Preface

This is the final report of a study of economic optimal design of Core-loc® breakwaters by Pieter-Martijn Flamink. The study was carried out as the final thesis of the study of Civil Engineering at the Faculty Civil Engineering and Geosciences of Delft University of Technology.

I want to thank the members of the examiner committee for their time and criticism. Furthermore, I want to thank my student colleagues at Boskalis Westminster for the support, interesting discussions and the refreshing lunch walks along the river. Special thanks go to my private chauffeurs who made possible -almost accident free- the journeys to and fro Papendrecht. Finally, I owe many thanks to my second readers, Cathelijne Flamink, Maarten Zanen and Stijn Kruijsen, for their effort and positive feedback.

Pieter-Martijn Flamink

Delft, August 2003

Additional note: in this report the patented armour Core-loc® is often mentioned. Although the author put in a lot of effort to comply to the request of the U.S. Army Corps of Engineers, Engineer Research and Development Center, Coastal and Hydraulics Laboratory (CHL) to make use of the proper CHL Core-loc® trademark, it could occur that in this report the ® in Core-loc® is mistakenly missing. In that case the reader is requested to read Core-loc® instead.

Summary

Deterministic design methods are commonly used to determine preliminary breakwater designs. Partial safety factors take into account previous experiences and provide a robust preliminary design. However, local circumstances can prove to differ considerably compared to average design conditions and stochastic variations in breakwater strength parameters are commonly neglected. With new armouring techniques, such as Core-loc® armouring, the uncertainties about the armour strength are relatively large. Design guidelines include a safety factor, but often an additional safety margin is applied in the final design of the armouring to ensure stability. This can result in structure strengths more, or less, than locally required. The economic optimum geometry with the lowest costs is possibly not achieved. These costs consist of the initial construction cost, the collapse damage cost and the economic damage cost due to downtime. To include the damage cost or risk (= failure probability x economic consequence) of breakwater collapse and functional failure, a probabilistic approach can be used to determine the failure probabilities.

In Veracruz, Mexico, the port authority of the Port of Veracruz investigates the feasibility of a large port extension next to the existing port of Veracruz. In the preliminary layout a Core-loc® armoured breakwater is anticipated to provide shelter at a container terminal and quay location.

Deterministic design methods result in an element weight of 18.7t (8.5m^3). Two construction methods are evaluated: a water-based and a land-based construction method, with crest heights of 3m +SWL and 11m +SWL respectively. In this deterministic evaluation the economic consequences of functional failure are not taken into account, but both alternatives fulfilled the harbour tranquillity restrictions by the port authority: a maximum downtime of 5%. The water-based construction method is elected as the best construction method, due to lower construction costs of 110.7 $ million.

The deterministic breakwater design is optimised with a probabilistic method for the most important parameters: the weight of the Core-loc® elements and the breakwater crest height. And a progressive deterioration over time of the strength of the Core-loc® armour is taken into account. The probabilities of collapse and functional failure of the breakwater and the economic consequences of failure are determined for 56 combinations of element weight and crest height. The probability of collapse is composed of two failure mechanisms: the Core-loc® armour and the toe structure. The probabilities of failure and economic consequences are time dependent, due to the sea level rise, the deterioration of the breakwater armour and the economic development of the port over the lifetime of 50 years. Therefore, all alternatives have different probabilities of failure for each year. Discounting all costs to a single year the economic optimal design geometry over the total lifetime proves to have a Core-loc® element weight of 30.8t (14m^3) and a crest height of 7 m +SWL. The construction costs of this geometry are 153 $ million.

A crest height of 7m +SWL complies with an allowable downtime of approximately 0.2%. The downtime costs are of considerable more influence than estimated by the port authority. Also the consequences of a breakwater collapse result in a 65% heavier element weight.

The discounted total costs over the lifetime of the breakwater are 219 $ million for the probabilistic design and 468 $ million for the deterministic design. The collapse costs and downtime costs have a significant influence on the total costs over the lifetime and therefore on the economic optimal geometry of the breakwater. A more robust design than deterministically derived can reduce the total cost over the lifetime by almost 50%.

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List of symbols

Symbol	Definition	Unit
C_{cl}	Stability coefficient for Core-loc®	[-]
C_{cl}	Stability number for Core-loc®	[-]
C_{il}	Interlocking stability contribution	[-]
C_w	Weight stability contribution	[-]
D	Costs of repair and downtime	[$ million]
d	Costs per day of interrupted operation	[$/day]
d_{15B}	Diameter passed by 15% of base material	[m]
d_{15F}	Diameter passed by 15% of filter material	[m]
d_{85B}	Diameter passed by 85% of base material	[m]
D_n	Nominal diameter of elements	[m]
D_{nx}	Sieve diameter passed by x% of the material	[m]
f_{Hs1}	Uncertainty parameter dependent on the wave height	[-]
f_{Hs2}	Uncertainty parameter independent on the wave height	[m]
g	Acceleration of gravity	[m/s^2]
h	Water level	[m]
$H_{allowed}$	Maximum allowable wave height in the port basin	[m]
$H_{entrance}$	Wave height transmitted through the entrance	[m]
h_{max}	Still water level	[m +CD]
$H_{port\ basin}$	Wave height transmitted into the port basin	[m]
H_s	Significant wave height	[m]
$H_{s,\ 0}$	Significant wave height at deep water	[m]
$H_{s,\ basin}$	Significant wave height transmitted into the basin	[m]
$H_{s,\ shore}$	Significant wave height at breakwater depth	[m]
h_t	Depth of the crest of the toe below water level	[m]
$H_{transmission}$	Wave height transmitted through and over the breakwater	[m]
I_0	Construction costs	[$ million]
k	Layer thickness coefficient	[-]
K_D	Damage coefficient in Hudson formula	[-]
K_D	Stability factor	[-]
$K_{entrance}$	Transmission coefficient for the wave intrusion via the port entrance	[-]
$K_{transmission}$	Transmission coefficient for the wave intrusion via the breakwater	[-]
L	Lifetime of the breakwater	[year]
M	Maintenance costs	[$ million]
n	Number of elements in a layer	[-]
N	Number of waves	[-]
N	Percentage of broken legs	[-]
N	Operational days per year	[day]
N_{od}	Damage level	[-]
P	Permeability of breakwater	[-]
$p_{collapse}$	Probability of collapse of the breakwater	[-]
$p_{intranquillity}$	Percentage of time functional failure due to wave transmission occurs	[-]
q	Average overtopping discharge	[l/m/s]
r	Discount rate	[-]
R	Strength	[-]
R_c	Crest height above still water level	[m]
R_{top}	Crest level	[m +CD]
S	Damage level	[-]
S	Load	[-]
SLR	Sea level rise	[m]
t	Layer thickness	[m]

Symbol	Definition	Unit
t	Year in the lifetime of the breakwater	[year]
T_m	Mean wave period	[s]
T_p	Peak wave period	[s]
V_{cl}	Core-loc® element volume	[m^3]
W	Weight of element	[t]
W_0	Initial weight of the complete element at time t=0	[t]
$W_{50,\ primary}$	Characteristic weight of an element of the primary armour	[t]
$W_{50,\ secondary}$	Characteristic weight of an element of the secondary armour	[t]
z_{bed}	Bed level	[m +CD]
z_{surge}	Storm surge	[m]
z_{toe}	Toe crest level	[m +CD]
α	Weight stability at t=0	[-]
β	Interlocking stability at t=0	[-]
γ	Reduction factor for the roughness of the slope	[-]
γ_{br}	Depth breaking coefficient	[-]
γ_r	Energy dissipation coefficient	[-]
Δ	Relative density ($=\rho_s- \rho_w)/ \rho_w$)	[-]
ξ	Iribarren parameter	[-]
ξ_{mc}	Critical value of Iribarren parameter	[-]
ρ_r	Mass density of rock	[t/m^3]
ρ_c	Mass density of concrete	[t/m^3]
ρ_w	Mass density of water	[t/m^3]

1 Introduction

1.1 General

In Veracruz, Mexico, the port authority APIVER of the Port of Veracruz has plans for a major extension to the existing port. The location of Veracruz is given in Figure 1.

Figure 1 Location Veracruz

To provide a sheltered basin area for this extension breakwaters are necessary. The extension will be constructed in phases. The proposed final layout of the extension of the Port of Veracruz is given in Figure 2. APIVER preferred the largest part of the breakwaters to be armoured with Core-loc® elements. This thesis only covers the east-west orientated Core-loc® section of the eastern breakwater. In Figure 3 an example of a breakwater with Core-loc® armour is given and in Appendix VII the principal dimensions of a Core-loc® element are provided.

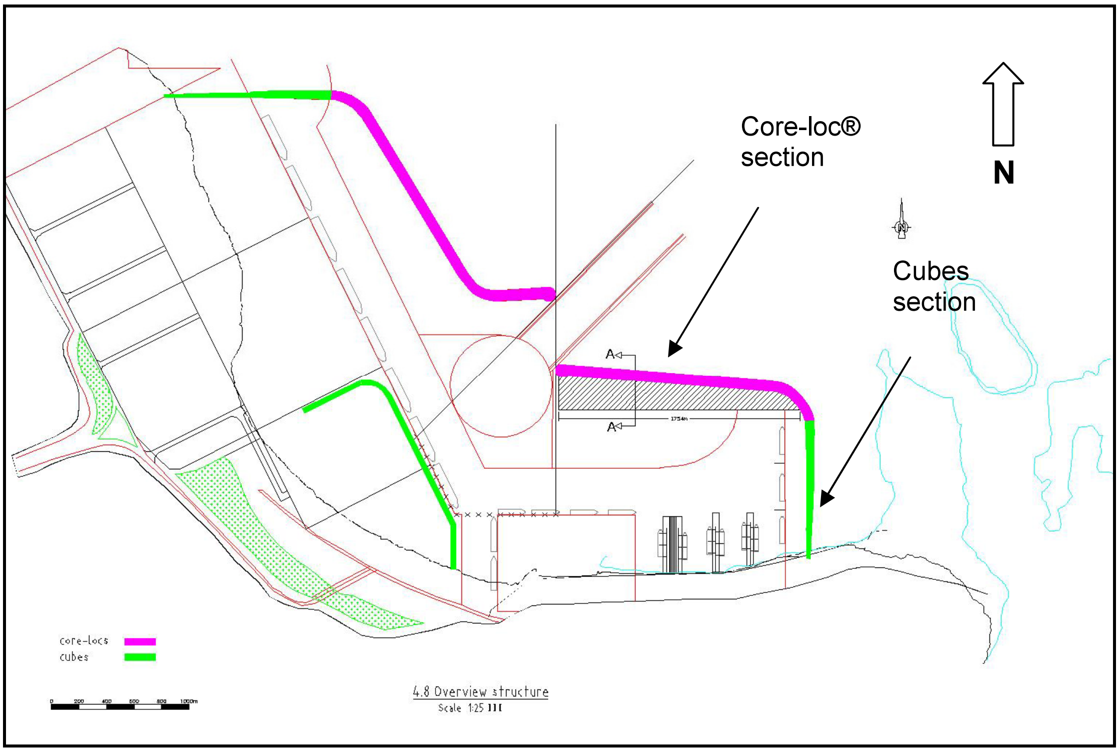

Figure 2 Layout of the proposed extension of the Port of Veracruz

Figure 3 Breakwater armoured with Core-loc® elements

At this moment an economic optimum design is lacking for the planned Core-loc® breakwater and no general approach for the determination of the economic optimum design of the Core-loc® breakwater exists. The amount of wave transmission into the port basin is uncertain. Also knowledge of the damage development, influencing the probability of collapse, is only limited available due to a lack of experience and test data with the Core-loc® units.

1.2 Objective of the study and working method

The total costs over the total lifetime of the breakwater must be minimised to determine the economic optimum design.

To define the economic optimum design, first a deterministic design is made to determine a preliminary design. This design is subsequently optimised for the most important parameters: the weight of the Core-loc® elements and the breakwater crest height. The probabilities of collapse and of functional failure of the breakwater are calculated, taking into account uncertainties. The economic consequences of the two types of failure are quantified and the total cost over the lifetime of the breakwater is derived for several alternatives. Finally, the most economic breakwater alternative is determined.

1.3 Outline of the report

In Chapter 2 the problem analysis is presented. The boundary conditions for the Veracruz port location are discussed in Chapter 3. Next in Chapter 4 the behaviour of Core-loc® is analysed and in Chapter 5 the deterministic design is given. Subsequently, in Chapter 6 the outline and composition of the probabilistic calculation is elaborated. The types of failure for the different limit states are explained in Chapter 7. The probabilities of failure are determined in Chapter 8 and the consequences of the two types of failure are described in Chapter 9. In Chapter 10 the alternative costs are given and evaluated. Finally, the conclusions are provided in Chapter 11 and recommendations are made in Chapter 12.

2 Problem analysis

2.1 Problem description

2.1.1 Limit states

The breakwater can fail to fulfil its sheltering function in two ways. First, the breakwater can collapse and fail to provide shelter. Secondly, the breakwater can stay intact, but is transmitting too much wave energy. The limit for which the breakwater collapses 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 in Figure 4. Important in this figure is that excessive wave height in the basin area can also be caused by waves entering the basin through the entrance. This implicates that the influence of alterations to the breakwater geometry is bounded. It is also shown that the collapse of the breakwater can be caused by several failure mechanisms.

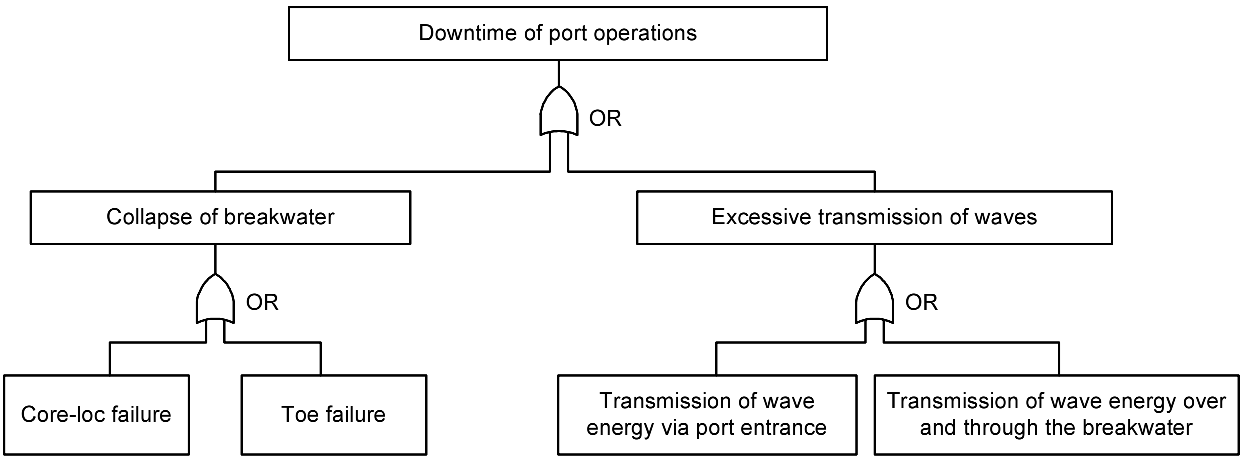

Figure 4 Fault-tree for a breakwater

2.1.2 Economic optimum

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

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

Construction costs

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

__Damage costs of the breakwater__

With traditional armouring the maintenance costs are dependent on the stability and the wave overtopping of the armour layer elements. Core-loc® armouring differs from the traditional rubble mound armouring. Core-loc® elements under wave attack also act as an integral layer, besides showing a reaction solely based on the stability of individual units. The difference between damage wave height and failure wave height is small (Van der Meer, 2002). This is due to the interlocking component of the stability of the Core-loc® elements. Progressive damage results due to a rapidly increasing lack of layer-stability. As damage starts, a fairly sudden failure, compared to a regular rubble mound breakwater, of the whole structure can occur. The difference between progressive damage and the damage development of a rubble mound breakwater is depicted in Figure 5.

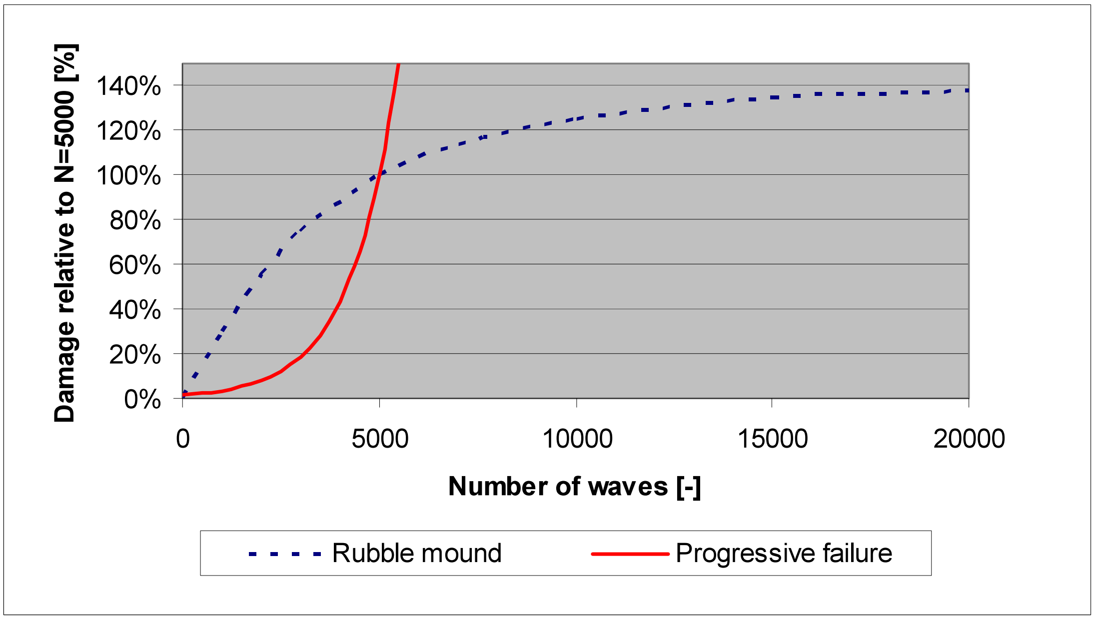

Figure 5 Progressive collapse

__Downtime and damage costs in the protected area__

Downtime and damage costs in the protected area behind the breakwater have two causes:

- Diffraction and refraction around the breakwater head
- Wave transmission through and over the breakwater

Only the second cause is influenced by the breakwater geometry. Therefore, the influence of the breakwater on the damage and downtime costs in the basin area plays a minor role.

2.2 Problem definition

A specific approach for the determination of the economic optimum design of a Core-loc® breakwater does not exist. The wave transmission and its economic effects are uncertain. Also knowledge of the damage development, influencing the probability of collapse and the maintenance and repair, is limited due to a lack of experience and test data with the Core-loc® units.

2.3 Objective

Determination of the economical optimum design process for the Core-loc® breakwater design based on the total costs over the total lifetime including the effects of the established damage and transmission response, taking into account uncertainties. The deterministic breakwater design is optimised for two parameters: the weight of the Core-loc® elements and the breakwater crest height.

3 Boundary conditions

3.1 Bathymetry

The foreshore at Veracruz is schematised in two parts based on data provided by APIVER, the Veracruz port authority (API, 2001). The deepest part, from 38.5 km up to 2.5 km in front of the shore, having a slope of 1/450 and the steeper, shallower part from 2.5 km to the shore having a slope of 1/125. The breakwater is to be positioned 2 km from the shore at a depth of –16 m +CD. More specific data is given in Appendix I.

3.2 Water level

Chart Datum (CD)

All water levels are given relative to Chart Datum level (CD), which is approximately the water level at low water.

Tide

The diurnal tide levels are approximated with a normal distribution with a mean of 0.31 m +CD and a standard deviation of 0.25 m. Additional tide data is provided in Appendix II.

Storm surge

Due to the deep foreshore the increase in water level induced by a storm is negligible. Therefore, a storm surge will not be taken into account in the design. More information is given in Appendix III.

Sea level rise

A linear sea level rise of 0.15 m at the end of the breakwater lifetime is taken into account.

3.3 Waves

To provide realistic wave loads for the breakwater calculations use is made of satellite data available on the internet at the wave data site Argoss. In Appendix IV this source of wave data is elucidated.

Wave direction

The most important wave directions are given in Table 1 and are visualised in Figure 6. In the windrose the distance from the centre indicates the percentage of waves coming from that direction.

Table 1 Wave directions

Direction	% of all waves
NNW	19%
N	30%
NNE	30%
NE	8.5%
Other	12.5%

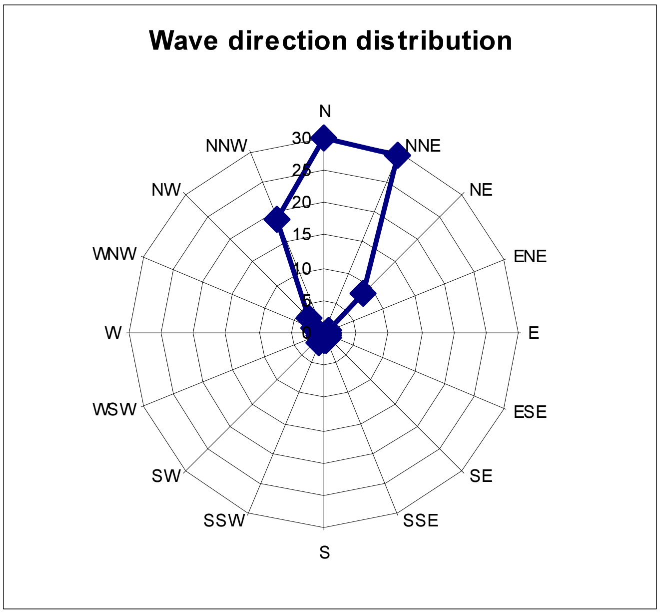

Figure 6 Windrose

The main direction is north. Therefore, wave attack on the east-west orientated breakwater is assumed to be perpendicular.

