PROBABILISTIC DESIGN OF THE CROSS-SECTION OF THE NEW ROTTERDAM BREAKWATER

(MAASVLAKTE II)

Master´s Thesis

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Summary

Nowadays the most important sectors in the strategy for the future of the port of Rotterdam are container handling, chemicals and distribution. To offer these sectors the opportunity to grow and renew, space is needed. By means of the construction of Maasvlakte II this space can be given.

This land reclamation is planned to be located in the area between the Euro-Maasgeul in the north and the current Maasvlakte in the west and the extended demarcation line in the south. The planned extension will be done in two phases. The first one includes 700 ha with a length of breakwater of 2.7 km, whose construction is planned to start in 2006/07 and the second phase, which includes 300 ha more, with a length of breakwater of 1.3 km is planned to start in 2013/2023.

In this study the cross-section of the breakwater, which protects the new area of the land reclamation, is analysed.
Classical deterministic design could provide a preliminary geometry for the breakwater, but the dimensions of it are too big and also the costs. Therefore a probabilistic optimization could be made in order to check if a reduction or growth, in the geometry, can provide an economical optimum geometry with a substantial save.

First a classical deterministic design is made. The most important elements of the cross-section are determined with the classical formulas and design guidelines.

The following elements are analysed:

- The armour layer
- The toe
- The secondary armour
- The core
- The filter system to establish the supporting bottom material
- The crest height

When the dimensions are given for all the elements, the geometry of the breakwater is established for the deterministic design. Afterwards the construction costs are determined for the breakwater solution.
The deterministic design results in an element weight of 18.8 tons (6.6 m3) and crest height of NAP+18 m. Economic consequences of the different failure mechanisms are not taken into account. The crest height is normally dependent on the construction method. In that case, the construction method does not produce a sensitive reduction in the breakwater geometry because the security level required in the determination of the crest height is too restrictive.

After the deterministic design, the probabilistic optimization takes place.

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1 INTRODUCTION

1.1 General

Rotterdam started as a small village on the river Rotte. Around 1250 the mouth of the river was closed by dams to prevent that too much salt water was able to penetrate inland. However, these dams did not let shipping traffic. This made it necessary to carry cargo over the dam, loading it from one ship to another. The dam therefore turned out to be an outstanding location for the trading of cargo. Thanks to the herring fishing industry, the village grew into a city. Around 1600, the port was able to accommodate as many as 100 herring ships. Rotterdam developed into a mercantile port. Merchant ships sailed from Rotterdam to South America and the Dutch East Indies and back. Ships would anchor right in the heart of the city to among other things discharge tobacco and spices. These products were stored in the warehouses on the quays.

In the nineteenth century, the age of the Industrial Revolution, the port drastically changed. Ships were increasingly made of steel instead of wood, steamers replaced sailing ships. Manual labour made way for machines such as steam cranes and steam trains. The port became too big. The construction of new port basins for the first time also took place on the south bank of the Nieuwe Maas, also referred to as the left bank. Three developments heralded the success of Rotterdam. The first one was the rise of the German Ruhr area. The German steel industry needed vast quantities of ores. Thanks to the Rhine, Rotterdam was the perfect port of supply for this. Barges carried the ores, but also coal and other products, to the cities on the River Rhine in Germany. From there, goods would also return to Rotterdam. Usually, sea-going vessels would move these goods from the port to overseas destinations. The second development was the opening of the Nieuwe Waterweg ('New Waterway'). Up to the second half of the nineteenth century, ships often had to take a long detour in order to reach Rotterdam. This was because the sea approaches Rotterdam silting it up slowly. Engineer Pieter Caland came up with the suggestion to cut through part of the dunes at Hoek van Holland and to in that way create a new link with the sea. In 1872, this 'Nieuwe Waterweg' was taken into use. It gave a new impulse to the growth of the port. From now on, it was much easier for ships to call at Rotterdam. The third development was the Mannheim Treaty of 1868, which gave everybody free access to the River Rhine.

At the end of the nineteenth century, people all over the world started to discover the importance of petroleum, for example for the production of gasoline. Right from the start, most of all the oil for Western Europe was supplied via Rotterdam. Western Europe in those days hardly had any petroleum of its own. The extraction of oil using rigs on the continental shelf of the North Sea did not start until later. But even nowadays, most of the oil is still imported. The construction of the first petroleum ports took place prior to World War II.

In World War II, roughly forty percent of the port was destroyed. Following the war, a lot of energy was invested in the reconstruction of the flattened port. Soon, the port was doing so well, that there was not enough room to accommodate all the companies and ships. The decision was made to expand in a westerly direction: the area between Rotterdam and Hoek van Holland. And south of the Nieuwe Waterweg, the Eemhaven and Botlek emerged.

Petroleum increasingly became more important to the economy after World War II. Shipbuilding yards constructed vast tankers for the transport of oil, which were also called mammoth tankers. Due to their drafts, these vessels could however not enter the existing harbor basins. The construction of the Europoort meant that Rotterdam retained its accessibility. The ports in the Europoort have a depth of more than twenty meters.

Various companies also established themselves in the Europoort. Until there was no more space and the sea was reached. In order to expand, the decision was made to create land in the sea. For this, a section of the sea was fenced off, the water was drained and the enclosure was raised by spouting up sand. In 1973, the first ship moored at a company at the Maasvlakte.

The most important sectors in the strategy for the future of the port of Rotterdam are container handling, chemicals and distribution. To offer these sectors the opportunity to grow and renew, space is needed. This space can be realized by means of the construction of Maasvlakte 2, a new top location for port-related activities boasting an excellent infrastructure.

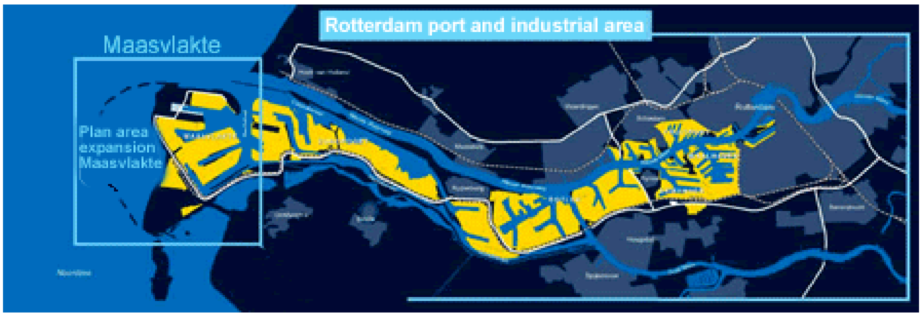

Figure 1 Planed area for expansion of Maasvlakte

The intended result of the land reclamation constituent project is a new, 1,000-hectare port and industrial site (net allocation) in the North Sea. This land reclamation can be realised in the area between the Euro-Maasgeul in the north and the current Maasvlakte in the west and the extended demarcation line in the south. For the construction of the Maasvlakte, it was agreed that the demarcation line would serve as the boundary between the port and wildlife areas. This line, which coincides with the southwestern boundary of the existing harbour at the Maasvlakte, will be extended seaward with land reclamation
The current disposition of Maasvlakte I is shown in the following picture:

Figure 2 Current Maasvlakte

And in the following picture is shown the aspect of the planned Maasvlakte 2:

Figure 3 Future Maasvlakte II

The planned extension will be done in two phases. The first one includes 700 ha with a length of breakwater of 2.7 km, whose construction is planned to start in 2006/07 and the second phase, which includes 300 ha more, with a length of a breakwater of 1.3 km is planned to start in 2013/2023.

1.2 Objective of the study and working method

In this project the target will be to define the cross section of the breakwater, which protects the new area of the land reclamation. Firstly a classical deterministic design will be done. Then, a probabilistic optimization will be made. By means of Vap, which is a computer program, Monte Carlo simulations will be done in order to get the probabilities of failure of the armour elements and the toe structure. In that way it can be possible to take into account uncertainties. Also a risk analisis will be made in the probabilistic optimization. Costs will be calculated in both designs so as to find out the most economical option.

2 Problem analysis

2.1 Problem description

2.1.1 Limit states

The breakwater can fail to fulfil its function in two ways. First, the breakwater can collapse and fail to provide shelter. Secondly, the breakwater can stay intact, but the overtopping can be excesive. The limit for the breakwater to collapse is called the Ultimate Limit State (ULS). The limit for insufficient functioning of the breakwater is called the Serviceability Limit State (SLS).

Both limit states are depicted in a fault-tree. This implicates that the influence of alterations to the breakwater geometry is limited. It is also shown that the collapse of the breakwater can be caused by several failure mechanisms (actually there are more failure mechanisms, but in this study only the most important considered, are taken into account).

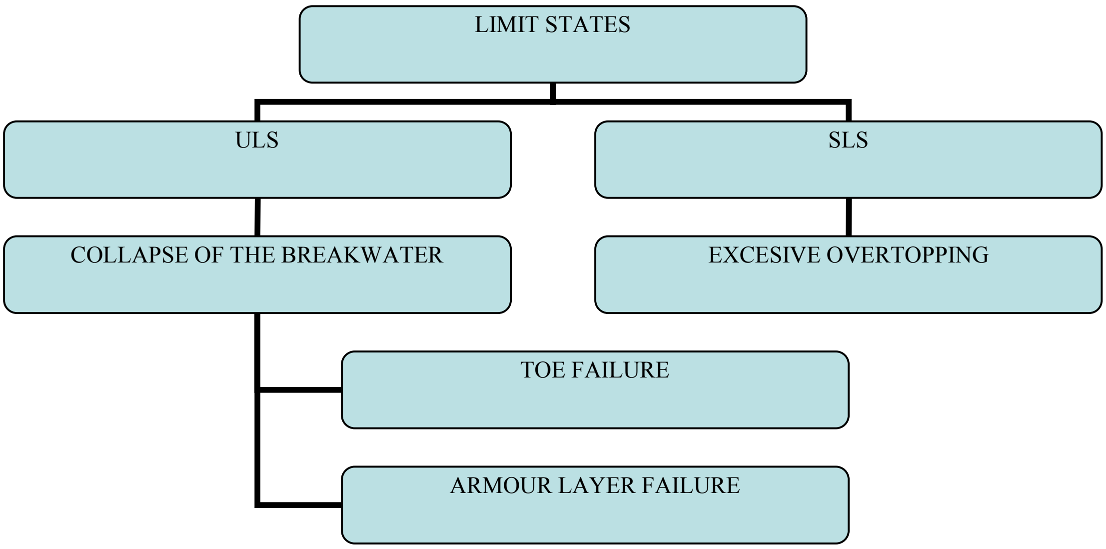

Figure 4 Fault-tree for the breakwater

2.1.2 Economic optimum

To derive an economic optimal design for the breakwater, establishing knowledge of costs is imperative. The following costs will be discussed in this report:

- Construction costs
- Repair of collapse and maintenance costs of the breakwater
- Downtime and damage costs in the protected area

- Construction costs

The construction costs are dependent on the breakwater geometry. A more conservative design will implicate higher construction costs, but will provide a breakwater with lower maintenance costs.

- Damage costs of the breakwater

When the armour layer consists of cubes, the maintenance costs are dependent on the stability of the armour layer elements. Furthermore, in this kind of breakwater the development of failure is slow compared with other elements with interlocking like Dolos, Terapods,etc. Nevertheless, failure with less waves is bigger at the beginning. That means that it is easier to detect a failure and failure is less sudden.

- Downtime and damage costs in the protected area

Downtime and damage costs in the protected area behind the breakwater are the cause of overtopping. The more overtopping discharge behind the breakwater, the more damage costs and, therefore, the more downtime costs.

3 Boundary conditions

3.1 Water level

All water levels are relative to NAP (Normal Amsterdam Level), which is a reference level in The Netherlands.

The extreme water levels including wind setup and storm surge considered at Maasvlakte 2 for each frequency and, according to the Port of Rotterdam report (Doc.nr.:AA-02-330), are as follows:

Frequency	1/year	1/10year	1/100year	1/1000year	1/10000year
max. water level (NAP+m)	2.3	2.89	3.52	4.21	4.95
min. water sea level(NAP+m)	0.48	1.07	1.7	2.39	3.13

Table 1 Maximum and minimum waterlevels

The function distribution of the maximum water levels has also been found out of long term data of the Hook of Holland (Breakwaters and closure dams, Appendix 10 Example of the determination of a design storm).

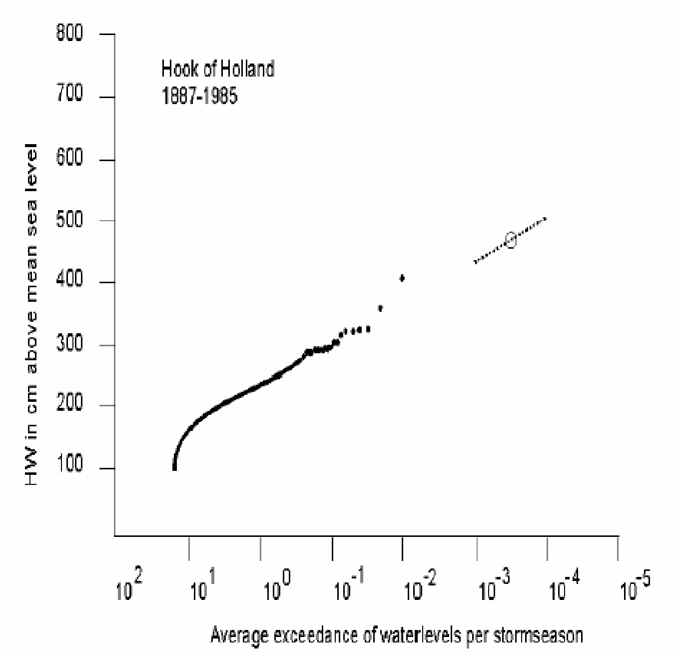

Figure 5 Maximum waterlevels Hook of Holland

The exceedance is given by the Gumbel distribution:

$$Q = 1 - \exp\left[-\exp\left(-\frac{h_{surge} - \gamma}{\beta}\right)\right]$$

Where:

γ=2.3 and β=0.3 are the parameters of the distribution of the maximum water levels per storm season.
The different frequencies were checked and the values are approximately the same.
For the minimum water levels the distribution is the same but the parameters change and its values are: γ=0.47 and β=0.27 .
It is observed that the difference between maximum and minimum water levels is 1.8 m per storm. This value correspond with the spring tidal range.

3.2 Waves

For this study it has been used the data of the waves from the report made by the Port of Rotterdam and the data of the Europlatform (EUR) because this is the only platform in the surroundings of Maasvlakte II which includes wave directions in the measurements. The platform is situated at 9.963 m(x) and 447.601 m (y) in the Dutch grid, where the local depth is 32 m. The geographical co-ordinates are:
NB: 51° 59’ 55.595’’
OL: 3° 16’ 30.721’’
In the next picture are shown the measurement stations of The Netherlands.

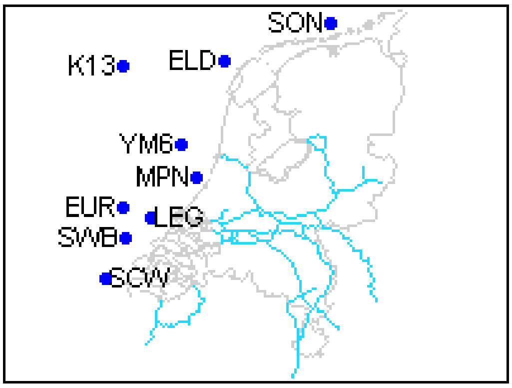

Figure 6 Measurement stations in the North Sea

For the breakwater design, it will be necessary to get the wave data nearshore since the data mentioned before comes from 55 Km offshore.

3.2.1 Direction

In the area of the study the northwesterly direction is the one that produces the most severe storms. The maximum Fetch, which produces the maximum wave heights, comes from that direction.
It is because of this, that the contruction of the breakwater in the north-west area of the land reclamation is needed. We will assume that the waves will be perpendicular to the breakwater since that is the worst design condition.
The following graph, based on the data of the europlatform, shows the exceedance of waves higher than 7 m for each direction.

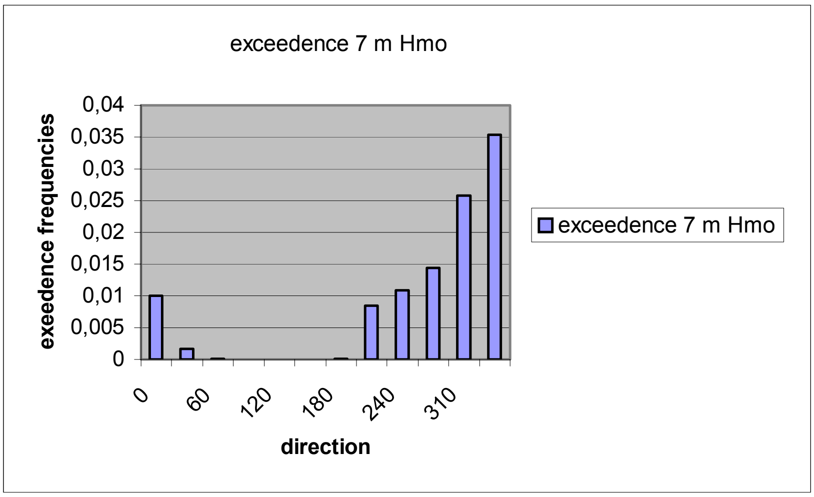

Figure 7 Exceedance of waves higher than 7 meters for each direction

The peaks of the storms with a wave height bigger than 4.5 meters were also analyzed and the result was similar. Fifty-two storms with that characteristics were found in twenty-two years of data gathering and the direction distribution was the following: All the peaks where between 210 and 71 degrees.0

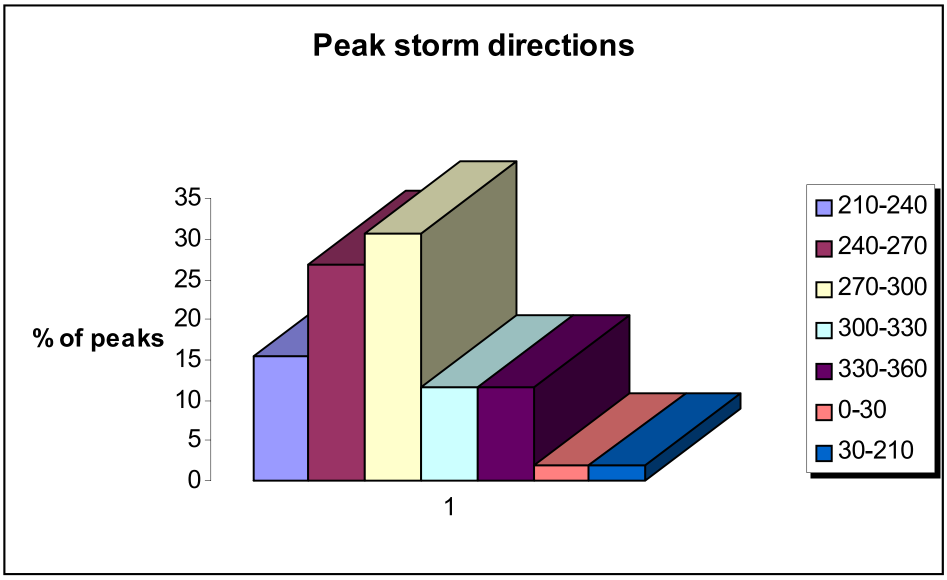

Figure 8 Percentage of storm peaks higher than 4.5 meters for each direction

3.2.2 Wave height

Many sources have been consulted:

The data of Global Wave Statistics are not taken into account because they are based in visual observations and the accuracy is not quite good. Moreover, the area covered for each series of data is big. That is the reason why these data are useful in open ocean coasts, but they are not the most appropriate for the North Sea.
Another consulted source was the website www.golfklimaat.nl, where data from 1979 until 2001 are available from some measurement stations.
Europlatform is the chosen station because it provides the direction of the waves. With data of that one there is an study made by the Port of Rotterdam (Doc.nr.:AA-02-330) which gives the following results:

Frequency	1/year	1/10year	1/100year	1/1000year	1/10000year
Hmo (m)	5,1	6,2	7,05	7,8	8,4
Tm (s)	7,1	7,8	8,3	8,8	9,1
Tp (s)	9,5	10,6	11,4	12,1	12,7

Table 2 Wave heights, wave priods

That analisys estimates an average storm duration of 6 hours.

