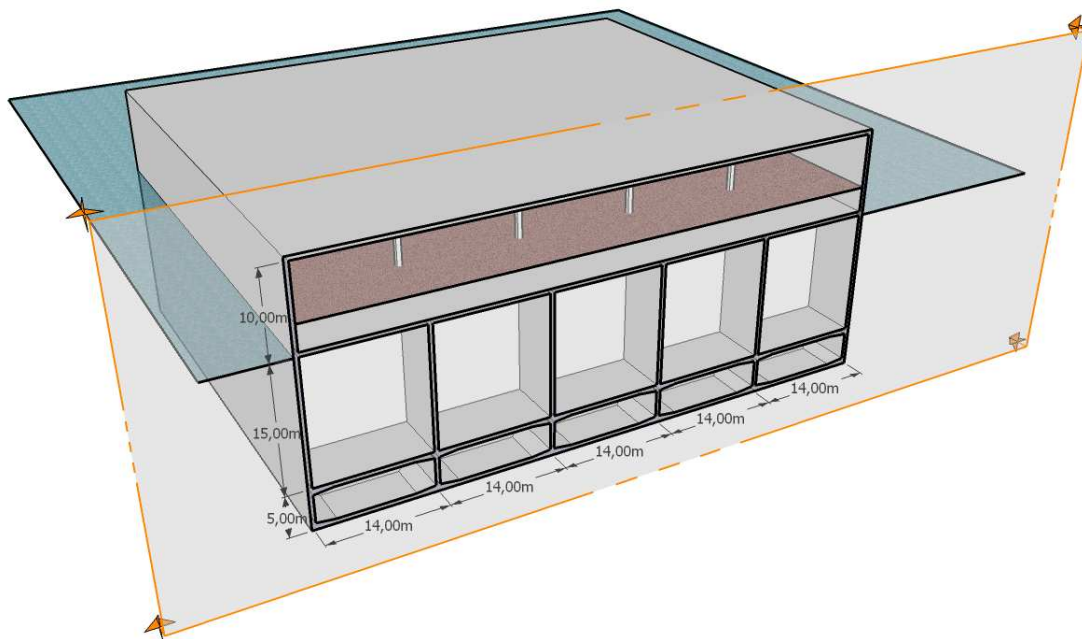


Figure G.20 Longitudinal cross-section of the normal section

G.4 Turret mooring section

The turret mooring section is similar to the normal sections and will have the same dimension as shown in figure G.19 and figure G.20. The main difference is that the turret mooring section is equipped with the mooring system. The mooring system itself will be elaborated in section G.6. In figure G.28 an impression is shown of the turret mooring section.

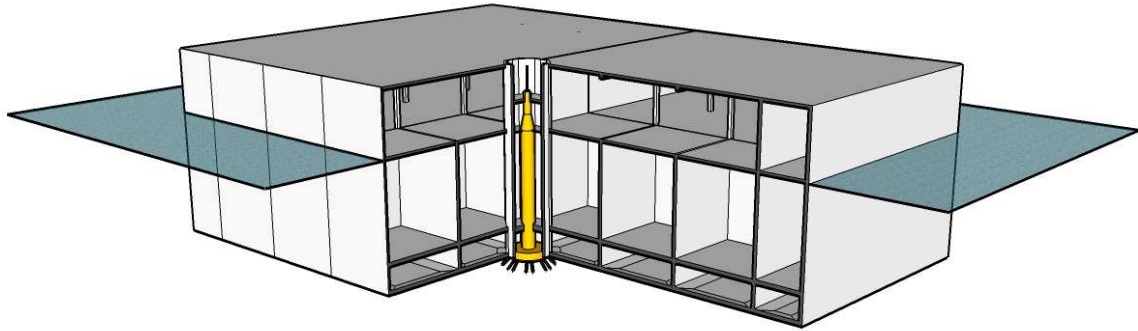


Figure G.21 Impression of the turret mooring section

G.5 Bow and stern sections

The bow and stern elements will have the same lay-out of the normal section, but it is constructed heavier to withstand the higher wave induced pressures. Also it has an extra side, the bow, which has to be strengthened to take up the wave-loads when the FB is loaded by head-on waves during the ultimate limit state (ULS).

The same procedures and formulas will be used as have been used for the normal sections. The tables are presented below without explanation here; the explanation can be found in paragraph G.3.

G.5.1 Bottom

In figure G.22 the hydrodynamic pressure distribution on the bottom of the bow/stern section is given. Due to the head-on waves and the diffraction of these waves by the structure the hydrodynamic pressure is high in the cells located near the bow, in figure G.22 denoted with a 2, see also table G-19. Therefore these cells have a thicker bottom slab, 700 millimetres against 600 millimetres for cells denoted with a 1. These dimensions come from the calculation performed in table G-20 and table G-21. In the final design it can be further elaborated for optimizing the material usage and required labour.