Deep water waves

The wave load of interest on the breakwater for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) differs. For the functional failure the wave height distribution in percentage of the time are important. The percentage of time in which too much wave energy is transmitted and consequently the breakwater cannot perform its function properly. For the failure due to collapse the highest waves heights are of interest. These waves occur very rarely and a forecast of their occurrence can be made by use of extrapolation.

ULS deep water waves due to 'Nortes'

The largest waves occur during storms coming in directly from the north and are called the 'Nortes'. The deep-water waves can be described with the Gumbel distribution:

$$Gumbel\ probability\ of\ exceedence = Q = 1 - \exp\left[-\exp\left(-\frac{H_s - \gamma}{\beta}\right)\right].$$

The fitted distribution is provided in Figure 7 with the coefficients are given in Table 2. The derivation is given in Appendix V.

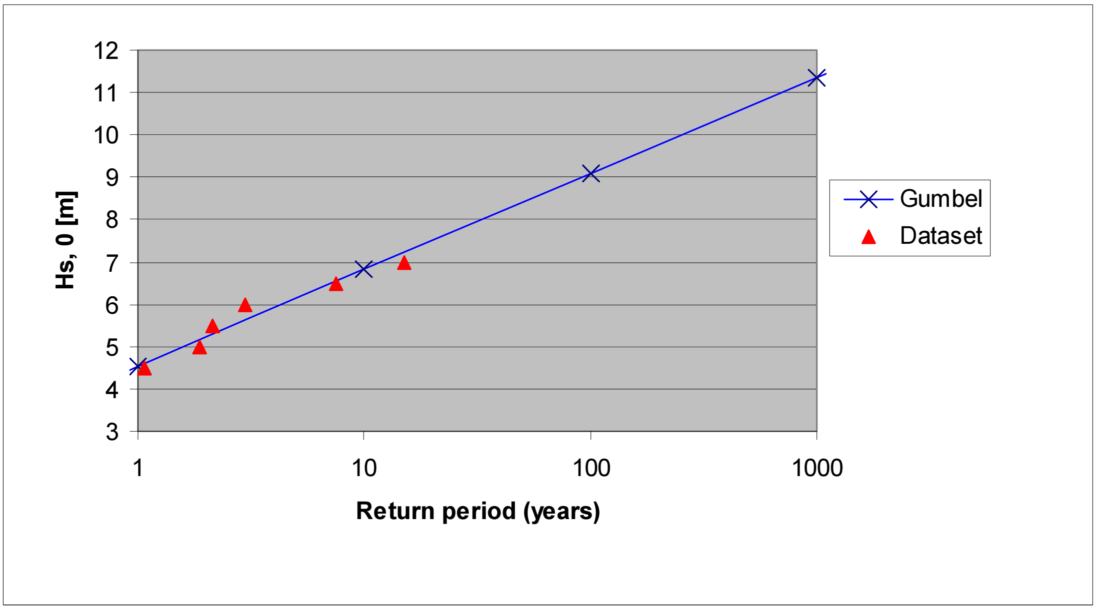

Figure 7 ULS significant wave height exceedence probability

Table 2 Gumbel coefficients

	Gumbel
Correlation with dataset	0.995
Beta	0.979
Gamma	1.544

ULS deep water waves due to hurricanes

Hurricanes, severe tropical storms, can occur in the Gulf of Mexico. In Appendix XVI the probability of occurrence of an intense hurricane is derived to be 0.005 per year. The occurring wave height is established to be depth limited and determined by the storm surge caused by the tropical depression. A storm surge of 3 m is assumed if a hurricane occurs and for probabilistic calculations a standard deviation of 1 m is additionally assumed to take into account the large uncertainty in the assumed storm surge.

SLS deep water waves

The SLS deep water waves can be described with the Weibull distribution:

$$Weibull\ \ probability\ of\ \ exceedence = Q = \exp\left[-\left(\frac{H_s-\gamma}{\beta}\right)^{\alpha}\right].$$

The fitted Weibull distribution for the SLS condition during the year is given in Figure 8 and Table 3. For the derivation see Appendix V. The distribution provides a good fit, especially in the range of 1.5 to 4 m wave heights.

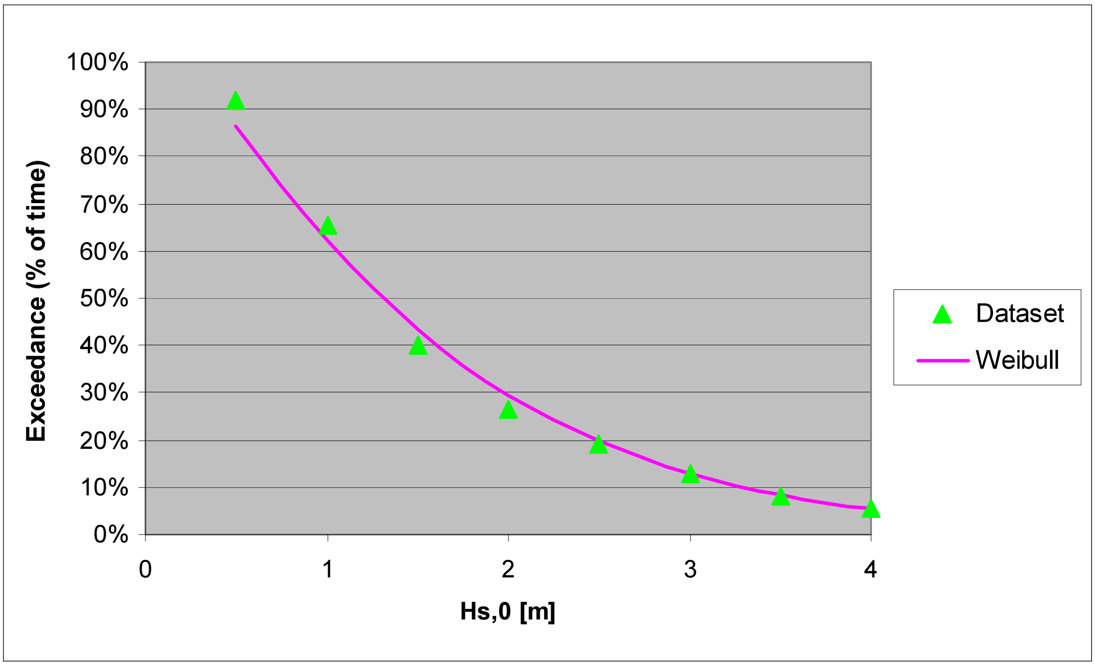

Figure 8 Significant wave height distribution in % of the time

Table 3 Significant wave height distribution in % of the time

Yearly conditions	Weibull
Correlation with dataset	0.997
Alpha	0.9382
Beta	0.8512
Gamma	0.3948

In Table 4 the distributions for the different wave seasons are provided. The wave seasons vary in period of time, this is indicated as well. The average significant wave heights are additionally depicted in Figure 9. There is a clear distinction between the lowest wave season from August till September and the highest wave season from October till January.

Table 4 Seasonal significant wave height distribution

Season	Average Hs [m]	Months	Distribution	Beta	Gamma	Alpha
Feb-Apr	1.67	3	Gumbel	0.6274	1.0624	
May-Jul	1.28	3	Weibull	1.2111	-0.0458	2.3590
Aug-Sep	1.12	2	Gumbel	0.3488	0.6546	
Oct-Jan	1.90	4	Weibull	1.5028	0.2123	1.1561

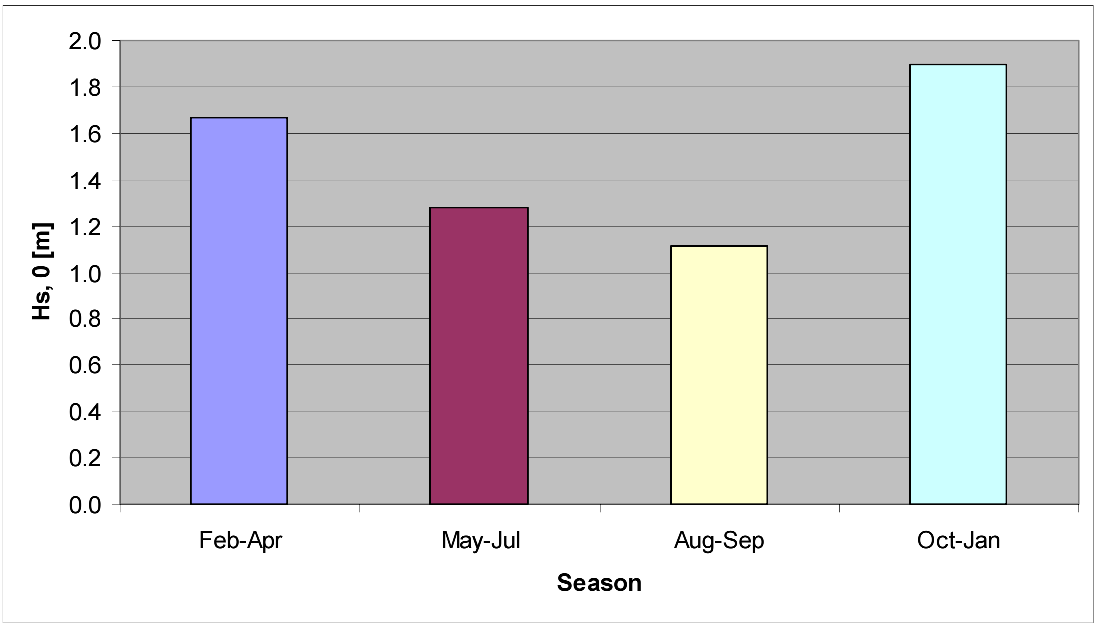

Figure 9 Average significant wave height per wave season

Translation of deep water waves to shallow water waves

Wave propagation calculations are conducted to translate the deep-water waves to shallow waves, taking into account the bathymetry of the breakwater location. Background of the calculation is given in Appendix VI. As an example the wave energy dissipation for a significant wave height of 10 m at deep water is provided in Figure 10. The significant wave height at the breakwater location 2000 m in front of the shoreline is highlighted. At the breakwater location the depth is -16 m +CD.

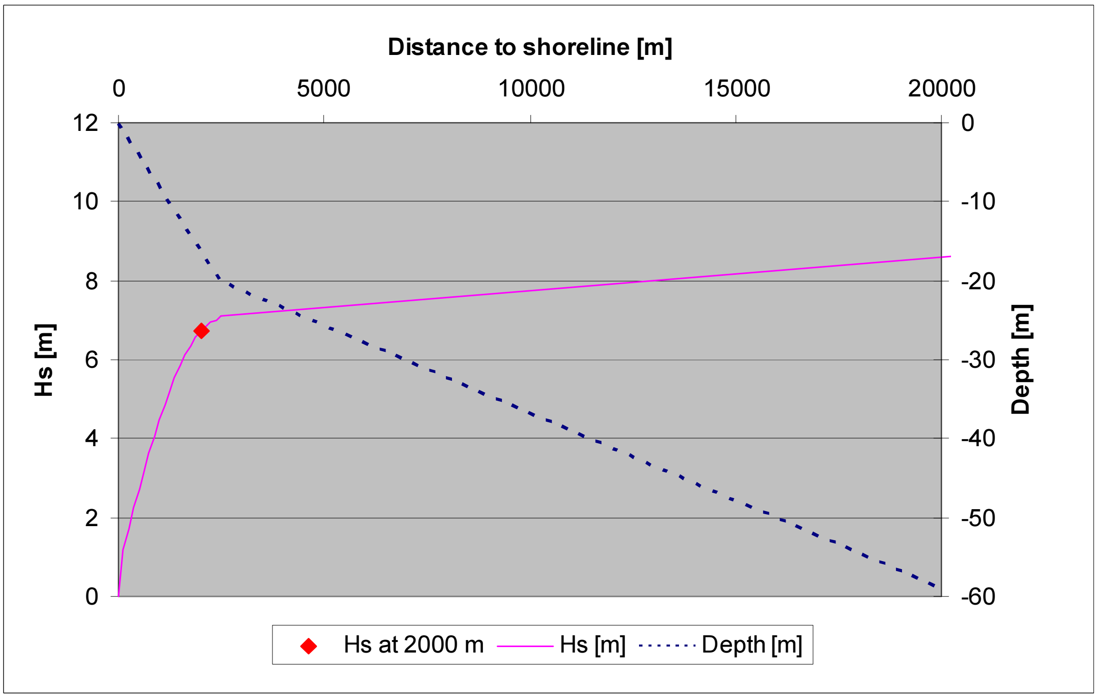

Figure 10 Wave energy dissipation

The results of all calculations at the depth of -16 m +CD are given in Appendix VI and also depicted in Figure 11.

Note: the calculations are based on a heavily simplified two-dimensional bathymetry and further study is recommended.

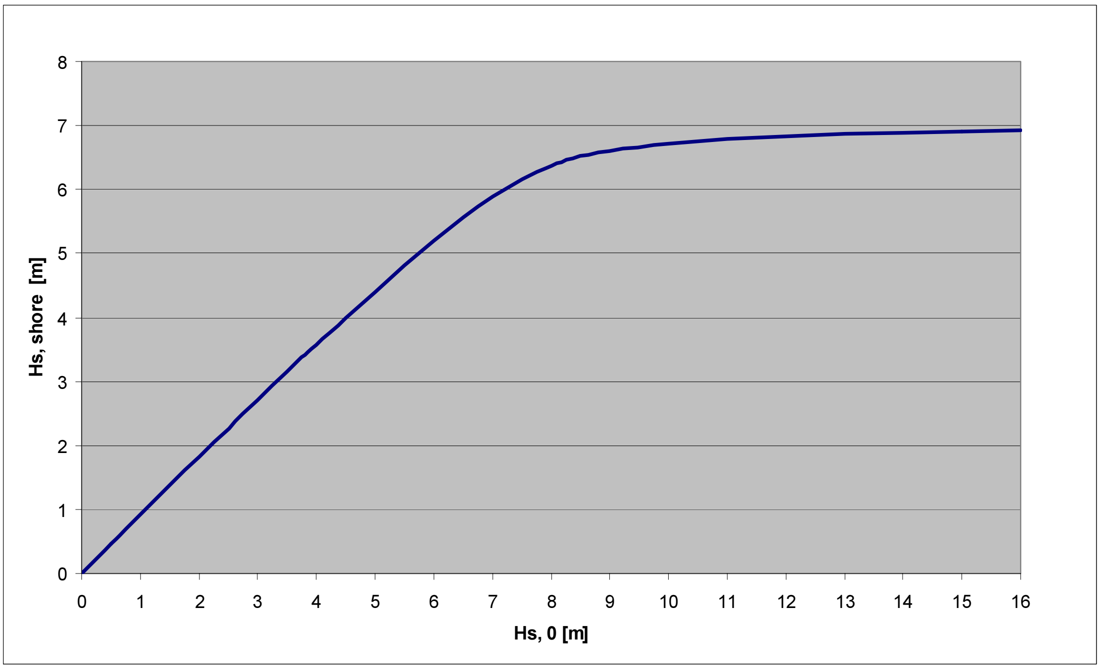

Figure 11 Relation significant deep and shallow wave height at -16 m+CD

Three sections can be distinguished. In the first section about 14% of the deep water wave energy is dissipated. In the intermediate section for medium wave heights a balanced combination of bottom friction and depth limited breaking occurs. In the last section with very high deep-water waves the depth limits the shallow water wave height.

A simple linear schematisation is applied to the transformation, splitting the waves in two types: high and low waves. The low, not depth limited, waves are approximated with a linear equation taking into account the 14% energy dissipation. For the deterministic design the depth-limited waves conversely are approximated with a wave height/ depth ratio of 0.45.

The distinction in these two classes of wave breaking is also useful in the context of ULS and SLS calculations. The not depth limited waves will occur during the SLS situation and the depth limited waves will be of interest during ULS situations. The formulas for the translation of the shallow water wave height at the breakwater location ($H_{s,\ shore}$) from the deep-water wave height ($H_{s,\ 0}$) are provided in Table 5. The resulting deep and shallow water wave height relation is given in Figure 12.

Table 5 Wave translation formulas

Limit state	Depth limited	$H_{s,shore}$
ULS	Yes	0.45*water depth
SLS	No	0.86*$H_{s,\ 0}$

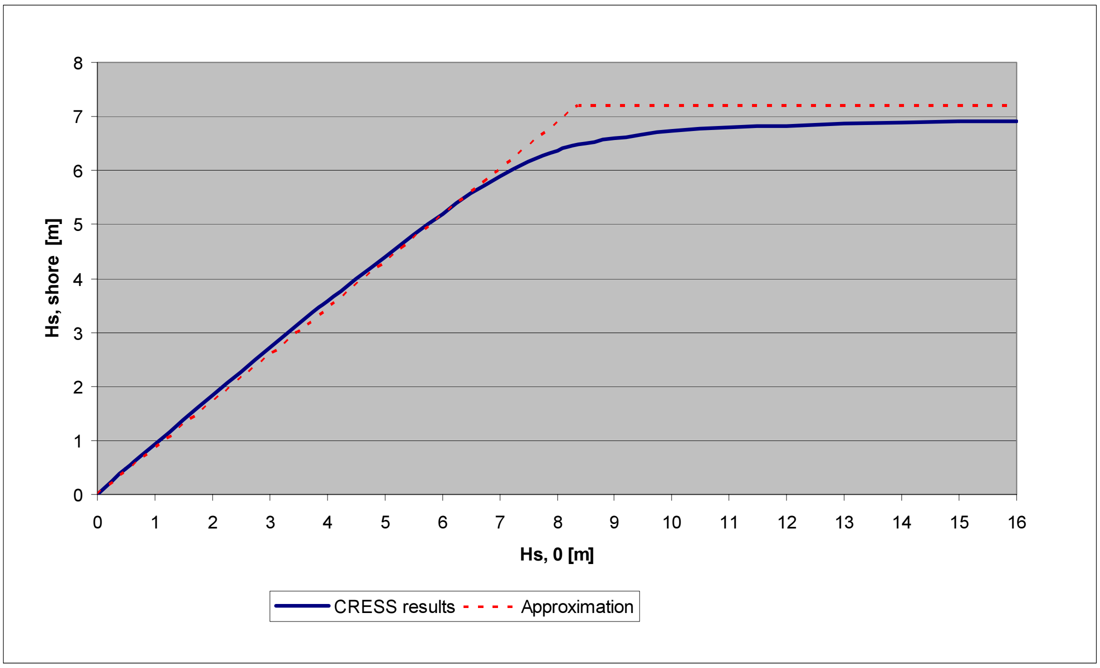

Figure 12 Comparison CRESS results and approximation

The bathymetry of the foreshore of the breakwater can be influenced by the construction of the breakwater. Erosion can lead to a deeper foreshore and consequently, the depth limited waves can exert higher wave loads on the breakwater. Due to the absence of a reliable coastal morphology study no erosion prediction can be made at this moment. Therefore, the effects will be neglected in this study.

Wave period

The wave period is difficult to establish as satellite data measurements used by Argoss are based upon the added wave energy of sea waves and swell. In API (2001a) results of wave period measurements at the Veracruz location are given. These are based on buoy measurements done from July 1995 until November 1995. The data is provided in Figure 13.

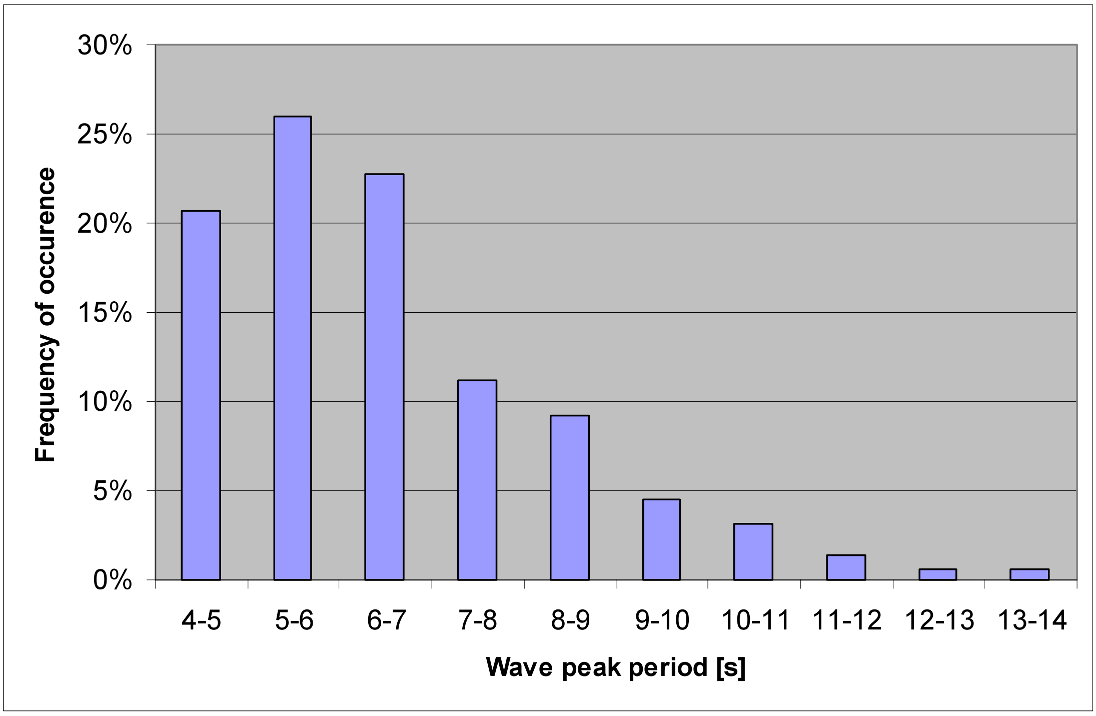

Figure 13 Wave period distribution Jul/95-Nov/95

The observed period covers partly the most calm wave season, but also a part of the storm season. The short observation period does restrict the validity of extrapolation of the results for the whole year. However, no large contribution of swell waves can be observed in the measurement period.

The wave heights and periods are provided independently in API (2001a) and no relation is available to establish the wave steepness from the corresponding wave height and period combinations. However, API uses a design wave height of 6.7 m combined with a mean period of 8.5 s.

3.4 Quarry

For the breakwater a large amount of rock will be required for the construction of the core and filter layers and as concrete aggregate. According to a preliminary quarry analysis (Boskalis 2002) the Balzapote quarry appeared to be the best location to acquire the required rock grading.