A PoT (peak over threshold) analisys has been done,with data of 22 years (1979-2001), both to ensure that the data are correct and to take into account the uncertainties in the probabilistic design (see Appendix I). It is because of this last reason that it is necessary to ascertain the function distribution followed by the wave height.

The exceedance frequencies in a period of a year from the Europlatform are available in www.golfklimaat.nl and are shown below:

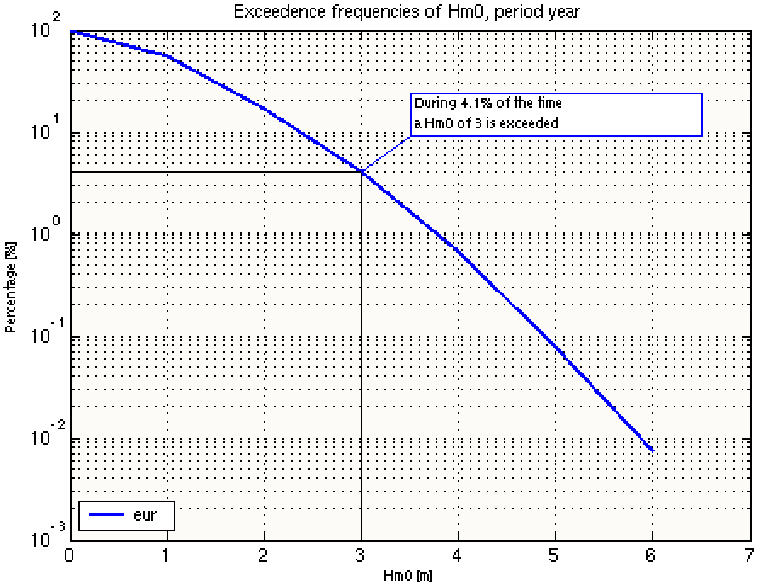

Figure 9 Exceedance frequencies of Hmo yearly

This graphic, just represent the distribution of single waves and not the peaks of the storms. But is useful to determine the workability during construction.

3.2.3 Wave period

The period varies for each wave height. Mean period and peak period are used to make calculations, depending on the basis of the formula. Van der Meer formula for the stability of the armour units is based on the mean period, and the formula of the overtopping is based on peak periods, so both periods will be used in this study. The Port of Rotterdam report (Doc.nr.:AA-02-330) provides the relations between peak periods (Tp) and mean periods (Tm), which for wave heights are estimated at one to ten meters with a stepsize of 0.25 m. Next graph show the results:

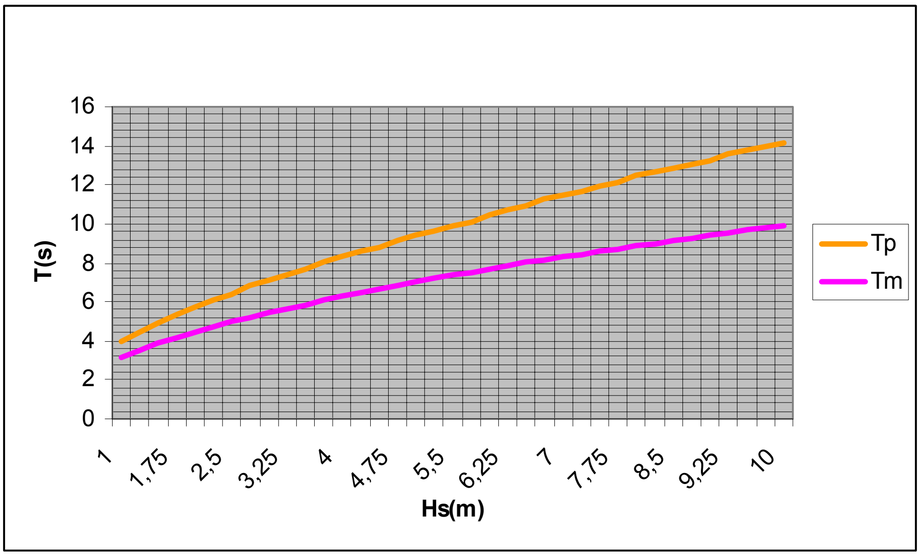

Figure 10 Mean and peak periods

The relations between the peak period, mean period, wave height and depth are the following:

$$T_m = 3.14 * \sqrt{H_{m0}}$$

For: $H_{m0} / d < 0.14$ $\quad T_p / T_m = 1.33$

For: $0.14 < H_{m0} / d < 0.35$ $\quad T_p / T_m = 0.5 * H_{m0} / d + 1.26$

Where:

Hmo=significant wave height
d=depth(at the Europlatform is 32 m)
Tp=peak period
Tm=mean period

It has to be said that, for each wave height the wave period vary. That means that it has large standard deviatons. The above relations try to approximate the mean and peak periods in order to make feasible the calculations.
Related with the period is the wave steepnes, used in the stability formula of the armour units.
Wave steepness is calculated with the following formula:

$$S_0 = 2 * \pi * H_{m0} / (g * T^2)$$

In which T, is the mean period.

3.3 Bathymetry

The bathymetry of the area of Maasvlate was got introduced in a program called Delft3D. In it the bathymetry is represented as a grid with the depth points. The depth of the axe of the breakwater was followed seeing the depth points. Almost all the breakwater bottom is at NAP–17.60 m, except the connection with the new sand dam, which is a little bit deeper NAP–18.60 m.
At the area of the current Maavslakte the depth is –12.60 NAP arriving till –17.60 NAP in the firsts 800 m of the new breakwater.
For calculations in this study the depth of -17.6 m will be taken into account.
Cross-section of the bathymetry, perpendicular to the breakwater, has been done in the connection of the new sand dam, which will follow the breakwater.

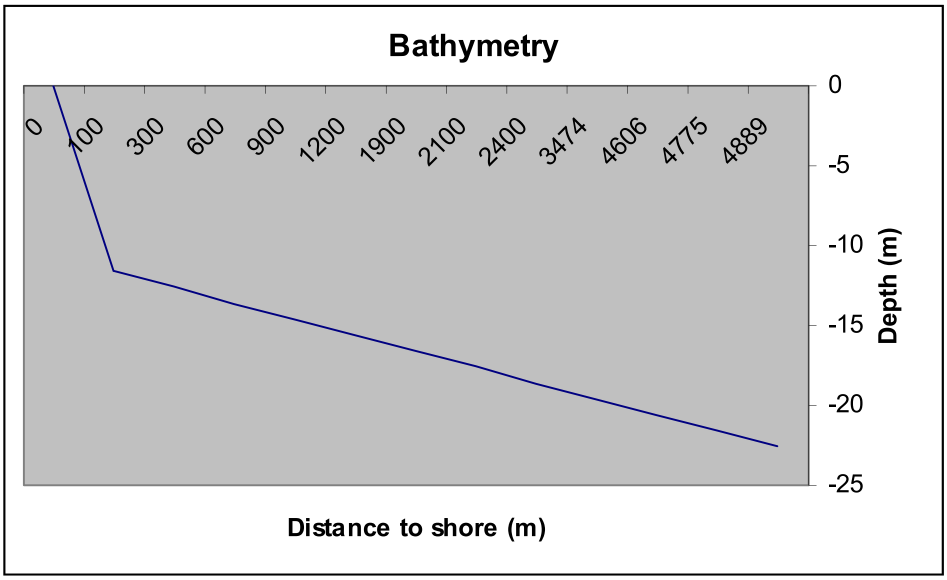

Figure 11 Bathymetry cross-section

In the following picture is shown the shape of the bathymetric lines:

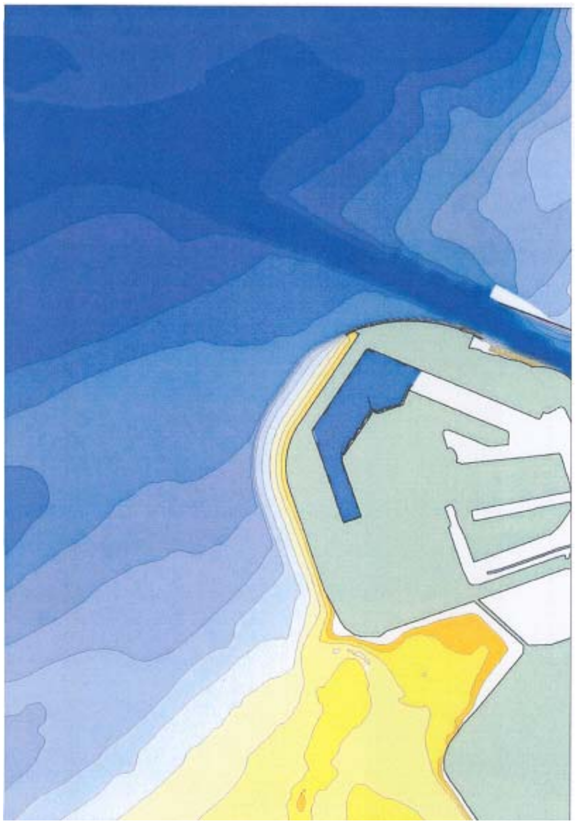

Figure 12 Bathymetry

Except in the entrance channel of the port the bathymetric lines are curved and sensitively paralel to the breakwater. In the calculations in which the angle of incidence of the waves plays a significant role, it will be assumed that they are perpendicular to the breakwater.

3.4 Soil classification

The soil below the breakwater is consists basically of sand. The size of the sand grains is D50 200-250 μm. Settlements are expected but are out of the scope of this study.

3.5 Quarry materials

Quarries can be found all over Europe, but only the quarries in Great Britain, Scandinavia, Belgium, France and Germany are useful for hydraulics works in Holland. Other quarries in Europe, Canada and South-America are not competitive because of the high transport costs. The quarry has to be in the neighbourhood of the water, because the costs of road transport are very high.
Marine gravels are a good source of materials. Maps and charts can be used as a starting point to locate gravels and other materials. Optimum depths of deposits bellow the water surface are between 10 and 30 meters. Depths of 40 meters are possible for some dredgers, but are not the best option, because the production is so low and the costs are high. Also a wide area of a few meters thick is most economically dredged because each load can be dredged without much manoeuvring, but there are conflicts with economical requirements to minimize the area of seabed destroyed. For gravel production two principal types of dredgers can be used: cutter suction dredgers and trailer hopper suction dredgers. Sea-going suction hopper trailer dredgers have been built up to 20,000-tonne capacity and are suited to high-volume contracts.
In the following table is shown the European Standard (EN-13381) grading of stones:

Grading	Class designation (Kg)	Dn
Light grading	10-60 kg	0.16-0.30
	10-200 kg	0.16-0.43
	60-300 kg	0.3-0.49
Heavy grading	300-1000 kg	0.49-0.72
	1000-3000 kg	0.72-1.04
	3000-6000 kg	1.04-1.31
	6000-10000 kg	0.31-1.55

Table 3 Grading of quarry stone (EN-13381)

4 Deterministic design

4.1 Introduction

The determination of a classical deterministic design is needed if a probabilistic design is going to be made. The resulting dimensions and costs are not only a good starting point for the probabilistic design, but also provide a check on the realism of the results to the probabilistic design.

The deterministic design is mainly focussed on the following components:

- The armour layer of cubes
- The toe
- The secondary armour
- The core
- The filter system to establish the supporting bottom material
- The crest height

Next figure shows a definition sketch of the elements of a breakwater:

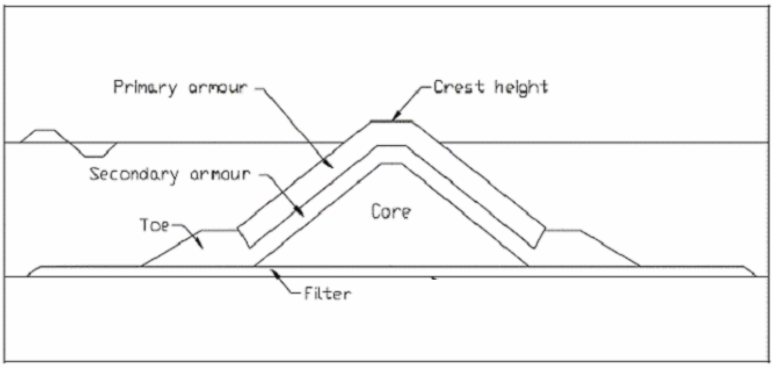

Figure 13 Definition sketch of a breakwater

4.2 Design wave height

In the deterministic design, an ultimate limit state design wave height has to be used. By choosing an acceptable probability of collapse during the lifetime of the breakwater the return period of this design wave height can be obtained. Consequently the return period can be used to determine the wave height.
The lifetime of the breakwater will be assumed in 50 years because the growth of the port in 50 years, probably will require more space. For this combination of values the frequency of the wave height is determined in the following way:

$$f = -1/t_L * \ln(1-p) = -1/50*\ln(1-0.05) = 1/975 \simeq 1/1000$$

In wich:

f = Frequency of the wave height
t_L =Design lifetime
p =Probability of failure during the lifetime.

For this frequency the combination of the design storm parameters are:

Hmo (m)	7.8
Tm (s)	8.8
Tp (s)	12.1
max. water level (NAP+m)	4.21
min. water sea level(NAP+m)	2.39

Table 4 Charactesistics of the design wave height

The limitation of the wave height by the depth in front of the breakwater has to be checked.
The average depth below NAP in front of the breakwater is -17.6 m , so the maximum wave height that could be in front of the breakwater will be assuming a breker parameter of γ_{br} =0.5 and an average water depth of 17.60. Hmax=**8.8 m**.
It is shown that there are no limitations of the wave height due to depth, at least in the return periods that are used in that project.
The not limited wave height by depth implies that the waves will not breaks by depth, but we have to study the decay of the wave energy when the waves comes from deep water to relatively shallow water.
The wave propagating calculations are performed by the wave propagation method with shoaling /refraction calculation Coastal and River Engineering Support System (CRESS) based on the Battjes and Janssen (1984) approach. The water level fluctuations are neglected for the determination of the translation of deep to shallow water waves.

After introducing the parameters of the design wave height in CRESS the result of the wave height in front of the breakwater is 6.36 m.

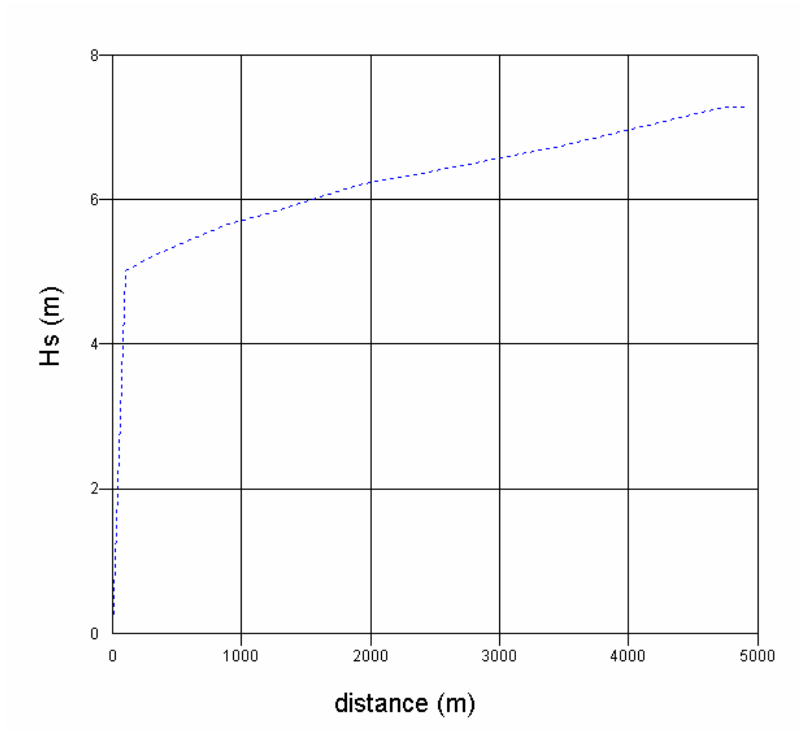

Figure 14 Energy disipation of the design wave height

4.3 Armour layer

4.3.1 Introduction

The armour layer is the most important part of the breakwater, because it protects the total structure and it takes care that the breakwater fulfils its functions. The armour layer has to withstand wave forces and internal forces. In cooperation with the filter layers the armour layer prevents core material from washing out.
The use of heavy weigh of armour units and a large number of them is expected. It is recommendable to use concrete armour units because of that.
Many concrete armour units exists at the market. Most of them have shapes for trying to mobilise stability contribution due to interlocking, in order to reduce the weight of the elements and therefore the quantity of concrete reducing the costs of material. But with this kind of elements, if there is a failure, the layer behaviour is like that of a rigid structure. The progress of the failure is really quick. Moreover, replacing that kind of elements is quite difficult. Concrete cubes have not got that problem, although the weight is bigger.

4.3.2 Slope

When the concrete armour units are used, it can easily be demonstrated that a steep slope (1:1.5) leads to the most economic design, since the work volume is the minimum. Steeper slopes could be dangerous for the breakwater stability.

4.3.3 Stability

There are some formulas to check the stability of the concrete cubes. The most used ones nowadays are the Hudson formula, and the Van der Meer formula.
First, the weight of the elements will be calculated with the Hudson formula to later compare the results with the Van der Meer formula.

*Hudson:

$$W \geq \rho_r * g * H^3 / (\Delta^3 * K_D * \cot\alpha)$$

In which:

W=Weight of the blocks (N)

ρ_r =density of the blocks (kg/m3)

g=gravity acceleration=9.8 m/s2

H=design wave height=6.36 m

Δ=relative density of the blocks= $(\rho_c / \rho_w - 1)$

K_D=Influence coefficient=7.5 for non-breaking waves and 6.5 for breaking waves (SPM).

cotα= slope =1.5

The formula is applicable for slopes not steeper than 1:1 and not gentler than 1:4. So that case are in the range of validity.
The coefficient Kd represents many different influences and has been determined on the basis of experiments. The Shore Protection Manual (1984) gives values for some elements. In the below table are shown some of them:

	STRUCTURE TRUNK			STRUCTURE HEAD	
type of block	Number of layers(N)	Breaking wave	Non breaking wave	Breaking wave	Non breaking wave
tetrapod	2	7	8	4.5	5.5
dolos	2	15.8	31.8	8	16
cube	2	6.5	7.5		5
akmon	2	8	9	n.a.	n.a.

