Breakwater Design for protection of floating houses in Haringvliet Region

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1. Introduction

1. 1 Problem Description

A floating house development project to be constructed in the coast of Hellevoetsluis, in the municipality of Voorne-Putten, is under consideration. Within a project of this nature, several aspects need to be taken into account in order to safeguard the infrastructure against additional loadings to which regular houses are not exposed to.

Hydrodynamic forcing could directly or indirectly cause damage and unpleasant displacements on the houses, as they float on the waters of the Harringvliet estuary. The above can be undermined by protecting the area against waves, the most relevant loading for this site, by means of a wave wall or breakwater. By doing so, waves impacting directly on the houses are avoided; additionally calm waters behind the structure guarantee adequate condition for living facilities.

1.2 Floating Houses

Floating house developments in The Netherlands and throughout the world are increasing in popularity. A floating house differs mainly in one aspect from other terrestrial house: the house rests on the water instead of pile foundations. For this reason their design requires a more thought out design as opposed to its land counterparts, as it was mentioned above.

A floating house can typically sway to and fro in case of external forcing. However, a certain level of balance needs to be maintained by controlling the degree of its tilt. The Floating constructions committee of the Netherlands Standardization Institute (NEN), defined in the Dutch Technical Agreement (NTA, October 2011) an acceptable tilting of the houses of four degrees as long as the distance to the adjacent house remains at least three meters (Witsen, 2012).

For the elaboration of this report, the following characteristics of a floating house development in the IJburg district in East Amsterdam (Floating Houses IJburg, 2011), were considered:

Total Area: 50 000 m^2
Minimum width of houses: 5 meters
Design lifespan: 50 years

1.3 Boundary condition

According to the preceding, a tilting of a maximum of four degrees will be allowed for the houses in question; this can be later translated into a wave height of 0.7 m.

1.4 Project Objective

1.4.1 General Objective

To provide information regarding the potential technical solutions that are required to ensure adequate site conditions for the construction and realization of the floating houses in Hellevoetsluis.

1.4.2 Particular Objectives

- To design the cross section of three different type of breakwaters that meet the stability safety standards and that satisfy the functionality requirements (i.e. reducing any incoming wave to a height of a maximum 0.7 meters).

- To determine a breakwater layout that will give a protected area against waves of approximately 50,000m^2.

- To carry out a multicriteria analysis to point out which type of breakwater is the most convenient for the project in question.

2. Site description

2.1 Geography

The Haringvliet is a large inlet of the North Sea, in the province of South Holland in the Netherlands. It belongs to the lower estuarine part of the Rhine and Meuse Rivers. It is closed off near Goedereede from the North Sea by Haringvliet dam. The dam with its sluices was built as a part of the Delta Works sea barrier protection works, and allow for a brackish ecological environment

2.2 Surroundings:

2.2.1 Islands

Tiangemeten (natural reserve): Tiangemeten is an island estuary of the rivers Rhine and Meuse, just south of Rotterdam.

Slijkplaat Haringvliet (protected area): The uninhabited tiny island Slijkplaat is part of (the archipelago) Haringvliet Islands. The island is situated in a closed estuary. The surface of the island is 0.51 km^2. The island lies in the Haringvliet and the total coastal length is about 4 km and the length of the island is about 1.6 km.

Fig.1. Locations of islands near the Haringvliet zone

2.2.2 Ports

There are some small ports in the vicinity of the Haringvliet zone. The ports are mainly used for tourism (yachts) and for aquatic sports.

- **A. Helliushaven**
- **B. Haavn het grote dok**
- **C. Koopvaardijhaven**
- **D. Tramhaven**

Fig.2. Locations of ports near the Haringvliet zone

2.3 Ecological aspects

As a result of the construction of the Haringvliet dam in 1970, the tide and ebb of the flow practically ceased. The effect has had serious consequences for the vegetation and fauna of the Haringvliet, including the island of Tiangemeten. In 1990, the Dutch government decided to incorporate Tiangemeten in the Ecological Main Structure (EHS), the network of existing and new nature reserves in the Netherlands.

2.4 Important players

a. *Owners of floating homes:*

The prospective owners of the floating houses will be important stakeholders as they are the intended customers of this project. Their role could be instrumental in deciding the way the project shapes up. For example, the aesthetics of the surroundings will have to be retained in such a way that the owners are pleased with the view.

b. *Construction company:*

The construction company is an important player for two reasons, firstly because it will be responsible for the design and building of the final project and secondly because it will be the prime financial beneficiary from the project. For the company, the focus will be to deliver the most satisfactory project while maximizing its profits.

c. Government:

The government will be an important participant in the project since the project concerns the waters and terrestrial surroundings of the Netherlands. As mentioned earlier, the government will have interests in protecting the areas it has designated as ecological reserves.

d. Environmental activists:

The environmental organisations may play a role in ensuring that all environmental safeguards are in place while the project is being executed.

e. Tourists:

Lastly, as noted earlier, a few areas nearby are meant for leisure activities along the coast. Therefore, tourists visiting these areas may expect that the project does not affect their areas and activities adversely.

3. Determination of design load conditions

3.1. Wind data compilation

Having obtained the data (KNMI website) for maximum hourly wind-speed and wind direction, the data was further classified into sections based on the angle of orientation of wind and the speed. 16 sections for the directions were made, as 0-22.5 degree, 22.5-45 degree up to 337.5-360 degree. The data was then sorted based on the wind direction value into one of these sections.

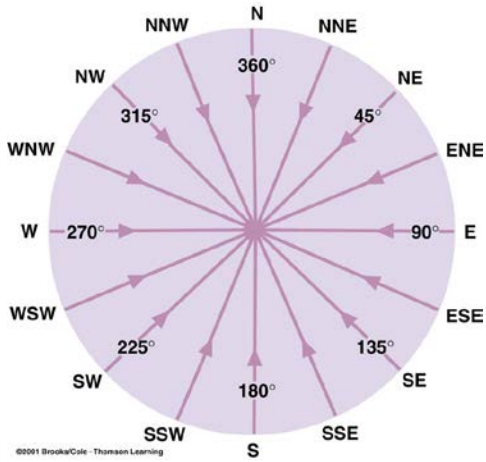

Fig.3. Division of directions to 16 regions

After having obtained this classification, a decision was made as to which of these sections will be taken into consideration for further analysis. After observing the geography of the region assigned for the project and the location of the sluice, it was decided that the angles ranging from lower than 90 to higher than 270 will be eliminated (see Appendix A for more details). These would be mostly winds blowing from the east or from west in the northern half of the 360 degree circle. Given the location of the sluice and the Haringvliet zone where the breakwater is proposed, these directions are the ones from which most the wind would be either blocked by the sluice or if not, would have negligible effect on wave creation. Therefore, the wind orientations between 90 to 270 degree were considered for analysis. The sorted data reflected that most of the dominant wind-speeds lay in the 180 degree to 270 degree bracket and particularly nearer to 270, i.e. from south-west or west.

Angle Range	**Number of Readings**
0-22,5	956
22,5-45	1053
45-67,5	760
67,5-90	739
90-112,5	664
112,5-135	520
135-157,5	483
157,5-180	753

180-202,5	1228
202,5-225	1590
225-247,5	2024
247,5-270	1490
270-292,5	1016
292,5-315	854
315-337,5	760
337,5-360	813
	15703

Table 1. Wind-data

3.2 Sorting of data on the basis of wind speed

Taking the values in these orientations, the data was again divided based on wind speed into sections such as 0-5 m/s, 5-10 m/s and so on up to 25-30 m/s. Based on the highest values and number of recurrences of those high values, an estimate could be made of the highest wind/ wave condition i.e. the worst case scenario. Wind-rose diagram (figure 4) gives more insight into this:

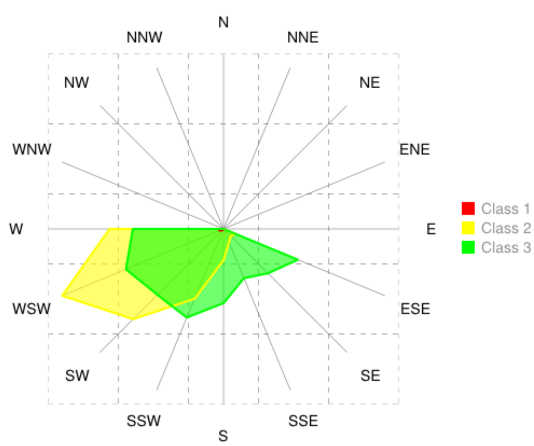

Fig.4. Wind-rose diagram

Wind Speed(upper limit in m/s)	No. of Readings per bin
5	479
10	4218
15	3298
20	704
25	82
30	5

Table 2. Wind-data (contd.)