The quarry yield curve is fitted with a Rosin-Rammler equation with $n = 0.75$ and $x_c = 2.68$. This curve is provided in Appendix VIII.

With this Rosin-Rammler curve the characteristics of the standard sieve gradings and the yield density curve have been determined. The graphs are given in Appendix VIII.

The quarry will not solely serve as a dedicated quarry for the Core-loc® breakwater investigated in this study. The north south orientated cubes armoured breakwater, see Figure 2, is simultaneously constructed with the Core-loc® breakwater. The cube breakwater necessitates a large amount of the 1-3t rock class and quarry-run. For quarry exploitation optimisation both breakwater demands should be observed simultaneously. As no data is yet available for the cube breakwater no optimisation is possible, regarding quarry output and breakwater demand.

3.5 Soil classification

On the beach granular loose sand, with fragments of shells and occasionally coral intrusions (API, 2001a) is found with a D_{50} = 200 μm. This soil is assumed to be able to support the breakwater without significant settlement.

3.6 Functional boundary conditions

The breakwater and protected basin area have to comply with the following demands:

- Lifetime of the breakwater is 50 years
- Bed level inside the port basin is –16 m +CD
- Length of the breakwater is 1500 m
- Type of vessels: container vessels; maximum 98,000 DWT; 6,600 TEU
- Maximum size of vessels: 14.5 m draft, 28 m width, 347 m LOA
- Allowable downtime in port basin ≤ 5% of the time

4 Core-loc®

4.1 Structural behaviour of Core-loc® armour layers

The structural behaviour of interlocking concrete elements, like the Core-loc® element, differs from the behaviour of rock under waves. Both structural behaviours are shown in Figure 14.

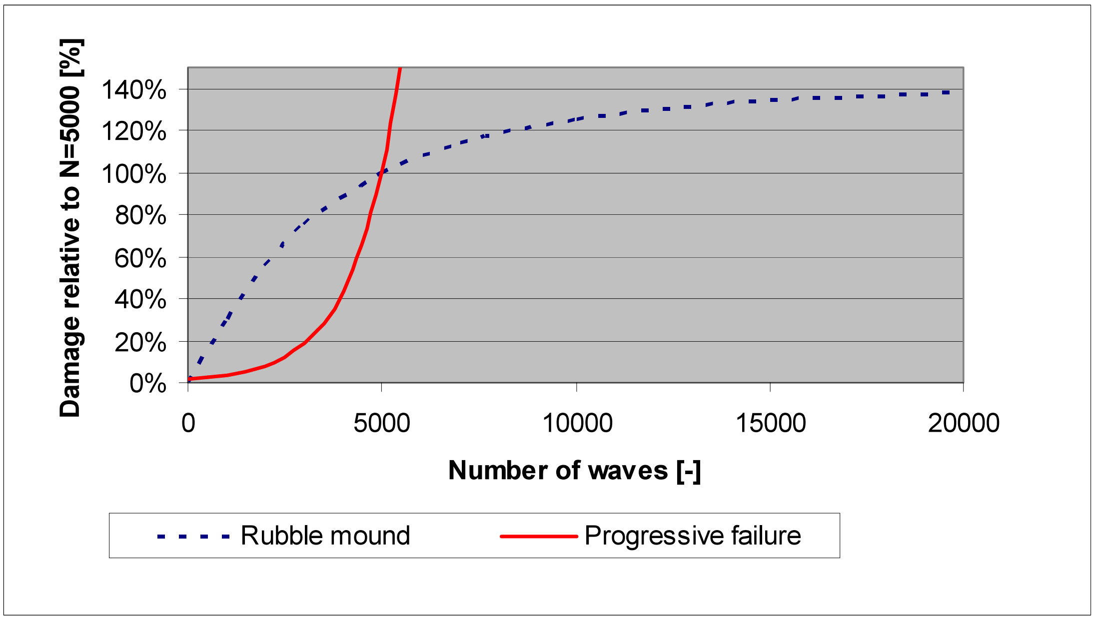

Figure 14 Evolution of damage as a function of time

Rock armoured layers show with an increasing development of damage self-repairing properties. The concrete units conversely, show progressive damage as more damage reduces the interlocking and therefore the stability of the elements. A load able to initiate damage is also able to cause total layer failure if the breakwater is exposed to a sufficient number of loads in a certain period of time. This period of time can be relatively short and could be even shorter than a single storm. The number of loads a breakwater is exposed to is dependent on the wave period and the period of time the breakwater is severely attacked. A peak period of 9 s and a storm duration of 4 hours induces 1600 wave loads on the breakwater. Lower peak periods will expose the breakwater to even higher numbers. This could indicate that intensive repair strategies hardly increase the strength of a breakwater, because no repair is difficult during a storm season. However, if the storm has just sufficient strength to cause minor damage the breakwater can be weakened considerably. Even if no severe damage at layer level at element level minor damage can occur, by breakage of an individual leg. In that case no direct failure results, but weakening of the protective layer is instigated. This could mean that the failure probability could increase during the lifetime of the breakwater if no adequate action is taken.

The Core-loc® breakwater can fail due to extreme wave loads. This failure at the Core-loc® layer level is instigated by failure of a part of the layer. This failure can only occur if lack of stability at the element level exists. Failure behaviour of a Core-loc® layer is therefore determined by the behaviour of the individual elements. Unfortunately, knowledge of the behaviour at element level is lacking. Only stability at the level of the complete layer is known. This stability is determined by use of a limited number of 3-D model tests. Numerous 2-D flume tests are performed, but did not prove to reflect a reliable stability of the Core-loc® layer as the 3-D models failed at significantly lower wave heights (Turk and Melby, 1997b).

4.2 Stability number

The Hudson formula can be used for Core-loc® breakwaters to determine the stability at layer level (Van der Meer, 1999):

$$\frac{H_s}{\Delta D_{n50}} = C_{cl}$$

In which

H_s = significant wave height at the location of the breakwater [m]

Δ = density of armour material relative to the water [-]

= (ρ_c / ρ_w)-1

ρ_c = mass density of concrete [kg/m^3]

ρ_w = mass density of water [kg/m^3]

D_{n50} = characteristic diameter of armour elements [m]

C_{cl} = stability number, constant for Core-loc® [-] for a particular slope angle

The stability number used to describe the strength of the Core-loc® layer expresses the resistance against wave attack of the total layer in a single number given a particular slope.

As mentioned earlier, derivation of the stability number for Core-loc® elements has proven difficult, as the numerous 2-dimensional flume tests did not provide reliable results when compared with the limited amount of 3-dimensional tests. And because Core-loc® breakwaters are only recently constructed not much field experience is available as well.

However, for preliminary design purposes Turk and Melby (1997b) provided a K_D-factor for several slopes. This K_D-factor is also a stability parameter and is translated into the stability number via the following formula:

$$C_{cl} = \left(K_D \cdot \cot\alpha\right)^{1/3}.$$

For trunk slopes of 1:1.5 and a K_D-factor of 16, the design stability number is 2.88. This design guideline includes a safety factor. Taking into account a safety factor of 1.5, the stability number for the initiation of damage is determined at 4.33. This number will be used as the mean of the stability number in the probabilistic calculations.

4.3 Uncertainty of the stability number

Introduction

Not only the mean value is important for probabilistic design, but also the variation of the stability number. To derive an indication of the variation the constitution of the stability will be examined. There appear to be numerous factors with significant influence, which are yet not fully represented in the stability number. These factors will be addressed qualitatively; consequently the standard variation is determined.

Wave period

The influence of the wave period is neglected in the stability formula. However, the wave period influences the number of waves and therefore the extent of the damage progression. Moreover, the wave period, in combination with the wave height and slope angle of the layer, determines the type of wave breaking on the layer. The wave period can hardly be neglected with the determination of the stability. But as no data is available and tests with similar concrete elements, Accropodes, showed, up to now, no direct influence of the wave period in tests, the wave period is assumed to be not decisive and is neglected.

Toe stability

Several model tests showed an unusual failure mechanism (Turk and Melby, 1997b and Mohammad and Jensen, 2002). Deterioration of the armour layer around still water level did occur. This is due to the concentration of wave breaking action on that particular slope area. However, the Core-loc® layer failed after settlement of the bottom (toe) part of the Core-loc® layer. The wave action instigated the lower elements to assume denser packing. However, elements higher up did not slid down the slope. This resulted in a decrease in interlocking between the elements around the still water level. This is the area most exposed to wave action and progressive failure resulted.

Ensuring the stability of the toe units is therefore very important. Various solutions are proposed and used. From the orientation of the bottom elements in a cannon-like fashion, (Turk and Melby, 1997b) to the more costly measures to use grout (Mohammad and Jensen, 2001) or concrete pins to fixate the toe elements in place to hinder movement.

Element size

The size of the concrete elements is also not taken into account. Assuming a similar concrete type for all sizes, the maximum allowable stress does not change with increasing element size. The weight of the elements increases with increasing volume. The internal stresses are increased due to the increasing weight. These stresses should be added to the stresses imposed by wave action.

Breakage of legs

Most tests conducted concern the stability of the complete undamaged units. Breakage of units is often not considered, because the effects of breakage of legs, due to exceeding stresses, are difficult to determine as a result of strength differences due to scaling effects. Two important effects of breakage can be distinguished. The most important effect that also affects the stability of surrounding units is the reduction in interlocking with other elements. The other effect is the reduction in weight by 10% of the original construction weight.

Dynamic loading

As mentioned before, test results provided by Turk and Melby (1997a) indicate that for static loading a splitting tensile stress of 3.5 MPa should be sufficient, even for large Core-loc® elements. However, during transportation and placement already dynamic loads are imposed on the elements. And if rocking of some elements should occur after placement, these loads can not be simplified as static. Experiences during the construction of a Core-loc® breakwater in Tuxpan, Mexico, confirmed that the concrete elements break during construction loads.

Armoured waves

The progressive collapse of the Core-loc® breakwater, due to the weakening effect of increasing lack of interlocking between elements, is already mentioned. Yet, there can be another mechanism that accelerates the deterioration of a Core-loc® layer. If failure occurs due to breakage of elements, the legs that are broken of can be picked up by the waves and smashed back into the armour layer. This imposes large dynamic loads on the elements and failure could result rapidly.

Some Core-loc® element volumes are provided in the following Table 6. The volume of a leg is approximately 10% of the total weight of the element and the maximum stresses in the element due to loading occur at the base of the legs (Turk and Melby, 1997a). Therefore, the weight of an individual leg that is broken off is approximately 10% of the total element weight.

Table 6 Leg weight and nominal diameter

Element Volume	Element Weight	Leg Weight	Element Dn50
[m3]	[t]	[t]	[m]
3.9	8.6	0.86	0.73
6.2	13.6	1.36	0.85
8.5	18.7	1.87	0.95
11	24.2	2.42	1.03
15.4	33.9	3.39	1.16

The formulae provided by Van der Meer (1987) are used to provide an indication of the stability of the broken legs during extreme wave conditions. The Van der Meer formulae for stability of a breakwater with an armour layer of rock are:

For plunging waves: $$\frac{H_s}{\Delta \cdot D_{n50}} = 6.2 \cdot P^{0.18} \cdot \left(\frac{S}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5}$$

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

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

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

Van der Meer used S to describe damage. S = 2-3 equals a 'no-damage' level. However, to determine the instability of the broken parts, an extreme damage level of S = 20 is used in the calculation. For a limited number of waves (N = 50) and the design storm conditions used in the deterministic design already a minimal nominal diameter of 1.43 m is resulting from the Van der Meer formula. The nominal diameter of the broken legs is much smaller, even for the larger Core-loc® size legs.

In conclusion, the broken legs have sufficient weight to cause serious damage and the waves have sufficient strength to throw the legs around.

Standard deviation

The above mentioned factors combined with the limited number of 3-dimensional test and field experience give an uncertainty in the expected stability of the Core-loc® elements over a long period of time.

Van der Meer (1988) conducted numerous tests with the concrete armour element Accropode. The stability number for failure (N_{od} = 0.5) of the Accropode was established at 4.1 (Core-loc® 4.33). The laboratory tests showed a standard deviation of 0.2 for the stability number. The Accropode armour system has the same progressive collapse behaviour as the Core-loc® armour system and has a similar structure. The standard deviation of the Accropode armour is assumed to reflect the variation of the stability number of the Core-loc® element as well. To take into account the uncertainty of field conditions compared to standardised laboratory circumstances the standard deviation is chosen to be 0.4. This results in a variation coefficient (the standard deviation divided by the mean) of approximately 0.1.

4.4 Stability contributions

Contributions

The stability of Core-loc® elements is assumed to be composed of two main contributions. One is the individual stability due to the weight of an element and the other is the layer stability due to the interlocking with other elements. Interlocking is also partly based on friction between the elements and therefore also dependent on the weight.

The stability formula can be adjusted to this distinction in stability contributions:

$$\frac{H_s}{\Delta D_{n50}} = C_{cl} = C_W + C_{IL}$$

In which

C_W = weight or individual stability number

C_{IL} = interlocking or layer stability number

Comparison with other elements

For a particular geometrical and hydraulic situation several armour layer design parameters can be assumed constant. In this situation several other armour elements can be compared with the Core-loc® armouring. As the highest possible wave gives the lowest contribution of the weight the highest possible wave height for the Veracruz breakwater location of approximately 7 m is used. The other parameters for the Veracruz situation are provided in Table 7. In Table 8 the stability numbers of various armour elements are given. The stability formulas are derived from Van der Meer (1999) and are shown in Appendix XI.

Table 7 Situation parameters

Parameter	Dimension	Value
Slope angle (1:x)	[-]	1.5
Permeability breakwater	[-]	0.4
Number of waves	[-]	7000
Damage level (S)	[-]	1
Damage level (Nod)	[-]	0
Spec. density rock	[t/m3]	2.200
Spec. density water	[t/m3]	1.025
Nominal diameter	[m]	2.04
Element weight	[t]	18.7
Mean wave period	[s]	8.45
Mean wave length	[m]	111
Wave height	[m]	7.00
Iribarren parameter	[-]	2.66
Wave steepness	[-]	0.063

Table 8 Stability numbers

Element type	Ctotal
Core-loc®	4.3
Accropodes	3.7
Tetrapods	1.5
Rubble mound	1.3
Cubes (single layer)	3.0
Cubes (double layer)	1.3

Complex elements with legs such as Core-loc® and Accropodes attribute a large part of their stability from interlocking. Cubes and rubble mound rock layers derive their stability mainly from the weight of the elements. Especially cubes placed in double layers seem to be lacking interlocking stability. This can be concluded from the higher stability of cubes placed in a single layer.

Comparison of the different elements should be done with care. The stability of cubes placed in a double layer and rubble mound armoured layers is assumed to be dependent on the wave period. Armour of concrete elements as Accropodes, and therefore presumably also Core-loc®, are assumed not to be dependent on the wave period. Still, Table 8 can provide a good indication for the upper bound of the contribution of the weight to the stability of Core-loc® units. The stability value for double layer cubes provides a reasonable upper bound estimation of the weight induced stability number. To compensate for the influence of friction between the cubes, the C_W is reduced with an assumed reduction factor of 0.9. The weight stability contribution of an element is thus calculated as 0.9*1.3 = 1.2. The interlocking stability contribution can subsequently be determined by subtracting the weight stability contribution from the total stability. This leads to the indicated C_W and C_{IL} in Table 9 for the different element types.

Table 9 Stability contributions

Element type	Ctotal	Cweight	Cinterlocking
Core-loc®	4.3	1.2	3.1
Accropodes	3.7	1.2	2.5
Tetrapods	1.5	1.2	0.3
Rubble mound	1.3	1.2	0.1
Cubes (single layer)	3.0	1.2	1.8
Cubes (double layer)	1.3	1.2	0.1

4.5 Time dependent breakage of legs

Both the weight stability and the interlocking stability the development in time are time dependent on the breakage development of the legs. Therefore, first the breakage of legs is discussed and subsequently the development of the weight stability and the interlocking stability in time will be elaborated.

The percentage of broken legs depends on the wave loads exerted on the Core-loc® elements and the strength of the elements. At this moment the relation between wave loads and strength of the elements is not known. Unfortunately, also no field data is available on the breakage rate of legs given an average number of storms per year with a given significant wave height. Turk and Melby (1997a) report 2% breakage after eleven years (1986-1997) of monitoring of a Dolos breakwater at Crescent City for elements of 38t. The Dolos is a more slender and vulnerable element than the bulky Core-loc® and tests seem to indicate a better interlocking for Core-loc® elements without rocking. Therefore, the breakage rate of Core-loc® is assumed lower than that of Dolos. Also an assumption on the breakage development has to be made. The approximation should reflect the increasing instability of the element, and an increased rate of breaking, if more legs are already broken. Therefore, in this study the breakage rate is assumed to be an in time exponentially increasing function:

$$N(t) = a \cdot e^{b \cdot t} - a .$$

The values of a and b are to be chosen to provide a reasonable reflection of the assumed percentage of broken legs over time. The values provided in Table 10 are evaluated in this study. The variation of coefficient b reflects the possibilities of progressiveness of the progressive collapse. The assumed development over the lifecycle of the breakwater is also depicted in Figure 15. The coefficients are not based on field or test experience. No such data exists at this moment. However, elements will break and the breakage rate will increase progressively in time. The exponential function is used to reflect the progressive collapse, but the shape of the curve is not based on test results and therefore disputable.

Table 10 Parameters exponential breakage development

a	b	N(t=50)
0.001	0.0487	1%
0.001	0.0691	3%
0.001	0.0787	5%

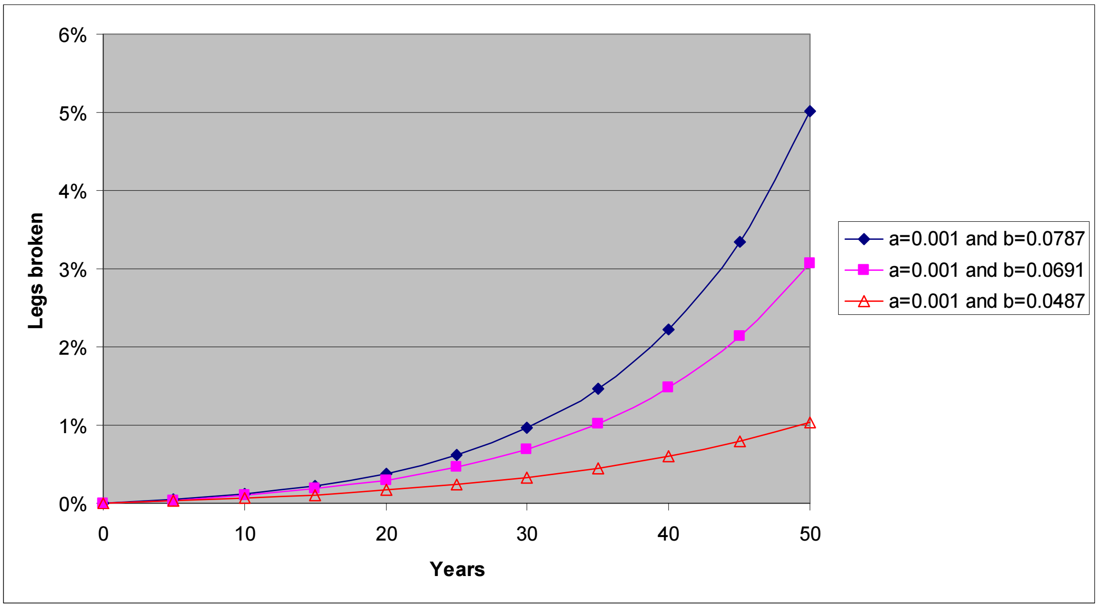

Figure 15 Development of leg breakage

4.6 Weight stability

The contribution to the stability number by the element weight is a constant, namely the earlier derived 1.2. The weight of an element is not a constant. For every weight a constant α is determined to express the relation for that particular weight with the weight stability:

$$C_W(t) = \frac{W(t) \cdot \alpha}{W_0}.$$

In which,

$C_w(t)$	= weight contribution of the stability number = weight stability = 1.2 [-]
$W(t)$	= value of the weight at time t [t]
W_0	= initial weight of the complete element at time t=0 [t]
α	= constant = weight stability at t =0 =1.2 [-]

The weight of an element decreases as legs break off. A leg contains approximately 10% of the total initial weight of a Core-loc® element. An element has six legs after construction. The following formula can therefore be used to describe the weight stability:

$$C_W(t) = \frac{\alpha \cdot W(t)}{W_0} = \alpha \cdot \frac{W_0 - 6 \cdot W_{leg} \cdot N(t)}{W_0} = \alpha \cdot (1 - 0.6 \cdot N(t)).$$

In which,

W_{leg} = weight of one leg = $0.1 \cdot W_0$ [t]

$6 \cdot W_{leg}$ = weight of six legs [t]

$N(t)$ = percentage of broken legs at time t [-]

With the assumed development of leg breakage the development of the average element weight over time can also be determined and is shown in Figure 16.