Table 5 Coeficient Kd (Hudson) SPM

After calculation the weight of the elements is derived for different densities with the Hudson formula:

density of the concrete (kg/m3)	2400	2800	3800
Weight (t)	22.73	12.3	4.37
Dn (m)	2.11	1.64	1.05

Table 6 Element weight, Hudson formula

*Van der Meer

The formula of Van der Meer defines a clear measurable definition of damage:

$$S = A / D^2_{n50}$$

S=Damage level, related with the eroded area.

A=the erosion area in the cross section in m2

$D_{n50} = (W_{50} / g * \rho_r)^{(1/3)}$

W_{50} = mean weight of armour stones in N.

ρ_r =density of the armour stone in Kg/m3.

Also the Van der Meer found a clear influence of the storm duration. The longer the storm, the more damage, because in a irregular wave field, a longer storm duration leads to a higher probability of occurrence of extremely high waves which are, apparently the responsible for ongoing damage.

For concrete elements the value of S is related with Nod, which is the number of displaced blocks. S=Nod/2

The following stability formulae, provided by Van der Meer for cubes, is used to determine the required weight:

$$Hs / \Delta Dn = (6.7 * Nod^{0.4} / N^{0.3} + 1.0) * s_{om}^{-0.1}$$

In which:

Hs=Significant wave height at the location of the breakwater= 6.36 m

Dn= side of the cubes (m)

Nod=Character of the damage=0.5 (-)

Damage development	Nod
Initial damage(needs no repair)	0-0.5
Intermediate damage(needs repair)	0.5-1.5
Failure(core exposed)	>2

Table 7 Character of damage armour stability

N=Number of waves=2500 (-)

Som=wave steepness in deep water=0.065 (-)

$\Delta = (\rho c / \rho w) - 1$= density of the concrete armour units relative to the water (-)

ρc= mass density of concrete (Kg/m3)

ρw=mass density of water=1.025 (Kg/m3)

Van der Meer formula uses values of the period in deep water so the wave steepnes will be based on the mean period and the significant wave height in deep water.

$$S_0 = 2 * \pi * H_{m0} / (g * T^2)$$

The wave steepness obtained with the wave data is 0.065.
The average storm duration considered is 6 hours and therefore the number of waves can be derived, just dividing the storm duration into the period. The number of waves is 2500.
Now the weight of the elements can be calculated.
Table 8 summarize the results of Van der Meer for different densities of the concrete:

Concrete density	2400(kg/m3)	2800(kg/m3)	3800(kg/m3)
Weight (ton)	34.4	18.6	6.61
Dn (m)	2.4	1.88	1.2

Table 8 Element weight, Van der Meer formula

The reduction of the weight and the size is so comparable when one vary the density. But we have to keep in mind that the number of elements will be bigger, and the price of the concrete too.
When concrete cubes have nominal diameters bigger than two meters, problems with retraction craks usually appear. So the density of 2800 kg is chosen to make the calculations.
Comparing the results with the Hudson formula, it is seen that the cubes are bigger aplying Van der Meer. This is because the number of waves is taken into account.

4.3.4 Minimum depth of the armour layer

As a rule-of-thumb the minimum depth of the primary armour on the seaside slope should be 1.5 times Hs. The minimum depth is also dependent on the toe stability.
Having a wave height of 6.36m the minimum depth of the primary armour layer it should be 9.54 m (data to take into account in the toe stability).

4.3.5 Layer thickness

The armour layer will consist of two layers of cubes with a nominal diameter of 1.88 m. The formula to determine it is the following:

$$t = n * k * D_n$$

In which:

t = layer thickness

n = number of layers

k=coefficient given by the SPM, that for two layers of cubes with a random placement is 1.10.

D_n =nominal diameter

After calculation the layer thickness obtained is: **t=4.1 m**

4.3.6 About the construction of the armour units

For the construction of the cubes it is important to have a production and storage area in the neighbourhood of the construction site. That reduces the transport costs. The Maasvlakte I is a good place to establish that area.
Attention has to be paid to the quality certificates and environmental licenses which are required.
The daily quantities will determine the dimensions of storage of the aggregate materials, cement silos and cement mixers. The different aggregate has to be transported separately to prevent mixing during the transport. Also the pollution of the aggregates has to be prevented.
Coarse material has to be stored in layers and the dump height has to be modest. During the transport and storage of cement, contact with humidity has to be prevented.

The activities, which carry out the production process, are the following:

-Preparation of the moulds for production process.
-Pouring of concrete in moulds.
-Hardening of concrete
-Striking the mould
-Numbering the units (production date and number)

-Transport from production line to storage area by shovel or gantry crane.
-Up to 28 days of hardening after the casting on storage area.

Minor defects shall be repaired. The strength and durability of the mortars for that shall be at least the same as the armour unit. The construction shall include measures to prevent shrinkage and ensure adherence.

The moulds have to withstand the load of the concrete mixture in the first part of the hardening process. A concrete pump or a bucket can be used for the pouring of the concrete and then the concrete has to be compacted with mechanical procedures. In summer the concrete reaches enough strength after hardening of one day. When the evaporation of water in the mixture is too fast, the chemical reaction will delay or stop, so evaporation of water has to be prevented.
The following measures could prevent the evaporation:

-Keep the units moist.
-Use curing products.
-Keep the concrete longer in the moulds.
-Cover the units with plastic sails.

During the winter, the units have to stay two days in the moulds, because the hardening process is slower due to the lower temperatures. In winter special attention has to be paid for the hardening process when the freezing point is reached, because the chemical reaction will stop and the quality of the concrete will not be the required. To prevent the freezing of the mixture we can heat the concrete mixture or cover with plastic sails and steam the units.

Attention has to be paid to the retraction cracks that could appear in the elements, because when we have a big volumes of concrete are easy to appear. Changing the density of the concrete the volume is modified and this problem can be minimized.

4.4 Layer under cubes

4.4.1 Stability after construction

The weight ratio between the armour layer and the first under layer it must be 10 (d´Angremond, Van de Rode). This leads to a first under layer of 1860 kg (rock class 1000-3000) having an armour layer weight elements of 18,6 tons. Filter rules of Terzaghi are also checked in order to ensure that the layers are geometricaly closed.

4.4.2 Stability during construction

To determine the size of possible damage during construction, the following formulas, provided by Van der Meer (1993), are used to calculate the stability of the secondary armour.

For plunging waves: $$\xi_{mc} = \left[6.2 * P^{0.31} * \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

For surging waves: $$\frac{H_S}{\Delta * D_{n50}} = 1.0 * P^{0.13} * \left(\frac{S}{\sqrt{N}}\right)^{0.2} * \sqrt{\cot\alpha} * \xi_m^P$$

In which,

Hs = significant wave height [m]

Δ = density of armour material relative to the water [-]
= (ρc /ρw)-1

ρc = mass density of concrete = 2.650 [kg/m3]

ρw = mass density of water = 1.025 [kg/m3]

P = permeability of the breakwater = 0.4 [-]

N = number of waves = 2500 [-]

S = damage level [-]

ξm = Iribarren parameter, describes the type of wave breaking on a slope= $\frac{\tan\alpha}{\sqrt{\frac{2\pi}{g}*\frac{H_s}{T_m^2}}}$

g = acceleration of gravity [m/s2]

The transition from plunging to surging waves can be calculated using a critical value of ξ_{mc} :

$$\xi_{mc} = \left[6.2*P^{0.31}*\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

Making calculations it is shown that for the stone class of 1000-3000 (Dn50=0.89 W50=1103Kg) chosen for the first under layer, the stability for a damage level of S=2 starts with a wave height of 2.2m and this wave height occurs 14% of the time.
However for the damage level of S=5 a wave height of 2.5 m has to occur. That wave height is exceeded 8% of the time. If we assume a loss of material of 10% for that damage level, the expected loss of material during construction per year will be 0.8%.
The wave heights distribution used to make this calculations was based in the yearly distributions of the Hmo given by www.golfklimaat.nl .

4.4.3 Layer thickness

A minimum thickness of three times the diameter of the larger stones in the filter distribution is considered. To be effective the filter system is required at least two, but losses of material from the core could cause breakwater instability.

Underlayer thickness: 3*Dn50underlayer=3*0.88=2.64m

4.5 Bed protection

The base of the breakwater consists on soil protection to prevent scour in front of the structure, which could initiate geotechnical instability. The soil protection can be granular filter or a geotextile filter. In our case we will use a granular filter because geotextile is not well known. The durability is uncertain and it is difficult to fit.

4.5.1 Filter system

The granular filter is calculated following the filter rules of Terzaghi:

For stability of the interface between base material and filter material:

$$\frac{d_{15f}}{d_{85b}} < 5$$

For an adequate permeability of the filter layer to avoid it uplifting:

$$\frac{d_{15f}}{d_{15b}} > 5$$

For an adequate internal stability:

$$\frac{d_{60}}{d_{10}} < 10$$

In which:

d_{15f} = sieve diameter passed by 15% of the filter material.

d_{85b} = sieve diameter passed by 85% of the base material.

A $\frac{d_{60}}{d_{10}}$ ratio of 10 is more or less equivalent to ratio $\frac{d_{15f}}{d_{85b}}$ of 12-15. This approximation will be used in this study.

The filter system is summarized in the table:

	natural material [mm]	first filter [mm]	second filter [mm]	core
d15	0.14	1.34	13.44	70.00
d50	0.20	2.12	23.27	440.00
d85	0.28	3.36	40.31	490.00
d85/d15	2	2.5	3	7
dn50	0.168	1.78	19.54693	369.6
dn50 [m]	0.000168	0.00178	0.019	0.369
Filter rules:				
d15f/d85b		4.75	4	1.7
d15f/d15b		9.5	10	5.2
Layer thickness		0.5m	0.5m	

Table 9 Filter system

4.5.2 Layer thickness and length

In underwater placement, bedding layer thickness should be at least two to three times the size of the larger quarry stones used in the layer, but never less than 30 cm thick to ensure that bottom irregularities are completely covered. Considerations such as shallow depths, exposure during construction, construction method, and strong hydrodynamic forces may dictate thicker filters, but no general rules can be stated. For deeper water the uncertainty related to construction often demands a minimum thickness of 50 cm.
In our case the best option is the last one. Seventeen meters depth is quite deep.
It is common practice to extend the bedding layer beneath rubble-mound structures at least 1.5 m beyond the toe of the cover stone to help reduce toe scour.

4.5.3 Construction aspects

Granular filter construction above water creates no special problems, and accurate placement is straightforward. However, constructing a filter bellow the water surface is more problematic. If small-size filter material with a wide gradation is dropped into place, there is a risk of particle segregation by size. This risk can be decreased using more uniform material and minimizing the drop distance. Another problem is to maintain the adequate layer thickness during underwater placement. This leads to fit a bigger thickness than the recommended, being greater than required by the geometric filter criteria.

Finally, filter or bedding layers placed underwater are exposed to eroding waves and currents until the following layers are placed. Depending on site-specific conditions, this factor may influence the construction sequence or the time of year chosen for construction.

4.6 Toe

The toe structure lends stability to the armour layer of the breakwater. The toe structure can be composed of quarry material or of a few armour units. In our case quarry material is used.
The stability of the toe is checked with the formula provided by Van der Meer, d'Angremond and Gerding [1995], where the design wave height is the same as in the stability of the armours units.

$$H_s / \Delta D_{n50} = (0.24 * h_t / D_{n50} + 1.6) * N_{od}^{0.15}$$

In which:

Hs= Design wave height at toe. (m)

Δ = density of toe material relative to the water (-)= (ρr /ρw)-1

ρr=density of the rocks (Kg/m3)

ρw=density of the water (Kg/m3)

D_{n50}=nominal diameter of the stones (m)

ht=depth of the toe crest bellow the low still water level (m)

Nod= character of the damage

Critical values for Nod:

0.5 Start of damage
1.0 Acceptable damage
4.0 Filiure

These values are valid for a standard toe, with a height from 2 to 3 D_{n50} and a width of 3 to 5 D_{n50} .

The validity range is:

0.4<ht/h<0.9

3<ht/Dn50<25

h=depth of the bed(m)

Making calculations in the way that all the restrictions are fulfilled, the toe that is found have the following characteristics:

Dn50(m)	0.88
Ht (m)	14.96
Nod (-)	0.5
H (m)	17.6
Width (m)	4.4
Height (m)	2.64

Table 10 Toe characteristics

4.7 The core

4.7.1 Stability after construction

The core forms the largest part of the breakwater and it consists in fine quarry material, which can be dumped easily by vessels.
Following the filter rules again the stability of the core is checked, and a rock class of 1-300 kg with a D_{n50}=0.38 m is used in it. With that very wide gradation (quarry run) could be problems of segregation, so special attention has to be paid, in case of use it. However, cheaper price is obtained.

4.7.2 Stability during construction

The same procedure as in the secondary armour is employed. The initiation of damage (s=2) starts with a wave height of 0.8 m and this wave height is excceded during 65% of the time.
With (S=5) the wave height is 1.1 m and this wave height is excceded during 50% of the time per year.
Loss of material is expected. If we assume a loss of material of 10% for a damage level of (S=5) the expected loss of material per year will be 5%. The core has to be covered as soon as possible for trying to minimize the losses.

4.8 Crest height

The crest height depends on the construction method, so has to be calculated for both construction methods(water-based and land-based). Depending on the final design crest height the construction method is decided.

* Construction methods

The choice of the construction method depends on different factors:

-Dimensions of the breakwater.
-Enviromental conditions.
-Bathimetry.
-Accesibility of the breakwater.
-Availability of equipement.

Within these boundary conditions a choice can be made between different type of construction methods and types of equipement. Production rates and fitness of equipment play an important role in the decision-making process.

4.8.1 Water-based construction

The breakwater can be constructed from pontoons positioned alongside the breakwater. This construction method has the advantage that no demands are imposed for a sufficiently high and broad working area on top of the breakwater. However, the costs of operation are higher and downtime due to wave conditions can be substantial. Water-based construction is thus only interesting if great reductions in the breakwater geometry can be achieved.
A combination of floating equipment and rolling equipment can be used for improve the schedule of the works or minimize the costs.
The crest width for water-based construction the CEM (2002) suggests a minimum crest width of 3 times the nominal diameter for concrete elements.
In water-based operations a maximum wave height of 1 m is allowed. The 58% of the time this wave height is exceeded. This is another data to take into account for the schedule of the work and the cost that could implicate.

The crest height in that construction method will be determined by the overtopping discharge that produce unsafe situation for the traffic in the protected area.
0.2 l/s*m is considered that could initiate that situation.

The overtopping discharge is also calculated with Coastal and River Engineering Support System (CRESS) based on the formulas provided by TAW(2002). The run up level considered is 1,28%.

q = a * exp (b * Rc)

The coefficients a and b are still functions of the wave height, slope angle, breaker parameter and other influence factors. The complete formulae are:

$$\frac{q}{\sqrt{g*H_{m0}^{3}}} = \frac{0.067}{\sqrt{\tan\alpha}} * \gamma_b * \xi_0 * \exp\left(-4.3 * \frac{Rc}{H_{m0}} * \frac{1}{\xi_0 * \gamma_b * \gamma_f * \gamma_\beta * \gamma_V}\right)$$

and a maximum of:

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

where:

q = average wave overtopping discharge (m3/s per m)

g = acceleration due to gravity (m/s2)

Hmo = significant wave height at toe of dike (m)

ξ_0 = breaker parameter = tan(α)/EFSo (-)

So = wave steepness =2*π*Hmo/g•Tp (-)

Tp = Peak period of the spectrum at toe of dike (s)

Tan(α)= slope (-)

Rc = free crest height above still water line (m)

γ = influence factors for influence of berm, roughness elements, angle of wave attack, and vertical wall on slope.

The wave height considered is the same as for the calculations of the stability of the armour units and the toe, because for the deterministic design is considered that the construction behind the breakwater have to be protected with the same security level as the breakwater itself. The economic consequences are not taken into account.

Calculations were carried out and the crest freeboard was Rc=NAP+18 m.
That means a total height from the bottom of 35.6 m.

4.8.2 Land-based construction

In that construction method cranes are positioned on top of the construction and trucks or pontoon supply them with quarry material or armour units. When the crane has built a new part of the breakwater, it moves to the end of the construction and starts again. For this construction method, the crest has to be high and wide enough(9 meters at least) to drive with two trucks, and also has to be thought the construction of turner platform for them. If the breakwater is too long, as this case, the transport time over the breakwater increase and could be uneconomical. The other fact is that the volume of the work is bigger because the breakwater is wider.
The overtopping of the breakwater under construction requires a minimum crest height of the core and indirectly the breakwater crest level.
For safety and proper working conditions, 0.01 l/s/m is assumed to be a maximum allowable overtopping discharge, under storm conditions.
During construction it is necessary to determine the minimum crest height of the core, which determines the total crest height of the breakwater.

To determine the crest height of the core has to assumed a downtime of the construction operations, so the crest height of the core is calculated for many downtimes.

Downtime(%)	Rc (m+NAP)
60	2.8
30	4.4
20	5.2
17	6
14	6.7
8	7.44
5	8.15
1	11.1

Table 11 Crest freboard-downtime construction operations

With a downtime of 1% the crest height of the core it must be 6.7 m. If we add the size of 3 diameters of the filter layer and the 2 diameters of the armour units the total freeboard for the land-based method is 18m.
It is shown that for land-based method the total crest is the same as for water-based method. So it does not implicate higher freeboard, just a crest wider.

Shown that, is feasible to combine water-based operations with land-based operations with prices not too much higher.

Besides when the land-based method is used the settlements goes faster because of the loads of the trucks and crains during construction.

4.8.3 Conclusions about the construction method

Because it is seen that independently of the construction method, the crest height must be at least NAP+18 m a combination of water-based and land-based operations is the best option. The geometry of the breakwater derived of the deterministic design could vary only if water-based method is aplied. Still, the dimensions of the breakwater, lead to a combination of both operations. Land-based operations take place and therefore the crest width must be width enough (12 meters).

4.9 Geometry of the breakwater

The breakwater will be constructed mainly to protect the land reclamation. At the inner slope side, will be covered with sand or other material, so the inner slope can consists of core material in the determination of the geometry.