3.3. Wind to wave conversión

3.3.1 Determination of fetch

Similar to wind data interpretation, 22.5° intervals were used to determine different fetches affecting the waves up to the desired breakwater construction site. Seven intervals ranging from 112.5° to 270° were relevant for wave formation and growth. Fetch was measured at the upper limit of the interval from the breakwater site to the nearest coastline obstacle in each section.

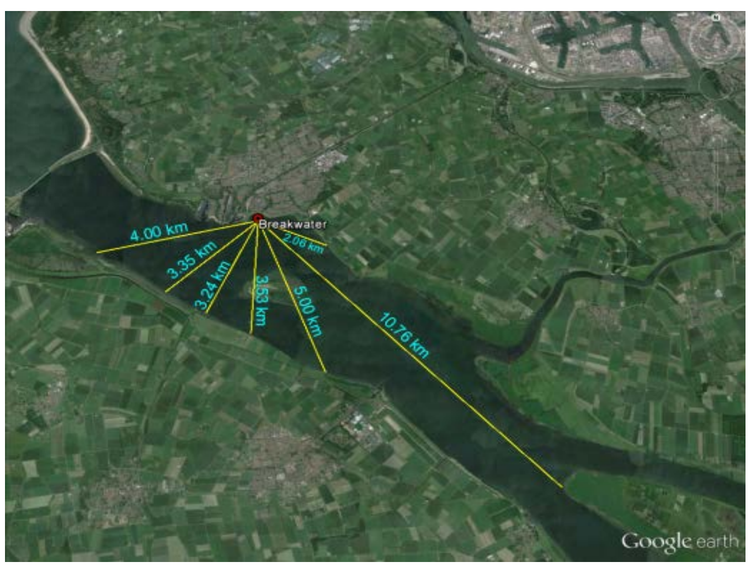

Fig.5. Fetch diagram

3.3.2 Using bathymetry to calculate average depth

Once the distances over which the estimated wind condition would blow were known, the bathymetry along these transects was analysed. Since variations were not too big, a representative water depth value was determined for each transect based on their predominant water depth . The software Map 610 Max was used for this purpose.

3.4. Determination of design significant wave height

The wind speeds from the predominant direction were then classified into three groups 0-10 m/s, 10-20 m/s and 20-30 m/s. Number of readings from the given directions in each of these categories were found out. Dividing these values by the total number of readings gave the corresponding probabilities of occurrences. Given the direction and the wind speed category, the significant wave heights were calculated using Young and Verhagen's formula, 1996 (see appendix A).

The obtained significant wave heights were then arranged in an ascending order and their respective cumulative probabilities were calculated. Conversely, their respective exceedance probabilities were also calculated (see appendix A for details)..

Now, the aim was calculate the maximum significant wave height for a return period of 475 years. The return period of the design storm, 475 years, was determined by proposing a structure with a life time of 50 years and by allowing a 10% probability of failure, which is assumed if the design condition occurs ($f = -\frac{1}{50}\ln(1-0.1)$).

Since the wind-speed readings used above were not exactly periodic, a specific storm period of 3 hours was assumed based on the estimated period of wind fluctuations in the Netherlands. Now, a 3 hour storm period means 8 periods in a day and 8*365 in a year. Therefore, probability of one-storm per year will be 1/8*365 i.e. approximately 1/3000. The wave height of 0.75 m was giving probability of 0.038 which would be nearly a 10 storm per year probability. A curve was then plotted for wave height versus the logarithm of exceedance probability. Extrapolating the curve, the wave height corresponding to 1 storm in 475 years was calculated to be 1.2 m.

4. Determination of the boundary condition

The allowable wave height criterion for the floating houses is maximum allowable tilt of 4^o. Assuming a complete wavelength of sinusoidal wave, it represents an angular spectrum of 0^o to 360^o. If the house is tilted due to wave action, the centre of the tilted base will rest on the point of inflexion of the wave. Noting that a minimum house width will lead to the smallest value of 'highest allowable wave height' and wider houses will effectively be able to take on higher waves, the width of the house is set to its minimum at 5 m. Then, half-width of house is 2.5 m. Therefore, the height to which the house is tilted (if tilt is 4^o) is 2.5*sin(4 degree). Also the projection of the tilted half-width on to the horizontal axis will be 2.5*cos(4^o) which is almost 2.5 as cos(4^o) is approximately 1.

Now, as explained earlier, the entire wavelength (in this case, wavelength is 30 m) corresponds to 360^o. Therefore, the angle corresponding to 2.5 m is 360/12 that is 30^o. Now, the full amplitude of the wave A (which is also 0.5*H) corresponds to a 90^o angle. A 36^o angle will therefore correspond to a local wave elevation of A*sin36^o for sinusoidal wave.

This A*sin(30^o) which is the water level elevation should be equal to 2.5*sin(4^o), the maximum allowable height to which the house is tilted.

Therefore, $A*\sin(30^o)=2.5*\sin(4^o)$ i.e. $0.5*H*\sin(30^o)=2.5*\sin(4^o)$

»» $H = 2*2.5*\sin(4^o)/\sin(30^o) = 0.7$ m

Thus, the maximum allowable wave height for the floating house becomes 0.7 metre.

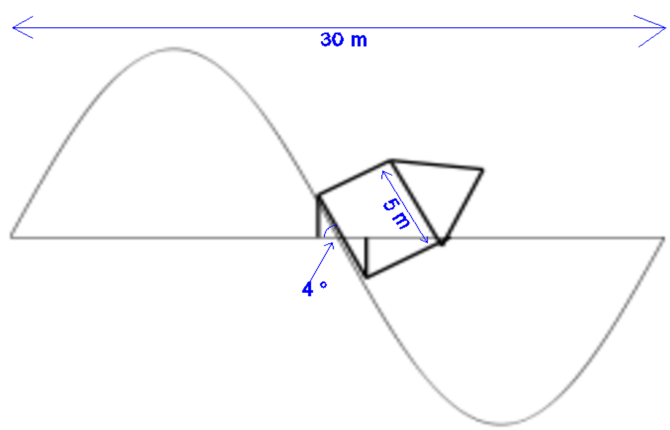

Fig.6: Maximum allowed tilting of house due to wave

5. Design of breakwaters

5.1 Possible alternatives

Once all the design basis and boundary conditions has been investigated in previous sections, the cross-sections of three different type of breakwaters will be designed. Functionality comparison, cost and maintenance analysis will be discussed in later sections.

First, two types of rubble mound breakwater will be considered. Since no quarry is found in the proximity of the constructing area, a concrete rubble mound breakwater is also put under consideration. A monolithic caisson cross-section is considered next. At last, literature suggests the use of floating breakwater in conditions of short waves and deep water, which sounds reasonable for the conditions within this project (Verhagen, H. J., & Angremond, K., 2009).

5.2 Rubble mound breakwater using stones

5.2.1 Dimensions:

Van der Meer Formula has been used for this design because it includes more parameters and it gives better results as compared to Hudson as it takes into account wave period, storm duration and the permeability of the breakwater structure. As per calculated design boundary conditions, notional permeability value (P) is chosen to be 0.4, damage level (s) = 1 (usually 1-3 is a safe range) , design wave height =1.2 metres , slope of 2:1.

$$\frac{H_{sc}}{\Delta d_{n50}} = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \varepsilon^{-0.5}$$

Where, P= Permeability of structure

N= Number of waves

S = Damage Level

ε = Iribarren Number

Using equation mentioned above , calculated stone size is found to be 0.5 m (Dn50) which corresponds to stone class HM_a 300-1000 Kg.

Also, the crest width is stipulated to be at least three stones and therefore in this case will be 1.5 m.

The layer thickness is the diameter multiplied by the number of stones across the layer. It is usually recommended to have 2-3 stones across the layer. Taking into account overtopping constraints, it was decided in this case to have 3. Therefore, the layer thickness was calculated to be 1.5 m. The first under layer and the toe berm is projected to use stones of weight 20 kg (ratio of 1/15 as compared to the armour layer) and a diameter of approximately 0.2 m. Furthermore, the core layer will use stones 15 times lighter than the under layer and therefore of mass between 1-3 kg. Also, a filter may be proposed under the breakwater on the seaward side with a filter layer thickness not less than 0.5 m.