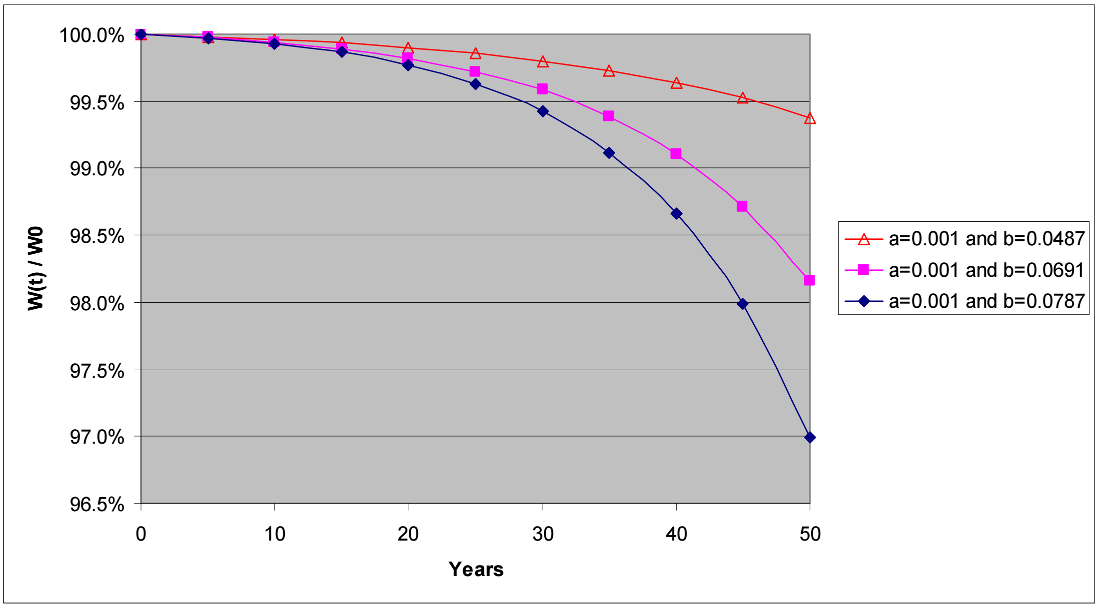

Figure 16 Development of element weight

The influence of progressive deterioration on the element weight is limited. In the worst evaluated scenario 5% of the legs is broken after 50 years, but still 97% of the original weight remains. The weight stability is linearly dependent on the weight via the constant α. The influence of breakage on the element weight stability of the elements is limited. The time dependent development of the weight stability is given in Figure 17.

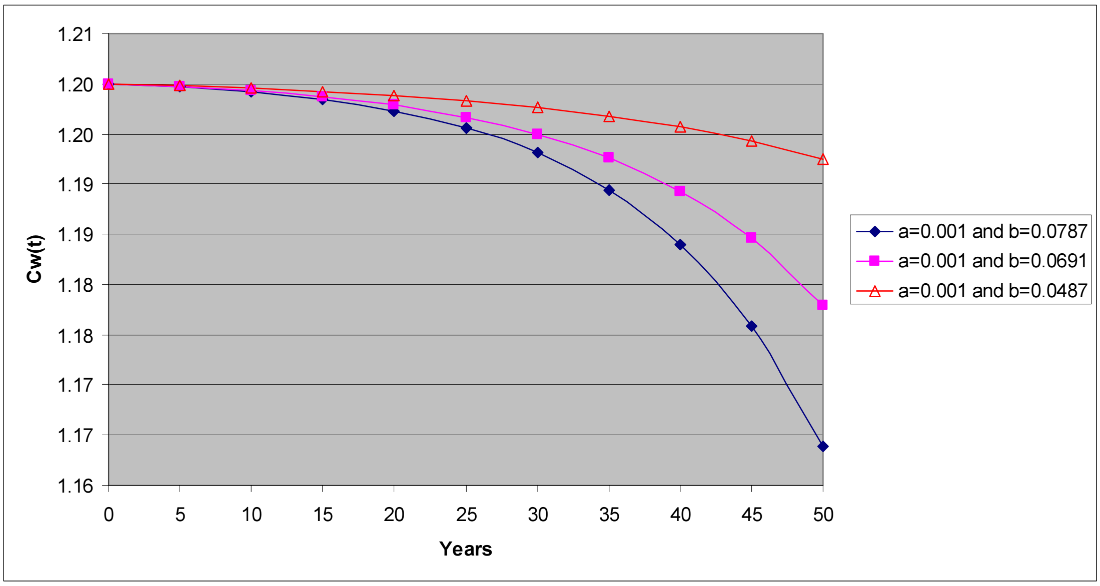

Figure 17 Development of weight stability

4.7 Interlocking stability

The interlocking stability of a Core-loc® element can be described as the average interlocking of six legs together. The interlocking stability will decrease if legs break off. One interlocking connection interlocks two elements. Thus, if on average one percent of the legs break the interlocking for the average element is reduced with twice that percentage.

$$C_{IL}(t) = \beta \cdot (1 - 2 \cdot N(t)).$$

In which,

$C_{IL}(t)$	= interlocking contribution of the stability number [-]
β	= constant = interlocking stability at t=0 =3.1 [-]
N(t)	= percentage of broken legs at time t [-]

The time dependent development of the interlocking stability is visualised in Figure 18.

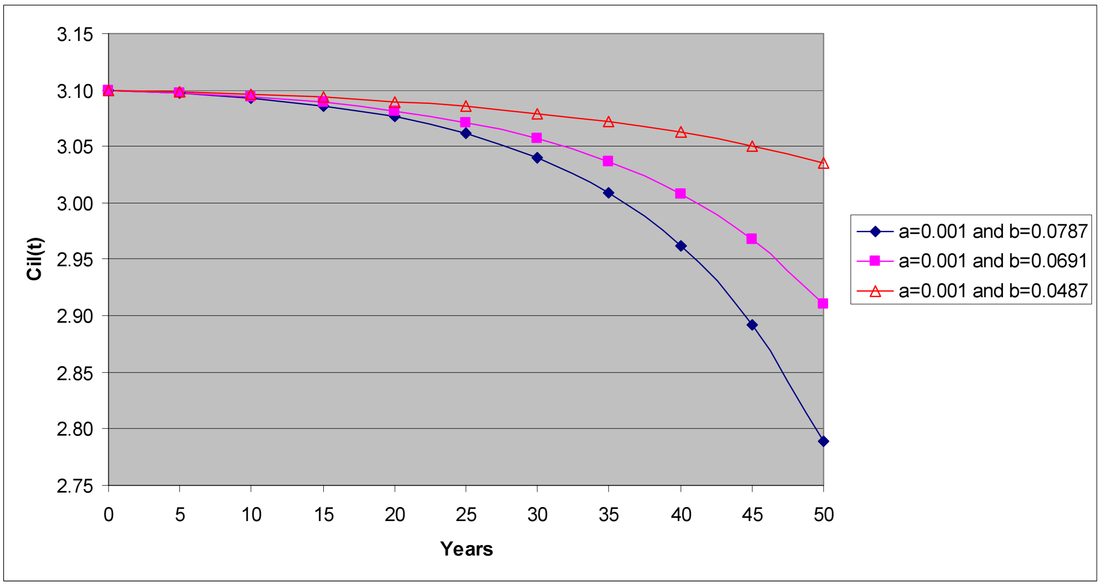

Figure 18 Development of interlocking stability

4.8 Total time-dependent stability number

For every element weight the total time-dependent stability number can be determined. The total stability is calculated by summarising the time-dependent stability derived for the weight stability and the time-dependent interlocking stability:

$$\frac{H_s}{\Delta D_{n50}} = C_{cl}(t) = C_W(t) + C_{IL}(t) = \alpha \cdot (1 - 0.6 \cdot N(t)) + \beta \cdot (1 - 2 \cdot N(t)).$$

In which $N(t) = a \cdot e^{b \cdot t} - a$ provides the development of leg breakage over the lifecycle of the breakwater.

The development in time of the stability number for Core-loc® elements is shown in Figure 19.

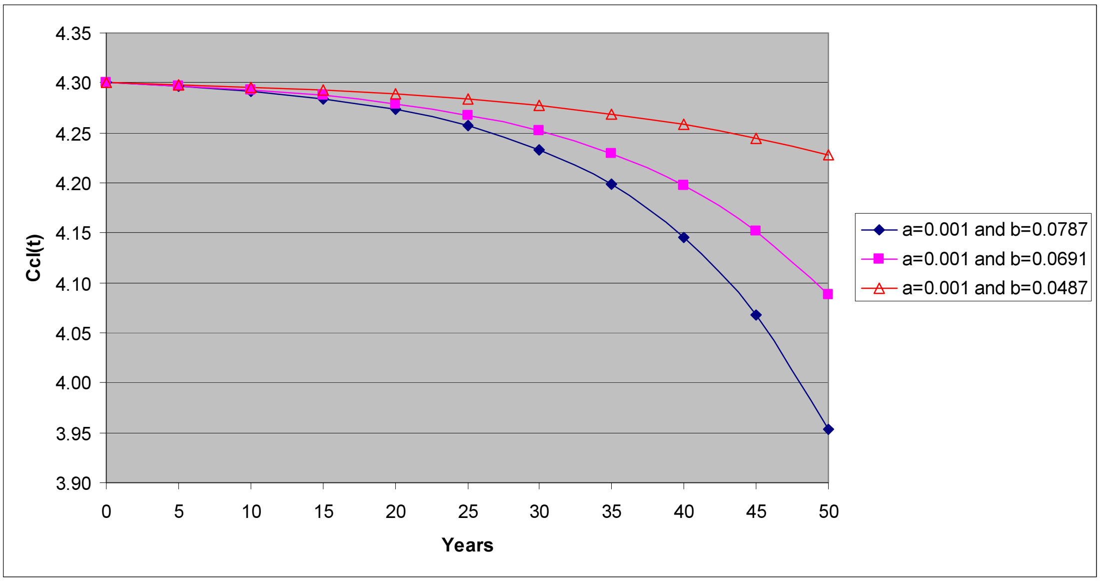

Figure 19 Development of Core-loc® stability number

In this research the values of parameters a and b are assumed to be respectively 0.001 and 0.0691. This represents a situation with three percent of the legs broken at the end of the 50 year lifetime of the breakwater as shown in Figure 15.

4.9 Representation progressive collapse

Progressive behaviour of collapse of a Core-loc® armour system is reflected in two ways. First, the stability number of Core-loc® for initiation of damage is assumed to reflect total failure of the Core-loc® layer during a storm with an exceeding wave height. This effect is the result of the progressive collapse behaviour. Secondly, the decrease of the stability number, due to the exponential increase of leg breakage, includes the progressive deterioration of the system strength.

5 Deterministic design

5.1 Introduction

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

The deterministic design is mainly focussed on the following components:

- The primary armour of Core-loc®
- The supporting toe
- The secondary armour
- The core
- The filter system to stabilise the supporting bottom material
- The crest height

The components are also indicated in Figure 20.

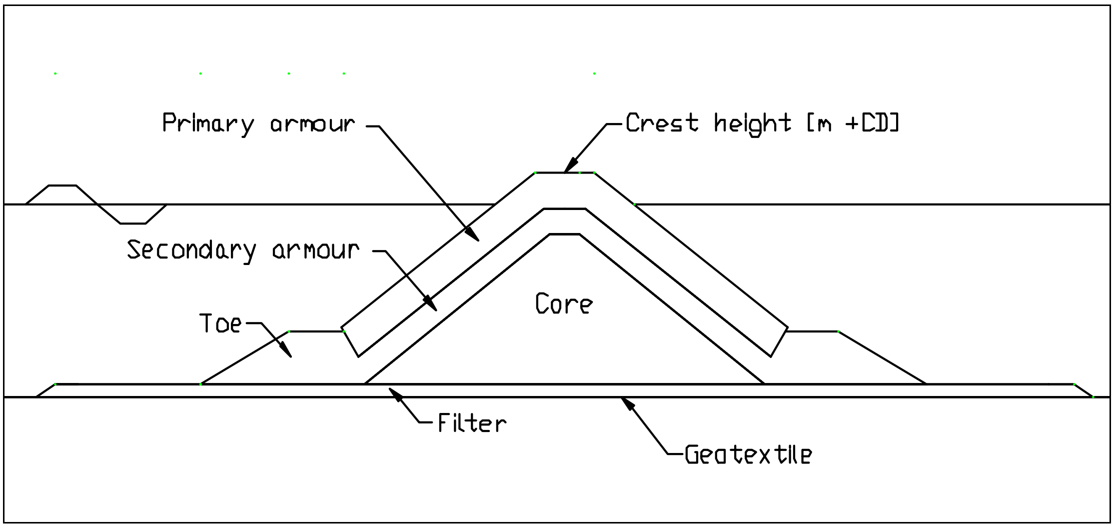

Figure 20 Breakwater components

In Appendix X the relevant design guidelines and dimensions, used in the design process, are depicted.

5.2 Design wave height

In the deterministic design process 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 derived. Consequently the return period can be used to determine the wave height. However, the wave height at the breakwater location is depth limited. If no storm surge occurs the significant wave height has a maximum shallow water significant wave height of 7.2 m. The shallow water wave height is depth limited if the significant deep water wave height exceeds 8.4 m. This deep water wave height has a return period of 50 years, equal to the lifetime of the breakwater. Appendix IX the probability of failure during the lifetime of the breakwater is determined to be 63% with the return period of 50 years. A probability of failure of the breakwater of 63% during the lifetime is assumed to be acceptable. This is relatively high, but not uncommon according to PIANC (1992).

As a result a return period of 1000 years provides the same design wave height as a return period of 50 years. Therefore, the influence of the stochastic variation of the extreme wave height is in this study limited for regular storms. The influence of the stochastic variation of other parameters, affecting the load and strength of the breakwater, could be very important. In the probabilistic design method these stochastic variations will be observed and taken into account to determine the probabilities of failure for the breakwater

However, two ULS situations exist in Veracruz. The common normal storm and also the rare hurricane can occur. This hurricane has an estimated return period of 200 years with results according to Appendix IX in a probability of failure during the lifetime of 50 years of 22%. The hurricane causes a significant storm surge of three meters and the maximum depth limited shallow water significant wave height is increased by 3 m x 0.45 = 1.4 m to a total significant wave height at the breakwater location of 8.6 m.

In the deterministic design the significant design wave height is chosen at 7.2 m for further determination of the breakwater design. However, the probabilistic results will be used to evaluate the chosen design wave height.

5.3 Hydraulic stability primary armour

Armour weight

The following stability formula, provided by Turk and Melby (1997b), is used to determine the required weight of the Core-loc® elements:

$$\frac{H_s}{\Delta \cdot D_n} = (K_D \cdot \cot \alpha)^{1/3} = C_{CL}$$

In which

H_s = significant wave height at the location of the breakwater [m]

Δ = density of armour material relative to the water [-]

= (ρ_c / ρ_w)-1

ρ_c = mass density of concrete [t/m^3]

ρ_w = mass density of water = 1.025 [t/m^3]

D_n = nominal diameter of armour elements [m]

= $D_n = \left(\frac{W}{\rho_c} \right)^{1/3}$

W = weight of element [t]

cot α = cotangent of slope angle = 1.5 [-]

K_D = design value for Core-loc® elements = 16 [-]

C_{CL} = stability number for N_{od} = 0 for Core-loc® elements for a 1:1.5 slope = 2.88 [-]

N_{od} = damage level (0.5 = initiation of damage) [-]

The stability formula can be rewritten:

$$W = \frac{\rho_c \cdot H^3}{K_D \cdot \Delta^3 \cdot \cot \alpha}.$$

In Mexico concrete with a specific weight higher than 2.2 t/m^3 is much more expensive than the common 2.2 t/m^3. Variation of the specific density is therefore not considered. The chosen element weight should be equal or more than the demanded weight. For an economic design the surplus of weight should be minimised. In Figure 21 the weight is plotted against the shallow water significant wave height. The sloping line gives the demanded weight to ensure stability at a given wave height according to the design guideline provided by Turk and Melby (1997b).

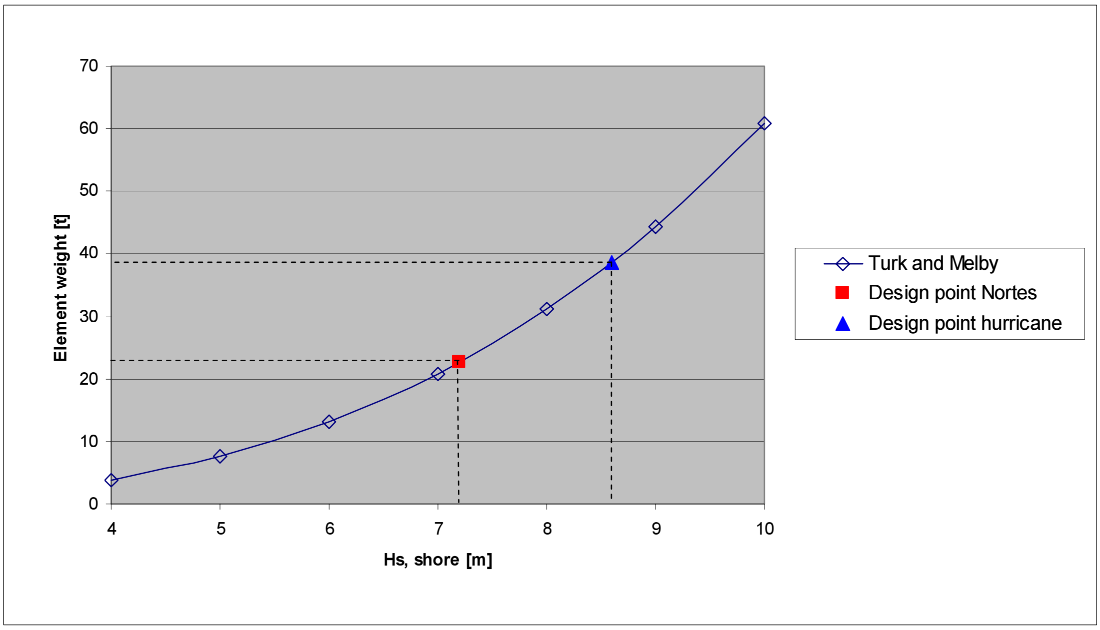

Figure 21 Weight demand and significant wave height

A combination of a specific density of concrete of 2,2 t/m^3 and an element weight of 22.7 ton provides a design with a minimum of surplus weight. Consequently, the volume of the element is 10.3 m^3. According to the design guidelines of Turk and Melby (1997b) the Core-loc® armour is thus designed for a design wave height of 7.2 m. This design point is also depicted in Figure 21.

The design wave height for hurricanes is 8.6 m. This significant wave height demands an element weight of 38.7 ton, this leads to an element volume of 17.6 m^3. The design point for the hurricane condition is given in Figure 21. As earlier mentioned the deterministic design is not designed at hurricane wave height level.

Layer thickness

The layer thickness is, according to Turk and Melby (1997b), determined as:

$$t = n \cdot k \cdot D_n .$$

In which

t = layer thickness [m]

n = number of elements in layer = 1 for Core-loc® [-]

k = layer thickness coefficient = 1.51 for Core-loc® [-]

D_n = nominal diameter = 2.18 [m]

The layer thickness results in 3.3 m.

Minimum depth

As a rule-of-thumb the minimum depth of the primary armour on the seaside slope should be 1.5 times H_s. The minimum depth is also dependent on the toe stability. The minimum depth is determined in paragraph 5.4 simultaneously with the toe stability.

5.4 Toe stability

Filter limits

For the transition between the primary armour and toe material a certain maximum weight or ratio has to be observed.

In Appendix VIII the available standard stone size distribution curves and their characteristic diameters are derived. The characteristic diameters are repeated in Table 11.

Table 11 Class characteristic diameters

Class	% of total	Dn15	Dn50	Dn85	Dn85/Dn15	W50
[t]	[-]	[m]	[m]	[m]	[-]	[kg]
0.001-1	56.6%	0.10	0.23	0.47	4.56	36
0.3-1	9.1%	0.49	0.56	0.64	1.32	530
1-3	6.2%	0.72	0.81	0.93	1.29	1649
3-6	2.8%	1.02	1.10	1.20	1.17	4124
4-7	2.0%	1.12	1.19	1.27	1.14	5186

Taking into account the transition of secondary/ toe material to primary concrete armour the SPM (1984) provides the following ratio:

$$\frac{W_{50,primary}}{W_{50,secondary}} = 5.$$

The W_{50} of the 3-6t stone class is 4120 kg. With 18.7t Core-loc® units this is well above the demanded weight of 3740 kg.

Toe stability formula

For the determination of the toe stability the formula of Van der Meer (1993) is written as follows:

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

In which

H_s	= significant wave height [m]
Δ	= relative density [-]
	= (ρ_c / ρ_w)-1
ρ_c	= mass density of concrete = 3.090 [t/m^3]
ρ_w	= mass density of water = 1.025 [t/m^3]
D_{n50}	= characteristic diameter of toe elements [m]
h_t	= depth of the crest of the toe below water level [m]
h	= water level [m]
N_{od}	= dimensionless damage level = 0.5 (initiation of damage) [-]

In Appendix X the dimensional parameters are elucidated.

This formula can be used in the range:

$$0.4 < \frac{h_t}{h} < 0.9$$

$$3 < \frac{h_t}{D_{n50}} < 25$$

The toe depth possibilities are limited by three factors: relative density, rock size, water level and wave height. The specific weight of water and rock are constants. The large Core-loc® elements and filter rules oblige the use of 3-6 t rock weights. For the water depth a still water level of 0.3 m +CD is assumed. The significant design wave height is the same as used for the Core-loc® stability and is 7.2 m.

Minimum depth to ensure toe stability

The minimum allowable depth is therefore dependent on the combination of water level and wave height. The toe depth is calculated for the still water level of –0.3 m +CD and the significant wave height of 7.2 m. The minimum depth is –9.6 m +CD for toe stability.

Minimum depth to ensure primary armour stability

The required depth of the primary armour also gives a minimum depth for the toe. As a rule-of-thumb the minimum depth should be 1.5 times H_s. The combination of water level and wave height provides the minimum depth. With a still water level of –0.3 m +CD and a significant wave height of 7.2 m the toe depth should be more than –10.5 m +CD to ensure primary armour stability.

Evaluation of minimum toe depths

The toe depth should be more than –10.5 m +CD for the primary armour stability and more than –9.6 m +CD for toe stability. Therefore, a toe depth of –10.5 m +CD is used as the minimum toe depth.

Toe dimensions

The crest of the toe is defined at –10.5 m +CD. The height and width of the toe are to be 2.2 m (=2*D_{n50}) and 3.3 m (=3*D_{n50}) minimal (SPM 1984). As mentioned Core-loc® layers are sensitive to movement at the toe section, thus a larger toe dimension is advisable. This allows for more toe erosion before the stability of the Core-loc® layer is affected. Therefore, a toe width of 5.5 m instead of 3.3 m is preferred.