GEOMETRY	Dimensions breakwater
Crest height	NAP+18m
Crest width	12m
Foundation depth construction	NAP-17.6m
Slope primary armour	1:1.5m
Slope toe	1:2m
Armour layer	
Weight of the cubes	18.6 t
Layer thickness	4.1m
Depth primary layer	NAP-14.96m
First underlayer	
Rock class	1000-3000
Layer thickness	2.64m
Core	
Crest height	10.96m
Stone class	1-300
Toe	
Rock class	1000-3000
Depth	14.96m
Height	2.64m
Width toe crest	4.4m
FILTER SYSTEM	
Bed stone size(d50)	0.2mm
1st filter layer	
rock class(dn50)	1.78mm
thickness	50cm
length before toe	2m
2nd filter layer	
rock class(dn50)	19.5mm
thickness	50cm
length before toe	1.75m

Table 12 Geometry of the breakwater

5 Costs

5.1 Introduction

The main objective of this study is to compare both different methods (deterministic and probabilistic) of design. Prices have to be established in order to compare both methods. But using the same prices in both designs the difference between both methods can be compared. The prices used in this study are merely approximated and mostly come from other related projects made in The Netherlands, which has been used as a reference point in this respect. It is also known, that the prices of material varies in time, place of supply and placement.
With the geometry of the breakwater it can be approximated the quantity of material needed and then, multiplying by a standard price of each material, an approximated price can be achieved.
The prices found are established in price per ton. In order to calculate the construction costs is necessary to find the amount of tones for each material used in the construction of the breakwater.
To calculate the tones of each material, needed for the construction, the following procedure has been done:
Firstly the area of the section is calculated. Then the bulck density as a laid with the formula:

$$n_v = 1 - (\rho_b / \rho_r)$$

In which:

n_v =volumetric porosity (-)

ρ_b =bulck density (Kg/m3)

ρ_r =density of rock (kg/m3)

It has to be known that the determination of the bulck density is not simple because of the errors made at the boudaries of the mesured volume. In situ tests are recommended to ascertain the actual values of the bulck density.
D'Angremond and Van Roode (2001) had collected values for the porosity of layers for different kind of rock. In this study these values are used in order to calculate the bulk density. In that way, it can be know the number of tonnes per meter of breakwater and then multiply for the price.

The demand of quarry material and concrete elements are calculated. Then a standard price is applied. Multiplying the quantities per the unitary price, the construction costs are calculated.

5.2 Demand and price of concrete

The concrete price has to be determined by the topography, market situation and acces to infrastructure. In Limburg the cost of the concrete is lower than the rest of holland, because of the availability of agregates there.
When more concrete is required, the prices use to change in a positive way.
The density of the concrete, also determine the price of the concrete. When is used a density of more than 3200 kg/m3, all agregates (sand,gravel) have to be removed of the mixture and finner Magnadense is added to the mixture. Bellow the density of 3200 kg/m3 coarser Magnadense has to be added to the mixture. So, that differences cause also diferences in the price.

Production costs of the concrete (Wagner,2004) of different densities are as:

Density (Kg/m3)	Price(euro/m3)
2400	80
2800	120
3800	250

Table 13 Production costs of concrete armour elements

The concrete used in calculations is the 2800 kg/m3 of density because leads to an optimum armour units elements. Since the deterioration is lower compared with the 2400 kg/m3 concrete, because of the measures. The weight is lower too and therefore the placement prices are lower, because cranes with lower capacity is required. In the study Research of costs in armour units the use of the heaviest concrete (3800 Kg/m3) leads to uneconomical option. But the use of concrete 2800 seems the best option.
Placement cost of the concrete armour depends on the weight element, the way of placing (water-based or land-based equipement) and placement rates. So, the lower the weight is, the chaper the price.

In The Netherlands and Spain concrete cubes have been used as armour unit on breakwaters.
The breakwaters of Scheveningen and Hook of Holland have a double layer of cubes. In Scheveningen the cubes were placed by two mobile land cranes and in Hook of Holland by two ships. The ships were tested were specialy designed for this project and tested in laboratories. The ships were very stable and a gantry crane was fixed on the ship. The gantry had two trolleys, which both had hydraulic clamps to pick up the concrete cubes. The placement rates of these gantry crane were high, approximately 20 cubes per hour.

Because the cubes of Scheveningen were 25 tonnes the same rates can be used for our project. It could be possible to rent that crane.
The price per week (5 labour days) of a crane with the capacity required is about 35000 euro/week.

Production		water-based	land-based
Concrete (2800kg/m3)	euro/m3	120	120
	euro/t	42.85	42.85
Placement			
Rate	1/hour	20	20
Operative hours/day	h	8	8
Working days/week	day	5	5
Dowtime reduction		0.4	0.9
No. of elements		61793	61793
Placement rate of the elements/week	1/week	644	644
Rate/crane	1/week	320	720
No. of cranes		2	1
Equipement	euro/week	70000	35000
Cost/cube	euro/cube	108.7	54.3
Cost/ton	euro/t	4.66	2.33
Production+placement	**euro/t**	**47.5**	**45.2**

Table 14 Total costs of armour units

It is seen that the variation of the placement method does not imply a very big difference in the price, so a standard price of 46 euro/ton is choosen to calculate the price of the armour layer.
With the geometry of the breakwater and the volumetric porosity, the bulk density of the armour layer is calculated. The quantity of tonnes per meter of breakwater required is 366 ton.
It has to be said that placement rates are approximated, they can vary quite a lot. It is just, for having an idea of the prices for further comparison with the probabilistic design.

5.3 Demand and price of quarry material

With the geometry of the breakwater the demand of quarry material for every layer is calculated and is shown in the following table:

Demand of rock	Area of cross-section (m2)	Bulk density (ton/m3)	Demand/m (ton/m)	Total demand (ton)x1000
CORE	1470,05	1,7	2499	9996
UNDERLAYER	147,01	1,6	235	940
BED PROTECTION	111,64	1,8	200	803
TOE	25,56	1,6	40	163

Table 15 Demand of quarry material

Actually the bed protection does not come from a quarry. Normally this material comes from dredge. But is included in the same table because a standard price will be applied.
Prices of quarry material are composed by supply and placement. Supply includes the price of production in the quarry and transportation.
Placement rates, varies for each material because the equipment is different and have different costs. Next table summarize the prices:

Stone class	Supply(euro/ton)	Placement(euro/ton)	Total (euro/ton)
1-300	25	6	31
1000-3000	27	6,5	33,5
3000-6000	30	7	37
bed protection and filter system	20	5	25

Table 16 Price of quarry material

5.4 Summarize of the costs

Total construction costs of the breakwater are calculated and summarized in the next table:

	(Euro/m)	Total (Million Euro)
CORE	77471	309.9
UNDERLAYER	7879	31.5
ARMOUR LAYER	16847	67.4
FILTER SISTEM	5023	20.1
TOE	1369	5.5
TOTAL Construction costs	108592	434.36

Table 17 Construction costs deterministic design

And the percentage of each element of the breakwater is shown in figure 15:

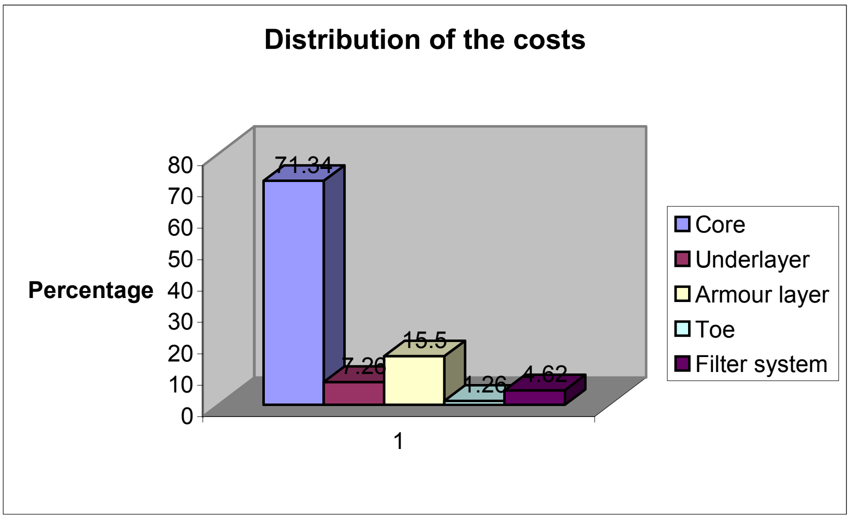

Figure 15 Distribution of construction cost deterministic design

In the distribution of percentage of the price is shown that, 71 percent of the costs correspond to the core material. This percentage is too big, but normal because the breakwater has a very high crest height and also the depth is too large. Reductions in the crest height can reduce the dimensions and therefore the construction costs. In the probabilistic optimization a risk analis with the crest height will be made in order to chek if it is profitable to reduce the crest height, or even increase it depending on the total costs.
The percentage of the armour layer is 15.5 percent. This is the second higher percentage in the breakwater. The percentage of the other parts are low compared with that both.
The toe structure has the lowest percentage of construction costs. However a failure of the toe could produce the collapse of the breakwater, because is in part the responsible of the armour layer stability.

5.5 Conclusions

It is seen that the crest height of the breakwater is 18m independent on the construction method beacause behind the breakwater the level of safety aplied in the deterministic design is quite high. The volume of material is very big and therefore the cost is also quite high.
The no construction of the land reclamation brings about a quite considerable loss of expected income. In the following chapters a probailistic optimization by taking into account damage costs and downtime costs in the protected area, will be done in order to find the most economical option.
Further types of breakwater (caisson breakwater, berm breakwater…) , should be studied in order to compare the prices.

6 Probabilistic optimization

6.1 Introduction

Experiences with similar structures and model tests provided knowledge about the behavior of a breakwater. The failure data of these experiences were combined with a safety factor and translated into design guidelines for future designs. Therefore, the deterministic design that results from these guidelines contains a safety margin. Multiple geometry calculations can be performed to determine the optimal breakwater dimensions. This can also include the economic consequences of functional failure of the breakwater or collapse of the breakwater.
However, with deterministic design methods the influence of the variation of the strength of the breakwater components and the variation in the load on the breakwater is often neglected or a partial safety factor is used.
With probabilistic design methods these variations can be included. The distribution of strength and load is taken into account when calculating the probability of failure for the breakwater, both for functional failure as for breakwater collapse. If these probabilities are established and are associated with the consequences of the two types of failure (risk), the total costs over the lifetime of a specific design can be determined. These total costs over the lifetime consist of the construction costs, the costs for maintenance, the costs due to collapse and the costs due to functional failure. By means of comparing the total costs of several design alternatives the most economic design can be selected.

***Risk analisys**

In order to judge wether a thechnical system such as a breakwater satisfies the requirements that the users and society expect with regard to its function, including safety and economy, it is possible to use the risk analisis methods. The term risk comprises the probability of occurrence of an undesirable event (failure due to a storm) and the consequences of the occurrence of that event (economic loss, maintenance or repair). By risk analisis may be understood the whole set of activities aimed at quantifying, on the one hand, the probability of the occurrence of the undesirable event and, on the other, the consequences of the occurrence of that event.(PIANC, 1992)

***Construction costs**

Construction costs are dependent on the dimensions of the breakwater. In this study only the crest height (Rc) and the element weight (W) are varied:

$$\text{Construction costs} = I_0(W, Rc)$$

Actually, each alternative represents a complete design alternative. This means that a change in the weight of the armour units represents a change in the subsequent layers and in the toe structure. These variations in this study will be neglected. As it was previously seen when carrying out the deterministic design, in principle the construction costs of the breakwater increases with the element weight and the crest height. For all the combinations of the crest height and element weight the construction costs have to be calculated. Variations in the element weight, only has consequences for the costs of the cubes (in fact, it also has consequences in other elements, like underlayer and other components, but in this study those will not be considered because the consequent change in the price is assumed to be low). However, variations in the crest height imply variations in all the components and therefore they have to be recalculated.

***Repair of collapse costs**

The most important failure is the collapse of the breakwater. With heavier blocks the possibility of collapse is lower.

The risk of collapse is determined by multiplying the damage costs with the probability of collapse, which is assumed to be a function of the element weight. The influence of the crest height on the probability of collapse is neglected. The failure of the toe or the primary armour layer is assumed to give the failure probability of the breakwater.

To take into account the effects of interest and inflation rates, the costs during the lifetime of the breakwater are discounted to the year at the lifecycle of the breakwater.

$$R_{isk,collapse} = \sum_{t=1}^{L} (P_{collapse}(W) * D_r(W, Rc) * \frac{1}{(1+r)^t})$$

$P_{collapse}$ =probability of collapse of the breakwater

D_r = costs of repair the collapse

r= discount rate=5%

t=year in the lifecycle of the breakwater

L=lifetime of the breakwater

Rc=crest freeboard

***Costs due to functional failure**

The second failure event occurs when the shelter provided by the breakwater is not enough to protect the area behind the breakwater. That sheltering function can be improved by increasing the crest height of the breakwater in order to reduce the overtopping. It can also be decreased, by obtaining the capitalized risk in order to find the most economical option.

To determine the risk of functional failure, the damage behind the breakwater has to be quantified (costs) and the probability of excedance of certain volume of overtopping discharge.

$$R_{isk,overtopping} = \left(P_{exceed,volume}(Rc) * D_o(O_{vertopping-discharge})\right)$$

$P_{exceed,volume}$ = Probability of exceedance of a certain overtooping volume.

D_o =Sum of damage in the protected area plus the cost of downtime.

***Maintenance costs**

Maintenance costs are dependent basicaly on the element weight. The heavier the weight of the element is, the less probability of minor damage, and therefore lower maintenance costs.

The discounted maintenance costs will be calculated with the following formula:

$$\text{Maintenance} = \sum_{t=1}^{L} (M * \frac{1}{(1+r)^t})$$

In which:

M=Maintenance costs

Maintenance costs are quantified by calculating the probability of damage with Nod=0.5 in the armour layer and multiplying by the costs of repair this damage. The maintenance is also dealt as a risk. A more element weight, less maintenance costs.

***Total costs**

The discounted value of the total costs results from the addition of the investment (Io) to the total risk component and maintenance costs:

In the calculations the influence of the deterioration of the armour elements (and therefore the loss of weight) is neglected. On top of this, when the costs of downtime are calculated, the growth of the incomes is neglected. In fact, all these values are variables and the sum has to be calculated for every year. In this study all that values have been assumed to be constant, and in that case, the formula of the total costs can be written as follows:

$$TC = I_0(W, Rc) + \left[P_{collapse}(W) * D_r(W) + P_{exceed,volume}(Rc) * D_o(V_{overtopped}) + M \right] * \sum_{t=0}^{L} \left(\frac{1}{(1+r)^t} \right)$$

6.2 Breakwater alternatives

The breakwater lifecycle costs are calculated for several alternatives. The alternative with the lowest total cost over the lifetme provides the optimal crest height and element weight combination.

6.2.1 Element weight

The deterministic design provided an element weight of 23,3 ton. The element weight of the alternatives, result from a variation of the element volume from 4 to 12 m3. Taking into account specific density of the concrete of 2800 kg/m3 the element weight varies from 11.2 ton to 33.6 ton.
The evaluated volumes, nominal diameters and weights are shown in the following table:

Volume (m3)	Dn (m)	Weight(tn)
4	1,58	11,2
6	1,8	16,8
8	2	22,4
10	2,15	28
12	2,28	33,6

Table 18 Alternatives of cubes

6.2.2 Crest height

The deterministic calculations provide a crest height of 18 m. The economic consecuences of downtime were no taken into account in the deterministic calculations. Several options will be calculated regarding the crest height in order to find the most economical option.
From 14 to 20 m the crest height will be varied with an incremental stepsize of one meter.

6.2.3 Alternatives

The total number of combined alternatives to be calculated, for the five weights and the seven crest heights, results in the 35 alternatives indicated in the below table.

	crest heigth (m + NAP)						
element volume (m3)	14m	15m	16m	17m	18m	19m	20m
4 m3	1	2	3	4	5	6	7
6 m3	8	9	10	11	12	13	14
8 m3	15	16	17	18	19	20	21
10 m3	22	23	24	25	26	27	28
12 m3	29	30	31	32	33	34	35

Table 19 Breakwater alternatives

6.3 Probabilities of failure

To determine the probability of failure of the failure mechanisms the following steps will be made. First, the failure formula is established. This is the same formula as used in the deterministic calculations. This formula is subsequently rewritten as a reliability function. A reliability function is a function of the following form:

Z=R-S=R(x1,x2,…,xm)-S(xm+1,…,xn)

In which:

R = strength
S = load

x1,…xm,…xn represent all random variables involved in the streght and load.

By defining this reliability function, the failure zone, no-failure zone and failure limit can be indicated. The following statements are valid:

Z > 0, no-failure zone;
Z = 0, failure limit;
Z < 0, failure zone.

After defining the reliability function for the failure mechanism, the behaviour of the variables is given. This behaviour can be assumed deterministic or stochastic. If the behaviour is stochastic, the distribution of the variable is provided based on available numerical data and on expert judgement.

Monte Carlo method simulations are used to get the probabilities of failure of the armour units and the toe structure. This method follows the following procedure: For all the parameter is taken a random number of values, taking into acount the probability distribution of this parameter. That means, that a value with a high probability density, will appear more often. When all the parameters have a value, the resulting value for G (reliability function) is computed for the equation of the reliability function.
This procedure is repeated N times after which P(failure) simply is Nf/N. Nf is the number of times that the reliability function is less than 0. The procedure is simple but the number of repetitions is very high. The higher the number of simulations, the more accurate the result is.
The number of simulations to be done depends also on how small the probability of failure is. A probability of failure with an order of magnitude of 10^{-4} requires a minimum of about 10^{-6} to 10^{-7} simulations, according to the following equation: (CUR 190,1997)

$$n > 400(1/Pf-1)$$

Probabilities of failure are calculated with the computer program called VaP. For more details about this program see Appendix IV.

6.4 Armour units

6.4.1 Reliability function

The formula for the determination of the reliability function is the same used in the deterministic design.

$$Hs/\Delta Dn = (6.7 * Nod^{0.4} / N^{0.3} + 1.0) * s_{om}^{-0.1}$$

But now the wave height is not a fixed value that is determined. The distribution of probability is introduced. The distribution of the wave height was obtained with the data of the Europlatform. To represent the energy dissipation of the waves, a coeficient multiplying the wave height has to be introduced. Rewriting the equation and introducing that parameter the reliability function turns out as follows:

$$G = (A * N_{od}{}^{0.4} / N^{0.3} + 1) * S^{-0.1} * \Delta * D_n - H_S * \gamma_r$$

6.4.2 Distribution of the parameters

Wave height Hs

Two function distributions were compared with the dataset in order to fit the one with the best correlation.

$$\text{Weibull distribution: } Q = \exp\left[-\left\{\frac{H_{ss}-\gamma}{\beta}\right\}^{\alpha}\right]$$

$$\text{Gumbel distribution: } Q = 1-\exp\left[-\exp\left(-\frac{H_{SS}-\gamma}{\beta}\right)\right]$$

Where Q is the exceedance probability. The wave height was assumed that follows a Weibull function distribution because this has the best correlation coefficient. The procedure to obtain the function distribution is explained in Appendix I.

The results are summarized in the next table:

Distribution	Parameters	Correlation with dataset
Gumbel	β=0.627 γ=1.88	0.9875
Weibull	α=1.8 β=1.867 γ=0.477	0.9984

Table 20 Correlation of the extreme wave height distributions

Wave height nearshore

The function distribution of the wave height was acquired out of data from the Europlatform, relatively deep water. Yet, to fit the function distribution in the reliability function, the wave height has to be reduced in order to take into account the energy dissipation of the wave height. To make this fact feasible, the wave height is multiplied by the energy dissipation coefficient γ_r.