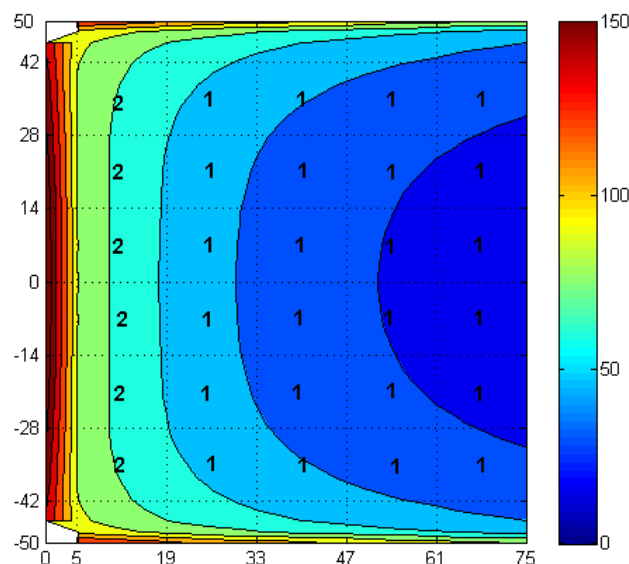


Figure G.22 Hydrodynamic pressures on the bow/stern section in kN/m^2 (from Appendix F.1.5)

Table G-18 Dimensions of panel numbers 1 and 2

Dimensions	Panel 1	Panel 2
Width (y-direction)	14 m	14 m
Length (x-direction)	14 m	14 m
L_y/L_x	1 [-]	1 [-]
Concrete quality B75	45 N/mm ²	45 N/mm ²
Minimum cover	150 mm	150 mm

Table G-19 Distributed loads on panels 1 and 2

Load - Distributed loads	Panel 1			Panel 2		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 20 meter Rho 1025 kg/m ³	201 kN/m ²	1	201 kN/m ²	0 kN/m ²	1	0 kN/m ²
Wave load	60 kN/m ²	1	60 kN/m ²	90 kN/m ²	1	90 kN/m ²
Own weight Thickness 0,6 meter Rho 2400 kg/m ³	14,4 kN/m ²	0,9	13 kN/m ²	0 kN/m ²	0,9	0 kN/m ²
Total			248 kN/m ²			90 kN/m ²

Table G-20 The required pre-stressing and dimensions for panel 1

Panel 1	Field moments		Support moments	
	M_{fx}	M_{fy}	M_{sx}	M_{sy}
Resistance				
Thickness	600 mm	600 mm	800 mm	800 mm
$F_{p,max}$	9450 kN	9450 kN	12600 kN	12600 kN
Eccentricity	100 mm	100 mm	133 mm	133 mm
M_{exc}	945 kNm	945 kNm	1680 kNm	1680 kNm
M_{conc}	945 kNm	945 kNm	1680 kNm	1680 kNm
M_{total}	1890 kNm	1890 kNm	3360 kNm	3360 kNm
Sollicitation				
M_d	872 kNm	872 kNm	2469 kNm	2469 kNm
Design				
Thickness	600 mm	600 mm	800 mm	800 mm
F_p	9400 kN	9400 kN	9400 kN	9400 kN
Eccentricity	100 mm	100 mm	133 mm	133 mm
$M_{total,d}$	1880 kNm	1880 kNm	2507 kNm	2507 kNm

Table G-21 The required pre-stressing and dimensions for panel 2

Panel 2	Field moments		Support moments	
	M_{fx}	M_{fy}	M_{sx}	M_{sy}
Resistance				
Thickness	700 mm	700 mm	800 mm	800 mm
$F_{p,max}$	11025 kN	11025 kN	12600 kN	12600 kN
Eccentricity	117 mm	117 mm	133 mm	133 mm
M_{exc}	1286 kNm	1286 kNm	1680 kNm	1680 kNm
M_{conc}	1286 kNm	1286 kNm	1680 kNm	1680 kNm
M_{total}	2572 kNm	2572 kNm	3360 kNm	3360 kNm
Sollicitation				
M_d	977 kNm	977 kNm	2769 kNm	2769 kNm
Design				
Thickness	700 mm	700 mm	800 mm	800 mm
F_p	10500 kN	10500 kN	10500 kN	10500 kN
Eccentricity	117 mm	117 mm	133 mm	133 mm
$M_{total,d}$	2450 kNm	2450 kNm	2800 kNm	2800 kNm

G.5.2 Sides

The thickness of the side of the FB section is here 750 millimetres, as is with the normal sections, but it requires heavier reinforcement, see table G-23 to table G-28 for panel 1 and table G-29 to table G-34 for panel 2. Also the outer compartments cannot be used for ballast as the combination of the applied pre-stress (especially the geometry of the tendons), the pressure from the ballast water and a wave trough will cause tensile forces in the concrete.

Table G-22 Dimensions of the panels

Dimensions	Panel 1		Panel 2	
	Upper panel	Lower panel	Upper panel	Lower panel
Height (y-direction)	14 m	5 m	14 m	5 m
Width (x-direction)	5 m	14 m	5 m	14 m
L_y/L_x	2,8 [-]	0,357 [-]	2,8 [-]	0,357 [-]
Concrete quality B75	45 N/mm ²	45 N/mm ²	45 N/mm ²	45 N/mm ²
Minimum cover	150 mm	150 mm	150 mm	150 mm

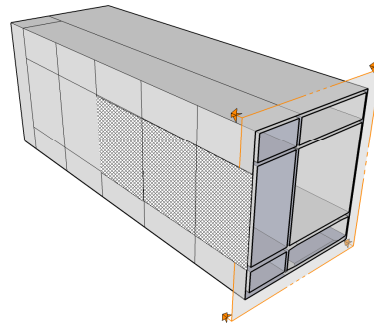


Figure G.23 The location of the considered panel (shaded)