5.5 Secondary armour stability

Stability after construction

Considering the 23 ton Core-loc® elements, the 1-3t class of rock provides a W_{50} of 1650 kg. The recommendation of the CEM (2002) is used:

$$\frac{W_{50,primary}}{W_{50,\sec ondary}} \leq 5.$$

With primary armour of 18.7t Core-loc® units this does not suffice for the 1-3t rock class. The 3-6t class does provide a stable situation. The available amount of the higher 3-6t weight class is significantly lower in the quarry but for construction use of the same material for both toe and secondary armour is favourable.

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: $$\frac{H_s}{\Delta \cdot D_{n50}} = 6.2 \cdot P^{0.18} \cdot \left(\frac{S}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5}$$

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

In which,

H_s = significant wave height [m]

Δ = density of armour material relative to the water [-]

= (ρ_c / ρ_w)-1

ρ_c = mass density of concrete = 3.090 [t/m3]

ρ_w = mass density of water = 1.025 [t/m3]

P = permeability of the breakwater = 0.4 [-]

N = number of waves = 7000 [-]

S = damage level [-]

ξ_m = 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/s^2]

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

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

For the 3-6t rock size, with a D_{n50} of 1.1 m, initiation of damage (S = 2) starts with a design wave height of 3.2 m. The construction of the secondary armour is assumed to continue during all wave seasons. Using the translation formulas provided in Chapter 3.3 and the wave data given in Appendix V, this wave height appears to be exceeded during 3% of the construction time. Significant loss of material is not to be expected. Additional quarry material has not to be taken into account.

5.6 Core filter stability

Filter stability after construction

As core material, a rock grading of 1-1000 kg is used. The secondary armour of 3-6t should function, according to the following filter rules of Terzaghi, as a proper filter for the core material. The grading properties are provided in Table 11.

Permeability rule: $\frac{d_{15F}}{d_{15B}} > 5$.

In which

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

d_{15B} = sieve diameter passed by 15% of the base material

To the permeability rule is complied by a D_{15} smaller than 0.2 m, namely 0.10 m.

Stability rule: $\frac{d_{15F}}{d_{85B}} < 5$.

In which

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

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

The stability rule is respected with a D_{85} greater than 0.2 m, namely 0.470 m.

The above mentioned results justify the use of the 3-6t stone class filter for the core material.

Rock stability during construction

The same procedure is followed for the potential loss of material during construction as for the secondary armour layer. For quarry run the D_{n50} is 0.23 m and the wave height to initiate damage is 0.7 m. In the situation with severe damage leading to deformation of the slope (S = 15) the wave height is 1.0 m. Using the translation formulas provided in Appendices V and VI these wave heights appear to be exceeded respectively 70%, 50% and 40% of the time. Significant loss of material is to be expected. The percentages of loss of unprotected material during wave conditions causing damage are indicated in Table 12. If ten percent of the material is assumed to wash away during conditions of level 5 and higher the total expected loss in a year is 50% of that value. Additional quarry material has to be taken into account. Five percent of additional material is assumed to be sufficient to compensate for the loss of material.

Table 12 Expected loss of material

Damage level	S	Period of time	Loss during period of time	Expected loss/year
Initiation	2	70%	0%	-
Average	5	50%	10%	5%
Severe	15	40%	25%	-

5.7 Geotextile

Geotextile is applied to prevent the bottom material to wash out. This geotextile retains the bottom material if it complies with the following filter rule:

$$O_{90} < d_{90B}$$

In which

O_{90} = size of openings in the geotextile passed by 90% of a certain diameter [m]

d_{90B} = sieve diameter passed by 90% of the base material

The bottom material has a d_{50} of 200 µm. With an assumed grading of d_{90}/d_{50} of the bottom material of 1.5, the d_{90} is determined at 300 µm. A geotextile with an O_{90} of 300 µm will be sufficient to retain the bottom material.

To keep the geotextile in place and to provide protection against the larger pieces of rock in the core, toe and secondary material a layer of 10-60 kg of rock is placed on top of the geotextile.

5.8 Crest height

5.8.1 Wave transmission

The crest height determines the amount of transmission of wave energy over the Core-loc® breakwater. Increasing the breakwater height limits the amount of wave overtopping and decreases the transmission of wave energy into the harbour basin. However, wave energy also bypasses the breakwater, by entering the basin via the port entrance, and flows through the breakwater. The total wave energy allowed in the port is composed of the wave energy transmission through, over and around the breakwater. The transmitted wave energy is proportionally with the quadratic transmitted wave height. This proportionality is elaborated in Appendix XV. The total transmitted wave height into the basin is described by the following formula:

$$H_{port\ basin} = \sqrt{\left(H_{entrance}\right)^2 + \left(H_{transmission}\right)^2} = \sqrt{\left(K_{entrance} \cdot H_{s,\ shore}\right)^2 + \left(K_{transmission} \cdot H_{s,\ shore}\right)^2}$$

$H_{port\ basin}$ = wave height transmitted into the port basin [m]

$H_{entrance}$ = wave height transmitted through the entrance [m]

$H_{transmission}$ = wave height transmitted through and over the breakwater [m]

$H_{s,\ shore}$ = incoming wave height at the breakwater location [m]

$K_{entrance}$ = transmission coefficient for the wave intrusion via the port entrance [-]

$K_{transmission}$ = transmission coefficient for the wave intrusion via the breakwater [-]

5.8.2 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.

Boundary conditions

The minimum allowable crest height is determined from the allowable port basin conditions and the occurring incoming wave heights at the port location. Accordingly to API (2001a) downtime of port operations occurs if the wave height in the basin exceeds 0.5 m. During 5% of the time downtime is allowed in the port basin (API 2001a). The incoming wave height of interest is the SLS wave height that is exceeded 5% of the time. This shallow water wave height is determined to be 2.7 m.

Wave transmission

The transmission coefficient for the wave intrusion via the port entrance is according to API (2001a) 0.08 at the quay location.

The transmission due to overtopping over and flow through a Core-loc® breakwater is described by the following conditional formulas of Melito and Melby (2002) and is valid for both submerged and low crested breakwaters:

$$K_{transmission} = 0.95 \qquad \text{for } R_c / H_{s,\,shore} < -1.0$$

$$K_{transmission} = 0.56 - 0.39 \cdot R_c / H_i \qquad \text{for } -1.0 < R_c / H_{s,\,shore} < 1.3$$

$$K_{transmission} = 0.05 \qquad \text{for } R_c / H_{s,\,shore} > 1.3$$

In which,

$K_{transmission}$ = transmission coefficient, defined as: $K_{transmission} = \dfrac{H_{transmission}}{H_{s,\,shore}}$ [-]

$H_{transmission}$ = transmitted wave height [m]

$H_{s,\,shore}$ = incoming wave height at the breakwater location [m]

$R_c/H_{s,\,shore}$ = dimensionless crest height [-]

R_c = crest height above still water level [m]

The $K_{transmission}$ can be calculated from the incoming wave height of 2.7 m. Subsequently applying the transmission formula of Melito and Melby to determine the required crest height results in a crest value of 3.0 m +CD, including the consequences of a sea level rise of 0.15 m. Because of this, the top of the secondary armour is located at –0.1 m +CD underneath the mean sea level.

Propagation of long waves through the open structure of the Core-loc® layer should be further investigated. Without additional knowledge of this intrusion mechanism a minimum crest height of the secondary armour up to the level of the highest high water level of 0.9 m +CD can be applied. This would prevent the potential unhindered transmission of long wave energy through the upper Core-loc® layer. The current available data shows hardly any swell in the area. Even though, measurements of the wave spectrum are limited and reliability is therefore relatively low, the propagation effects of long waves will be neglected in this study.

The construction operation is likely to suffer downtime due to excessive wave height, which hampers water based operations. The seaside of the breakwater should be constructed during very calm weather periods and the leeside during rough conditions. The secondary armour layer provides a relatively stable temporary protection for the breakwater. The Core-loc® elements, which require very accurate placement, could be positioned during the low wave season from January till September. The various downtimes with accompanying allowed maximum wave heights are provided in Table 13.

Table 13 Downtime and wave heights

Downtime	Hs shore
[-]	[m]
90%	0.41
80%	0.49
70%	0.58
60%	0.70
50%	0.83
40%	1.01
30%	1.23
20%	1.56
10%	2.12
5%	2.70
1%	4.07

A maximum wave height of 1.0 m is allowed for all water-based operations based on Duijvestijn (1995). From the data in Table 13 is deducted that during 40% of the time the maximum wave height is exceeded.

5.8.3 Land based construction

Land based construction imposes constraints on the breakwater height and width. The trucks and cranes operate at the level of the crest of the core of the breakwater. Thus, the overtopping of the breakwater under construction requires a minimum crest height of the core and indirectly the breakwaters crest level. The 3-6t secondary armour and the Core-loc® elements do not provide a flat working area or transportation possibilities. In the CIRIA/ CUR (1991) a guideline is given on the impact of overtopping on workability and safety for equipment and personnel. This graph is provided in Appendix XII. For safety and proper working conditions, 0.01 l/s/m is assumed to be a maximum allowable overtopping discharge.

For calculation of the overtopping discharge Van der Meer (1994) gives the following formula for rubble mound and concrete armoured breakwaters:

For $\xi_{0p} > 2$

$$\frac{q}{\sqrt{g \cdot H_s^3}} = \frac{0.2}{1000} \exp\left(-2.6 \frac{R_c}{H_s} \cdot \frac{1}{\gamma} \right).$$

In which,

ξ_{0p} = Iribarren parameter, describes the type of wave breaking on a slope

q = average overtopping discharge [l/s/m]

γ = reduction factor for slope roughness = 0.55 [-]

g = acceleration of gravity = 9.81 [m/s^2]

For several crest heights the amount of downtime is calculated with the overtopping formula of Van der Meer for a maximum overtopping discharge of 0.01 l/s/m. The results are shown in

Table 14.

Table 14 Influence crest height on construction downtime

R_c	Downtime of construction operations
[m +CD]	[-]
7	65.1%
8	39.2%
9	24.5%
10	15.6%
11	10.2%
12	6.8%
13	4.5%
14	3.0%
15	2.1%
16	1.4%
17	1.0%

The analysis of the SLS wave data shows a considerable seasonal variation in the distribution of the wave height. The distribution of the occurring downtime over the seasons is provided in Figure 22. This figure gives the contributions of the four wave seasons to the total yearly downtime. For example, for a crest height of 7 m +CD Table 14 informs that downtime can be expected during 65% of the time. From Figure 22 is subsequently deducted that in the Oct-Jan season the largest part of the downtime occurs and in the Aug-Sep season the smallest contribution. For a crest height of 12 m +CD and higher Figure 22 shows, downtime is to be expected only in the seasons Feb-Apr and Oct-Jan, with the most downtime from October till January.

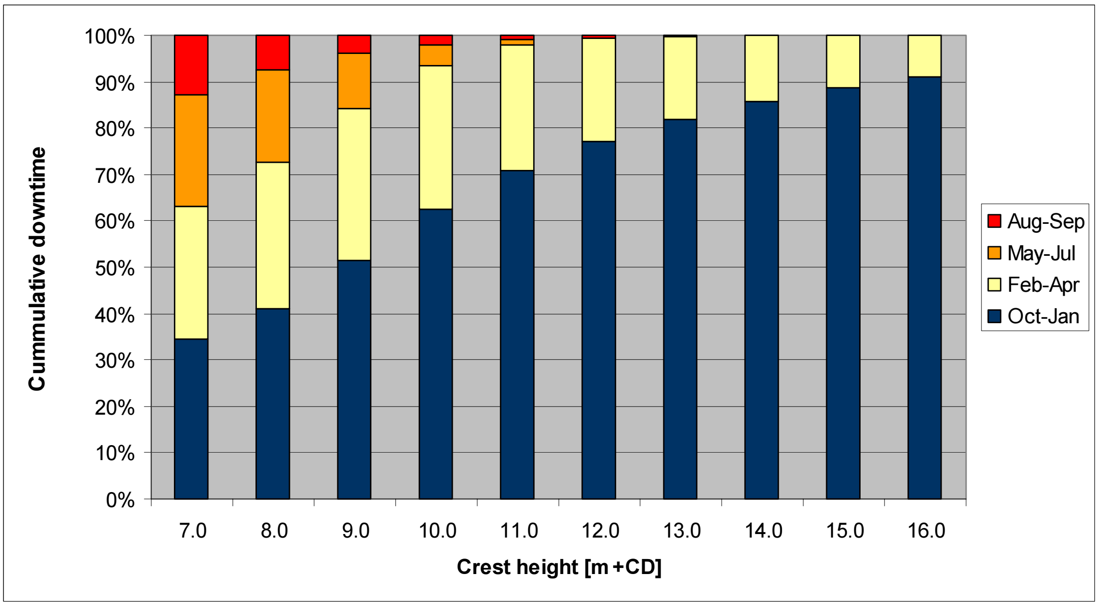

Figure 22 Seasonal downtime distribution

In the storm season from October to January the works could be suspended. This would save costs because this prevents damage to the equipment and the construction. Observing the third column in Figure 22, the column for a crest height of 9 m +CD, the total downtime per year would be more than halved. However, the suspension of works during October till January also reduces the construction time by 33%. An increase up to a crest height of 11 m +CD showed a reasonable decrease in downtime down to 10%. An increase in crest height gives a disproportional increase in the quantity of material needed and thus in the construction costs. Therefore, a height of 11 m +CD is assumed to be the optimal crest level based on this limited amount of information. A cost optimisation could be performed to determine the best option. This is considered to be out of the scope of this thesis.

5.9 Crest width

Water- based construction

The CEM (2002) suggests a minimum crest width of 3 times the nominal diameter for concrete elements. 22.7t Core-loc® units have a nominal diameter of 2.04 m. Therefore, a width of 6.1 m is chosen.

Land based construction

The minimum width for trucks and cranes to operate on the core crest is 9.0 m, according to Schiereck (2001). This results in a width of the Core-loc® crest of 12.2 m.

5.10 Summary geometry breakwater

In Figure 23 the cross-sections of both the land- and water-based constructions are given. In Appendix XIV the cross-sections are provided in more detail. In Table 15 the main characteristics of the breakwaters are shown.

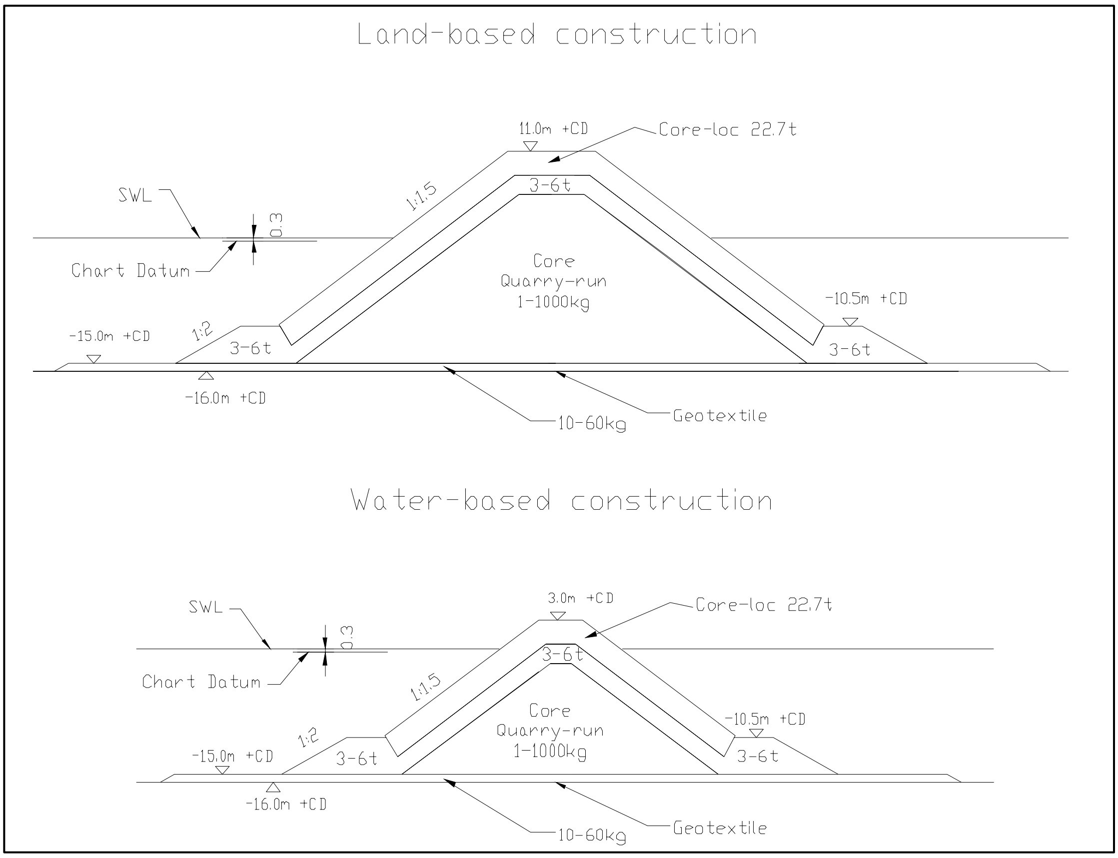

Figure 23 Breakwater cross-sections

Table 15 Summary geometries

	Water-based	Land-based
Dimensions breakwater:		
Crest height	3.0 m +CD	11 m +CD
Crest width	6.1 m	12.2 m
Foundation depth construction	-16 m +CD	-16 m +CD
Slope primary armour	1:1.5	1:1.5
Slope toe	1:2	1:2
Primary armour:		
Weight Core-loc® elements	22.7t	22.7t
Layer thickness	3.3 m	3.3 m
Depth primary layer	-10.5 m +CD	-10.5 m +CD
Secondary armour:		
Rock class	3-6t	3-6t
Layer thickness	2.2 m	2.2 m
Core:		
Stone class	1-1000kg	1-1000kg
Toe:		
Rock class	3-6t	3-6t
Crest toe	-10.5 m +CD	-10.5 m +CD
Height toe	4.5 m	4.5 m
Width toe crest	5.5 m	5.5 m
Filter:		
Stone class	10-60kg	10-60kg
Layer thickness	1 m	1 m
Length before toe	15 m	15 m
Geotextile:		
Width	109 m	139 m

5.11 Construction costs

5.11.1 Costs per quantity

Costs of rip-rap

Rip-rap is the rock quarried and used for the filters and core of the breakwater. The costs of rip-rap are composed of the costs for production in the quarry, the transportation to the site and the placement at the site.

Little information is available about the exploitation costs of the quarry, the transportation costs and the placement of the rock at the site. Assumptions are made to approximate the total costs for the construction with quarry rock. These assumptions are based on results of similar projects (Pals, 1998).

The production costs for rip-rap is assumed to be independent of the size of the rock classes and covers all activities at the quarry site, including local transportation and storage.

The rock is transported from the Balzapote quarry to the breakwater location over water. The quarry is located alongside the shore and a distance of 80 km has to be sailed. The cost of transportation consists of the costs of loading and actual transportation over water and the additional storage and re-handling near the construction site. The cost of transportation is assumed to be dependent on the rock size.

The placement cost is also dependent on the weight of the rock elements. Larger units require heavier equipment and individual handling and are consequently more expensive to place than the smaller bulk material.

The total cost per ton of rip-rap is given in Table 16. The costs are based on experiences of Redecon Nedeco Consultants (1990). Distinction is made between the costs for the water-based and the costs for the land-based alternative due to the differences in downtime, equipment and transportation method. The costs for the water-based alternative for transportation and placement are a factor 1.5 higher.

Table 16 Constitution of costs per ton of rip-rap

Rock class	Production	Transportation	Placement	Subtotal	Additive for losses	Total
Water-based	[$/t]	[$/t]	[$/t]	[$/t]	[$/t]	[$/t]
3-6t	9	12	38	59	3	62
1-1000kg	9	8	24	41	4	45
10-60kg	9	6	11	26	1	27
Land-based	[$/t]	[$/t]	[$/t]	[$/t]	[$/t]	[$/t]
3-6t	9	8	25	42	2	44
1-1000kg	9	5	16	30	3	33
10-60kg	9	4	11	26	1	27

Costs of Core-loc®

The construction cost of Core-loc® elements consists of the costs of production and placement.

The production cost of Core-loc® elements are 130 $/t. This is based on experience with a Core-loc® breakwater, with the same specific density of 2.2 t/m^3, in Tuxpan, Mexico.

The placement cost are derived from the placement rate and equipment cost per week. The placement rate of a crane is dependent on the size of the elements. The same concrete is used for large and small elements and consequently both have the same concrete strength. The larger, and heavier, elements demand a slower placement rate to avoid breakage during placement. The assumed placement rate is depicted in Figure 24. The placement rate is based on experience with the Core-loc® breakwater in Tuxpan, Mexico.

Figure 24 Placement rate as a function of element weight

The element weight of 22.7 t provides a placement rate of 3.33 elements per hour. Ten working hours per day and seven working days per week, combined with a downtime due to excessive wave height of 40% of the time and an additional downtime of 5% of the time give a placement rate of 133 elements per week per crane. With land-based construction, suffering a downtime due to excessive wave height of 10% of the time, a placement rate of 199 elements per week per crane is feasible.

The element weight, reach and additional safety determine the necessary crane capacity shown in Figure 25. The 22.7 t Core-loc® unit requires a crane with a capacity of approximately 267 ton.