This factor is obtained comparing many wave heights in the Europlatform with their values in front of the breakwater.

The coefficient is obtained by dividing the wave height nearshore by the wave height offshore for several values. Then the average and the standard deviation is calculated.

The wave heights taken into account are biger than five meters with a stepsize of 0,25 meters until arriving to a wave height of eight meters. This is so owing to the fact that it has been taken into account low wave height values, and thence, the reduction coefficient

becomes higher and does not represent the reduction of the highest waves, which are considered the most important.
The mean (0,86) and the standard deviation (0,02) is calculated and is assumed that follows a normal distribution.

Parameter A

Parameter A is assumed that follows a normal distribution with a mean of 6.7 with a standard deviation describing the accuracy of the equation itself. According to Van der Meer the value of the standard deviation for it is approximately the 10% of its value.

Damage level Nod

The damage level Nod is dealt with as a deterministic parameter, since it is known.

Wave steepness

Considering the heavier storms from all the wave data it is possible to get the average wave steepness and the standard deviation. The assumed distribution for this parameter is the normal distribution.
The storms considered to get the average wave steepness were the ones with a wave height higher than 4.5 meters. Fifty-two storms were found in the data from 22 years. With the following formula the wave steepness is calculated:

$$s = \frac{2 * \pi * Hs}{g * T^2}$$

And the mean and standard deviation derived was:

Mean=0,069   Standard deviation=0,006

In the deterministic design the average wave steepnes adopted was 0.065 based on the relations mentioned there. Now, when calculating the average wave steepness for the highest waves, one can see that the average it is too similar.

Number of waves

The average period calculated from the wave steepness is 6,8 s. If the average duration of this storms are 6 hours, we can calculate the number of waves in that storms.
There are 3000 waves in a storm. It is important to take into account that the duration of the storm can vary quite a lot, up to 20 hours, which means that could be around 10000 waves. That fact produces large standard deviations in the number of waves. The normal distribution cannot be fitted here because with high standard deviations the number of waves could become negative. Thus, a Log-Normal distribution will be asssumed. The standard deviation considered is 4000 waves.

Nominal diameter

Small variations can suffer the diameter of the cubes and that parameter will be considered as a deterministic value, because the influence in the results is slight.

Relative density

Because of the small variations of the density of the water, the relative density of the cubes will be assumed to be distribuited with a normal distribution with a mean value of 1,73 and a standard deviation of 0,05.

Variables normal distributed	Mean	Standard deviation
A	6,7	0,67
Som	0,069	0,006
Δ	1,73	0,05
γ_r	0,86	0,02

Table 21 Variables Normal distributed armour layer

Variables Weibull distributed	Parameters
Hs	α=1,8 β=1,867 γ=0,477

Table 22 Variables Weibull distributed armour layer

Variables Log-normal distributed	Mean	Standard deviation
N	3000	4000

Table 23 Variables Log-Normal distributed armour layer

Variables deterministic	Values
Nod	0,5 and 2
Dn (m)	1.58 , 1.82 , 2 , 2.15 , 2.29

Table 24 Variables deterministic armour layer

The deterministic parameters have different values, because for each element volume of the alternatives, the probabilities of failure are calculated for the diferent damage levels.

6.4.3 Probability of failure for Nod=0,5

The probability of failure Nod=0.5 is calculated for all the different alternatives of cubes. These probabilities are calculated in order to quantify the maintenance costs.

Volume (m3)	Weight(tn)	Dn (m)	P((G<0)/year) (Nod=0.5)
4	11.2	1.58	0.05022
6	16.8	1.82	0.01116
8	22.4	2	0.001116
10	28	2.15	0.000372
12	33.6	2.29	0.000279

Table 25 Probabilities of failure armour units Nod=0.5

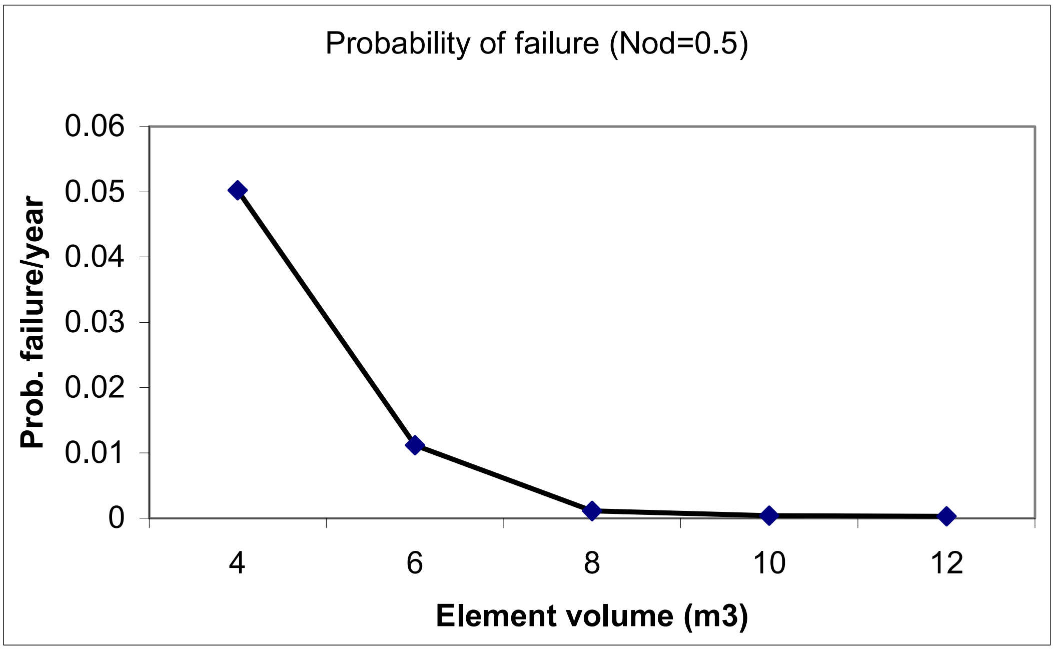

Figure 16 Probabilities of failure armour layer Nod=0.5 (Cubes)

Probability of failure decreases with the element volume. Probabilities of ocurrence of a minor damage (Nod=0.5) on the armour layer, are related to the maintenance frequency requiered. The more the probability of failure, the more maintenance required. With element volumes bigger than 8 cubic meters almost no maintenance is required, but, nevertheless, they entail higher construction costs.

6.4.4 Probability of collapse

The probability of collapse is calculated for all the different alternatives of cubes.
The collapse of the breakwater be ascertained when the damage level reaches the value of Nod=2.

Volume (m3)	Weight (tn)	Dn (m)	P(Collapse/year) Nod=2
4	11.2	1.58	0.0059706
6	16.8	1.81	0.000465
8	22.4	2	0.000093
10	28	2.15	0.00001023
12	33.6	2.29	0.0000093

Table 26 Probabilities of collapse armour layer

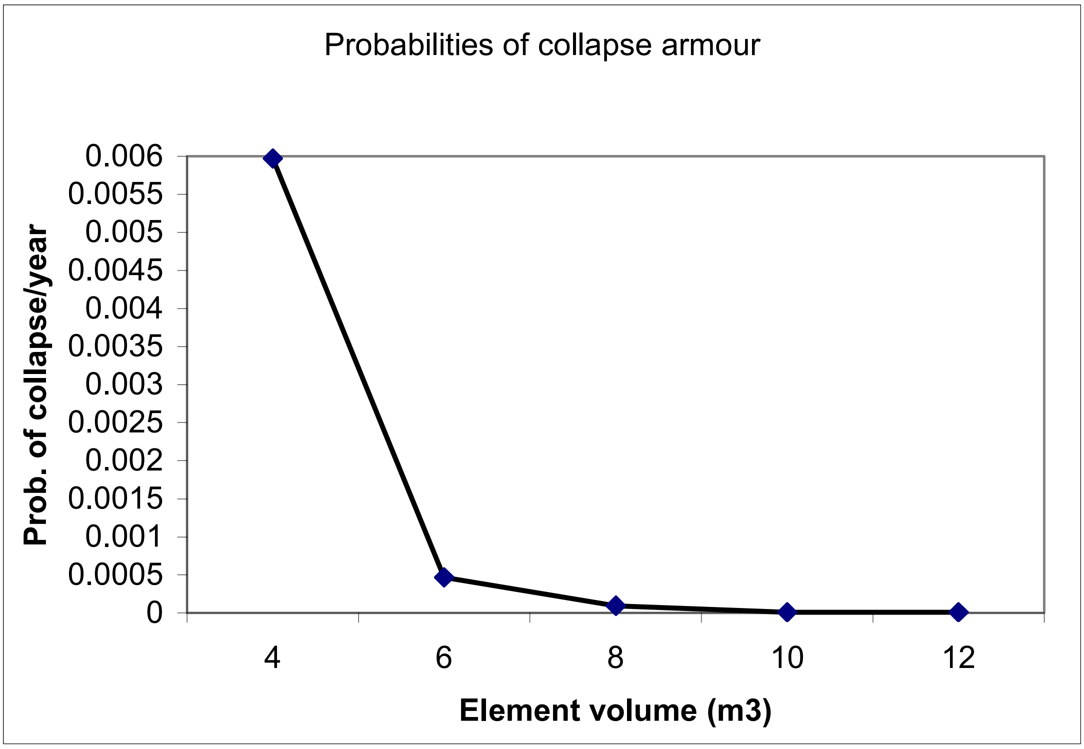

Figure 17 Probabilities of collapse armour layer

Probabilities of collapse for the armour units decreses very swift when the weight increases. Collapses imply very high costs of reconstruction. The probability of collapse is in other order of magnitude, far smaller than the probability of minor damage.

6.5 Toe structure

6.5.1 Reliability function

The probability of failure of the toe is independent from the armour layer units. The probabilities of failure for the toe structure are determined with the same method as the stability of the armour units (Monte Carlo simulations).

The formula used for probabilistic calculations is the same as the one used in the deterministic design:

$$H_s / \Delta D_{n50} = (0.24 * h_t / D_{n50} + 1.6) * N_{od}^{0.15}$$

In which:

ht=hbed + hsurge-htoe

hbed=bed level respect NAP.

hsurge=water level in a storm surge respect NAP.

htoe=height of the toe=(3 or 2)*Dn50

Rewriting the equation, the reliability function becomes:

$$G = (0.24 * (h_{bed} + h_{surge} - 3 * D_{n50}) / D_{n50} + 1.6) * Nod^{0.15} * \Delta * D_{n50} - Hs * \gamma_R$$

To take into account the uncertainty of the rock density the formula has to be written as follows:

$$G = (0.24 * (h_{bed} + h_{surge} - 3 * D_{n50}) / D_{n50} + 1.6) * Nod^{0.15} * (\rho_r / \rho w - 1) * D_{n50} - Hs * \gamma_R$$

The height of the toe is varied from 3 to 2 times the nominal diameter. With this variation, the best toe within the validity range of the formula can be selected.

6.5.2 Distribution of the parameters

Wave height(Hs)

The wave height distribution is the same as calculated in the stability formula, including the energy dissipation factor.

Nominal diameter(Dn50)

Nominal diameter of the stones Dn50 is assumed that follows a normal distribution with a mean value of 0.88, as obtained in the deterministic design. A standard deviation of 0,05 is assumed to include the uncertainty in the median stone diameter (Dn50).

Depth of the bed(Hbed)

The depth of the bed is a parameter that could vary enough because of the irregularities caused by long-shore transport. Also is assumed normal distributed with a mean of 17,6 m and a standard deviation of 1 m.

Rock density

The rock density is considered to follow a normal distribution with a mean of 2650 kg/m3 and a standard deviation of 200 kg/m3.

Water level

The water level due to a surge follows a gumbel distribution and is based on the data of the Hook of Holland (d'Angremond and Van Roode, 2001). In order to calculate the probability of failure the worst condition has to be taken into account. That situation takes place when the water level is the minimum. It was proved in foregoing sections of this study that the maximum water levels according to the Port of Rotterdam report (Doc.nr.:AA-02-330) followed the same distribution as the Appendix 10 of the book referenced of d'Angremond and Van Roode, but in that last the distribution of the minimum water levels was discarded. It is observed that the distribution of the minimum water levels follows the same distribution as the maximum. The final results, however, is 1.81 m less due to the maximal tidal range. The parameters of the distribution have been found simply by considering two values and two exceedance frequencies:

Frequency	1/year	1/10year	1/100year	1/1000year	1/10000year
max. water level (NAP+m)	2.3	2.89	3.52	4.21	4.95
min. water sea level(NAP+m)	0.48	1.07	1.7	2.39	3.13

Table 27 Water levels

Apling the gumbel distribution two times for diferent exceedance frequencies:

$$0.1 = 1 - \exp\left[-\exp\left(-\frac{1.07-\gamma}{\beta}\right)\right]$$

$$0.01 = 1 - \exp\left[-\exp\left(-\frac{1.7-\gamma}{\beta}\right)\right]$$

Solving the equation system the parameters are γ=0.47 and β=0.27 for the distribution of the minimum water levels.

Damage level

The damage level is assumed deterministic and it has the same values as the deterministic design.

Critical values for Nod are:

0.5 Start of damage

1.0 Acceptable damage

4.0 Failure

Next tables summarize the distributions followed by the parameters involved:

Variables normal distributed	Mean	Standard deviation
Dn50	0.88	0.05
hbed	17.6	1
gamma reductor	0.84	0.05
rock density	2650	200

Table 28 Variables Normal distributed Toe

Variables Weibull distributed	Parameters
Hs	α=1.8 β=1.867 γ=0.477

Table 29 Variables Weibull distributed Toe

Variables Gumbel distributed	Parameters
hsurge	β=0.3 γ=0.47

6.5.2.1 Table 30 Variables Gumbel distributed Toe

6.5.3 Probability of failure

In order to choose the optimum toe the probability of failure has been calculated for different toes.

Rock class	1000-3000 (kg)	3000-6000 (kg)
Ht=3*Dn50	Dn=0.88 m	Dn=1.175
Pf(Nod=0.5)	0.00465	0.001302
Pf(Nod=1)	0.001395	0.000279
Pf(Nod=4)	0.0000465	0.0000186
Ht=2*Dn50		
Pf(Nod=0.5)	0.00186	0.000372
Pf(Nod=1)	0.000372	0.000093
Pf(Nod=4)	0.0000186	0.0000093

Table 31 Probabilities of failure Toe structure

It is pointed out that for the toe chosen in the deterministic design the probability of start of damage is 0.47%. The probability of a severe damage (Nod=4) is quite low 0.005%.

As shown in the deterministic design the price of the toe structure is around 4% of the total cost. However, the failure of the toe implies a very high repair cost because is in part the responsible of the armour layer stability. Therefore a very low probability of failure has to be assumed for the toe structure, around ten times less than the armour units.
Increasing the rock stone class to 3000-6000 the probability of failure of toe structure decreases considerably. Also is shown that the depht of the toe is another important parameter that reduces the probability of failure.
Once at this point, the choice of the toe can be determined because of its repercution in the price.
The optimum toe found is the one, which is composed by stone rock class of 3000-6000 kg and a height of two times the nominal diameter.

6.6 Probabilities of failure of the breakwater

The sum of the independent probabilities of collapse of the breakwater components, armour layer and toe structure, are assumed to represent the probability of collapse for the combined probability of collapse. This upper fundamental bound is an overestimation of the actual probability, but gives a better approximation than the lower fundamental bound: the maximum value of the probabilities of failure.

When probabilities of failure for the toe structure where determined it was made obvious that the collapse of the toe is reached for a damage level of Nod=4.
The probability of collapse of the toe structure is independent of the armour units and the crest height.

Next table shows the total probability of collapse of the breakwater for each element volume:

Volume (m3)	Weight (tn)	Dn (m)	P(Collapse) Armour	P(Collapse) Toe	P(Collapse) Breakwater
4	11.2	1.58	0.0059706	0.0000093	0.0059799
6	16.8	1.81	0.0004650	0.0000093	0.0004743
8	22.4	2	0.0000930	0.0000093	0.0001023
10	28	2.15	0.0000102	0.0000093	0.0000195
12	33.6	2.28	0.0000093	0.0000093	0.0000186

Table 32 Probabilities of collapse of the breakwater

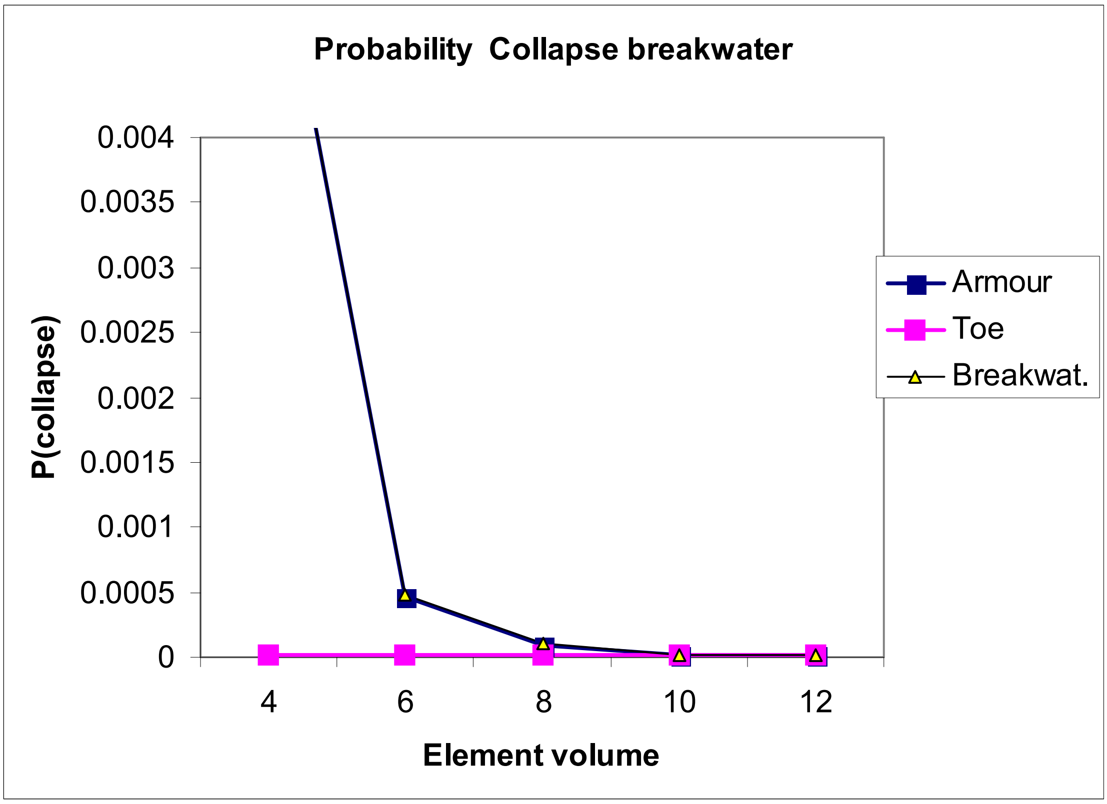

Figure 18 Probability of collapse of the breakwater

It is shown that the influence of the probability of collapse of the toe in the total probability of collapse of the breakwater does not have a big influence for smaller elements. That influence is apreciated when elements are bigger than 10 m3.