Table G-23 Distributed loads on upper part of panel 1

Load - Distributed loads Panel 1 - Upper panel	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 0 meter Rho 1025 kg/m ³	0 kN/m ²	1	0 kN/m ²			
Hydrostatic load Waterdepth 15 meter Rho 1025 kg/m ³				150 kN/m ²	1	150 kN/m ²
Wave load	150 kN/m ²	1	150 kN/m ²	105 kN/m ²	1	105 kN/m ²
Total			150 kN/m ²			255 kN/m ²

Table G-24 The design moments for the upper part of panel 1

Load - Moments Panel 1 - Upper panel	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	22 [-]	646,8 kNm/m	10,9 [-]	224,4 kNm/m	- [-]	871,2 kNm/m
M_{fy}	17 [-]	499,8 kNm/m	10,2 [-]	210,0 kNm/m	- [-]	709,8 kNm/m
Support moment						
M_{sx}	57 [-]	1675,8 kNm/m	30,2 [-]	621,8 kNm/m	- [-]	2297,6 kNm/m
$M_{sy,min}$	52 [-]	1528,8 kNm/m	18 [-]	370,8 kNm/m	- [-]	1899,6 kNm/m
$M_{sy,max}$	52 [-]	1528,8 kNm/m	36,4 [-]	748,4 kNm/m	- [-]	2277,2 kNm/m

Table G-25 The required pre-stressing and dimensions for the upper part of panel 1

Panel 1 Upper panel	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$	11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{conc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}	2953 kNm	2953 kNm	2953 kNm	2953 kNm	2953 kNm
Sollicitation					
M_d	871 kNm	710 kNm	2298 kNm	1900 kNm	2277 kNm
Design					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
F_p	9500 kN	9500 kN	9500 kN	9500 kN	9500 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$	2375 kNm	2375 kNm	2375 kNm	2375 kNm	2375 kNm

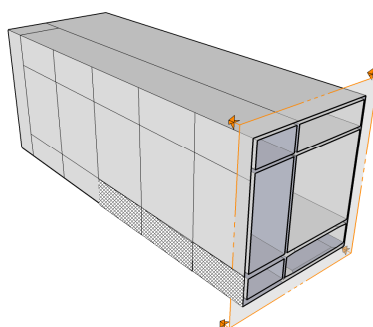


Figure G.24 The location of the considered panel (shaded)

Table G-26 Distributed loads on lower part of panel 1

Load - Distributed loads			Top			Bottom		
Panel 1 - Lower panel			p	γ	p_d	p	γ	p_d
Hydrostatic load			150 kN/m ²	1	150 kN/m ²			
Waterdepth	15 meter							
Rho	1025 kg/m ³							
Hydrostatic load						200 kN/m ²	1	200 kN/m ²
Waterdepth	20 meter							
Rho	1025 kg/m ³							
Wave load			105 kN/m ²	1	105 kN/m ²	90 kN/m ²	1	90 kN/m ²
Total					255 kN/m ²			290 kN/m ²

Table G-27 The design moments for the lower part of panel 1

Load - Moments		Distributed loads		Triangle loads		Total	
Panel 1 - Lower panel		Factor	Moment	Factor	Moment	Factor	Moment
Field moment							
M_{fx}		10 [-]	63,8 kNm/m	4,7 [-]	4,1 kNm/m	- [-]	67,9 kNm/m
M_{fy}		43 [-]	274,1 kNm/m	25 [-]	21,9 kNm/m	- [-]	296,0 kNm/m
Support moment							
M_{sx}		49 [-]	312,4 kNm/m	29,6 [-]	25,9 kNm/m	- [-]	338,3 kNm/m
$M_{sy,min}$		83 [-]	529,1 kNm/m	38,5 [-]	33,7 kNm/m	- [-]	562,8 kNm/m
$M_{sy,max}$		83 [-]	529,1 kNm/m	53,8 [-]	47,0 kNm/m	- [-]	576,2 kNm/m

Table G-28 The required pre-stressing and dimensions for the lower part of panel 1

Panel 1		Field moments		Support moments		
Lower panel		M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance						
Thickness		750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$		11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Excentricity		125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}		1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{conc}		1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}		2953 kNm	2953 kNm	2953 kNm	2953 kNm	2953 kNm
Sollicitation						
M_d		68 kNm	296 kNm	338 kNm	563 kNm	576 kNm
Design						
Thickness		750 mm	750 mm	750 mm	750 mm	750 mm
F_p		2000 kN	3000 kN	2000 kN	3000 kN	3000 kN
Excentricity		125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$		500 kNm	750 kNm	500 kNm	750 kNm	750 kNm

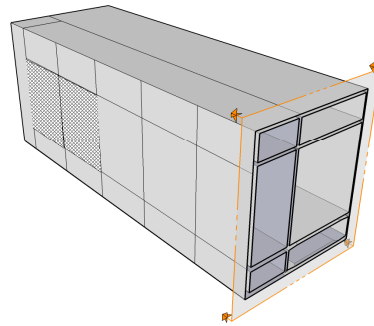


Figure G.25 The location of the considered panel (shaded)

Table G-29 Distributed loads on upper part of panel 2

Load - Distributed loads Panel 2 - Upper panel	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 0 meter	0 kN/m ²	1	0 kN/m ²			
Hydrostatic load Waterdepth 15 meter				150 kN/m ²	1	150 kN/m ²
Wave load	180 kN/m ²	1	180 kN/m ²	135 kN/m ²	1	135 kN/m ²
Total			180 kN/m ²			285 kN/m ²