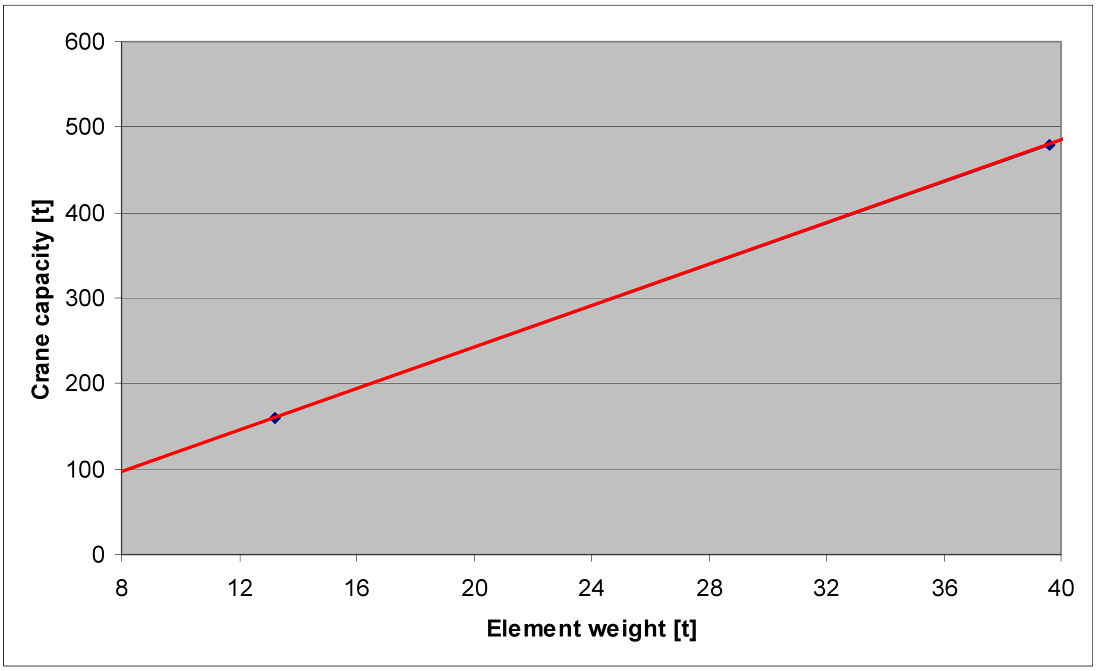

Figure 25 Crane capacity as a function of element weight

The number of 22.7 t elements used to protect the 1500 m breakwater is 9370 for the water-based breakwater and 14802 for the land-based breakwater. To be able to construct the breakwater within a year an average number of cranes of respectively 1.36 and 2.14 is required for both alternatives, taking into account the placement rate of 133 elements per week. The crane cost is dependent on the capacity of the crane as indicated in Figure 26. The crane cost includes the costs for insurance and (dis)embarkment costs. The equipment costs used in the calculations is the cost of one crane multiplied by three. This takes into account all other equipment necessary for the construction.

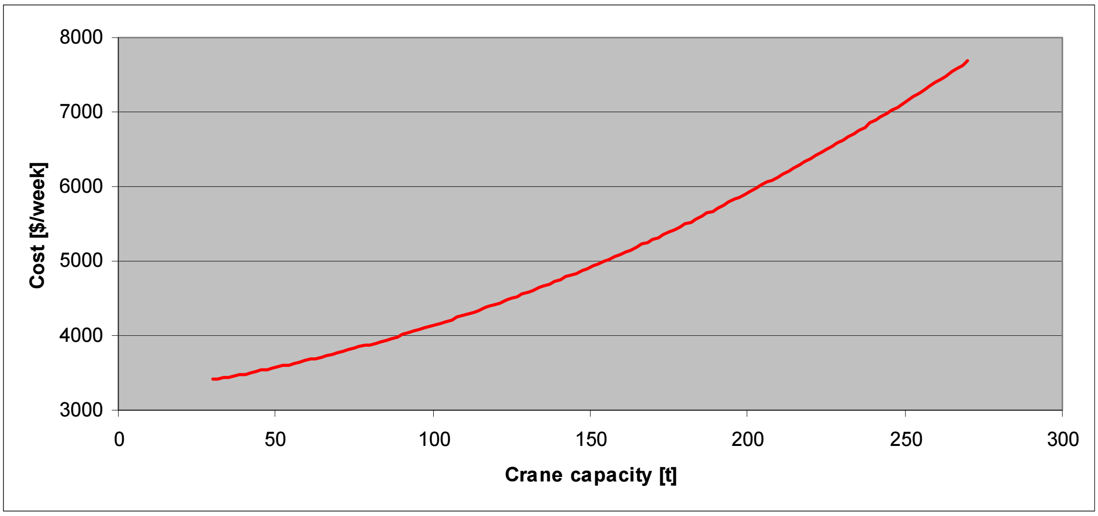

Figure 26 Crane Cost as a function of crane capacity

With the equipment costs per week and the necessary number of cranes the costs per Core-loc® and per ton of Core-loc® are calculated in Table 17.

Table 17 Constitution of costs per ton of Core-loc®

Production		Water-based	Land-based
Cost per ton	[$/t]	130	130
Placement			
Rate	[1/h]	3.33	3.33
Operative hours/day	[h]	10	10
Working days per week	[day]	7	7
Downtime reduction (40%)	[-]	0.60	0.90
Additional (5%)	[-]	0.95	0.95
No. of elements	[-]	9370	14802
Placement rate of elements	[1/week]	180	285
Rate/crane	[1/week]	133	199
No. of cranes	[-]	1.36	2.14
Equipment	[$/week]	30958	48713
Cost per Core-loc®	[$/CL]	172	171
Weight Core-loc®	[t]	22.7	22.7
Cost per ton	[$/t]	7.6	7.5
Total			
	[$/t]	138	138

Costs of geotextile

The costs of the geotextile are assumed to be 10 $/m^2.

5.11.2 Quantities

Demand of rock

From the cross-sections of the breakwater alternatives the following quantities of material per meter breakwater are derived and provided in Table 18. Also indicated are the estimated losses during quarry operations, transportation and placement, adding 5% to the demand of quarried rock. For the quarry-run, used for the core of the breakwater, an additional 5% is necessary to compensate losses due to instability during severe wave attack. In Table 18 the total demand of rock per class of rock for the 1500 m of breakwater is given.

Table 18 Quantity demand and distribution

Water-based	Per m length		Per 1500 m
	Demand	Demand	Total
Rip-rap	[m3]	[t]	[t *1000]
1-1000kg	167	515	773
3-6t	133	411	617
10-60kg	60	186	279
Total	360	1112	1668
Land-based	Per m length		Per 1500 m
	Demand	Demand	Total
Rip-rap	[m3]	[t]	[t *1000]
1-1000kg	420	1299	1949
3-6t	171	530	795
10-60kg	75	231	347
Total	666	2059	3089

Yield of quarry

The demand of rock based on the breakwater geometry is always lower than the quantity to quarry due to overburden and loss. The expected quantity, including loss of material, is provided Table 19. The losses of the 1-1000kg rock class are significantly higher than the losses of other rock classes. This is due to the instability of the core during construction determined in paragraph 5.6. These additional losses are included in the column ‘Losses’ in Table 19.

Table 19 Expected quantity demands to quarry

Water-based	Per m length					Per 1500 m
	Demand	Demand	Losses	Total	Distribution	Total
Rip-rap	[m3]	[t]	[t]	[t]	[-]	[t *1000]
1-1000kg	167	515	53	568	48%	852
3-6t	133	411	21	432	36%	647
10-60kg	60	186	9	195	16%	293
Total	360	1112	83	1195	100%	1,792
Land-based	Per m length					Per 1500 m
	Demand	Demand	Losses	Total	Distribution	Total
Rip-rap	[m3]	[t]	[t]	[t]	[-]	[t *1000]
1-1000kg	420	1299	133	1432	64%	2,148
3-6t	172	530	27	557	25%	835
10-60kg	75	231	12	243	11%	364
Total	666	2059	171	2231	100%	3,347

The Balzapote quarry sieve density curve is provided in Appendix VIII. The class of rock that exceeds the quarry yield curve the most, determines the necessary productivity of the quarry. An uneven match between the demand and yield distribution curve produces a surplus of certain rock class material. A surplus in the large stone classes could be secondary blasted to provide smaller rock classes. In the Veracruz situation a shortage of large elements exists and secondary blasting is useless. The 3-6t rock class will determine the amount of rock to be blasted in the quarry. The discrepancy between the demand and yield is elucidated in Appendix XIII. The superfluous quantity should be included in the calculation as an additional cost of $9 per ton overburden, i.e. the production costs for rock. However, the Core-loc® breakwater is not the only breakwater constructed for the port extension. The 1-3t rock class can probably be well used for the cubes breakwater in the leeside of the Core-loc® breakwater indicated in Figure 2, with an additional large amount of quarry-run for the core. As the wave attack at the location is perpendicular to the breakwater, a berm breakwater could be applied for the north-west breakwater, which is also indicated in Figure 2. As the quarry demand for these breakwaters is not known yet, a quarry optimisation is not feasible.

Demand of Core-loc®

The following formula is valid for the determination of the quantity of Core-loc® (Turk and Melby, 1997b):

$$N = \Phi \cdot V_{CL}^{-2/3} \cdot A .$$

In which,

N = number of Core-loc® elements in a layer area [-]

Φ = packing density, dependent on Core-loc® volume = 0.56 for 8.5 m^3 [-]

V_{CL} = volume of Core-loc® element [m^3]

A = area on breakwater to be protected [m^2]

The number of Core-loc®s multiplied by the weight of a Core-loc® element provides the total amount of tons of Core-loc® per meter breakwater. The results of the calculations are given in Table 20.

Table 20 Core-loc® demand

	Per m length		Per 1500 m	
	Demand	Demand	Total	Number of elements
	[m3]	[t]	[t *1000]	[-]
Water-based	62	137	206	9370
Land-based	102	224	336	14802

Demand of geotextile

The demand of geotextile depends on the width of the breakwater. The higher crest height and broader crest width necessitate for an increased amount of geotextile for the land-based breakwater.

	Per m length	Per 1500 m
	[m2]	[m2]
Water-based	109	163,500
Land-based	139	208,500

5.11.3 Costs and conclusion

The costs are derived for the two alternatives and in Table 21 the results are provided. In these costs the economic consequences of differences in downtime during construction are included, but the economic consequences of failure of the breakwater during the lifetime are neglected.

Table 21 Total breakwater costs

	Water-based	Land-based	Cost increase due to land-based construction
	[$ million]	[$ million]	[-]
Rip-rap	80.7	117.6	46%
Geotextile	1.6	2.1	28%
Core-loc®	28.4	29.7	5%
Total	110.7	149.5	35%

The water-based alternative is approximately $39 million less expensive. If more downtime had been allowed for the land-based alternative, which necessitates a lower crest height, the costs for the land-based breakwater may have been lower. However, in the calculation of the costs per ton the downtime effects on costs are included. Land-based operations have an advantage over the water-based operations because of the possibility to use the, relative small, local equipment and the greater demand for manpower. The positive benefits for the local economy are beyond the scope of this study. Based on the data available at this point the water-based construction is the most economic construction method for the breakwater and will be optimised in the probabilistic analysis.

6 Probabilistic optimisation

6.1 Background and introduction

Experiences with similar structures and model tests provided knowledge about the behaviour 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, 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. Comparison of the total costs of several design alternatives the most economic design can be selected.

Construction costs

The construction costs are dependent on the dimensions of the breakwater. In this study only the crest height (R_c) and the element weight (W) will be varied:

Construction costs = $I_0(W, R_c)$.

Qualitative consequences of these variations on the breakwater design are neglected. In reality an alternative element weight represents a complete design alternative. Therefore, besides changes in primary armour, changes in e.g. toe structure and secondary armour should have to be made as well. In principle the cost of the breakwater increases with the applied element weight and the crest height. For all investigated combinations of crest height and element weight the construction costs have to be calculated. However, the variation in element weight only has consequences for the costs of the Core-loc® layer. For the variation in crest height the necessary quantities of all components are recalculated.

Collapse and downtime costs

The first major failure event is collapse of the breakwater. Failure entails suspended harbour operations during the period of reconstruction and necessitates repair activities. Strengthening of the breakwater by applying heavier blocks and by adapting the cross-section reduces the possibility of collapse of the breakwater.

The risk of collapse is determined by multiplying the damage with the probability of collapse, which is assumed to be a function of the element weight and time. The influence of the crest height on the probability of collapse is neglected. The failure of toe or primary armour layer at the seaside is assumed to provide 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 the lifecycle of the breakwater starts.

The discounted value is calculated according to the following formula:

$$Risk_{collapse} = \sum_{t=0}^{L} \left(p_{collapse}(W,t) \cdot D(W,R_c,t) \cdot \frac{1}{(1+r)^t} \right).$$

In which,

$p_{collapse}$	= probability of collapse of the breakwater [-]
D	= costs of repair and downtime [$]
r	= discount rate [-]
t	= year in the lifecycle of the breakwater [year]
L	= lifetime of the breakwater [year]

The second major failure event occurs if the tranquillity provided by the breakwater under daily, SLS, circumstances is unsatisfactory. The tranquillity can be improved by increasing the crest height of the breakwater to reduce the overtopping.

To determine the risk for the intranquillity, the number of non-operational days has to be multiplied with the costs per day of interrupted operation:

$$Risk_{intranquillity} = \sum_{t=0}^{L} \left(p_{intranquillity}(R_c,t) \cdot N \cdot d(t) \cdot \frac{1}{(1+r)^t} \right).$$

In which,

$p_{intranquillity}$	= percentage of time functional failure due to wave transmission occurs [-]
N	= operational days per year [day]
d	= costs per day of interrupted operation [$/day]

The discounted value of the total risk over the planning period of M year:

$$Risk_{total} = \sum_{t=0}^{L} \left(\left[p_{failure}(W,t) \cdot D(W,R_c,t) + p_{intranquillity}(W,R_c,t) \cdot N \cdot d(t) \right] \cdot \frac{1}{(1+r)^t} \right).$$

Maintenance costs

The discounted maintenance costs are calculated with the following formula:

$$Maintenance = \sum_{t=0}^{L} \left(M(t) \cdot \frac{1}{(1+r)^t} \right)$$

In which,

M	= maintenance costs [$].

Total costs

The discounted value of the total costs results from the summation of the investment (I_0) and the total risk component and the maintenance costs:

$$TC = I_0(W, R_c) + \sum_{t=0}^{L}\left(\left[p_{failure}(W,t)\cdot D(W,R_c,t) + p_{intranquillity}(W,R_c,t)\cdot N\cdot d(t) + M(t)\right]\cdot\frac{1}{(1+r)^t}\right)$$

6.2 Breakwater alternatives

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

Element weight

The deterministic design provided an element weight of 22.7 ton and an element volume of 10.3 m^3. The element weights of the alternatives result from a variation of the element volume from 4 m^3 up to 18 m^3. Taking into account the specific density of concrete of 2200 kg/m^3 the element weight varies from 8.8 ton up to 39.6 ton. The evaluated volumes, nominal diameters and related weights are provided in Table 22.

Table 22 Evaluated element sizes and weights

Volume [m3]	Dn [m]	Weight [t]
4	1.59	8.8
6	1.82	13.2
8	2.00	17.6
10	2.15	22
12	2.29	26.4
14	2.41	30.8
16	2.52	35.2
18	2.62	39.6

Crest height

The crest height variation is only rational within certain boundaries. The deterministic calculation provided a crest height of 3.0 m +CD. The economic consequences of downtime were not taken into consideration in the deterministic design and a relative high percentage of downtime was allowed. Therefore, higher crest heights, from 3.0 m +CD up to 9.0 m +CD with an incremental stepsize of one meter, are evaluated in the probabilistic calculations.

Alternatives

The total number of combined alternatives to be calculated, for the eight weight sizes and seven crest heights, results in the 56 alternatives indicated in Table 23.

Table 23 Breakwater geometry alternatives

Core-loc® element volume [m3]	Crest height [m +CD] 3.0	4.0	5.0	6.0	7.0	8.0	9.0
4	1	2	3	4	5	6	7
6	8	9	10	11	12	13	14
8	15	16	17	18	19	20	21
10	22	23	24	25	26	27	28
12	29	30	31	32	33	34	35
14	36	37	38	39	40	41	42
16	43	44	45	46	47	48	49
18	50	51	52	53	54	55	56

6.3 Methodology

First, the probabilities of failure due to collapse and functional failure of the breakwater are derived. Next, the financial consequences of these failures are established. Consequently, the construction costs, the maintenance costs and the expected additional costs due to failure are summarised. A comparison is made of the alternatives to determine the economical optimal design. Finally the economical optimal designs of the various sea level rise and maintenance strategies are finally compared to determine the influence of these parameters.

7 Limit states

7.1 Introduction

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:

$$Z = R - S\,.$$

In which,

R = strength

S = 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.

7.2 Wave height

7.2.1 Translation deep to shallow water

Not depth limited

The shallow water significant wave height is described by the following formula if it is not limited by the depth:

$$H_{s,\,shore} = \gamma_r \cdot H_{s,\,o}\,.$$

In which,

$H_{s,\ shore}$ = shallow water significant wave height [m]

γ_r = energy dissipation coefficient = 0.86 [-]

$H_{s,\ 0}$ = deep water significant wave height [m]

Depth limited

The wave height is depth limited if the significant shallow water wave height, i.e. the deep water significant wave height reduced with the energy dissipation, is higher than allowed by the breaking depth. The depth is time-dependent due to the sea level rise during the lifetime of the breakwater. The depth limited wave height is therefore also time-dependent. The significant wave height allowed by the water depth is described by the following formula:

$$H_{s,\,shore}(t) = \gamma_{br} \cdot \left(\left(h_{\max} - z_{bed}\right) + SLR(t) + z_{surge}\right).$$

In which,

γ_{br} = depth breaking coefficient [-]

h_{max} = still water level (SWL) [m +CD]

z_{bed} = bed level [m +CD]

SLR = time-dependent sea level rise [m]

z_{surge} = storm surge [-]

7.2.2 Uncertainty of wave height

Nortes

The extreme wave height due to the Nortes is already described with the Gumbel distribution function in Chapter 3. The uncertainty of this wave height is yet still unknown. The uncertainty of the wave height, indicated by the wave data provider Argoss, is taken into account with the following mathematical addition:

$$H_{s,\,0} \cdot f_{Hs1} + f_{Hs2}.$$

In which,

$H_{s,\,0}$ = deep-water wave height [m]

f_{Hs1} = uncertainty parameter dependent on the wave height [-]

f_{Hs2} = uncertainty parameter independent on the wave height [m]

The f_{Hs1} is a normal distributed parameter with a mean of 1.00 and a standard deviation of 0.13. The standard variation is based on the reliability boundaries indicated by Argoss for the Veracruz wave data.

The f_{Hs2} is also a normal distributed parameter, but with a mean 0.00 m and a standard deviation of 0.15 m. The standard deviation reflects the inaccuracy in the satellite measurements and is independent on the wave height.

Hurricanes

The wave height during an intense hurricane is always depth limited, thus the water level determines the wave height. Therefore, the uncertainty of the wave height due to a hurricane is determined by the uncertainty of the storm surge due to a hurricane. In Appendix XVI the uncertainty of the storm surge is assumed to be described by a normal distribution. A mean of 3 m and a standard deviation of 1 m are assumed.

7.2.3 Independent occurrence Nortes and hurricanes

For all years during the lifetime of the breakwater the probability of collapse is calculated. The occurrence of the Gumbel distributed extreme wave heights and the occurrence of a hurricane are assumed independent. Every year the breakwater can collapse due to an extreme normal storm and due to a hurricane. Therefore, the collapse probability has to be calculated for both the normal extreme wave heights and for the hurricanes. The total collapse probability is the summation of both individual collapse probabilities.

7.3 Ultimate limit state

Failure due to collapse of the breakwater is defined as the incapability of the breakwater to perform the function of providing tranquillity in port basin.

The failure due to collapse is caused by the occurrence of one or more failure mechanisms. In Figure 27 several failure mechanisms are given.

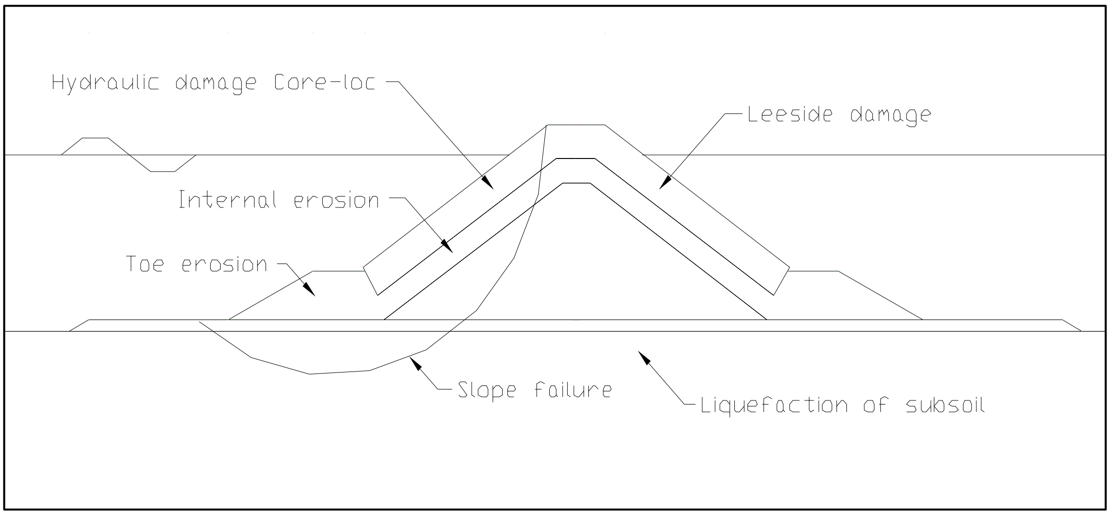

Figure 27 Breakwater failure mechanisms

For the determination of the probability of collapse of the breakwater, reliability calculations are carried out for the most important mechanisms:

- Hydraulic damage Core-loc®
- Toe erosion seaside

The other failure mechanisms are considered beyond the scope of this study.

7.3.1 Fault tree

The fault tree for the ULS conditions is given in Figure 28. The primary armour and toe failure mechanisms are indicated. Both mechanisms fail if either a hurricane or an extreme storm during the Nortes produces a wave height sufficiently high to exceed the breakwater strength.