6.7 Crest height Risk analisys

The serviceability limit state is defined as the exceedance of a critical value of overtopping discharge, which produces the downtime, minor damage or severe damage in the protected area. In that situation the structure does not collapse, but the function of the breakwater is not fulfilled.
The following iterative process has been followed in order to estimate the probability of functional failure.
For each crest height selected, the wave height near shore, which produces overtopping discharges of values 0,2l/s per m, 2 l/s per m, and 10 l/s per m are calculated with the same formula as in the deterministic design. These wave heights are translated into offshore, where the distrubution of the wave height was obtained. With the function distribution, the probability of ocurrence per year for a storm with this wave height is calculated.
In the below table are shown the wave heights, the probability of ocurrence and the discharges wich produces in each crest height:

Crest height (NAP+m)		0.2 l/s per m	2 l/s per m	10 l/s per m
20	Hs	6	7,8	NO
	Ocurrence	0,0047	6,7E-06	
19	Hs	5,65	7,3	NO
	Ocurrence	0,014	0,00005	
18	Hs	5,25	6,9	8,4
	Ocurrence	0,045	0,00023	0,0000005
17	Hs	5	6,25	7,7
	Ocurrence	0,091	0,011	0,0000125
16	Hs	4,51	5,75	7,1
	Ocurrence	0,33	0,011	0,00011
15	Hs	4,25	5,25	6,5
	Ocurrence	0,66	0,045	0,00091
14	Hs	4	4,4	5,98
	Ocurrence	1,05	0,43	0,0052

Table 33 Overtopping discharge, wave height and probability per year

With these values and the economic consequences, which produce each overtopping discharge, the risk is determined.

6.8 **Cost quantification**

6.8.1 Discount rate

The net interest rate, or discount rate, is of importance for the optimisation of the breakwater over the lifetime of 50 years. If the real interest (the nominal interest minus the inflation) decreases, the discounted value of costs made in the future increases. The real interest influences the choice of the appropriate geometry. In this study the real interest rate is assumed to be five percent and is constant during the lifetime of the breakwater. In chapter 6.9.4 the discount rate is varied in order to see the influence in the economic optimal alternative of the breakwater.

6.8.2 Construction costs

For all the alternatives the construction costs are calculated like in the deterministic design, with the geometry and the estimated prices per ton of material. The results are shown in the following table:

	Crest heigth (m + NAP)						
element volume (m3)	14m	15m	16m	17m	18m	19m	20m
4 m3	332	353	374.7	397	419.9	443.4	467.6
6 m3	338.7	359.9	381.8	404.4	427.6	451.4	475.8
8 m3	344	365.5	387.6	410.3	433.7	457.7	482.3
10 m3	348.5	370.1	392.4	415.3	438.8	463	487.8
12 m3	352.4	374.2	396.6	419.7	443.3	467.7	492.6

Table 34 Construction costs of alternatives in million Euros

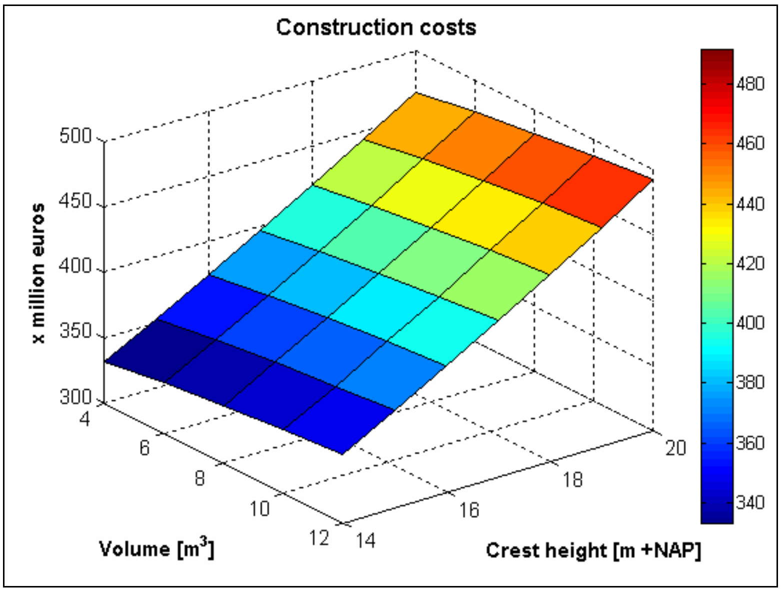

Figure 19 Constuction costs of alternatives

The alternative with the lowest considered crest height and the lowest element weight is the one with the lowest construction costs: 332 million Euros. These costs can be considered as the initial construction costs Io to be made, independent of the variation of crest height and tha element weight taken into account. A higher height and or heavier element weight leads to increasing construction costs. Dependent on the crest height and element weight additional costs have to be added to the initial costs. This is described with the formula for the initial investment costs, derived in Chapter 6:

Construction costs =Io(W,Rc)

In Vrijling (1998) these additional costs are linearised for the weight variation and the variation of crest height. Examining Figure 18, a linear variation of the construction costs also seems a good approximation. The derivatives of the calculated alternatives to both the crest height (I Rc) and element weight (I W) variation are given in the next Table.

Derivatives of construction costs:

	Crest heigth (m+NAP)							I Rc
element volume (m3)	14m	15m	16m	17m	18m	19m	20m	(Euro million/m)
4 m3	332	353	374,7	397	419,9	443,4	467,6	22,6
6 m3	338,7	359,9	381,8	404,4	427,6	451,4	475,8	22,9
8 m3	344	365,5	387,6	410,3	433,7	457,7	482,3	23,1
10 m3	348,5	370,1	392,4	415,3	438,8	463	487,8	23,2
12 m3	352,4	374,2	396,6	419,7	443,3	467,7	492,6	23,4
I W (Euro million/m3)	5,1	5,3	5,5	5,7	5,9	6,1	6,3	

Table 35 Derivatives of construction costs

The derivative for the construction costs due to the variation of the crest height is significantly influenced by the element weight and vice-versa. Therefore no single derivative can be assumed to represent the cost consequences of a variation in element weight or crest height. The derivations show that the influence of the crest height on the investment cost is considerable. This is due to the fact that a small reduction of the crest level decreases the area of a breakwater cross-section considerably. The influence of the element weight variation on the initial construction costs is smaller, but not negligible.

6.8.3 Repair of collapse costs

The collapse of the breakwater is mainly produced by the collapse of the armour layer. When probabilties of collapse for the breakwater were calculated, it was shown that the probability of collapse of the toe was almost no sensitive compared with the probabilities of collapse for the armour units smaller than 10 m3.
The eroded area of the armour units, for the damage level, which implies the collapse is analized.
It has been assumed that the collapse starts when the damage level is Nod=2.
The eroded area in that case can be calculated and therefore the number of blocks that produces the collapse.

$$S=Nod*2$$

In which S is the damage level for quarry stone and is compatible with the values of Nod. S is about double the value of Nod (d'Angremond and Van Rode 2001). Also is known that the damage level defines an eroded area:

$$S=\frac{A}{{D_{n50}}^{2}}$$

For the different alternatives of cubes the eroded area is calculated. The eroded area is independent of the crest height,it just depends on the weight of the elements. This is because because the eroded area belongs to the breakers area. Table 37 shows the results:

Element volume (m3)	Dn (m)	Eroded area(m2)
4	1,6	10.24
6	1,8	12.96
8	2	16
10	2,15	18.49
12	2,3	21,16

Table 36 Eroded area Nod=2

It is shown that a small eroded area (around the cross-section area of 5-6 cubes) produces the collapse of the breakwater.
When this situation occurs, the armour layer loses stability, and the cubes that are over the breakers area fall down. In that situation the layer under cubes is exposed to severe wave attack, which is not designed to withstand. Damage is also considered there.
The repair costs of collapse are different for diferent elements and crest heights. A heavier element derives into a more expensive reconstruction. The higher the crest height, the more elements to replace. A percentage of the construction costs seems a good aproximation to estimate the costs of repair in case of collapse of the structure. A 20 percent of the construction costs is the assumed price to repair the collapse.

6.8.4 Maintenance costs

Costs of maintenance are calculated in the same way as the collapse costs but for the damage level Nod =0.5. It is supposed that, when this damage level is reached the structure does not collapse but needs to be repaired after the storm.
The eroded area in this case is calculated as in the collapse situation

Element volume	Dn	Eroded area(m2)
4	1.6	2.56
6	1.8	3.24
8	2	4
10	2.15	4.62
12	2.3	5.29

Table 37 Eroded area Nod=0.5

In this case the eroded area is much smaler. The underlayer does not suffer damage but some cubes have to be replaced or recollocated, depending on the state of them. Here the costs of repair, depends also on the element volume and crest height. A two percent of the construction costs is taken into account.

6.8.5 Downtime costs in the protected area

As it is seen in the figure 19, the area of the chemicals and liquid bulk and container are far enough from the protected area of the breakwater. It is considered that the wave attack does not afect to this area directly, but this wave attack could create downtimes indirectly. The most important design condition for this area, which could afect it would be the water level.

But behind the breakwater there is a road, which connects the chemicals, liquid bulk and containers storage area. The transport of chemicals and liquid bulk will be all by pipeline. However part of the containers transport will be made by road.

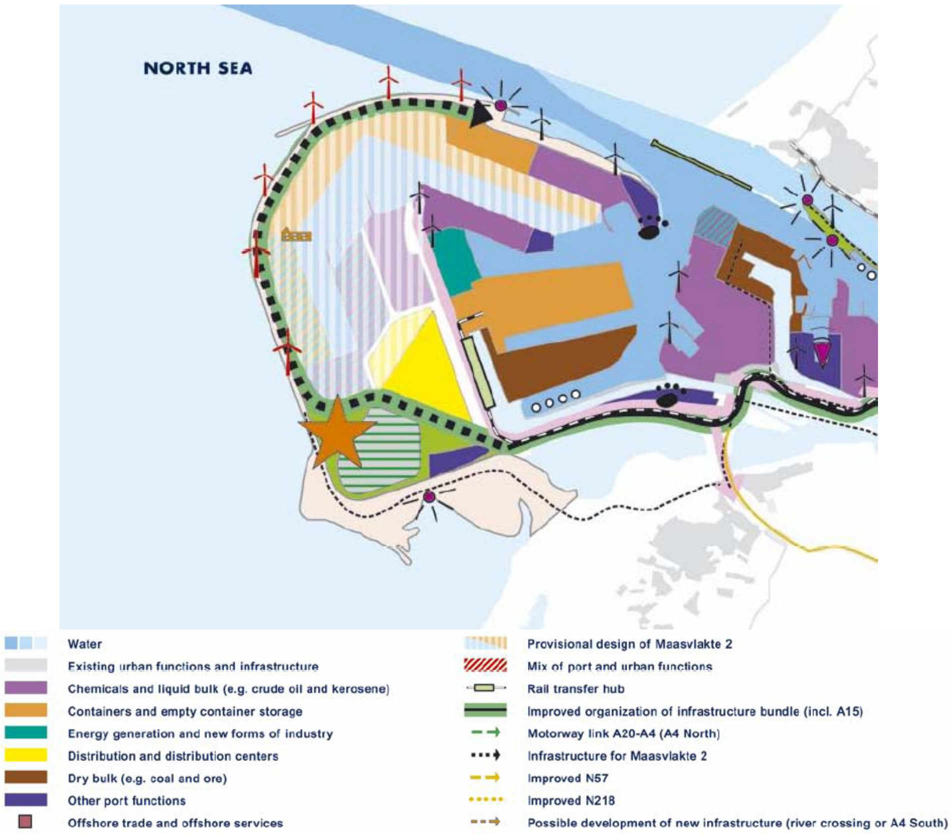

Figure 20 Distribution of the protected area

Downtime costs are produced when overtopping discharge exceeds a certain amount.
It is assumed that the operations behind the breakwater are stopped when the overtopping discharge is bigger than 0.2l/s per meter of breakwater. When 2l/s per meter of breakwater is exceeded is assumed that the road will be closed also and a certain damage in the protected area will occur, but after the storm, the damage considered does not implicates the non-use of the road and therefore no downtimes are expected.
Notwithstanding, when the amount of 10l/s per meter of breakwater is exceeded, the road of course will be closed during the storm, but also a downtime of 2 months is expected in order to restore the traffic by the road.
The expected volume of TEU is around 3000000 per year, in the year 2035 (at 25 years of the lifetime) in the new container terminal of Maasvlakte II. These data comes form the statistics given by the Port of Rotterdam (www.portofrotterdam.com). The lifetime of the breakwater is 50 years, and growth is also expected during the last 25 years. In order

to make the calculations the value of TEU per year chosen is that one (3000000 per year), which is assumed to be constant during the whole lifetime.
The incomes for the Port of Rotterdam are about 130 euro per TEU and therefore that is, in turn, the loss in case of downtime. In order to consider also the losses that downtime also produces for companies, a percentage of the costs per teu is applied.
Vrijling (1998) gives a multiplier of 1.5 for the indirect economic damage in case of downtime, so the damage estimated per TEU has a value of 195 Euro/TEU.
The transport of containers, nowadays take place by ship, train and road. Inland ship represents the 50 % of the container transport and therefore that percentage will not be taken into account in the downtime loses.
The losses per hour are calculated and are shown in the next table:

Item		Euro/hour
Loss of income direct	Expected income 170 TEU/hour	
	Port dues: 130 Eur/TEU	22100
Indirect loss	x 1.5	
	Total loss of income	33100

Table 38 Downtime costs per hour

6.8.6 Damage costs in the protected area

Damage costs in the protected area start when overtopping discharge is higher than 2l/s per meter of breakwater. The loses expected during that storm are not quite big because is an amount of water relatively small, although it might be enough to provide some damage in lights and other stuff. These costs are estimated at 200000 Euros.

However if the overtopping discharge is bigger than 10l/s per meter of breakwater the damage considered is 20000000 Euro in order to repair the damage caused in the inner slope revetment and also on the road.

Overtopping (l/s per m)	Type of damage	Quantification (Euros)
0,2	no damage	0
2	minor damge(lights,port equipement,…)	200000
10	severe damage(inner slope revetment,road,ligths,…)	20000000

Table 39 Damage costs in the protected area

6.9 Results

6.9.1 Element volume

The influences of the element volume and crest height on the costs are analysed. First, the influence of the variation of the element volume is determined and secondly the influence of the crest height.

The element weight is varied for the optimal crest height of 17m + NAP and the results are shown in figure 24.

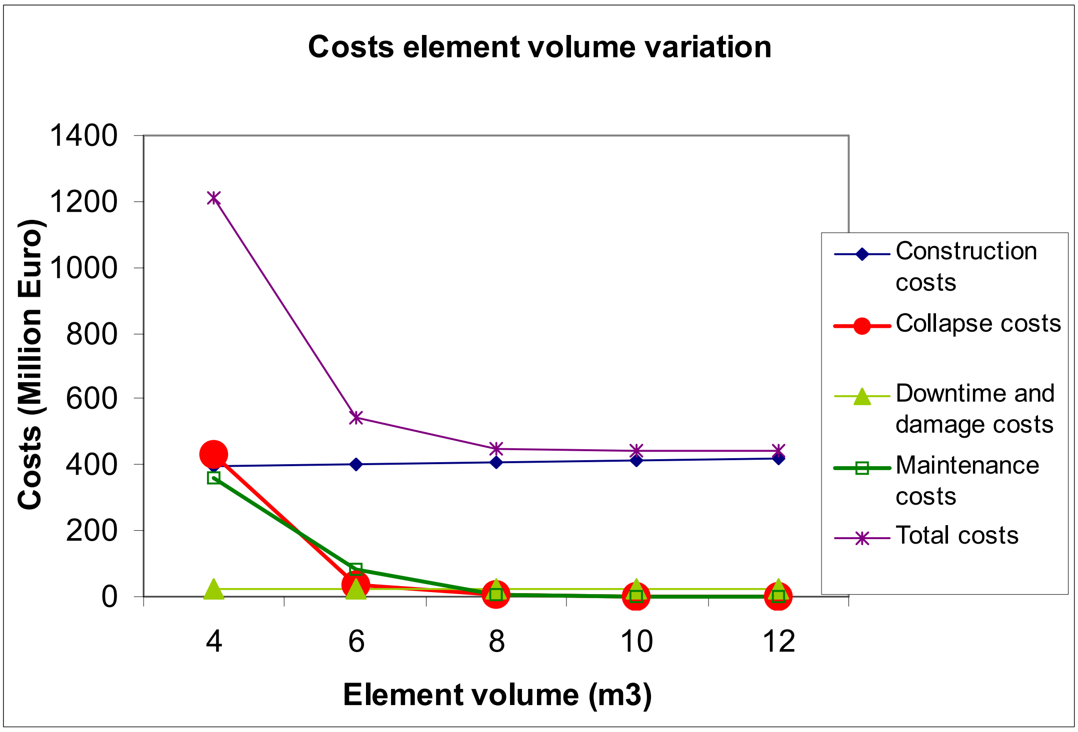

Figure 24 Costs for element volume variation

The increase of the construction costs, with the variation of the element volume, is not too sensitive. Is because of this, that the optimal probabilistic design differs to the deterministic design. By increasing the element volume from an element volume of 4 m3, the risk of collapse and maintenance decrease faster than the construction costs increase, arriving up to the element volume of 10 m3. In that point is found the optimum volume. By increasing the element volume the total costs grow, but very slight. It is also seen that when the element volume is larger than 8 m3 the construction is almost maintenance free.