Table G-30 The design moments for the upper part of panel 2

Load - Moments Panel 2 - Upper panel	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	22 [-]	776,2 kNm/m	10,9 [-]	224,4 kNm/m	- [-]	1000,6 kNm/m
M_{fy}	17 [-]	599,8 kNm/m	10,2 [-]	210,0 kNm/m	- [-]	809,8 kNm/m
Support moment						
M_{sx}	57 [-]	2011,0 kNm/m	30,2 [-]	621,8 kNm/m	- [-]	2632,7 kNm/m
$M_{sy,min}$	52 [-]	1834,6 kNm/m	18 [-]	370,8 kNm/m	- [-]	2205,4 kNm/m
$M_{sy,max}$	52 [-]	1834,6 kNm/m	36,4 [-]	748,4 kNm/m	- [-]	2582,9 kNm/m

Table G-31 The required pre-stressing and dimensions for the upper part of panel 2

Panel 2 Upper panel	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$	11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{conc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}	2953 kNm	2953 kNm	2953 kNm	2953 kNm	2953 kNm
Sollicitation					
M_d	1001 kNm	810 kNm	2633 kNm	2205 kNm	2583 kNm
Design					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
F_p	9000 kN	11000 kN	11000 kN	9000 kN	11000 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$	2250 kNm	2750 kNm	2750 kNm	2250 kNm	2750 kNm

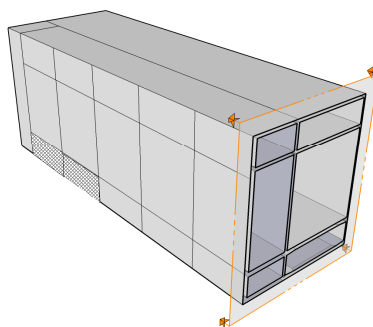


Figure G.26 The location of the considered panel (shaded)

Table G-32 Distributed loads on lower part of panel 2

Load - Distributed loads Panel 2 - Lower panel	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 15 meter	150 kN/m ²	1	150 kN/m ²			
Hydrostatic load Waterdepth 20 meter				200 kN/m ²	1	200 kN/m ²
Wave load	135 kN/m ²	1	135 kN/m ²	120 kN/m ²	1	120 kN/m ²
Total			285 kN/m ²			320 kN/m ²

Table G-33 The design moments for the lower part of panel 2

Load - Moments Panel 1 - Lower panel	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	10 [-]	71,3 kNm/m	4,7 [-]	4,1 kNm/m	- [-]	75,4 kNm/m
M_{fy}	43 [-]	306,4 kNm/m	25 [-]	21,9 kNm/m	- [-]	328,3 kNm/m
Support moment						
M_{sx}	49 [-]	349,1 kNm/m	29,6 [-]	25,9 kNm/m	- [-]	375,0 kNm/m
$M_{sy,min}$	83 [-]	591,4 kNm/m	38,5 [-]	33,7 kNm/m	- [-]	625,0 kNm/m
$M_{sy,max}$	83 [-]	591,4 kNm/m	53,8 [-]	47,0 kNm/m	- [-]	638,4 kNm/m

Table G-34 The required pre-stressing and dimensions for the lower part of panel 2

Panel 2 Lower panel	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
$F_{p,max}$	11812 kN	11812 kN	11812 kN	11812 kN	11812 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
M_{ecc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{conc}	1477 kNm	1477 kNm	1477 kNm	1477 kNm	1477 kNm
M_{total}	2953 kNm	2953 kNm	2953 kNm	2953 kNm	2953 kNm
Sollicitation					
M_d	75 kNm	328 kNm	375 kNm	625 kNm	638 kNm
Design					
Thickness	750 mm	750 mm	750 mm	750 mm	750 mm
F_p	2000 kN	3000 kN	2000 kN	3000 kN	3000 kN
Eccentricity	125 mm	125 mm	125 mm	125 mm	125 mm
$M_{total,d}$	500 kNm	750 kNm	500 kNm	750 kNm	750 kNm

G.5.3 Bow plane

The bow, or front end of the FB element, is loaded by the highest hydrodynamic pressures, see figure G.27. The required thickness of the bow is 900 millimetres and for the same reason as for the sides no ballast water can be contained in the outer compartments located at the bow.

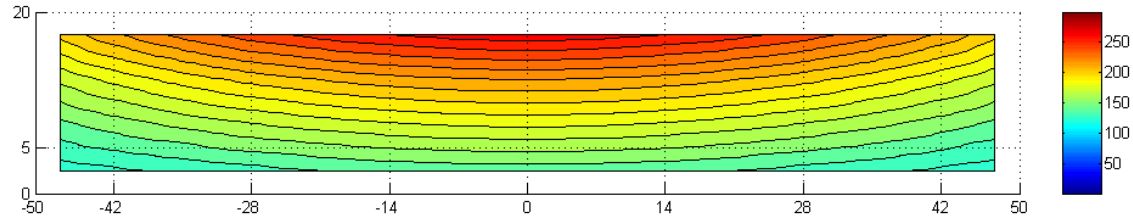


Figure G.27 Hydrodynamic pressures on the side of the bow/stern section in kN/m^2 (enlarged)

Table G-35 Dimensions of bow-panels

Dimensions		Upper panel	Lower panel
Height (y-direction)		15 m	5 m
Width (x-direction)		14 m	14 m
L_y/L_x		1,1 [-]	0,357 [-]
Concrete quality	B75	45 N/mm^2	45 N/mm^2
Minimum cover		150 mm	150 mm