5.2.2 Transmission:

Here, calculated transmission coefficient is 0.10 which is well within allowable transmitted wave height range. BreakWat Software has been used for above calculations.

5.2.3 Overtopping:

Overtopping is the quantity of water that is passing over the crest of the structure. It is usually expressed as specific discharge per unit length. In this case this value is around 10 l/sec/m , this is an order higher than 1 l/m/s limit specified in Eurotop Manual, however it is as close as design could get to required limit. The BreakWat software also gives a limiting output as the maximum overtopping volume. Maximum overtopped volume in this case is 3.03 m^3/m.

5.2.4 Rear side dimensions:

The stone diameter was found to be 0.2 m with the mass as 20 kg. This is in keeping with the observation that as long as the crest of the structure is tall enough to prevent overtopping, the armour units on the rear slope can be much smaller (and lighter) than those on the front slope. The rear side slope is taken to be 1:2.

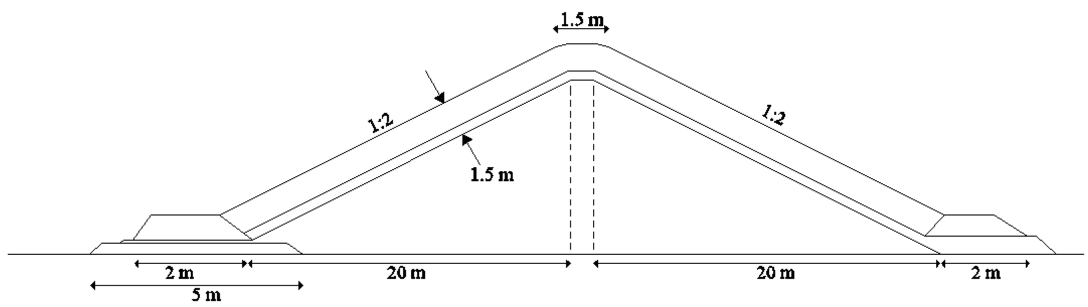

Fig.7. Cross-section of rubble mound breakwater using stones

5.3 Rubble Mound Breakwater using Concrete Cubes

5.3.1 Dimensions:

Now, for rubble mound breakwater, if concrete cube is used, using same procedure as mentioned above, calculated size of concrete cube is 0.4 m (D_{n50}) using Van der Meer Formula (see Appendix B).

Also, the crest width is stipulated to be at least three stones and therefore in this case will be 1.2 m.

The layer thickness is the diameter multiplied by the number of stones across the layer. It is usually recommended to have 2-3 stones across the layer. Taking into account overtopping constraints, it was decided in this case to have 3. Therefore, the layer thickness was calculated to be 1.2 m. The first under layer and the toe berm is projected to use stones of weight 10 kg (ratio of 1/20 as compared to the armour layer which is an acceptable limit) and a diameter of approximately 0.1 m. Note that choosing a relatively smaller stone size for the under layer may lead to requirement for increasing the size of the cubes in the armour layer. But, for concrete breakwaters, this is quite acceptable and usually, for increasing breakwater stability, the adhered norm is to increase the cube size as opposed to reducing slope as in the case of stone rubble mound breakwaters. Furthermore, the core layer will use stones 15 times lighter than the under layer and therefore of mass less than a kg. Also, a filter may be proposed under the breakwater on the seaward side with a filter layer thickness not less than 0.5 m.

5.3.2 Transmission:

Computed transmission coefficient K_t is 0.28.

5.3.3 Overtopping:

However, damage level being used in both (for stone and concrete cube) is different because using same damage level 2 (number of actually displaced units) gives a high overtopping, thus to reduce overtopping volume, crest height is increased and damage level is reduced further in case of 1. The specific overtopping discharge in this case is around 3 l/s/m which is close to acceptable value specified in Eurotop Manual.

The maximum overtopping volume for this design is 1.73 m^3/m. BreakWat software has been used here as well for calculations.

5.3.4 Rear side dimensions:

The stone diameter was found to be 0.2 m with the mass as 20 kg. The rear side slope is taken to be 1:2. It is worth noting that the inner slope could have been steeper but due to constraints of the software, it has been kept as 1:2.

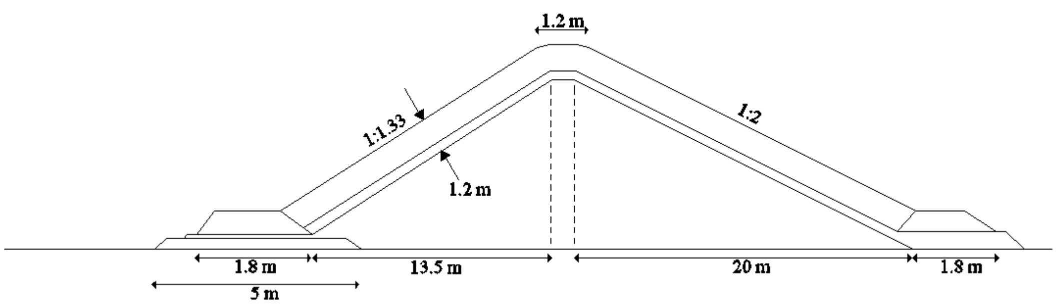

Fig.8. Cross-section of rubble mound breakwater using concrete cubes

5.4 Caisson Breakwater

Caisson type breakwaters are normally constructed as reinforced concrete caisson, built in a dry dock and towed to its position and lowered to a prepared stone bed.

5.4.1 Dimensions

Based on the boundary conditions discussed before, a caisson breakwater is designed. The dimensions of breakwater cross sections is shown in the figure 10 below, which is fit for design wave height 1.2m, period 4.6s and wave length 31m.

5.4.2 Stability

Making use of breakwater design website Cress, safety factors for sliding and overturning are calculated. Sliding safety factor is 2.3 and overturning safety factor is 1.22, which satisfy the requirement , larger than 1.2. Goda formula (see appendix B) is deployed for wave action calculation on vertical walls of caisson breakwater.

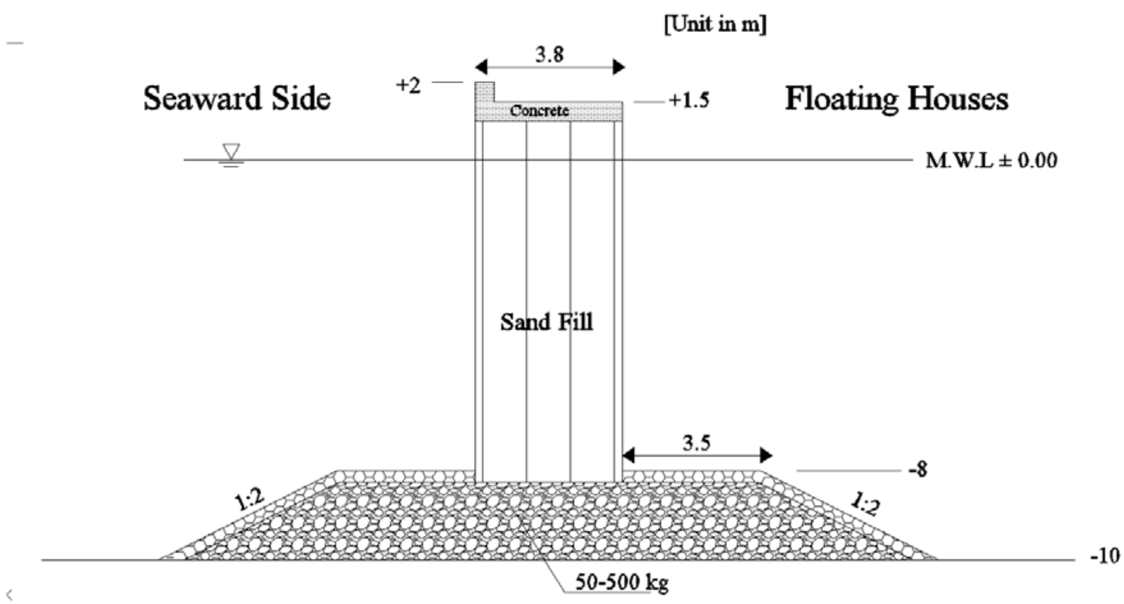

Fig.9. Cross-section of caisson breakwater

5.4.3 Wave overtopping

Overtopping is another important parameter to check caisson breakwater design. In order to calculate time mean discharge of nonbreaking wave, Van der Meer and Jansen (1995) formula, is used. Overtopping is 0.008 $m^3/s/m$. Again, this a notch higher than the .001

m^3/s/m limit listed by Eurotop Manual. One must note that limit value here relates to the effective overtopping at the building (floating houses). But the value calculated in design is at the breakwater itself.