6.9.2 Crest height

For the optimum element volume the crest height is varied and the costs are assessed. The following graph shows the results:

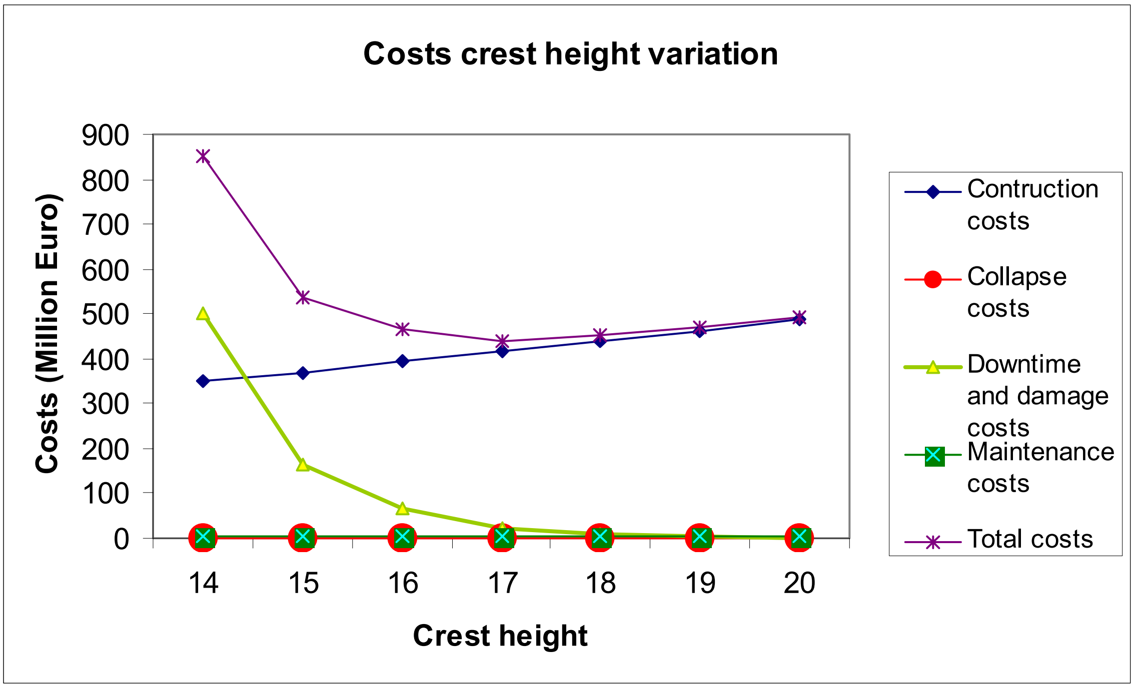

Figure 25 Costs for crest height variation

It is shown that the maintenance and risk of collapse costs are very low compared with the construction costs, and downtime and damage costs for the lowest crest heights. Downtime and damge costs in the protected area decreases rapidly from 14 meters crest height until 17m, but then the decrease is lighter. Total costs over the lifetime decreases rapidly until the minimum, but after that point the increase is not so quick.

6.9.3 Economic optimal design

The total discounted costs over the lifetime are calculated for all the alternatives and, thus most economical option is ascertained. A crest height of NAP+17 m and an element weight of 28 tons (volume of 10 m3) produce the minimum costs over the lifetime.

The following graph shows the total costs over the lifetime for all the breakwater alternatives.

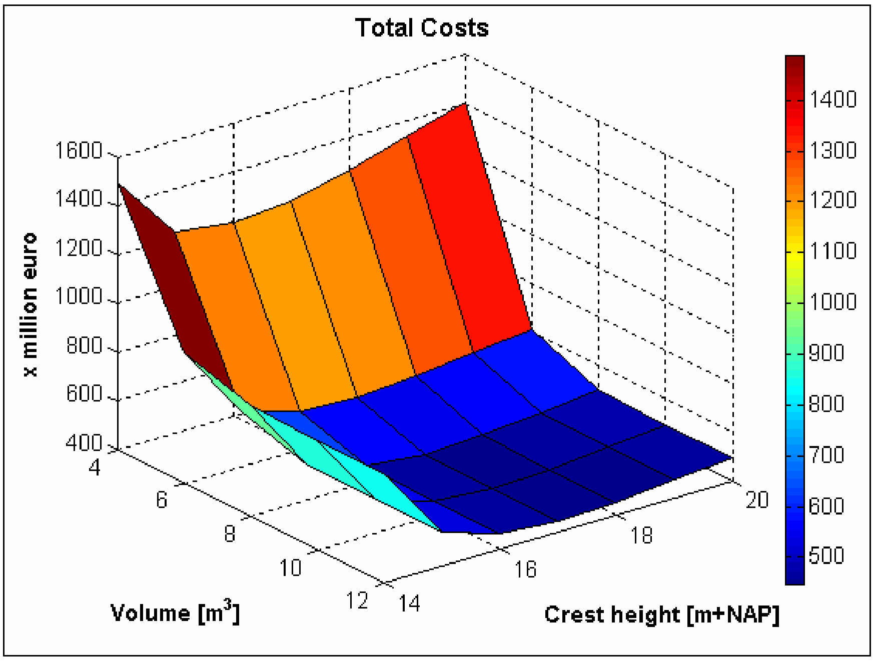

Figure 21 Total costs over the lifetime I

By making cross-sections of the 3D graph for each element volume, the point with the minimum total costs appears:

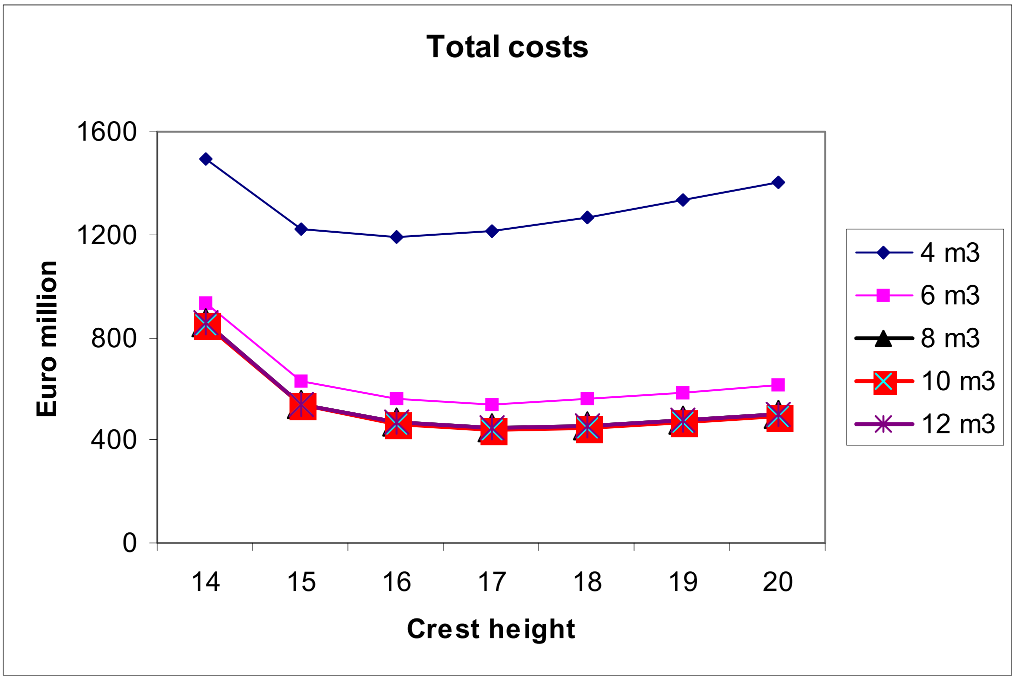

Figure 22 Total costs over the lifetime II

It is shown that for each element weight the total costs increase for low crest heights and for high crest heights. Low crest heights, produces high downtime and damage costs in the protected area. High crest heights, produce high construction costs. The bigger the element volume, the less risk of collapse and the less maintenance costs because the probability of occurrence is lower. When varying the crest heights for each element volume, it is observed that the total costs are the lowest for a crest height of NAP+17 m, except for the element volume 4 m3. This is so because repair of collapse costs and maintenance costs of the breakwater has high influence in the total costs and therefore makes a displacement in the curve to the left, producing a decrease of the crest height.
For element volumes of 8, 10 and 12, the total costs do not vary very much.
Table 40 shows the prices in million euros for all the alternatives:

	crest height (m + NAP)						
element volume (m3)	14m	15m	16m	17m	18m	19m	20m
4 m3	1497.6	1223.8	1193.1	1213.8	1270.1	1335.1	1406.1
6 m3	935.9	626.4	558.8	541.6	558.8	583.7	613.4
8 m3	857.2	542.6	469.8	447.1	458.8	477.9	501.8
10 m3	851.9	536.9	463.6	440.5	451.6	470.3	493.7
12 m3	855.3	540.3	467.2	444.1	455.4	474.2	4976

Table 40 Total costs over the lifetime in million Euros

For the most economical design, the distribution of the costs, in percentage, are shown in the figure 23:

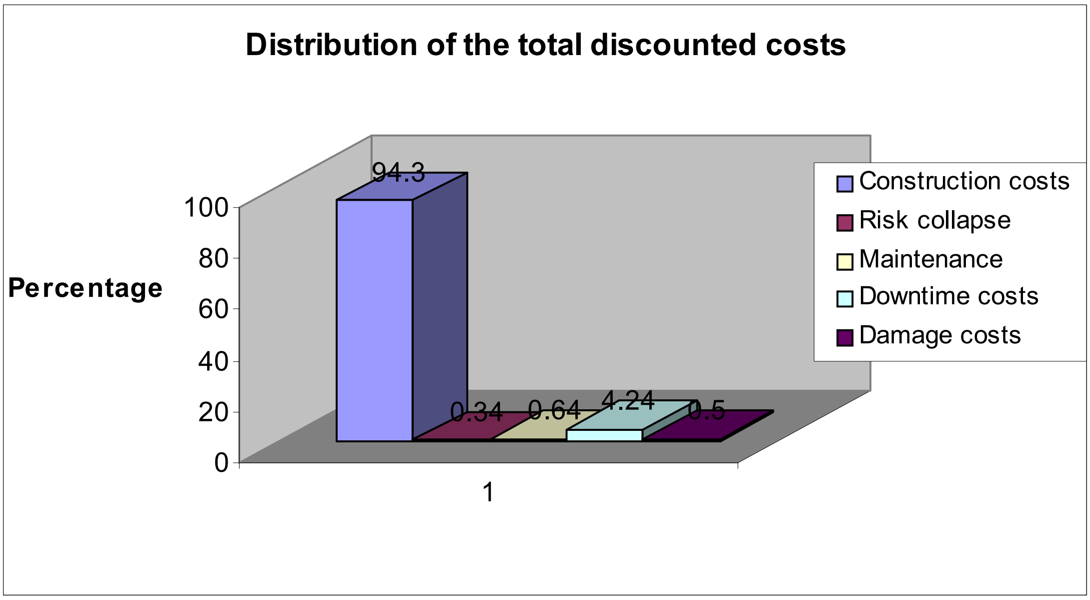

Figure 23 Distribution of the total discounted costs

For this option it is shown that the ninety-four percent of the costs are construction costs. The capitalized risk of collapse is the lowest percentage. Damage costs and maintenance, are in the same order of magnitude as the risk of collapse. Finaly, the downtime costs are about 5 %. This increase of downtime costs allows to reduce the crest height up to seventeen meters.

6.9.4 Discount rate

The discount rate was assumed constant as 5% over the lifetime. The effects of a discount rate varying from 0% to 10% are also evaluated.

Discount rate 0%

With a discount level of 0%, the discounted total costs over the lifetime increase, because the present value of future costs increases. The most economic alternative has a crest height of NAP+18 m and the same element volume(10 m3) as taking into account a 5 % of discount rate. But in this case the total costs of the breakwater with an element volume of 12 m3 is closer (there is less difference) than the one of 8 m3.

	Crest heigth (m + NAP)						
element volume (m3)	14m	15m	16m	17m	18m	19m	20m
4 m3	3523	2737	2615	2633	2747	2884	3037
6 m3	1974	1089	866	780	786	813	852
8 m3	1749	850	612	511	502	513	535
10 m3	1727	826	587	484	474	483	503
12 m3	1729	829	589	486	476	485	506

Table 41 Total costs million Euro (discount rate 0%)

Discount rate 10%

The discount rate of 10% decreases the present value of the discounted collapse and downtime costs. A breakwater with a crest height of 17 m +CD and an element weight of 10 m3 is the economic optimal design. Variation in the discount rate from 5% to 10% almost does not produce a variation in the breakwater geometry, nor element volume variation (or almost). But in this case total costs of an element volume of 8 m3 is closer to the most economical option than the one with 12 m3.

	Crest height (m + NAP)						
element volume (m3)	14m	15m	16m	17m	18m	19m	20m
4 m3	964	825	818	839	880	926	976
6 m3	662	504	477	478	498	523	550
8 m3	622	461	432	430	447	468	492
10 m3	621	460	431	428	445	467	491
12 m3	625	464	434	433	450	471	495

Table 42 Total costs million Euro (discount rate 10%)

7 Conclusions

7.1 Construction method

Normaly, the geometry can be reduced by means of the construction method because water-based method does not implicate a minimum crest height of the core.
In that case, the construction method does not provide almost reduction in the breakwater geometry, because the final design has a very high crest height. The construction costs could be reduced by means of the best combination of water-based and land based operations.

7.2 Discount rate

Higher discount rates leads to smaller geometry and lower discount rates to a stronger breakwater. Anyway, in this case, the discount rate does not influence very much in the economic optimal alternative of the breakwater.

7.3 Crest height

From the deterministic design the crest height obtained was 18 m+NAP, and was not taken into account the economic consecuences during the lifetime, but the safety level taken into account was too high. From the probabilistic design, the crest height derived for the most economical option is 17 m+NAP. Almost no reduction of the crest height is reached. Also if a sea level rise is taken into account during the lifetime of the breakwater, it could be better aproximation the crest height of the deterministic design, depending on the sea level rise taken into account.
It has to be taken into account that the estimation of the downtime costs and losses are merely approximative. And in case of a growth higher than the expected the most economical option could has bigger crest heights.

7.4 Element weight

In the case of the element weight, the deterministic calculations gave an element weight of 18.8 tons, and the probabilistic design gives an element weight of 28 tons. Taking into account the uncertainties (when calculating probabilities of failure), the costs of repair in case of collapse as well as the frequency of maintenance required, the most economical option for the lifetime of the breakwater is 10 tons bigger. When analising the total costs over the lifetime the difference between blocs of 8 m3 and 10 m3 was not differ too much. Contractors in this case would preffer elements of 8 tons because they are handy. Depending on the real costs, placement rates, and costs of the cranes, the optimal solution could change.

Deterioration was no taken into account and this fact, reduce the weight of the armour elements, increasing the probability of failure.

7.5 Optimum breakwater geometry

The economic optimal design of the breakwater has a crest height of NAP+17m and an element volume of 10 m3 (28 tons). The total discounted costs over the lifetime are 440.5 million Euros of which 415.3 million Euro are the initial construction costs. The economical optimum design it is almost maintenance free.
When is varied the element size and crest height around the optimal geometry does not give a substantial increase of the total costs. Change in the total construction costs, due to a variation in element size or a variation in crest height, are small. The contribution of the toe structure to the total probability of collapse is not dependent on the element size. The influence on the probability of collapse of the armour layer is therefore limited. The decrease of the failure probabilities, are therefore not substantial.

8 Recommendations

Shallow water waves

The translation of deep water wave height to shallow water wave height is based on a 2-dimensional simplification. Also when calculations were carried out, a short bathymetry near the coast was taken into account, producing very high wave heights. With the whole bathymetry from the Europlatform in a 3-dimensional model, which is more realistic could provide different results. Further investigation is recommended.

Cost quantification

The costs of the construction behind the breakwater are simplified to a road and some port stuff. Detailed study of the costs behind the breakwater could provide more realistic results in the damage costs. This fact could provide another breakwater geometry.
When downtimes are taken into account in the protected area, the flow of goods are assumed constant and based in an economical study made until 25 years of the lifetime. Changes in fow of goods produces changes in downtime costs and therefore in the total costs of the breakwater changing the optimal geometry.

Failure mechanisms

The failure due to collapse was simplified to two failure mechanisms. Other mechanisms should be evaluated in order to determine a more accurate probability of failure.

Lifetime

The breakwater has been designed for a lifetime of 50 years. Consequences for longer design should be investigated. Deterioration of the breakwater increase the probability of failure. Also the sea level rise increase the risk of downtime and is not taken into account. Study of the sea level rise should be investigated to derive the final crest height.

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INTERNET:

www.golfklimaat.nl

www.getij.nl

www.portofrotterdam.com

10 APPENDICES

10.1 Appendix I. Design wave height

The Peak over Threshold method has been used in order to determine the design storm. The data from the whole years (1979-2001) has been downloaded from www.golfklimaat.nl. A threshold of 1.5 m has been selected. The peaks observed of the storms, over the threshold, has been classified in wave height bins according to the maximum Hss of each storm. Then this data has been adjusted to a function distribution. The extreme value distributions Weibull and Gumbel have been checked in order to see which one has the best correlation with the database.
In order to make a regression analysis the Weibull and Gumbel distribution has been changed in a linear way. That step is shown bellow.

Gumbel distribution: $P = \exp\left[-\exp\left(-\frac{H_{SS}-\gamma}{\beta}\right)\right]$

Where β and γ are the parameters of the function that are found by regression analysis, P is the probability of non-exceedance and Hss is the storm wave height.
Taking two times the log, the expression has the following aspect:

$-\ln(-\ln P)=1/\beta*Hss-\gamma/\beta$

The left-hand side of the equation is called reduced variate G:

$G=-\ln(\ln(1/P))$

And in the other side $A=1/\beta$ and $B=-\gamma/\beta$

Now the regression analysis can be made.

In a similar way the Weibull distribution fitted to make the regression analysis:

Weibull distribution: $Q = \exp\left[-\left\{\frac{H_{ss}-\gamma}{\beta}\right\}^{\alpha}\right]$

Taking the log and rewriting the reduced Weibull variate is:

$W = -(\ln Q)^{1/\alpha} = 1/\beta*Hss-\gamma/\beta$

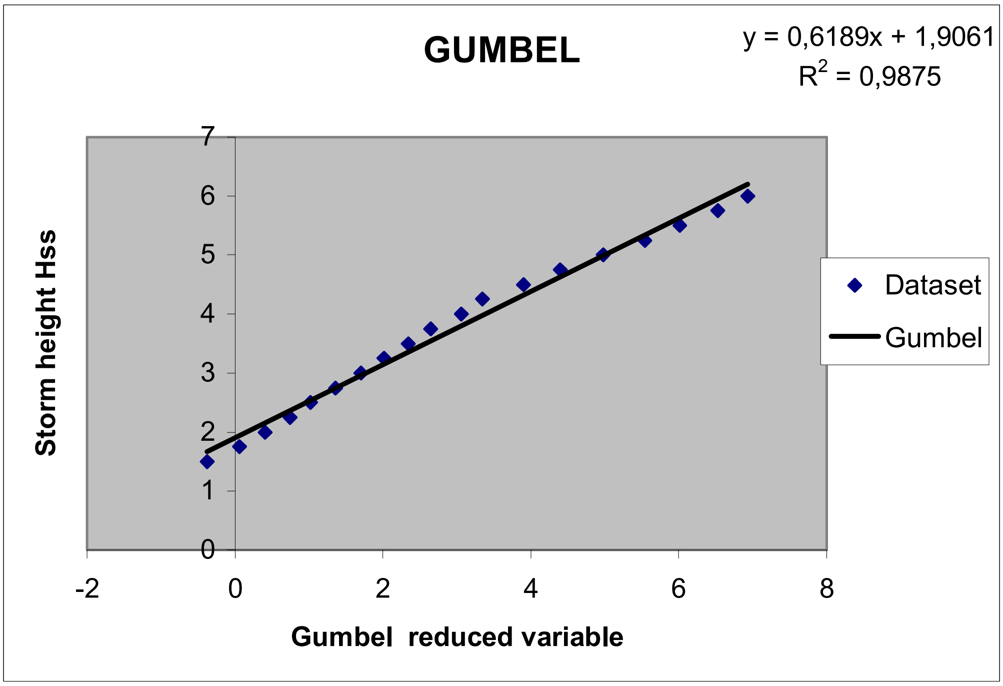
As in the Gumbel distribution A=1/β and B=-γ/β

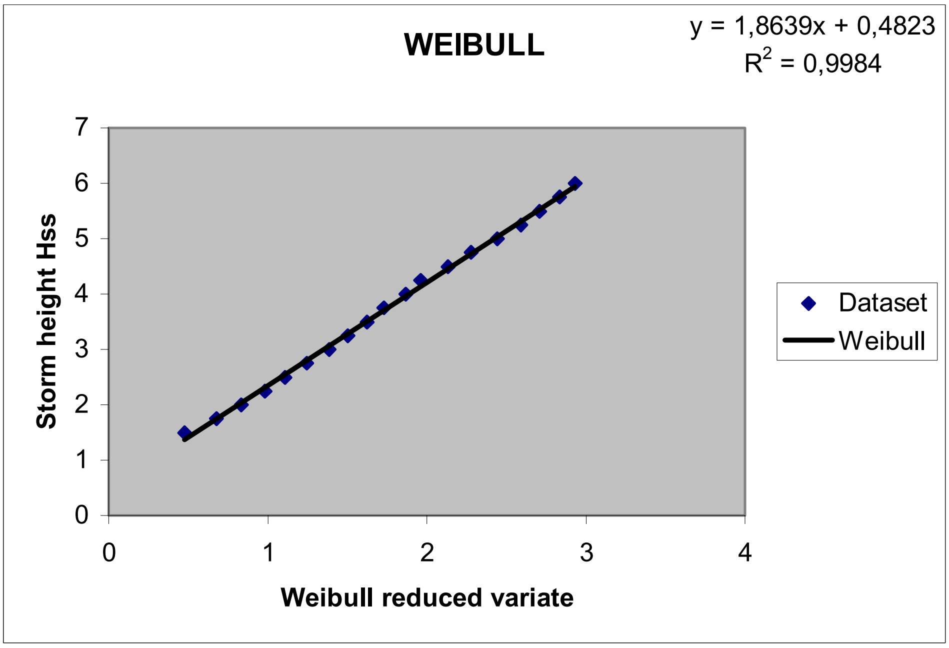
In the Weibull distribution there are three variables, so the determination of the variable α has been determined in a iterative way. The value of α, which gives the best correlation, has been selected and its value is α=1.8.