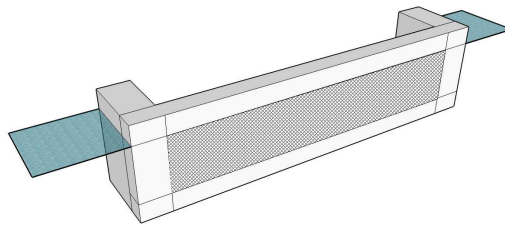


Figure G.28 The location of the considered panel (shaded)

Table G-36 Distributed loads on upper part of the bow panel

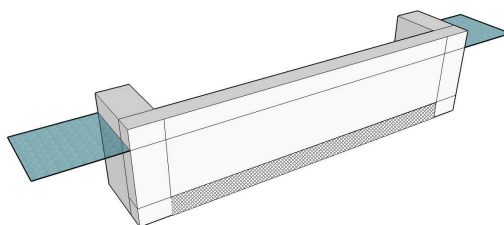
Load - Distributed loads Bow - Upper panel	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 0 meter	0 kN/m^2	1	0 kN/m^2			
Hydrostatic load Waterdepth 15 meter				150 kN/m^2	1	150 kN/m^2
Wave load	265 kN/m^2	1	265 kN/m^2	175 kN/m^2	1	175 kN/m^2
Total			265 kN/m^2			325 kN/m^2

Table G-37 The design moments for the upper part of the bow panel

Load - Moments Bow - Upper panel	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	22 [-]	1142,7 kNm/m	10,9 [-]	128,2 kNm/m	- [-]	1270,9 kNm/m
M_{fy}	17 [-]	883,0 kNm/m	10,2 [-]	120,0 kNm/m	- [-]	1003,0 kNm/m
Support moment						
M_{sx}	57 [-]	2960,6 kNm/m	30,2 [-]	355,3 kNm/m	- [-]	3315,9 kNm/m
$M_{sy,min}$	52 [-]	2700,9 kNm/m	18 [-]	211,9 kNm/m	- [-]	2912,8 kNm/m
$M_{sy,max}$	52 [-]	2700,9 kNm/m	36,4 [-]	427,6 kNm/m	- [-]	3128,5 kNm/m

Table G-38 The required pre-stressing and dimensions for the upper part of the bow panel

Bow Upper panel	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	900 mm	900 mm	900 mm	900 mm	900 mm
$F_{p,max}$	14175 kN	14175 kN	14175 kN	14175 kN	14175 kN
Eccentricity	150 mm	150 mm	150 mm	150 mm	150 mm
M_{ecc}	2126 kNm	2126 kNm	2126 kNm	2126 kNm	2126 kNm
M_{conc}	2126 kNm	2126 kNm	2126 kNm	2126 kNm	2126 kNm
M_{total}	4252 kNm	4252 kNm	4252 kNm	4252 kNm	4252 kNm
Sollicitation					
M_d	1271 kNm	1003 kNm	3316 kNm	2913 kNm	3129 kNm
Design					
Thickness	900 mm	900 mm	900 mm	900 mm	900 mm
F_p	9000 kN	10000 kN	12000 kN	10000 kN	12000 kN
Eccentricity	150 mm	150 mm	150 mm	150 mm	150 mm
$M_{total,d}$	2700 kNm	3000 kNm	3600 kNm	3000 kNm	3600 kNm

**Figure G.29** The location of the considered panel (shaded)**Table G-39** Distributed loads on lower part of the bow panel

Load - Distributed loads Bow - Lower panel	Top			Bottom		
	p	γ	p_d	p	γ	p_d
Hydrostatic load Waterdepth 15 meter	150 kN/m ²	1	150 kN/m ²			
Hydrostatic load Waterdepth 20 meter				200 kN/m ²	1	200 kN/m ²
Wave load	175 kN/m ²	1	175 kN/m ²	145 kN/m ²	1	145 kN/m ²
Total			325 kN/m ²			345 kN/m ²

Table G-40 The design moments for the lower part of the bow panel

Load - Moments Bow - Lower panel	Distributed loads		Triangle loads		Total	
	Factor	Moment	Factor	Moment	Factor	Moment
Field moment						
M_{fx}	10 [-]	81,3 kNm/m	4,7 [-]	2,4 kNm/m	- [-]	83,6 kNm/m
M_{fy}	43 [-]	349,4 kNm/m	25 [-]	12,5 kNm/m	- [-]	361,9 kNm/m
Support moment						
M_{sx}	49 [-]	398,1 kNm/m	29,6 [-]	14,8 kNm/m	- [-]	412,9 kNm/m
$M_{sy,min}$	83 [-]	674,4 kNm/m	38,5 [-]	19,2 kNm/m	- [-]	693,6 kNm/m
$M_{sy,max}$	83 [-]	674,4 kNm/m	53,8 [-]	26,9 kNm/m	- [-]	701,3 kNm/m

Table G-41 The required pre-stressing and dimensions for the lower part of the bow panel

Bow Lower panel	Field moments		Support moments		
	M_{fx}	M_{fy}	M_{sx}	$M_{sy,min}$	$M_{sy,max}$
Resistance					
Thickness	900 mm	900 mm	900 mm	900 mm	900 mm
$F_{p,max}$	14175 kN	14175 kN	14175 kN	14175 kN	14175 kN
Eccentricity	150 mm	150 mm	150 mm	150 mm	150 mm
M_{ecc}	2126 kNm	2126 kNm	2126 kNm	2126 kNm	2126 kNm
M_{conc}	2126 kNm	2126 kNm	2126 kNm	2126 kNm	2126 kNm
M_{total}	4252 kNm	4252 kNm	4252 kNm	4252 kNm	4252 kNm
Sollicitation					
M_d	84 kNm	362 kNm	413 kNm	694 kNm	701 kNm
Design					
Thickness	900 mm	900 mm	900 mm	900 mm	900 mm
F_p	2000 kN	3000 kN	2000 kN	3000 kN	3000 kN
Eccentricity	150 mm	150 mm	150 mm	150 mm	150 mm
$M_{total,d}$	600 kNm	900 kNm	600 kNm	900 kNm	900 kNm