5.5 Floating Breakwater

Box type breakwater is considered here. Design of floating breakwater has been done using the book *"Floating Breakwaters: A Practical Guide for Design and Construction*" by PIANC. The graphs based on researches done by Thompson (1989) regarding initial statement on attenuation properties and Jones (1971) based on effect of width and relative water depth for floating breakwater are used extensively in the design procedure which are further elaborated with design calculations in Appendix B.

Economically the best option found to be width, w=3m and drought, z=5m for box type floating breakwater. The length of each pontoon is considered to be 20.0 m.

Freeboard is decided according to accepted overtopping rate. It is considered that 2.0m of freeboard in this case will be a good assumption. For this freeboard, calculated wave overtopping (Pullen et al. 2007) is found to be same as that in case of caisson breakwater. So finally with the checked freeboard the final cross section with draught becomes 3m x 7m (for box type). A diagram of design cross section of the floating breakwater is presented below.

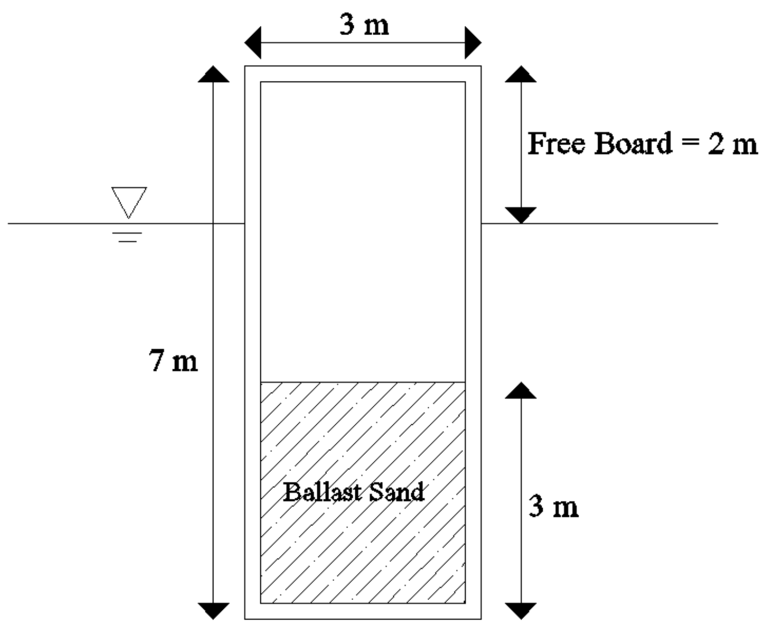

Fig.10. Cross-section of floating breakwater

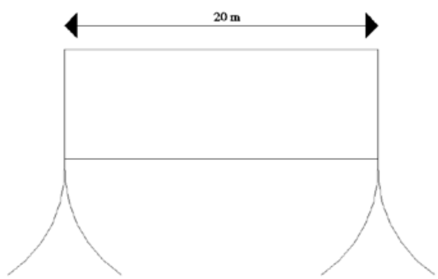

Fig.11. Front view of floating breakwater with moorings

5.6 Breakwater Layout

A breakwater layout was determined based on the protected area requirements. The floating house development mentioned in section 1.2 was taken into consideration, giving a calm area of 50,000 m^2 needed for the house development.

A breakwater with a length of 250 m, gives a protected area of over 50,000 m^2. The height for the incident waves diffracting on the head of the structure was checked using a wave diffraction diagram with a wave angle of incidence of 135^o (Shore Protection Manual, 1984). The diffraction coefficients in the protected zone are all below 0.2, giving a considerable reduction of the wave height, and fulfilling the restriction of wave height of 0.7 m (Figure 12)

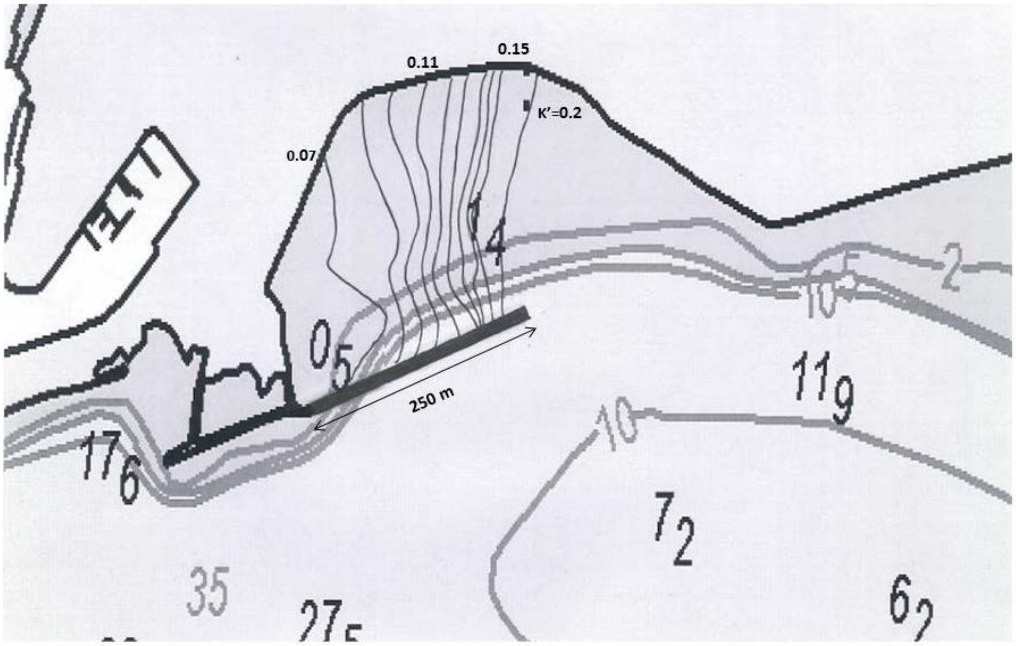

Fig.12. Diffraction determination

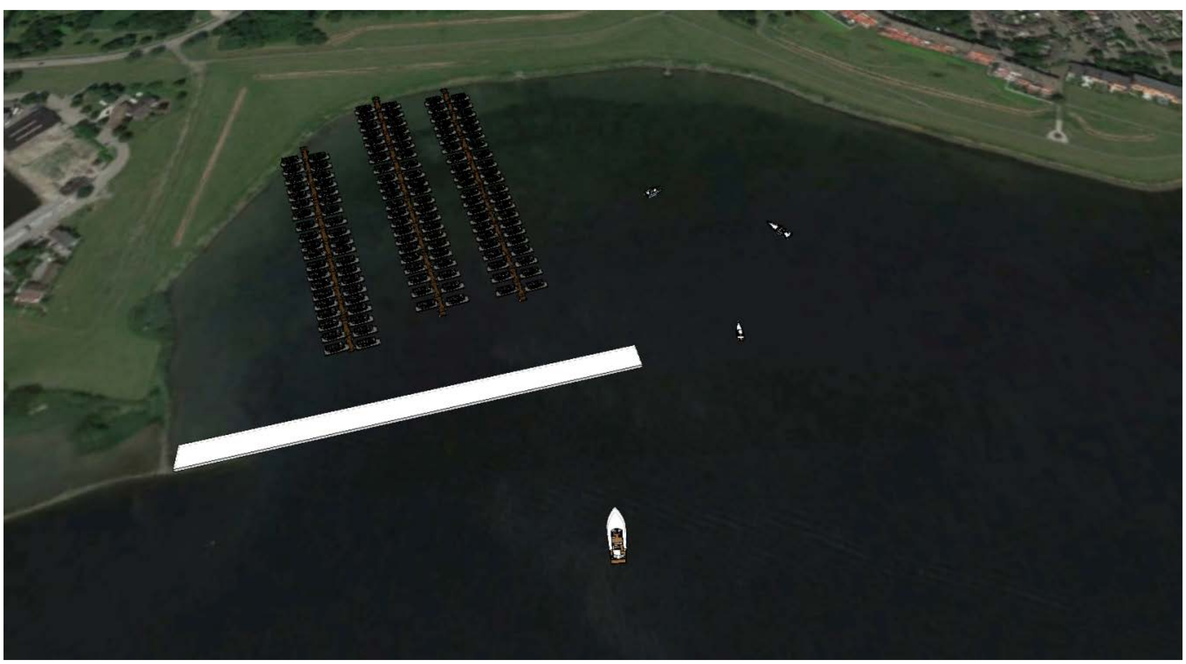

Fig.13. Layout of breakwater at the site

6. Comparison of alternatives

6.1. Functionality:

The aim of the project was to design the breakwater to carry out the required function of protecting the floating houses. Therefore, all the four breakwaters have been designed to satisfy this functional requirement. As a result, the breakwaters have been so designed that the transmission co-efficient and overtopping volume of each breakwater is within the allowable limit. Therefore, the intention was to design all breakwaters for the appropriate functionality and then compare on factors such as cost, maintenance, risk and environmental impact.