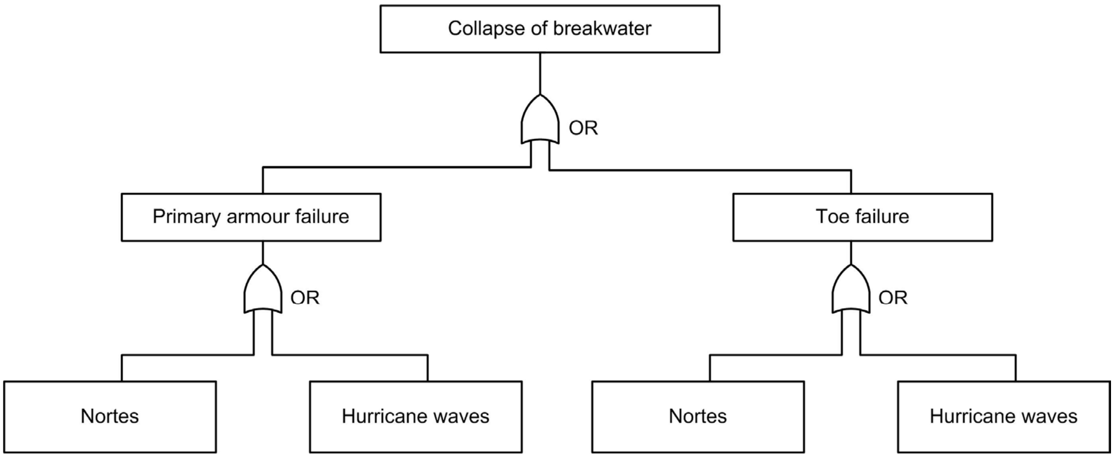

Figure 28 Fault tree for collapse of breakwater

7.3.2 Hydraulic damage Core-loc®

Failure formula

The hydraulic damage of the Core-loc® layer is the first main hazard.

As already elaborated in Chapters 4 and 5, the following formula is valid:

$$\frac{H_{s,\,shore}}{\Delta \cdot D_n} = C_{cl}$$

Several parameters in the formula are time-dependent in the probabilistic calculations and the formula is rewritten as:

$$\frac{H_{s,\,shore}(t)}{\Delta \cdot D_n} = C_{cl}(t).$$

Reliability function

Rewriting the failure formula gives the following reliability function:

$$Z(t) = C_{cl}(t)\left(\frac{\rho_c}{\rho_w} - 1\right) \cdot \sqrt[3]{V_{CL}} - H_{s,\,shore}(t).$$

In which,

V_{cl} = Core-loc® element volume [m^3]

7.3.3 Toe erosion

Failure formula

Another main hazard that may lead to failure is toe erosion. Also the probability of toe erosion is time-dependent. The failure formula describes the level of damage of the toe structure:

$$\frac{H_{s,\,shore}(t)}{\Delta \cdot D_{n50}} = \left(2 + 6.2\left(\frac{h_t(t)}{h(t)}\right)^{2.7}\right) \cdot N_{od}^{0.15}$$

Reliability function

Reformulation of the equation to the reliability function gives:

$$Z(t) = \left(2 + 6.2\left(\frac{h_t(t)}{h(t)}\right)^{2.7}\right) \cdot N_{od}^{0.15} \cdot \left(\frac{\rho_r}{\rho_w} - 1\right) \cdot D_{n50} - H_{s,\,shore}(t).$$

The storm surge during a hurricane and the sea level rise are used additionally to the variables used in the deterministic design:

h_t	= toe depth = h_{max} - z_{toe} + SLR(t) + z_{surge} [m]
h	= water depth in front of toe = h_{max} - z_{bed} + SLR + z_{surge} [m]
z_{toe}	= toe crest level [m +CD]
SLR	= time-dependent sea level rise [m]
z_{surge}	= storm surge (only in case of a hurricane) [m]
z_{bed}	= bed level [m +CD]

7.3.4 ULS variables

For each variable the distribution and values are determined, based on numerical data available for the design of a breakwater toe and expert judgement. The results are provided in Table 24.

Table 24 Input values for ULS conditions

Gumbel distributed	Shift parameter	Scale parameter
H_s, 1 year	1.544 [m]	0.979 [m]
H_s, 50 year	5.374 [m]	0.979 [m]
Normal distributed	**Mean**	**Standard deviation**
$C_{cl,0}$	Time dependent [-]	1.0 [-]
ρ_c	2200 [kg/m^3]	50 [kg/m^3]
ρ_r	3090 [kg/m^3]	300 [kg/m^3]
f_{Hs1}	1 [m]	0.13 [m]
f_{Hs2}	0 [m]	0.15 [m]
D_{n50}	1.10 [m]	0.05 [m]
h_{max}	0.31 [m +CD]	0.25 [m +CD]
z_{bed}	-16.00 [m +CD]	0.50 [m +CD]
z_{toe}	-10.50 [m +CD]	0.30 [m +CD]
z_{surge}	3 [m]	1 [m]
γ_{br}	0.45 [-]	0.02 [-]
Deterministic	**Value**	
SLR	0.00, 0.15, 0.30, 0.50 [m]	-
ρ_w	1025 [kg/m^3]	-
V_{cl}	4-16 [m^3]	-
N_{od}	0.5 [-]	-

For the nominal diameter, D_{n50}, a standard deviation is estimated and added to include the uncertainty of the quarry output. The toe crest height has a variation due to construction inaccuracies and the tide influences the still water level. The irregularities of the bed level in front of the breakwater are also taken into account. The volume of the Core-loc® elements will vary from 4 m^3 up to 16 m^3.

7.4 Serviceability limit state

Failure of the serviceability limit state is defined as the exceedence of the critical wave height in the port basin. The wave levels occurring in the sheltered port basin area originate from several causes. The most important causes are shown in Figure 29.

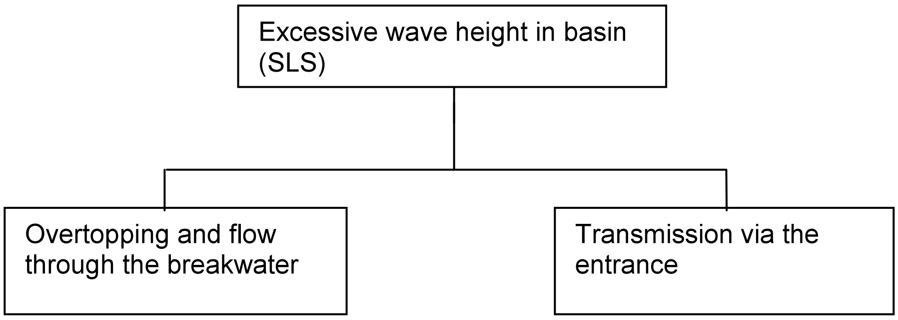

Figure 29 Wave transmission contributions

First, waves can bypass the breakwater via the harbour entrance. Second, waves overtop the breakwater transmitting wave energy into the port basin. Finally, waves can flow through the breakwater. Besides wave penetration from outside the basin also local wave generation due to wind or vessel movements can contribute to the wave energy in the basin, yet the contribution of the local generated waves is assumed negligible compared to the external contributions.

Failure formula

The transmitted wave energy is proportionally with the quadratic transmitted wave height. This proportionality is elaborated in Appendix XV. The total transmitted wave height into the basin is described by the following formula:

$$H_{port\ basin}(t) = \sqrt{(H_{entrance}(t))^2 + (H_{transmission}(t))^2} = \sqrt{(K_{entrance} \cdot H_{s,\ shore}(t))^2 + (K_{transmission}(t) \cdot H_{s,\ shore}(t))^2}$$

The formula is similar to the one used for the deterministic design, only the wave heights are time-dependent in the probabilistic calculations.

The transmission due to overtopping over and flow through a Core-loc® breakwater is also described by the same conditional formulas of Melito and Melby (2002) as in the deterministic design and is valid for the range of submerged up to high crested breakwaters:

For $R_c / H_{s,\,shore} < -1.0$ $\quad K_{transmission} = 0.95$

For $-1.0 < R_c / H_{s,\,shore} < 1.3$ $\quad K_{transmission} = 0.56 - 0.39 \cdot R_c / H_{s,\,shore}$

For $R_c / H_{s,\,shore} > 1.3$ $\quad K_{transmission} = 0.05$

R_c = crest height above still water level = R_{top} - h_{max} - SLR - z_{surge} [m]

R_{top} = crest level [m +CD]

h_{max} = still water level (SWL) [m +CD]

SLR = time-dependent sea level rise [m]

z_{surge} = storm surge (only in case of a hurricane) [m]

Melito and Melby also indicate a standard deviation of 0.07 for the $K_{transmission}$ based on the results of model tests.

Reliability function

The reliability function is a conditional reliability function due to the conditional transmission formulas:

$$Z(t) = H_{allowed} - \left(H_i \cdot f_{Hs1} + f_{Hs2}\right) \cdot \sqrt{\left(K_{entrance}\right)^2 + \left(K_{transmission} + f_{Ktransmssion}\right)^2} \ .$$

For $R_c / H_i < -1.0$ $\quad K_{transmission} = 0.95$

For $-1.0 < R_c / H_i < 1.3$ $\quad K_{transmission} = 0.56 - 0.39 \cdot R_c / H_i$

For $R_c / H_i > 1.3$ $\quad K_{transmission} = 0.05$

In which,

$H_{allowed}$ = maximum allowable wave height in the port basin = 0.75 [m]

R_c = crest height above still water level = R_{top} – h_{max} [m]

h_{max} = still water level (SWL) [m +CD]

R_{top} = crest level [m +CD]

$f_{Ktransmission}$ = uncertainty of $K_{transmission}$ [-]

7.4.1 SLS variables

The variables occurring in the reliability formula are described in Table 25.

Table 25 Input variables for the wave transmission

Weibull distributed	Shift parameter	Scale parameter	Shape parameter
$H_{s, shore}$	0.395 [m]	0.851 [m]	0.938 [-]
Normal distributed	**Mean**	**Standard deviation**	
$K_{entrance}$	0.08 [-]	0.02 [-]	-
$f_{Ktransmssion}$	0 [-]	0.14 [-]	-
h_{max}	0.31 [m +CD]	0.25 [m +CD]	-
z_{bed}	-16.00 [m +CD]	0.50 [m +CD]	-
f_{Hs1}	1 [m]	0.13 [m]	-
f_{Hs2}	0 [m]	0.15 [m]	-
γ_{br}	0.45 [-]	0.02 [-]	-
Deterministic	**Value**		
SLR	0.00, 0.15, 0.30, 0.50 [m]	-	-
R_{top}	3-9 [m +CD]	-	-
$H_{allowed}$	0.75 [m]	-	-

The crest height is given in the table as 4.0 m +CD and this value will also be replaced by the other element sizes in the different calculations. However, deviations occur in the constructed crest height. The standard deviation of the construction accuracy is estimated at 0.5 m.

The still water level distribution is described with the tide level distribution.

The transmission coefficient for the wave intrusion via the port entrance, resulting from the diffraction calculations of API (2001a), contains an uncertainty as well. The accuracy of the calculations is estimated with a variation coefficient of 20%. This leads to a standard deviation of 0.02.

The transmission formula of Melito and Melby (2002) fits test results with a standard deviation of 0.07. However, the test circumstances in a two-dimensional flume do not fully represent the more complex three-dimensional situation in practice. This uncertainty is taken into account by taking two times the test deviation in the probabilistic calculations, i.e. a standard deviation of 0.14.

The uncertainty of the accuracy of the wave height is also included, similar to the uncertainty of the ULS wave conditions and is assumed to be a normally distributed with a standard deviation of 0.13. This is deducted from the reliability data provided by Argoss (see Appendix IV for more information about ARGOSS).

8 Probabilities of failure

8.1 Introduction

The probabilities of failure are calculated with the computer program ‘Probmod’. This program is written in the computer language FORTRAN. In this program reliability functions and parameters with their distributions can be entered. The program has the possibility to calculate the failure probabilities with a level II FORM analysis or a level III Monte Carlo analysis. In this study the Monte Carlo analysis is used to provide the probabilities. It was not possible to conduct the level II analysis due to the conditional breaking of waves and the conditional occurrence of hurricanes. All scenarios are calculated for all variations in crest height and Core-loc® element sizes. The listings of the programs are given in Appendix XVII and XVIII.

The accuracy of the method is determined by the number of simulations. To limit the number of simulations a relative error of 0.01 within the 95% confidence interval is accepted.

8.2 Probability of collapse

8.2.1 Introduction

The sum of the independent probabilities of collapse of the breakwater components, Core-loc® armour and toe structure, and the wave origins, Nortes and hurricanes, 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.

8.2.2 Probability of collapse by component

The probability of collapse is composed of the probability of failure of the Core-loc® armour layer and the probability of failure of the toe structure. The probability of collapse is dependent of the time and of the element volume. In Figure 30 the influence of the deterioration of the Core-loc® strength is clearly visible. The absolute increase of the probability of collapse is larger for smaller elements.

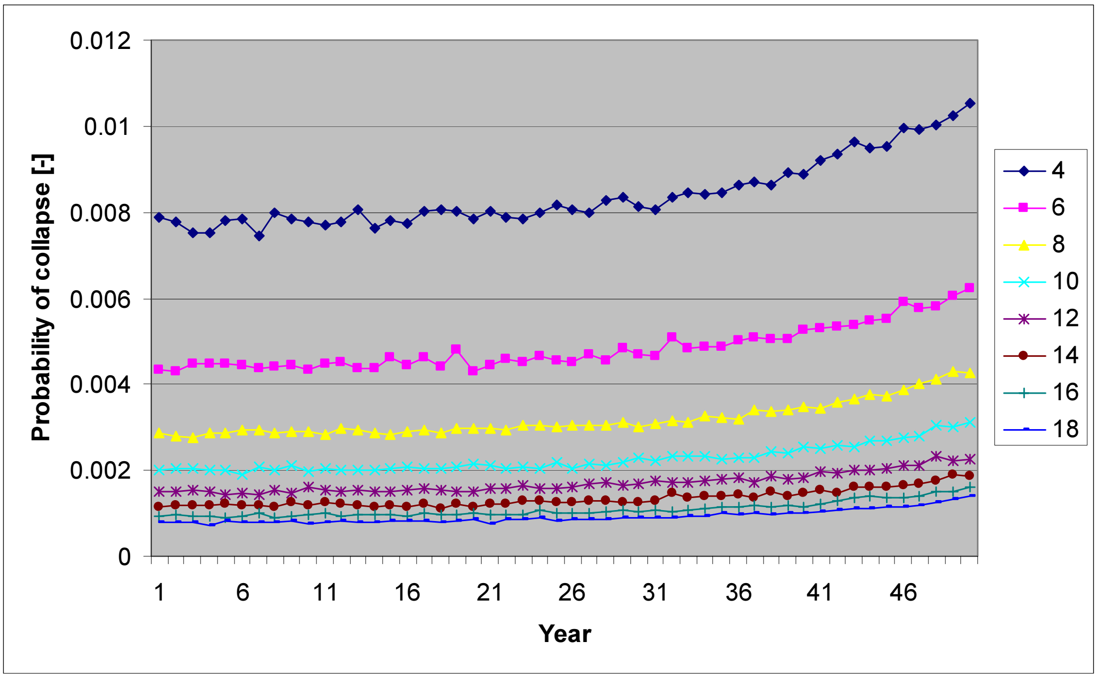

Figure 30 Probability of collapse due to Core-loc® armour failure for all element volumes [m^3]

The probability of collapse for the toe structure is depicted in Figure 31. The probability of collapse for the toe structure is independent of the Core-loc® element volume.

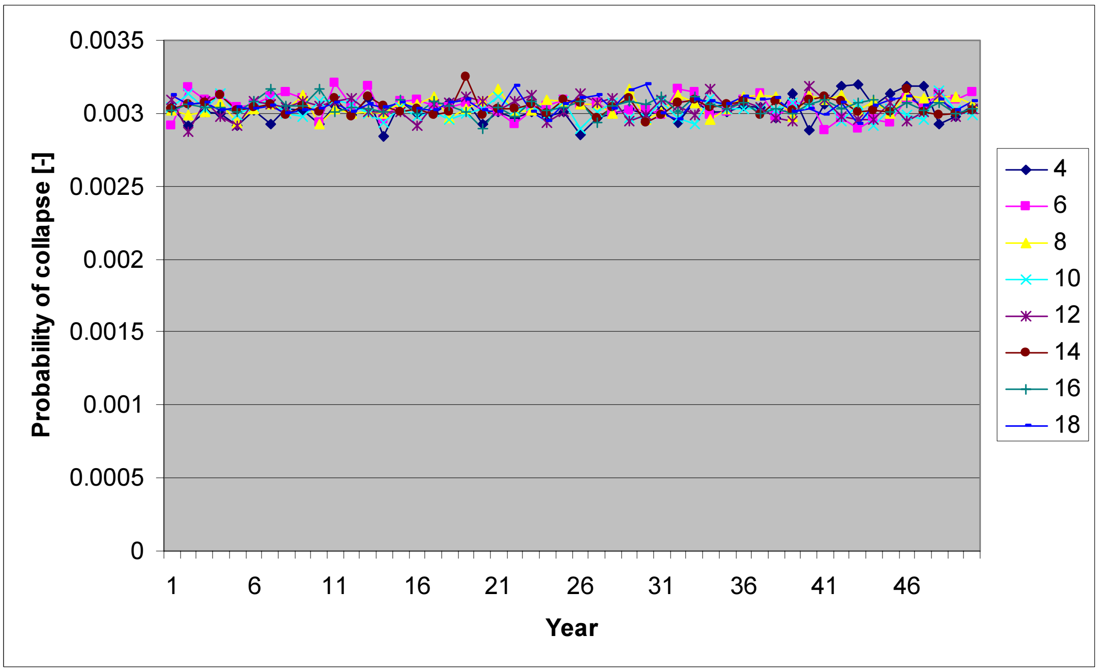

Figure 31 Probability of collapse due to toe structure failure for all element volumes [m^3]

For small elements the probability of collapse Core-loc® armour for is twice as much as for the toe structure, but for larger elements the probability of collapse is half of the probability of collapse of the toe structure. Generally the probabilities are of the same order of magnitude.

8.2.3 Probability of collapse by wave origin

The probability of collapse for the total system can also be split up into a contribution due to the Nortes and a contribution due to the occurrence of hurricanes. The probability of collapse due to the Nortes and hurricanes are given in respectively Figure 32 and Figure 33. Both probabilities of collapse are of approximately the same order. The probability of occurrence of hurricanes, determined in Appendix XVI, is of substantial influence on the system probability of collapse. If the actual hurricane occurrence would be lower than the estimated occurrence, the probability of collapse of the breakwater could be decreased up to half of the current probability.

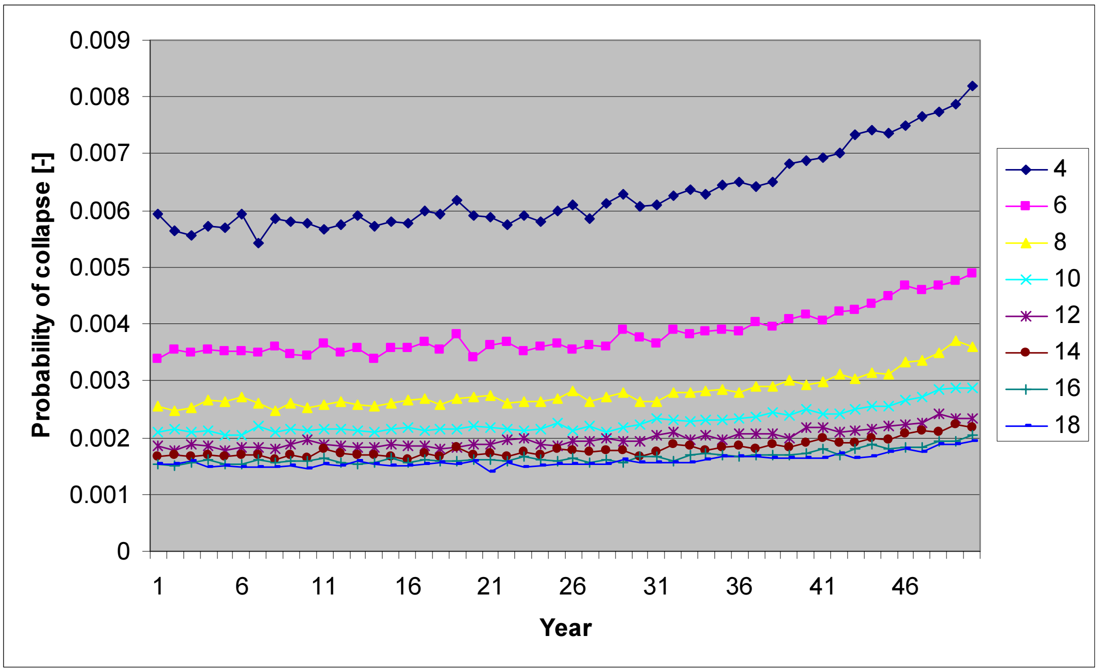

Figure 32 Probability of collapse due to Nortes for all element volumes [m^3]

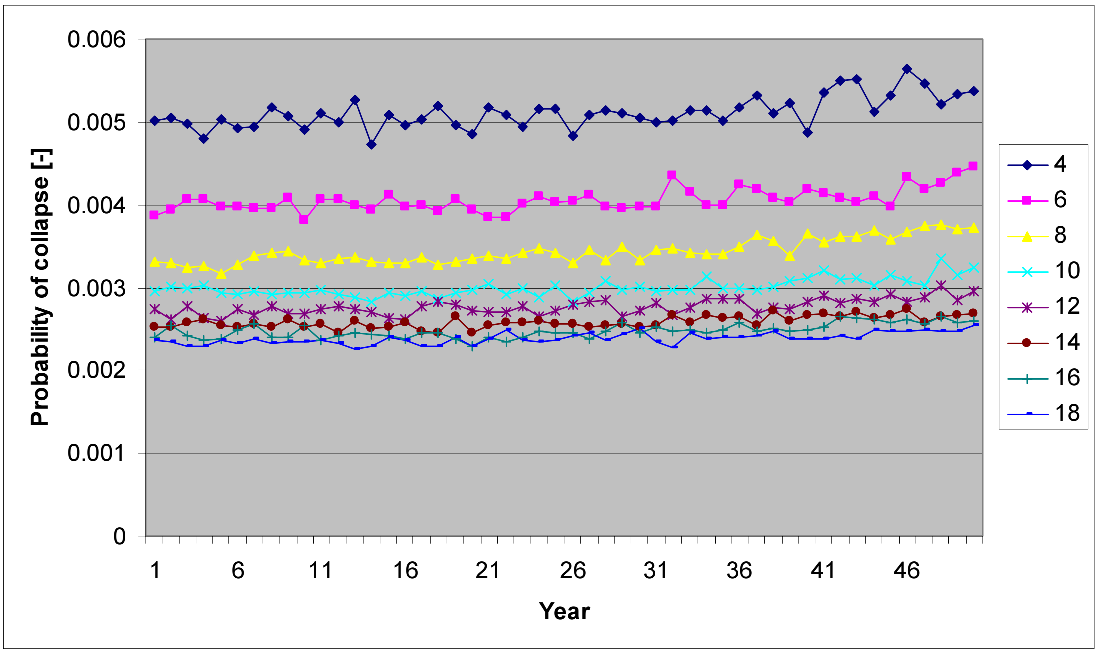

Figure 33 Probability of collapse due to hurricanes for all element volumes [m^3]

The probability of collapse due to hurricanes seems not substantially influenced by the deterioration of the Core-loc® strength as no significant increase of the probability of collapse is visible over the lifetime. If a hurricane occurs, collapse of the breakwater is likely. If the breakwater is already collapsing without deterioration, additional weakening of the Core-loc® armour due to deterioration will not increase the probability of collapse significantly.