G.5.4 Ballast

The bow-section is ballasted for the same purposes as the normal section is ballasted:

- To obtain the required draught
- To position the centre of gravity above the centre of buoyancy

While the compartments located between the inner and the outer hull can be used in a normal section, they can not be used in this section due to design of the pre-stressing system. The complete ballast of this element will be of sand. A part of the ballast may be placed in the form of ballast water in other elements. This will cause bending moments in the FB element as the weight will not be uniformly distributed anymore.

During construction and especially mating extra provisions are necessary to trim this element to a horizontal position because its weight is not uniformly distributed. The bow is heavier and thus the element will have the tendency to list to that side. During construction temporary ballast tanks in the bottom part may be used to counteract the list. During its service life these tanks should not be used because they are located low in the structure, causing a location of the COG which is too low.

G.5.5 Conclusion

In this section the bow/stern section has been designed. All important dimensions are found in the longitudinal cross-section shown in figure G.30. The transverse cross-section of the bow/stern section is similar to the transverse cross-section of a normal element as has been shown in figure G.19.

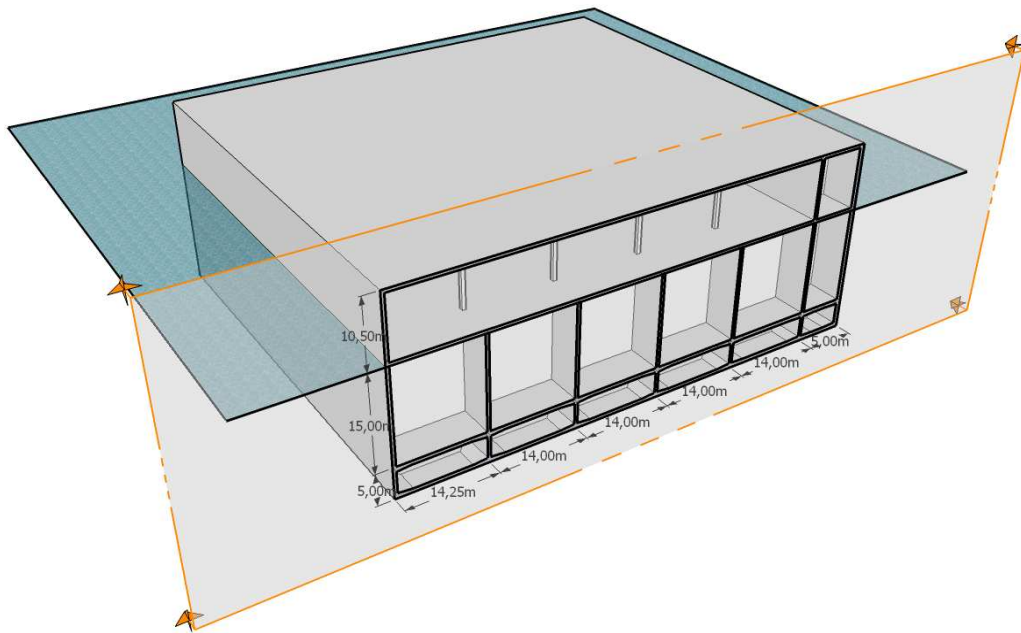


Figure G.30 Longitudinal cross-section of the bow/stern element

G.6 Mooring system

The mooring system is responsible for holding the FB element on its place. An equilibrium has to be sought between a very stiff system, with small deflections and large forces, and a very soft system, with large deflections and small forces. In the design of taut moored structures there are three types of synthetic ropes that can be used, see table G-42.

Table G-42 Material properties of mooring lines [Vryhof, 1999]

	Polyester	HMPE	Circular braided nylon
Material	High tenacity polyester	High modulus polyethylene	Nylon
Specific weight	1,38 [kg/m ³]	0,99 [kg/m ³]	1,14 [kg/m ³]
Elongation			
At 20% of MBL	3 [%]	0,8 [%]	16 [%]
At 50% of MBL	6 [%]	2 [%]	22 [%]
At break	12 [%]	4 [%]	>40 [%]
Tensile strength (MBL)			
Diameter 120 mm	6057 [kN]	3924 [kN]	3345 [kN]
Diameter 144 mm	8529 [kN]	5396 [kN]	4679 [kN]
Diameter 168 mm	11301 [kN]	6867 [kN]	6023 [kN]

Polyester has a high ultimate tensile strength combined with a moderate elasticity, HMPE and nylon have both a lower break strength. The elasticity of HMPE is so low that it can be considered as too stiff, resulting in higher forces and thus not suitable for this application.

When a water depth of 150 metres is considered, and the mooring line has an angle of 1:2, see figure G.31, the length of the mooring line can be determined with:

$$L_{moor} = \sqrt{x_{moor}^2 + z_{moor}^2} \quad (G.25)$$

Where:

- L_{moor} : The length of the mooring line;
- x_{moor} : The horizontal distance;
- z_{moor} : The water depth.