6.2. Cost:

The cost of each breakwater was estimated after considering material, transportation and labour costs. However, despite the cost calculations, specific uncertainties could arise due to price fluctuations and assumptions in these calculations. For example: The mooring cost for the floating breakwater was assumed to be equal to that of the main block (based on another floating breakwater calculation), which may not be the case. (d'Angremond et al., 1998). Based on these calculations, costs of breakwaters have been estimated to a certain order which should correspond to the ranges mentioned in the table below:

Rubble mound (stone)	Rubble mound (concrete)	Caisson	Floating
12-15 million (in EUR)*	7-9 million*	8-10 million*	1-2 million*
Costliest alternative	Intermediate costs	Intermediate costs	Cheapest alternative

Table 3. Cost Comparison table of different breakwaters

*Refer appendix C for calculations

6.3. Maintenance:

Maintenance is an important consideration when making a comparison between different breakwater structures. Some, like the rubble mound and floating breakwaters, may require consistent and periodic maintenance cycles and this could add considerably to the prospective cost of the project. The caisson breakwater on the other hand may not need much maintenance though the costs after complete failure may be severe. Listed below are some qualitative comments on the maintenance patterns that each of the breakwaters entail.

Rubble mound (stone)	Rubble mound (concrete)	Caisson	Floating
High	Medium (relatively less compared to stone)	Low (Almost negligible)	High
Could be a considerable proportion of the initial cost depending on extent of damage. Also worth pointing out is the fact that a solid cap block is not implemented which means floating equipment could be required for maintenance purposes.	Could be considerable proportion of the initial cost depending on extent of damage (Note though that the initial cost of the concrete rubble mound breakwater was low)	Almost zero maintenance compared to other breakwaters	Mainly focused on moorings and connections, hence could be a small portion of the initial cost.
Needs to be done if and when stones are washed away	Needs to be done if and when concrete blocks are washed away		Periodic inspection and maintenance is required

Table 4. Maintenance Comparison table of different breakwaters

6.4. Risk:

In case of complete failure, different breakwaters have been compared in table shown below:

	Rubble mound breakwater	Caisson breakwater	Floating breakwater
Effect on houses	Some stones and debris could be washed towards the floating houses but no major disturbance	Overturning of the caisson (complete failure) may cause the sand and concrete to spill out into the areas around the houses	In case of mooring-line failure (complete failure), the floating breakwater may float closer to houses and even hit them causing damage
Damage to the breakwater structure itself	In case of complete failure, most of the stones could be washed away	The caisson structure will topple down and undergo considerable damage to the extent that it cannot be reused	Except for the moorings, the main block of the floating breakwater may not be damaged significantly
Effect on environment	Finer rubble that is washed may affect water quality and suspended particles affect flora and fauna in top layer	Excess spilling of sand and concrete may lead to excessive sedimentation and deterioration of water quality	Relatively insignificant effect on surroundings
Type of repairs	Most of the stones that have been washed away will have to be replaced	The underlying concrete debris will have to be cleaned and an entirely new caisson breakwater ought to be built	In most cases, only the moorings will need replacement and strengthening but the structure as such may stay intact
Cost of repairs	++ Cost of new stones used for replacement (may depend on how much damage has been caused, could be a considerable percentage of initial cost)	+++ Cost will be high since an entirely new breakwater will have to be constructed (will be even higher than the initial cost because cost of clean-up will be added)	+ Repair cost will mainly comprise the cost of moorings (approximately 50-60 percent of the initial cost)

Table 5. Risk Comparison table of different breakwaters in case of complete failure

6.5 Advantages and Disadvantages of each alternatives:

Advantages of rubble mound breakwaters (stone) :

- It is easy to maintain.
- It provides a better transmission coefficient as compared to floating breakwater.

Disadvantages of rubble mound breakwaters (stone):

- Regular maintenance and inspection is required
- Expensive solution as compared to floating breakwater.

Advantages of rubble mound breakwaters (concrete):

- It required much smaller cubes (compared to stone), thereby making the breakwater lighter and cheaper.
- It provides a better transmission coefficient as compared to floating breakwater.

Disadvantages of rubble mound breakwaters (concrete)::

- Regular maintenance and inspection is required.
- Placement of the cubes needs to be considered and considerable planning & costs may be associated.

Advantages of caisson breakwater:

- It effects almost zero transmission.
- It has a relative cost advantage over rubble mound (stone) breakwater.

Disadvantages of caisson breakwater:

- Good bed soil quality is required for construction of the caisson breakwater to prevent it from settling. Therefore, prior investigation and associated costs needs to be considered.
- Including the crown, the breakwater juts 2 m above water level which could be aesthetically disturbing for the residents as it would affect their view.

Advantages of floating breakwater:

- In this case, the floating breakwater requires the least amount of material and therefore incurs least investment.
- Also, the floating breakwater is projected to affect the environment the least in case of failure.

Disadvantages of floating breakwater:

- It is tougher to convince the buyers of floating houses to purchase these behind the floating breakwaters since the floating breakwaters do not give a psychological sense of complete protection.
- They entail a relatively high transmission co-efficient, though within the allowable limits.

7. Conclusion:

The four breakwater alternatives proposed for floating-house protection have been designed and compared in the above sections. The choice of the final alternative for implementation depends on the specific constraints of the client.

For example, if financing is the major criterion, it is evident that the floating breakwater will be the choice as it provides good functionality at a relatively low budget. Given the fact that the initial investment for this type of structure is so low, a considerable amount of money can be destined to periodic inspections, maintenance and avoiding the risks involved in case of failure; which are the main concerns for this breakwater. However, as noted earlier, some residents may be uncomfortable with the thought of a floating breakwater protecting their houses (with the impending danger of hitting the houses in case of a mooring failure). In that case, the rubble mound breakwater with concrete cubes presents the next best alternative economically. Also, if complete protection (i.e. zero-transmission) is the deciding factor for the residents then they may choose the caisson breakwater at the cost of spending considerably more. The stone-based rubble mound breakwater, one may argue, is a good alternative from the point of view of aesthetics as it would be using only stones available in nature avoiding the visual pollution caused by concrete structures. However, it is an unlikely choice given that its costs are exorbitant when compared to any other alternative.

From the above discussion it would be safe to conclude that the choice of the breakwater to be constructed for this project and moreover the decision on whether the realization of this project is feasible whatsoever rests with the client, as different weight can be given to each of the comparing factors based on the client's interests. Through this report, however, an effort has been made to provide certain indications as to the consequences of a particular choice with respect to various factors. It is believed that these analyses will prove comprehensive in helping the clients make an appropriate decision according to their specific budget and liking.