The following table shows the storms, the probability of exceedance and non-exceedance, and the reduced variates of the weibull and gumbel distributions:

wave height	n.of storms	Cumulatve	P	Q	Qs(Q*Nstorms(=93)/year)	ln(Qs)	G	W (α=1.8)
1.5-1,75	475	475	0.2323	0.7677	71.39853301	4.2682773	-0.3783	0.47749
1.75-2	325	800	0.3912	0.6088	56.61858191	4.0363372	0.06343	0.67756
2-2.25	250	1050	0.5134	0.4866	45.24938875	3.8121892	0.40555	0.83345
2.25-2.75	221	1271	0.6215	0.3785	35.199022	3.5610183	0.74319	0.98411
2.5-2.75	157	1428	0.6983	0.3017	28.0591687	3.3343154	1.02409	1.10572
2.75-3	153	1581	0.7731	0.2269	21.10122249	3.049331	1.35736	1.24486
3-3.25	125	1706	0.8342	0.1658	15.41662592	2.7354465	1.7079	1.38496
3.25-3.5	83	1789	0.8748	0.1252	11.64205379	2.4546239	2.01185	1.5013
3.5-3.75	69	1858	0.9086	0.0914	8.504156479	2.140555	2.34448	1.62341
3.75-4	47	1905	0.9315	0.0685	6.366748166	1.8510888	2.64626	1.72977
4-4.25	46	1951	0.954	0.046	4.274816626	1.4527412	3.05642	1.86813
4.25-4.5	23	1974	0.9653	0.0347	3.228850856	1.1721263	3.34286	1.96085
4.5-4.75	30	2004	0.98	0.02	1.864547677	0.6230185	3.89947	2.13285
4.75-5	16	2020	0.9878	0.0122	1.136919315	0.1283222	4.39813	2.27881
5-5.25	11	2031	0.9932	0.0068	0.636674817	-0.4514962	4.98066	2.44089
5.25-5.5	6	2037	0.9961	0.0039	0.363814181	-1.011112	5.54175	2.58954
5.5-5.75	3	2040	0.9976	0.0024	0.227383863	-1.4811157	6.01249	2.7093
5.75-6	2	2042	0.9985	0.0015	0.136430318	-1.9919413	6.52381	2.83483
6-6.25	1	2043	0.999	0.001	0.090953545	-2.3974064	6.92952	2.93139
6.25-6.5	2	2045	1	0	0			

The graphs of the regression analysis are also shown below:





It is shown that the weibull distribution has the best correlation. For this reason is the distribution selected.
Making the regression analysis the values of β=1.867 and γ=0.477 are derived.

In order to transform the excedance in a probability per year the following equatiation is used for the weibull distribution:

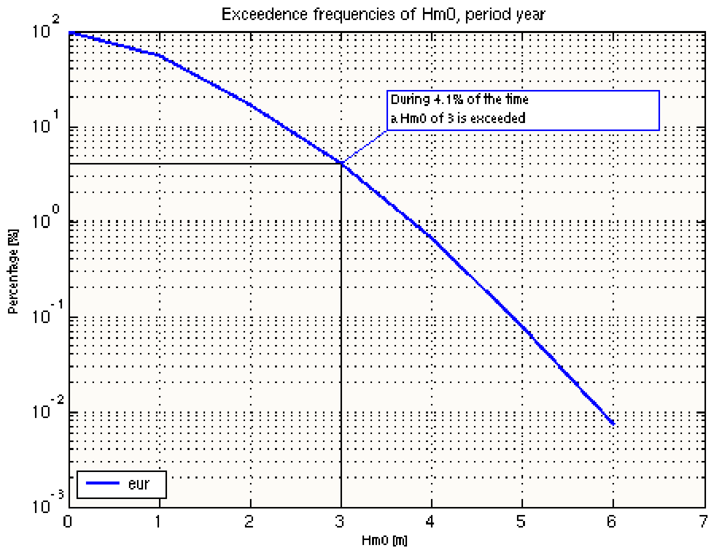
$$H_{SS} = \gamma + \beta \left\{ -\ln\left(\frac{Q_S}{N_S} \right) \right\}^{1/\alpha}$$

Where Q_S is the probability per year:

1/year Hss=4.79m
1/10 years Hss=5.9m
1/100 years Hss=6.86m
1/1000years Hss=7.7m
1/10000years Hss=8.4m

It is observed that the values are quite similar to the values given by the port of Rotterdam report.

The distribution of the wave height Hmo in a period of a year is given by www.golfklimaat.nl :



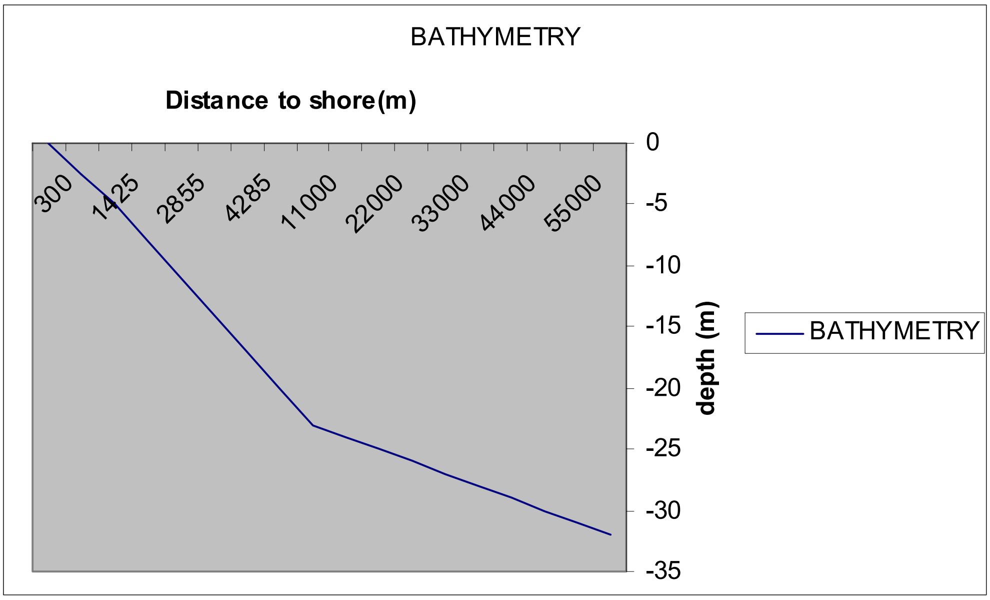
The distribution is used to determine the workability and downtimes in costruction operations.

10.2 Appendix II .Wave height near-shore

Waves, has to be translated from deep water to shallow water. When the disipation of the energy of the waves was calculated, the bathimetry taken into account was quite short just arriving at 5 km away from the coast. This fact influences all the results because almost all the design of the breakwater depends on the wave height. So the wave height is overestimated. Anyway, taking into account the simplification of 2-dimensinal model the results are different from a 3-dimansional model, which could represents better the reality.
The wave propagating calculations are performed by the wave propagation method with shoaling / refraction calculation Coastal and River Engineering Support System (CRESS) based on the Battjes and Janssen (1984) approach.

Hs,shore	Hs,offshore	frequency /year	Hs,shore	Hs,offshore	frequency /year
			4,51	5,36	0,33
8,7	10,35	0,00000016	4,4	5,23	0,43
8,4	10	0,0000005	4,35	5,17	0,55
8,35	9,94	0,0000006	4,25	5,05	0,66
8,2	9,76	0,0000017	4,2	5	0,66
7,8	9,28	0,0000067	4	4,76	1,05
7,7	9,16	0,0000125	3,85	4,58	1,24
7,3	8,69	0,00005	3,6	4,28	2,45
7,1	8,45	0,00011	3,58	4,26	2,45
6,9	8,21	0,00023	3,5	4,16	3,3
6,5	7,73	0,00091	3,3	3,92	4,54
6,45	7,67	0,0011	3,2	3,80	5,5
6,25	7,44	0,0022	2,85	3,39	10
6	7,14	0,0047	2,75	3,27	11,76
5,98	7,11	0,0052	2,7	3,21	12,5
5,75	6,84	0,011	2,5	2,97	18,18
5,65	6,72	0,014	2,25	2,67	22,2
5,45	6,48	0,025	2,2	2,61	26,3
5,25	6,25	0,045	2,13	2,53	28,5
5,25	6,25	0,045	1,78	2,11	41,6
5	5,95	0,091	1,76	2,09	43
4,95	5,89	0,125	1,4	1,66	90

Considering the following bathimetry the reduction of the wave height is much more sensitive. But has to be said, that anyway is a two dimensional model and a three dimensional simulation could vary also the results.



Hs	Hs,shore	Hs	Hs,shore
1	1	5,75	3,96
1,25	1,24	6	4,02
1,5	1,48	6,25	4,11
1,75	1,7	6,5	4,2
2	1,91	6,75	4,27
2,25	2,11	7	4,34
2,5	2,31	7,25	4,4
2,75	2,48	7,5	4,49
3	2,65	7,75	4,56
3,25	2,81	8	4,61
3,5	2,95	8,25	4,68
3,75	3,09	8,5	4,75
4	3,21	8,75	4,81
4,25	3,33	9	4,86
4,5	3,46	9,25	4,9
4,75	3,55	9,5	4,95
5	3,66	9,75	5
5,25	3,75	10	5,05
5,5	3,86		

10.3 Apendix III. Wave period

Wave period is calculated with the relations given by the Port of Rotterdam report (Doc.nr.:AA-02-330)

Tm=3,14*(Hmo)^(1/2)

For: $H_{m0} / d < 0.14$ $T_p / T_m = 1.33$

For: $0.14 < H_{m0} / d < 0.35$ $T_p / T_m = 0.5 * H_{m0} / d + 1.26$

Where:

Hmo=significant wave height
d=depth(at the Europlatform is 32 m)
Tp=peak period
Tm=mean period

Hs	Tm	Tp	Hs	Tm	Tp
			5,5	7,36	9,89
1	3,14	4	5,75	7,53	10,1
1,25	3,51	4,47	6	7,69	10,5
1,5	3,84	4,9	6,25	7,85	10,7
1,75	4,15	5,36	6,5	8	10,9
2	4,44	5,73	6,75	8,16	11,3
2,25	4,71	6,08	7	8,31	11,5
2,5	4,96	6,41	7,25	8,45	11,7
2,75	5,21	6,81	7,5	8,6	11,9
3	5,44	7,11	7,75	8,74	12,1
3,25	5,66	7,4	8	8,88	12,5
3,5	5,87	7,68	8,25	9,02	12,7
3,75	6,08	8,06	8,5	9,15	12,8
4	6,28	8,32	8,75	9,29	13
4,25	6,47	8,58	9	9,42	13,2
4,5	6,66	8,83	9,25	9,55	13,6
4,75	6,84	9,19	9,5	9,68	13,8
5	7,02	9,43	9,75	9,8	14
5,25	7,19	9,66	10	9,93	14,1

10.4 Appendix IV. Computer program VaP.

Function and Purpose of *VaP*

The **Va**riables **P**rocessor (***VaP***) enables the user of the program to deal with stochastic quantities, so-called variables, in some given mathematical expression. In view of one of the applications of the program, this expression is called a limit state function (LSF). The program lends itself to reliability analysis, but may be used in a much wider context when evaluating the influence of variables for problems encountered in other fields of engineering practice.

At first, the limit state function G(**X**) representing the problem at hand is defined using the usual mathematical notation and concrete terms for the basic variables **X**. The variables then have to be described by choosing among a set of several distribution types. The results can be produced as a probability of failure, as the first four moments, or as a histogram of the resulting stochastic quantity G.

Different methods of analysis are implemented in *VaP*. *VaP* calculates the moments E[G(**X**)n] following procedures proposed by Evans, the probability of failure pf=P[G(**X**)≤0] using the well known FORM procedures and is able to produce a histogram of G(X) based on Crude Monte Carlo techniques [Rubinstein, 1981].
FORM sometimes has difficulties with user defined variables, due to the particular shape of the corresponding histograms. Anyway in this study (Level III approach) Monte Carlo analisys is used.

The results displayed in the window are the first four moments, the probability of failure and a graphical representation of the results as a histogram.

In order to introduce the parameters of the function distributions, it must be taken into account the relations shown in the next tables:

Distribution type	Para-meters	Moments
Deterministic	$1: m$	
Rectangular $f_X(x) = \frac{1}{b-a}$ $a \le x \le b,\ a \ne b$	$1: a$ $2: b$	$m = \frac{a+b}{2}$ $s = \frac{b-a}{\sqrt{12}}$
Normal $f_X(x) = \frac{1}{s\sqrt{2\pi}} \cdot \exp\left(-\frac{1}{2}\left(\frac{x-m}{s}\right)^2\right)$ $s > 0,\ -\infty < x < +\infty$	$1: m$ $2: s$	
Lognormal $f_X(x) = \frac{1}{\zeta x\sqrt{2\pi}} \cdot \exp\left(-\frac{1}{2}\left(\frac{\ln x - \lambda}{\zeta}\right)^2\right)$ $\zeta > 0,\ 0 < x < \infty$	$1: \lambda$ $2: \zeta$	$m = \exp\left(\lambda + \frac{\zeta^2}{2}\right)$ $s = \exp\left(\lambda + \frac{\zeta^2}{2}\right) \cdot \sqrt{\exp\left(\zeta^2\right) - 1}$
sLognormal $f_X(x) = \frac{1}{\zeta(x-\varepsilon)\sqrt{2\pi}} \cdot \exp\left(-\frac{1}{2}\left(\frac{\ln(x-\varepsilon) - \lambda}{\zeta}\right)^2\right)$ $\zeta > 0,\ \varepsilon < x < \infty$	$1: \lambda$ $2: \zeta$ $3: \varepsilon$	$m = \varepsilon + \exp\left(\lambda + \frac{\zeta^2}{2}\right)$ $s = \exp\left(\lambda + \frac{\zeta^2}{2}\right) \cdot \sqrt{\exp\left(\zeta^2\right) - 1}$
sExponential $f_X(x) = \lambda \exp(-\lambda(x-\varepsilon))$ $\lambda > 0,\ \varepsilon \le x < \infty$	$1: \varepsilon$ $2: \lambda$	$m = \varepsilon + \frac{1}{\lambda}$ $s = \frac{1}{\lambda}$
Gamma $f_X(x) = \frac{b^p}{\Gamma(p)} \exp(-bx) x^{p-1}$ $b > 0,\ p > 0,\ 0 \le x < \infty$	$1: p$ $2: b$	$m = \frac{p}{b}$ $s = \frac{\sqrt{p}}{b}$

Beta $f_X(x) = \frac{(x-a)^{r-1} \cdot (b-x)^{t-1}}{(b-a)^{r+t-1} \cdot B(r,t)}$ $a \le x \le b$, $a \ne b$, $r,t \ge 1$	1 : a 2 : b 3 : r 4 : t	$m = a + (b-a) \cdot \frac{r}{r+t}$ $s = \frac{b-a}{r+t} \cdot \sqrt{\frac{rt}{r+t+1}}$
Gumbel (Largest) $f_X(x) = \alpha \exp\left(-\alpha(x-u) - \exp\left(-\alpha(x-u)\right)\right)$ $-\infty < x < +\infty$, $\alpha > 0$	1 : u 2 : α	$m = u + \frac{0.577216}{\alpha}$ $s = \frac{\pi}{\alpha\sqrt{6}}$
Frechet (Largest) $f_X(x) = \frac{k}{u-\varepsilon} \cdot \left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k-1} \cdot \exp\left(-\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{-k}\right)$ $\varepsilon \le x < +\infty$, $u,k > 0$	1 : u 2 : k 3 : ε	$m = \varepsilon + (u-\varepsilon)\Gamma\left(1-\frac{1}{k}\right)$ $s = (u-\varepsilon)\sqrt{\Gamma\left(1-\frac{2}{k}\right) - \Gamma^2\left(1-\frac{1}{k}\right)}$
Weibull (Smallest) $f_X(x) = \frac{k}{u-\varepsilon} \cdot \left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{k-1} \cdot \exp\left(-\left(\frac{x-\varepsilon}{u-\varepsilon}\right)^{k}\right)$ $\varepsilon \le x < +\infty$, $u,k > 0$	1 : u 2 : k 3 : ε	$m = \varepsilon + (u-\varepsilon)\Gamma\left(1+\frac{1}{k}\right)$ $s = (u-\varepsilon)\sqrt{\Gamma\left(1+\frac{2}{k}\right) - \Gamma^2\left(1+\frac{1}{k}\right)}$

So, for the distribution of the wave height (Weibull distribution) having α=1.8, β=1.867, γ=0.477 the change becomes:

$$1=u=\beta+\gamma=2.344$$
$$2=k=\alpha=1.8$$
$$3=\varepsilon=\gamma=0.477$$

And for the distribution of the waterlevels(Gumbel distribution), when calculating the probabilities of failure of the toe structure: γ=0.47 and β=0.27

$$1=u=\gamma=0.47$$
$$2=\alpha=1/\beta=3.7$$