The approximate maximum translation can be calculated with:

$$x_{trans;max} = L_{moor} \varepsilon_{break} \quad (G.26)$$

Where:

- $x_{trans;max}$: The maximum translation of the element;
 ε_{break} : The elongation at break, see table G-42.

This leads to the following maximum translations:

- For polyester : 40 metres;
- For nylon : >100 metres.

The largest possible translation with nylon will reduce the mooring forces, but will impose requirements on the distance between two free-floating elements as they may not hit each other. A larger possible translation will lead to a larger required distance and thus to a higher transmission of the whole system. All wave energy will pass the breakwater on places where no element is present. Therefore it is chosen to limit the motions of the breakwater and polyester is chosen.

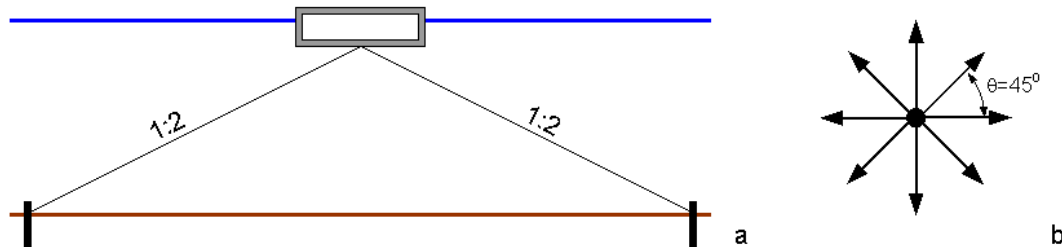


Figure G.31 Turret mooring system, side view (a) and plan view (b)

The lines contribute unequally to the total mooring force delivered as a result of the mooring spread plan. When a mooring line has an angle relative to the direction of deflection the elongation and the force is reduced with a cosine:

$$\begin{aligned} \varepsilon_{\theta} &= x \cos(\theta) \\ F_{moor} &= \varepsilon_{\theta} E \end{aligned} \quad (G.27)$$

Where:

- x : The motion of the element in arbitrary direction;
 θ : The angle between the considered mooring line and the direction of the motion;
 ε_{θ} : The resultant elongation;
 E : The elasticity modulus;
 F_{moor} : The contribution of the considered mooring line to the delivered mooring force;

The force that has to be withstood has been calculated in Appendix F.2. The magnitude of this force is 16242 kN. The design value can be determined with:

$$F_{moor;d} = \gamma F_{moor;s} \quad (G.28)$$

Where:

- $F_{moor;d}$: The design value of the mooring force, 24500 kN;
 γ : A safety factor, taken in this case as 1.5;
 $F_{moor;s}$: The mooring force as has been determined in Appendix F.2.

Table G-43 Mooring forces in the lines

Line nr.	MBL	Vertical direction		Horizontal direction		Resultant
		Angle	Red. cof.	Angle	Red. cof.	
1	8529 [kN]	1 : 2 [-]	0,89 [-]	-45 [°]	0,53 [-]	4007,455 [kN]
2	8529 [kN]	1 : 2 [-]	0,89 [-]	-45 [°]	0,53 [-]	4007,455 [kN]
3	8529 [kN]	1 : 2 [-]	0,89 [-]	0 [°]	1,00 [-]	7628,57 [kN]
4	8529 [kN]	1 : 2 [-]	0,89 [-]	0 [°]	1,00 [-]	7628,57 [kN]
5	8529 [kN]	1 : 2 [-]	0,89 [-]	45 [°]	0,53 [-]	4007,455 [kN]
6	8529 [kN]	1 : 2 [-]	0,89 [-]	45 [°]	0,53 [-]	4007,455 [kN]
Total				Required:	24500 [kN]	31286,96 [kN]

The required number of mooring lines is 16, divided in 8 pairs radiating away from the FB element in successive angles of 45°, see table G-43.

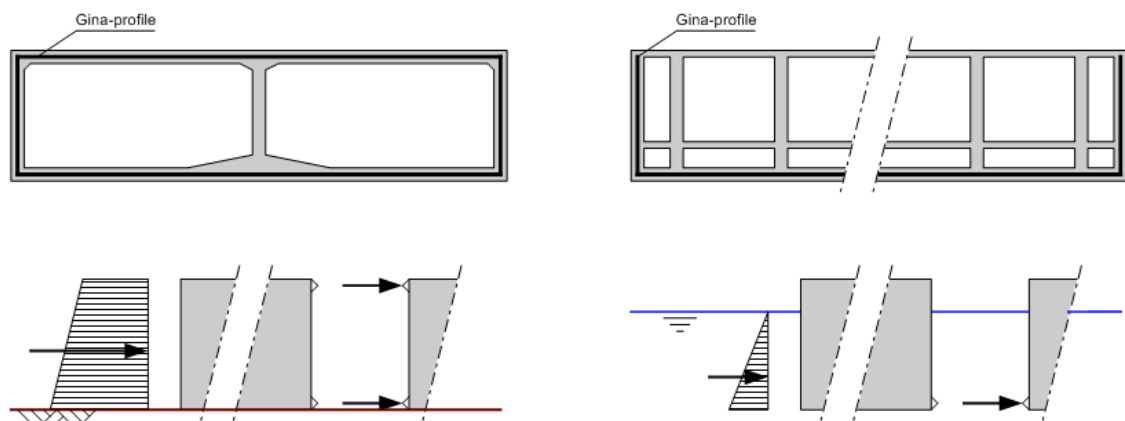
G.7 Connections

The sections will be combined to an element in a sheltered area. The connections between the sections should be completely stiff, i.e. the FB should not be influenced by the fact it is made out of several sections. Therefore extra attention has to be paid to the design of the connections between the sections.