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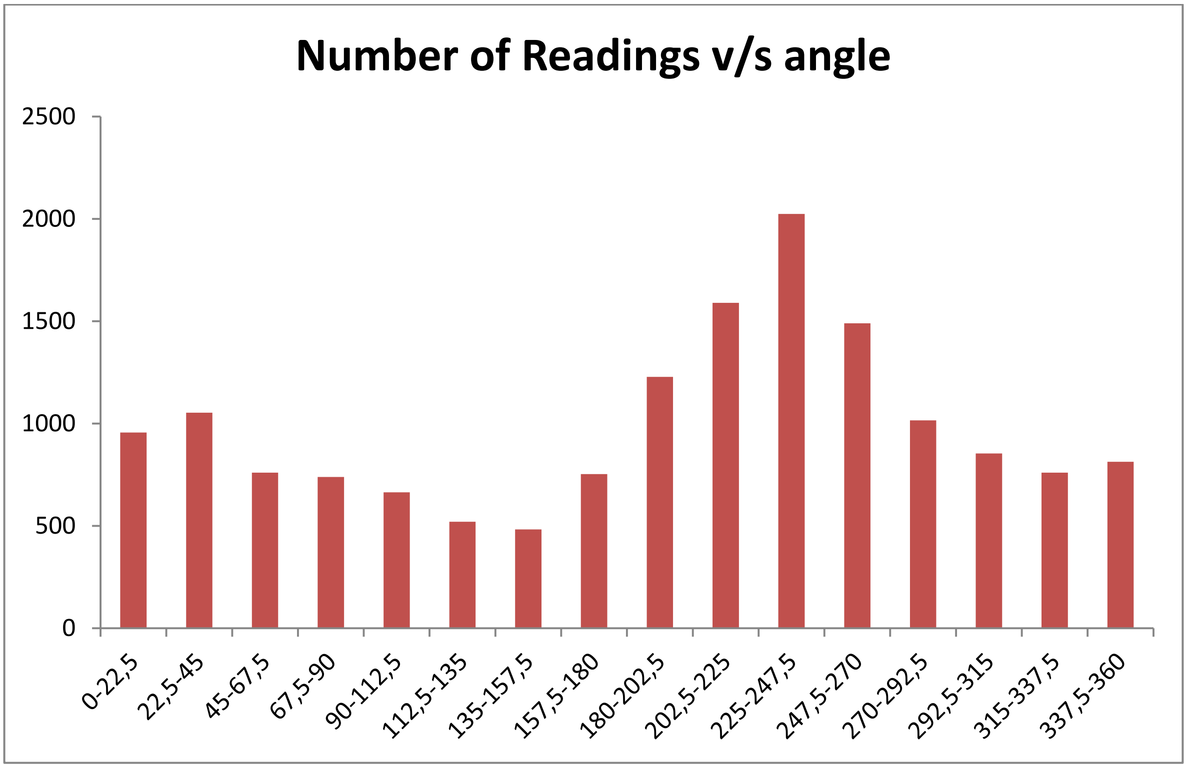
Appendix A

Classification of wind speed by direction and magnitude and the corresponding wave heights:

Wind direction	Wind speed	Fetch(m)	Depth(m)	Hs(m)	Tp(s)	Occurrence(%)	speed	Lambda
125-157.5	15	7800	10	0.64	3.36	0.507614213	9.904544	33.27927
	20	7800	10	0.85	3.82	0	9.904544	37.83536
	25	7800	10	1.06	4.22	0	9.904544	41.79718
157.5-180	15	4200	8	0.47	2.85	2.157360406	8.858894	25.24785
	20	4200	8	0.63	3.25	0	8.858894	28.7914
	25	4200	8	0.79	3.59	0	8.858894	31.80343
180-202.5	15	3400	6	0.43	2.69	9.010152284	7.672027	20.63775
	20	3400	6	0.57	3.06	0.634517766	7.672027	23.4764
	25	3400	6	0.7	3.38	0	7.672027	25.93145
202.5-225	15	3300	7	0.42	2.67	22.08121827	8.286736	22.12559
	20	3300	7	0.56	3.04	2.030456853	8.286736	25.19168
	25	3300	7	0.7	3.37	0	8.286736	27.9263
225-247.5	15	3700	9	0.45	2.76	30.58375635	9.396276	25.93372
	20	3700	9	0.6	3.15	3.680203046	9.396276	29.59827
	25	3700	9	0.75	3.48	0.253807107	9.396276	32.69904
247.5-270	15	4000	8	0.46	2.81	24.61928934	8.858894	24.89349
	20	4000	8	0.62	3.21	4.060913706	8.858894	28.43705
	25	4000	8	0.77	3.55	0.38071066	8.858894	31.44907

Frequency of wind-data based on angle



Wave heights and respective exceedance probabilities:

Hs(m)	Tp(s)	Occurrence(%)	Cumulative occurence %	Exceedence probablity	
0.42	2.67	22.08121827	22.08122	0.779187817	-0.2495
0.43	2.69	9.010152284	31.09137	0.689086294	-0.37239
0.45	2.76	30.58375635	61.67513	0.383248731	-0.95907
0.46	2.82	24.61928934	86.29442	0.137055838	-1.98737
0.47	2.85	2.157360406	88.45178	0.115482234	-2.15864
0.56	3.04	2.030456853	90.48223	0.095177665	-2.35201
0.57	3.06	0.634517766	91.11675	0.088832487	-2.421
0.6	3.15	3.680203046	94.79695	0.052030457	-2.95593
0.62	3.21	4.060913706	98.85787	0.01142132	-4.47227
0.63	3.25	0	98.85787	0.01142132	-4.47227
0.64	3.36	0.507614213	99.36548	0.006345178	-5.06006
0.7	3.38	0	99.36548	0.006345178	-5.06006
0.7	3.37	0	99.36548	0.006345178	-5.06006
0.75	3.48	0.253807107	99.61929	0.003807107	-5.57089

Young-Verhagen's Formula

The Young and Verhagen formula is a formula that predicts the energy of wind waves for a given fetch length (the distance over water over which wind blows), water depth and wind speed. The formula and its parameters are defined as follows:

$$\in = 3.64\ X\ 10 - 3 \left\{ \tanh(A1)\,.\tanh\left[\left(\frac{B1}{\tan(A1)}\right)\right]\right\} 1/n$$

Where, n=1.74

$$A1 = 0.292^{1/n}.\,\delta^{\frac{1}{n}} = 0.493.\,\delta^{0.75}$$

$$B1 = (4.396\ X\ 10^{-5})^{1/n}.\,X^{1/n} = 3.13\ X\ 10^{-3}.\,X^{.57}$$

Non-dimensional energy: $\in = gE/u_{10}{}^{4}$

Non-dimensional water depth : $\delta = gd/u_{10}{}^{2}$

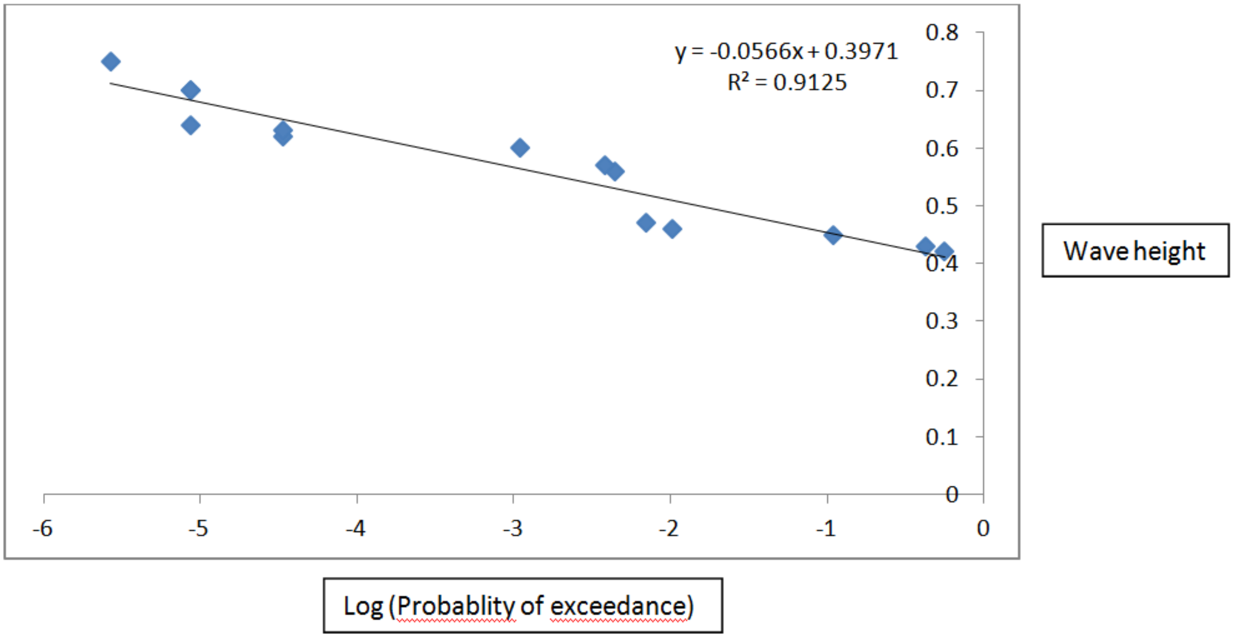
Non-dimensional fetch: $X = gx/u_{10}{}^{2}$