First the temporary connection will be discussed as this connection is necessary to provide a dry a safe working area in which the permanent connection has to be constructed.

G.7.1 Temporary connection

When a rubber seal is used to create a dry working area, one can compare the seal that is to be used with the Gina-gasket used in the immersed tunnel construction in The Netherlands. The most important difference between a connection in an immersed tunnel and the connection between two sections of a FB element is that a tunnel segment is completely submersed. This means the hydrostatic pressure can be used for compressing the Gina-gasket, while for the FB extra measures have to be taken to assure a watertight seal. In figure G.32 a comparison is given between a tunnel segment and a FB.

**Figure G.32** Comparison between a tunnel segment and a FB element

It must be ensured that over the complete height of the element compression is present in the Gina-gasket. The pressure in the connection can be found with:

$$\sigma_{Gina} = \frac{M_d}{I_{zz}} + \frac{N}{A} \leq 0 \quad (\text{G.29})$$

Where:

- σ_{Gina} : The pressure in the Gina-gasket;
 M_d : The moment due to the eccentricity of the hydrostatic pressure relative to the neutral line of the gasket;
 I_{zz} : The section area moment of inertia.
 N : The resultant force of the hydrostatic pressure;
 A : The section area of the Gina-gasket.

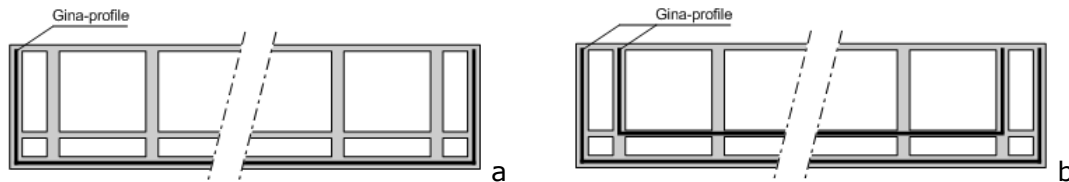


Figure G.33 Two designs for the Gina-gasket, one gasket (a) and a double gasket (b)

For two designs the minimum and maximum pressures on the Gina-gasket have been determined. The first design includes a single Gina-gasket which is located on the outer hull. The second design has a double Gina-gasket, one on the outer hull and one on the inner hull, see also figure G.33. All data on both designs is given in table G-44.

Table G-44 Minimum pressures and maximum pressures on the Gina-gasket for the two designs

	Single Gina-gasket	Double Gina-gasket
Element (all values from keel)		
Height	30 [m]	30 [m]
Beam	100 [m]	100 [m]
Location waterline	15 [m]	15 [m]
Gina profile		
Thickness	0,2 [m]	0,2 [m]
Neutral line	5,63 [m]	7,47 [m]
I_{zz}	2587,5 [m ⁴]	2696,0 [m ⁴]
Forces, moments and stresses		
Hydrostatic force	450000 [kN]	450000 [kN]
Pressure line	5 [m]	5 [m]
Moment	281250 [kNm]	1109694 [kNm]
Min stress (compression at top)	-11,41 [N/mm ²]	tension [N/mm ²]
Max stress (compression at bottom)	-14,67 [N/mm ²]	-10,73 [N/mm ²]

From table G-44 it can be seen that the application of a double gasket leads to tensile stresses at the top of the Gina-gasket. With a single Gina-gasket no tensile stresses are present. Because no tensile stresses are present, no mitigating measures have to be taken to guarantee a watertight connection. Therefore it is chosen to apply a single gasket as a temporary connection.

G.7.2 Permanent connection

The final permanent connection should be completely stiff and has to resist the global moments which have been determined in Appendix F.3. The connections have to be pre-tensioned as no tensile stresses are allowed. The method for determining the required pre-stressing force has been described in section G.2.

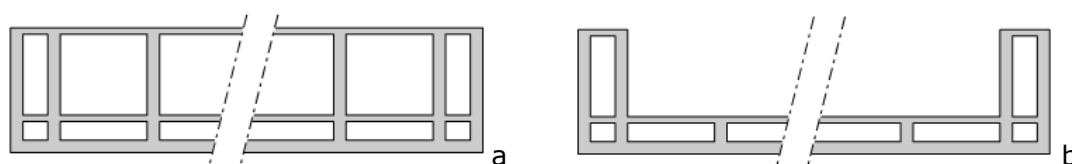


Figure G.34 Designs for the connection, inner walls and decks connected (a) and not connected (b)

For both designs presented in figure G.34 the required pre-tension stresses in the concrete is low:

- For the design where the inner walls of two sections are interconnected, see figure G.34a, the required stress is 0.46 N/mm^2 . This is equal to a pre-stress force of 168 MN in total.
- For the design where the inner walls of two sections are not interconnected, see figure G.34b, the required stress is 0.95 N/mm^2 . This is equal to a pre-stress force of 215 MN in total.

To ease the construction it is chosen not to connect the inner walls.

The ducts for the pre-stress cables are installed and the walls are poured in the dry working space. When the concrete has cured the pre-stress tendons are installed in the ducts and tensioned up to the required level. In this way a complete element, which is completely under compression in all conditions, can be assembled by individual elements.

G.8 References

Vryhof (1999), *"Vryhof anchor manual 2000"*, Vryhof Anchors b.v., Krimpen a/d IJssel