u_{10}= Wind speed above 10m water surface

x= Fetch length

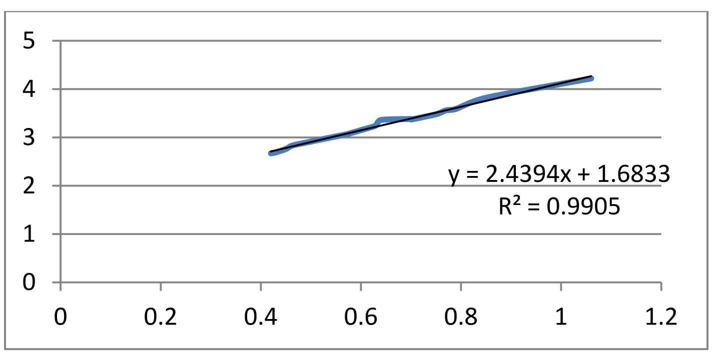
d= Water depth

g= Acceleration due to gravity



Determination of peak period:

Previously calculated values of H_s and T_p were plotted, after which the corresponding peak period value for the design significant wave height was extrapolated to be 3.8 s



Computed H_s vs Computed T_p

Appendix- B

Rubble Mound Breakwater:

Van Der Meer's formula:

Van der Formula's has been used for our design because it includes more parameters and it gives better results as compared to Hudson. As per calculated design boundary conditions, notional permeability value (P) is chosen to be 0.4, damage level (s) = 1 (usually 1-3 is safe limit) , design wave height =1.2 metres , slope of 2.

$$\frac{H_{sc}}{\Delta d_{n50}} = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \varepsilon^{-0.5}$$

Where, P= Permeability of structure

N= Number of waves

S = Damage Level

ε = Iribarren Number

Calculated stone size is found to be 0.5 m (Dn50) which corresponds to rock rubble mound breakwater while the Dn50 for the concrete rubble mound breakwater is found to be 0.4 m. Further, the mass of stones is obtained by multiplying volume of the stone ($Dn50^3$) by the density of rock.

Transmission:

The severity of wave transmission is described by the coefficient of transmission, Ct, defined
in terms of the incident and transmitted wave heights, Hi and Ht respectively: Ct = Ht / Hi. There are formulae for Ct mentioned in the BreakWat user manual which vary for narrow and wide structures. Depending on the inputs, the BreakWat software calculates the required Ct. The transmission co-efficients were calculated as 0.103 and 0.28 for rock and concrete rubble mound breakwaters respectively.

Overtopping for rubble mound breakwaters:

For deterministic design purposes, the following formula is applied for the dimensionless mean overtopping discharge:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\left(\alpha_{rep}\right)}}\gamma_b \xi_{s,-1} \exp\left(-4.3\frac{R_c}{H_{m0}}\frac{1}{\xi_{s,-1}\gamma_b\gamma_f\gamma_\beta\gamma_V}\right)$$

These values were found to be 0.01 and 0.003 for rubble mound stone and concrete breakwaters respectively.

The maximum overtopping is calculated using the formula below:

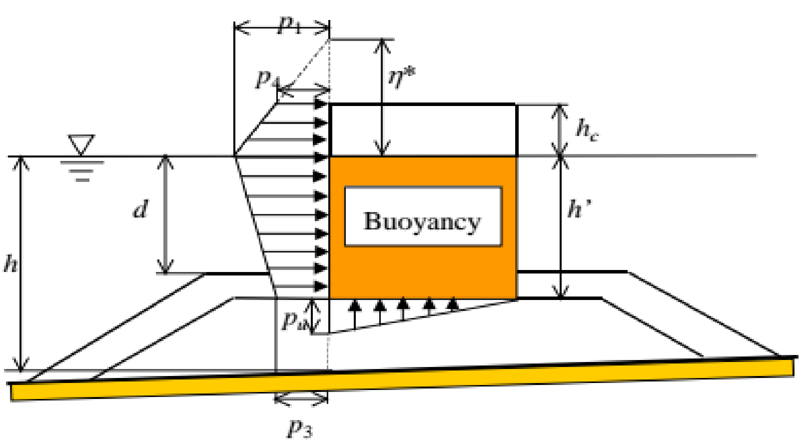
$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.3\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\gamma_\beta}\right)$$

Caisson Breakwater:

Stability Check for Caisson Breakwaters

Creess.nl [Software] carries out the stability calculations for a caisson breakwater according to the method of Goda (2000).

The wave forces on the vertical wall breakwater are integrated wave pressures from a standing like wave pressure and impulse breaking wave pressure. Goda formula is deployed for wave action calculation on vertical walls of caisson breakwater.



Pressure stability diagram of caisson breakwater

The elevation to which the wave pressure is exerted, denoted with η*, is given by

$$\eta^* = 0.75(1+\cos\beta)\, H_D = 3.07 \text{ m}$$

The pressures acting on breakwater are calculated

$$p_1 = 0.5(1+cos\beta)(\alpha_1\lambda_1 + \alpha_2\lambda_2 cos\beta^2)\rho g H_D = 12.8\ Mpa$$

$$p_3 = \alpha_3 p_1 = 4.43 \text{ MPa}$$

$$p_4 = p_1\left(1 - \frac{h_c}{\eta^*}\right) = 6.59\ MPa$$

The uplift exerted on the bottom of the main body is assumed to have a linear distribution with the maximum intensity of the following:

$$p_u = 0.5(1+cos\beta)\alpha_1\alpha_2\lambda_3\rho g H_D$$

Goda (2000) gives the following formulae for the total horizontal force and the overturning moment this force gives around the rear lower corner:

$$P = \frac{1}{2}(p_1+p_3)h' + \frac{1}{2}(p_1+p_4)h'_c$$

$$M_p = \frac{1}{6}(2p_1+p_3){h'}^2 + \frac{1}{2}(p_1+p_4){h'}^{h_c^*} + \frac{1}{6}(p_1+2p_4)h_c^{*2}$$

The total uplift pressure and its moment around the heel of the upright section,

$$U = \frac{1}{2}\, P_u B$$

$$M_u = \frac{2}{3} UB$$

Goda (2000) give formulas for safety factors for sliding and overturning, which has been modified to:

$$SF_{sliding} = \frac{\mu(Mg - U - Bouy)}{P}$$

$$SF_{overturning} = \frac{Mgt - M_u - Bouy\ t)}{M_p}$$

Overtopping Check for Caisson Breakwaters

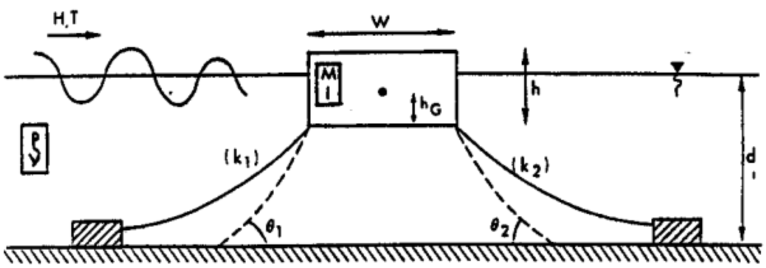
Overtopping for vertical structures is calculated using the following formula for wave impact under non-impulsive conditions which was derived on the basis of all existing tests on vertical breakwaters given by PIANC guidelines.

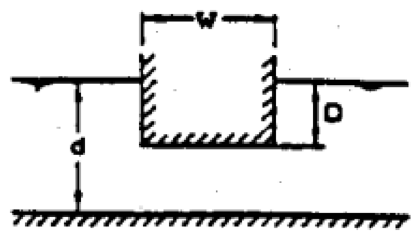
$$Q = \frac{q}{\sqrt{gH_s}} = 0.04 \exp\left(-1.8\frac{Rc}{Hs}\right)$$

In which Rc=2 m, Hs=1.2m

Q=0.0082 m^3/m/s

Floating Breakwater :





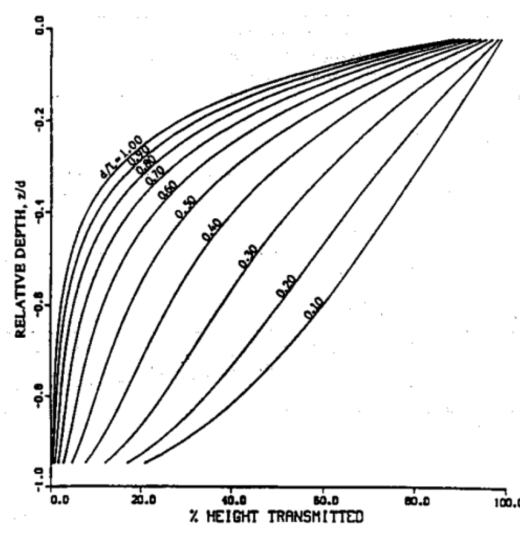
Now, Relative depth= z/d = ??

% height transmitted = 0.55/1.20 =45.83%

L= Wave length = 32 m

d/L= 10/32 =0.3125

From Graph; z/d = 0.5



Note that a conservative value of 0.55 (although allowable is 0.7) has been used for transmission coefficient to take into account possible changes in house dimensions or wave length. Also, since the conservative design itself gives a reasonably cheap structure, one may say that the consideration is justified.

Z= 0.5 x

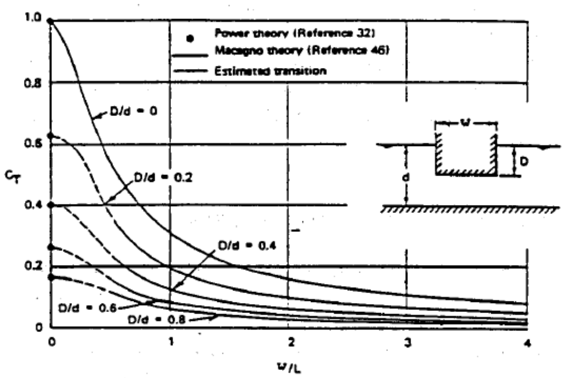


Figure 2.9 - Transmission coefficient for rigid, rectangular surface barrier (L/d = 2.5)

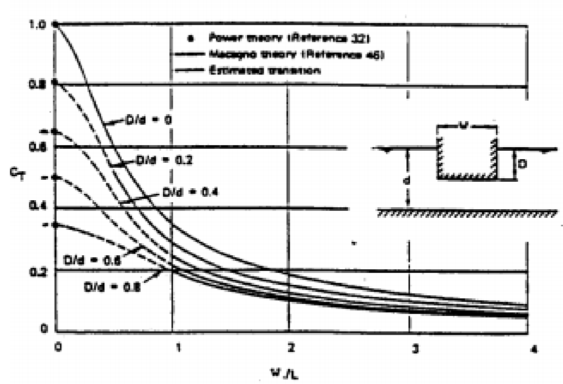


Figure 2.10 - Transmission coefficient for rigid, rectangular surface barrier (L/d = 5)

10 = 5.0 m

L/d = 32/10 = 3.2

Sl. No.	w	D	w/L	D/d	C_T for L/d=2.5	C_T for L/d=5	C_T for L/d=3.2	Remark
1	5	5	0.16	0.5	0.3	0.55	0.37	<46%; OK
2	5	4	0.16	0.4	0.38	0.63	0.45	<46%; OK
3	4	5	0.13	0.5	0.32	0.55	0.38	<46%; OK
4	3	5	0.10	0.5	0.34	0.58	0.41	<46%; OK
5	3	4	0.10	0.4	0.40	0.65	0.48	>46%; not OK
6	4	4	0.13	0.4	0.40	0.62	0.46	<46%; OK

Economically the best option found to be width, w=3m and drought, z=5m for box type floating breakwater. The length of each pontoon is considered to be 20.0 m.

wave overtopping with 2.0m freeboard = 0.008m^3/m/s

Final cross section with 2.0m freeboard = 3m x 7m (for box type).

Appendix C:

Cost Calculations for Rubble Mound Breakwater:

Activity	Prevailing unit cost	Tonnes used for RMB per m (Stone)	RMB (Stone)	Tonnes used for RMB per m (Concrete)	RMB (Concrete)
Quarry production (all gradings)	35 euro/ton		875000		332500
Transport of rocks (over land): 250 km					
< 300 kg	1.8 euro/tonkm	70	7875000	38	4275000
> 300 kg	2 euro/tonkm	30	3750000		
Construction costs					
Bedprotection	35 euro/ton	6	52500		
Core	22 euro/ton	40	220000	20	110000
Rock armour and filter layers	35 euro/ton	60	525000	18	157500
Concrete armour units (all in)					
Cubes	450 euro/m3	20			2812500
Total			13245000		7687500

*Note: Length of the breakwater is 250 m (used for the aggregate calculation)

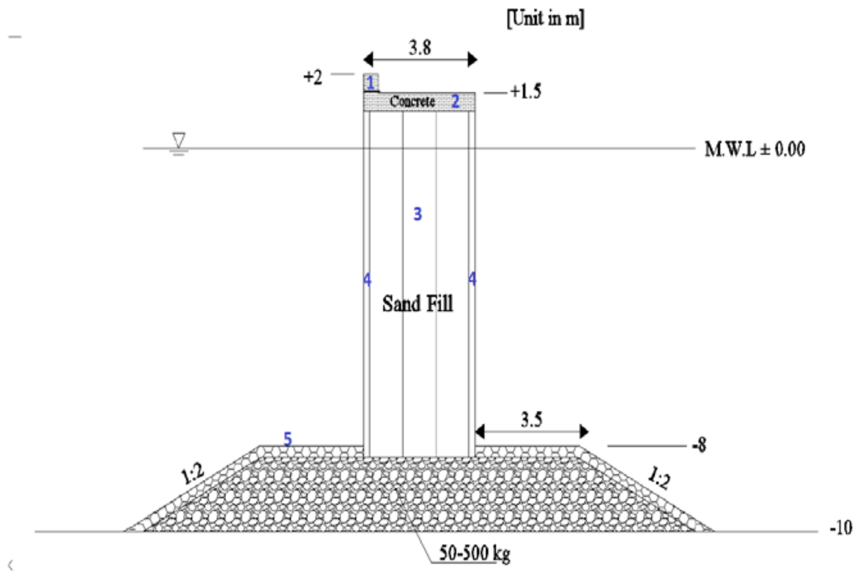
Cost Calculations for Caisson Breakwater:

The caisson was divided into 5 parts and the cost was calculated for each of the sections.

Cost calculation for the Caisson Break Water is shown in table below

Section	Material	Density (ton/m^3)	Cross Section area (m^2)	Volume (m^3)	Weight (ton)	Price (€/m^3)	Price (€/ton)	Cost including transport/ casting(€/unit)	Total Expenses (€)
1	Concrete	2.5	0.25	62.5	-	125	-	160	10,000
2	Concrete	2.5	1.9	475	-	125	-	160	76,000
3	Sand	1.5	26.4	-	9900	-	50	50	495,000
4	Concrete	2.5	7.8	1950	-	125	-	160	312,000
5	Rock	2.6	25.8	-	16770	-	35	450	7,546,500

Total cost	8,439,500



Section of caisson breakwater

Cost Calculations for Floating Breakwater:

Length of breakwater =250m
Volume of concrete from cross-section = 3.85cubic meter/running meter = 962.50 cubic meter

Reinforcement steel (assuming 2% of concrete volume) = 19.25 cubic meter = 151 Ton

Ballast sand (assuming bulk density=1500kg/m^3) = 7.30 cubic meter/running meter
= 1825 cubic meter =2737.50 Ton

Item		Volume	Unit	Unit Price	Total Price
Concrete		962	Euro/m^3	125	120250
Reinforcement (2%)		151	Euro/ton	582	87882
Ballast sand		2737.5	Euro/ton	50	136875
Construction of concrete part		962	Euro/m^3	35	33670
(includes Transport of raw materials, Casting & Labour)					
Formwork		962	Euro/m^3	30	28860
Mooring and Transportation		250	Euro/m	2000	500000
Anchoring and connections					45377
(assuming 5% of total cost)					
				Grand Total in Euro =	952915

Approximate total price in Euro = 1,000,000

Total price in Euro per running meter = 4000