8.2.4 Probability of collapse of the breakwater

The probabilities of collapse for the breakwater are provided in Figure 34.

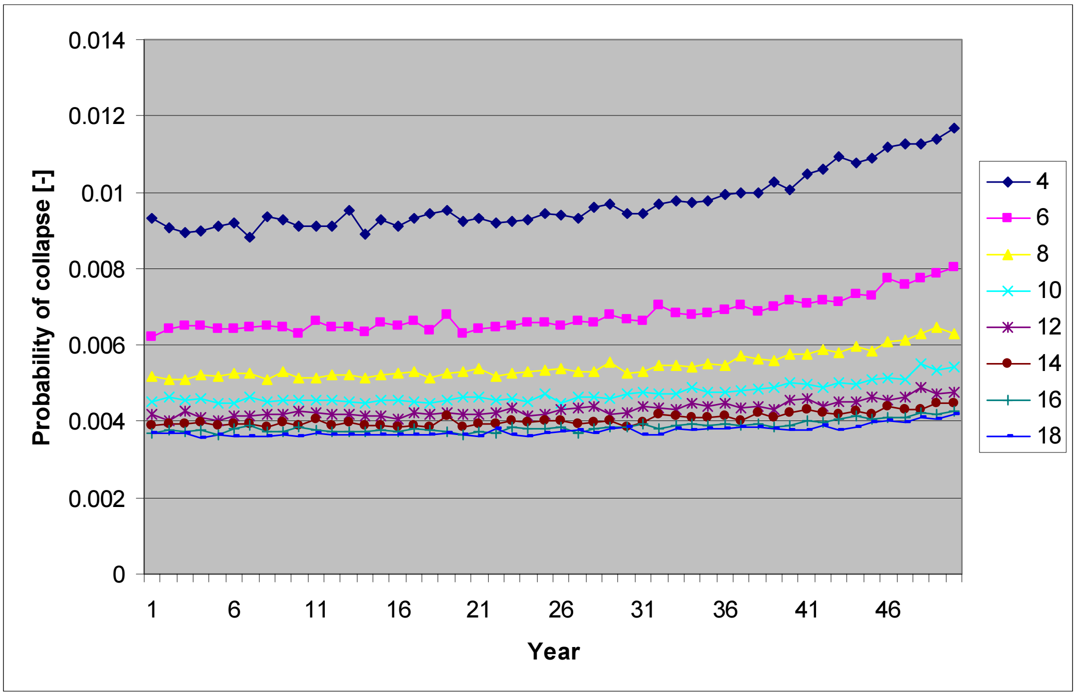

Figure 34 Probability of collapse for the breakwater for all element volumes [m^3]

8.2.5 Sea level rise

Without deterioration of the Core-loc® armour strength the probability of collapse due to the Core-loc® armour is constant over the years, if the influence of the sea level rise is negligible. In Figure 35 the probability of collapse for Core-loc® armour without deterioration of strength is provided. The probability seems to be almost independent of the sea level rise of 0.15m. The influence of sea level rise on the probability of collapse of Core-loc® armour is therefore assumed to be negligible. However, with a sufficient large sea level rise the depth limited waves will increase substantial and the sea level rise will possibly not be negligible.

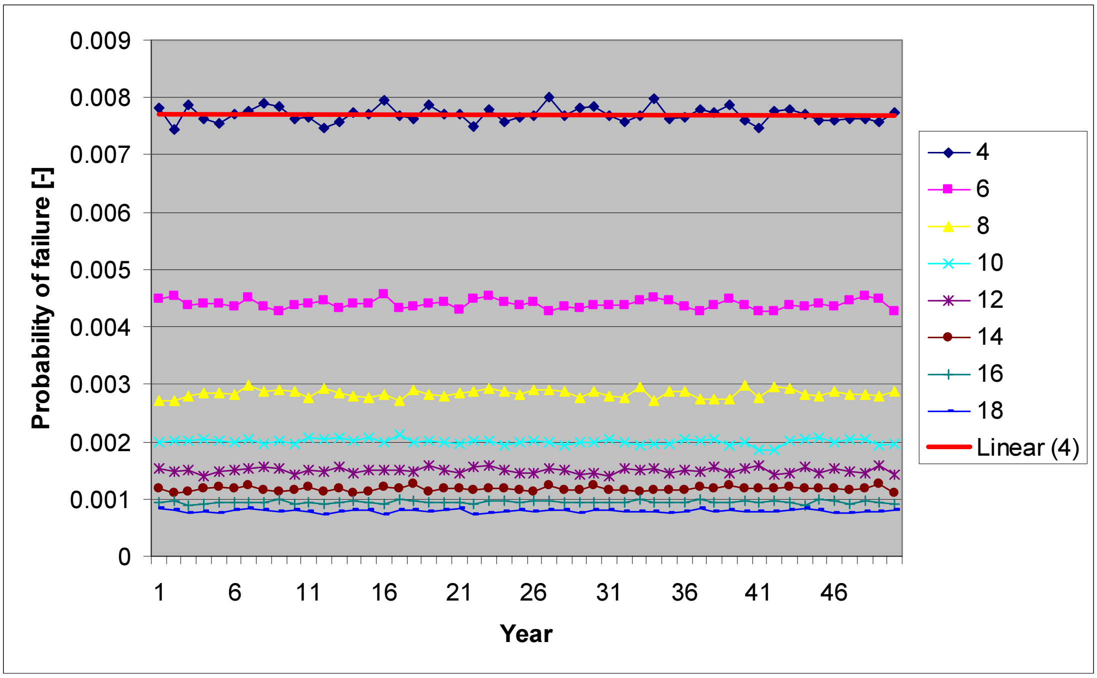

Figure 35 Probability of collapse for Core-loc® armour without deterioration of strength for all element volumes [m^3]

The sea level rise has no significant influence on the probability of collapse of the Core-loc® armour, but the probability of collapse of the toe structure could decreases with an increasing still water level. Due to the relative larger depth of the toe structure the wave action at the toe depth reduces. The depth limited waves can be larger, but not all waves are depth limited. Because reduction of the load is applied to all waves the probability of collapse of the toe structure can reduce. However, in Figure 31 no substantial reduction of the probability of the toe structure can be observed.

8.2.6 Maintenance strategy

The maintenance strategy results in a reset of the deteriorating Core-loc® armour strength to the initial strength at the beginning of the lifetime of the breakwater. The collapse probability of the toe structure is not influenced by this measurement. However, the accuracy of the Monte Carlo analysis conducted, is not sufficient to shown the effects in the first ten years of the deterioration. Due to the progressiveness of the deterioration the deterioration effects without maintenance are visible in Figure 30 for the second half of the lifetime. The maintenance strategy corrects the deterioration before significant damage is done. A more accurate calculation would provide a probability development in time as given in Figure 36.

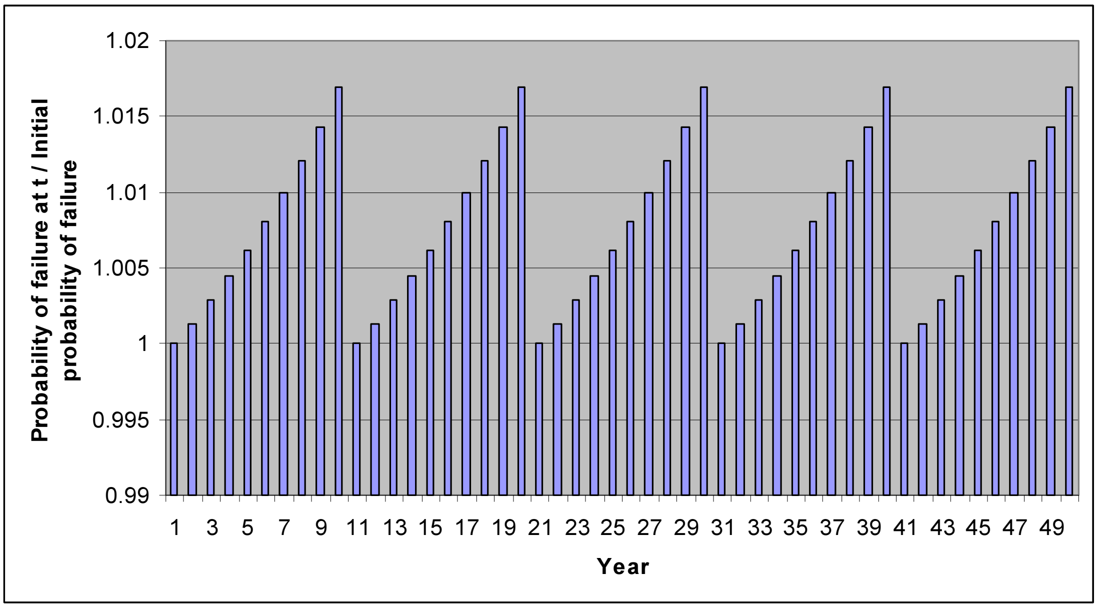

Figure 36 Fictitious probability of collapse for Core-loc® armour

8.2.7 Dependency of subsequent years

The wave height distribution and hurricane occurrence probability can be translated from their 1 year probability to a 50 year probability for the scenario without sea level rise and deterioration of the Core-loc® armour. Therefore, instead of the probability of collapse per year also the probabilities of collapse per 50 year can be calculated with the 50 year wave distribution and a 22% occurrence probability for a hurricane. The results are compared in Table 26.

Table 26 Probability of collapse for a 1 year and a 50 year period

Element volume	Observed period		
[m^3]	1 year	50 x 1 year	50 year
4	9.13E-03	4.56E-01	2.72E-01
6	6.39E-03	3.20E-01	2.11E-01
8	5.17E-03	2.58E-01	1.80E-01
10	4.53E-03	2.26E-01	1.63E-01
12	4.15E-03	2.08E-01	1.52E-01
14	3.90E-03	1.95E-01	1.45E-01
16	3.73E-03	1.87E-01	1.40E-01
18	3.63E-03	1.81E-01	1.37E-01

The probability of collapse in one year is of a different order of magnitude than the probability of collapse in 50 years. The uncertainty in the strength of the Core-loc® armour and toe structure is not high enough to make the subsequent years dependent. On the contrary, the high uncertainty in the wave height makes the subsequent years almost completely independent.

8.3 Probability of functional failure

8.3.1 Introduction

The probability of functional failure represents the fraction of time downtime occurs. The downtime is dependent on the crest height and the sea level rise. The sea level rise is time-dependent. For every year during the lifetime the downtime has a different value for every crest height, unless the sea level rise is assumed zero.

The crest height is varied from 3m +CD up to 9m +CD with incremental steps of one meter. The sea level rise is evaluated for linear increases of the still water level with respectively 0.00m, 0.15m, 0.30m and 0.50m per 50 years. The number of years is equal to the lifetime of the breakwater and is 50 years.

All calculated downtimes are discussed in this chapter. However, due to the long calculation times of the probabilities of collapse only the results of the scenarios without and with a sea level rise of 0.15m will be evaluated in the economic optimisation in Chapter 10.

8.3.2 Crest height

The effects of crest height variation on the downtime, for a sea level rise of 0.15m per 50 years, are given in Figure 37. An increase of the crest height of the breakwater decreases the transmission of wave energy into the port basin, which results in decrease of the downtime.

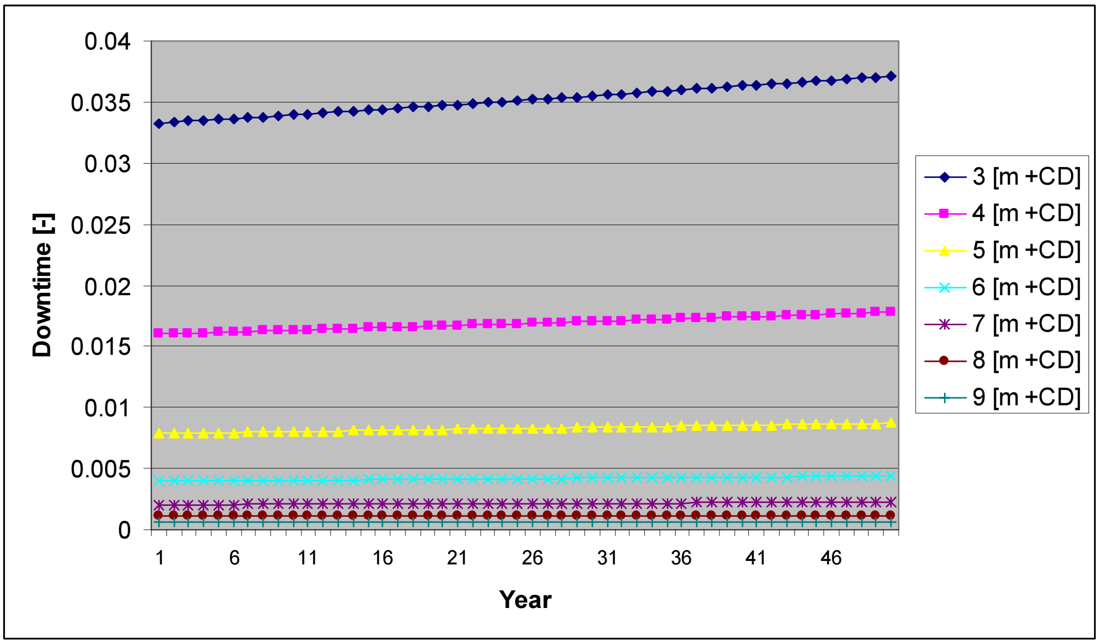

Figure 37 Influence of crest height on downtime for a sea level rise of 0.15 m per 50 year

8.3.3 Sea level rise

The influence of the sea level rise on the downtime is investigated for a sea level rise up to 0.50m per 50 year. In Figure 38 the downtime is given for a crest height of 6m +CD. The linear sea level rise results in a linear increase of the downtime level in the port basin. The results for the other observed crest heights show the same linear behaviour.

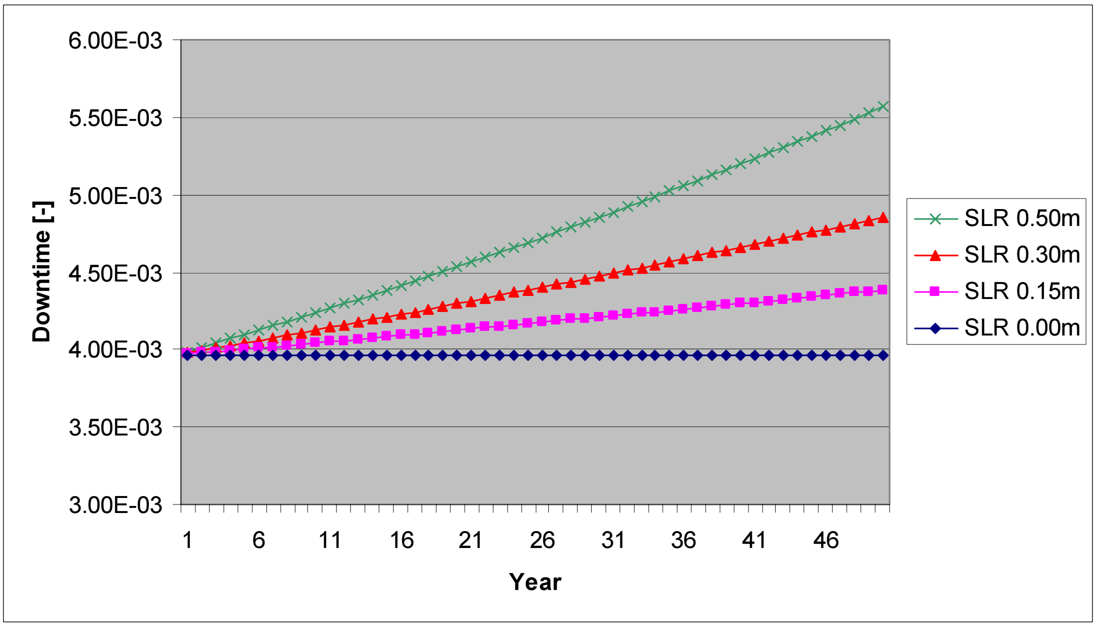

Figure 38 Influence of sea level rise on downtime for a crest height of 6m +CD

However, the lower crest heights show a relatively larger increase of downtime compared to the higher crest height. This is indicated in Figure 39. An absolute increase of the sea level gives a relative larger reduction for the lower crest heights. Because the downtime is already larger for the lower crest heights the absolute increase of downtime is even larger for the lower crest heights.

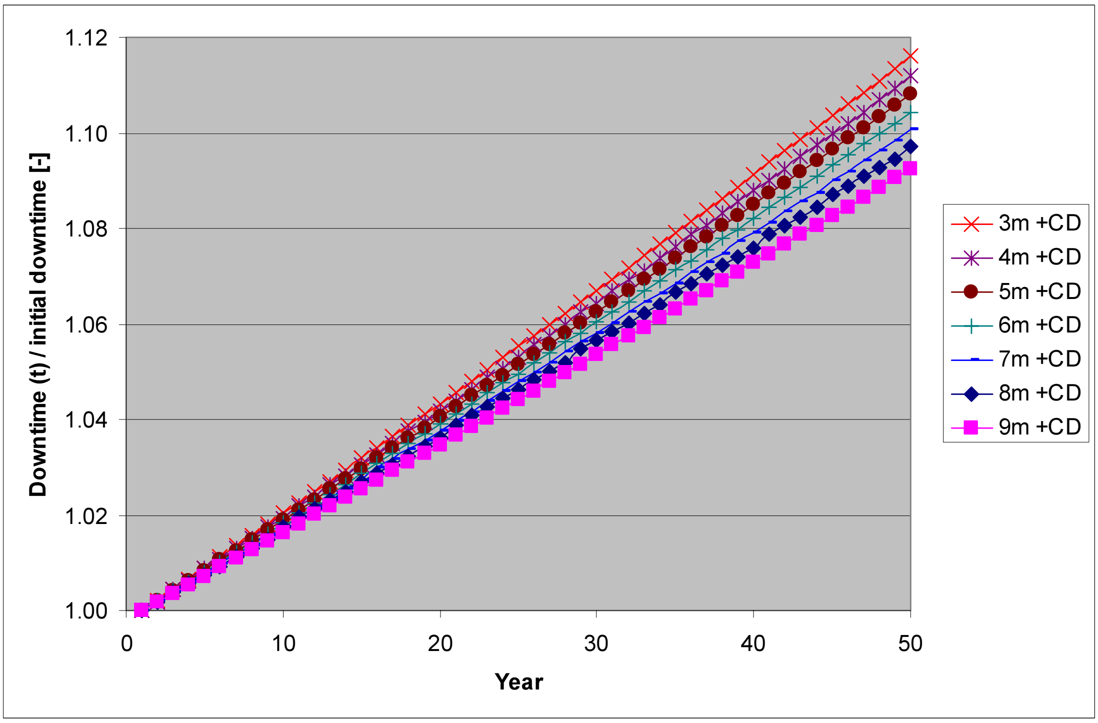

Figure 39 Relative downtime for all crest heights for a sea level rise of 0.15m +CD

For the scenario without sea level rise and deterioration of the Core-loc® armour the downtime due to collapse of the breakwater and due to functional failure can be compared. In Table 27 the downtimes are provided. It can be observed that for a crest height of 7m +CD most downtime will be caused by collapse of the breakwater.

Table 27 Downtime due to collapse and functional failure for a crest height of 7m +CD

	Downtime due to collapse		Downtime due to functional failure	
Element volume	Probability of failure per year	Downtime	Probability of functional failure per year	Downtime
[m3]	[-]	[day/year]	[-]	[day/year]
4	9.13E-03	3.33	2.04E-03	0.75
6	6.39E-03	2.33	2.04E-03	0.75
8	5.17E-03	1.89	2.04E-03	0.75
10	4.53E-03	1.65	2.04E-03	0.75
12	4.15E-03	1.52	2.04E-03	0.75
14	3.90E-03	1.42	2.04E-03	0.75
16	3.73E-03	1.36	2.04E-03	0.75
18	3.63E-03	1.32	2.04E-03	0.75

9 Cost quantification

9.1 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 considerably. In this study the real interest rate is assumed to be five percent and is held constant during the lifetime of the breakwater.

9.2 Construction costs

The costs are calculated analogous to the procedure followed in Chapter 4. A summary of the construction costs of the alternatives is given in Table 28 and are also graphically shown in Figure 40. E.g., the construction costs of the alternative with a weight of 10 m^3 and a crest height of 6m +CD is found by following the dashed lines over the grid.

Table 28 Construction costs of the alternatives in $ million

Element		Crest height						
Volume	Weight	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]
[m3]	[t]	3	4	5	6	7	8	9
4.0	8.8	104.1	112.6	121.5	130.8	140.5	150.6	161.0
6.0	13.2	106.6	115.3	124.4	133.9	143.8	154.0	164.6
8.0	17.6	108.6	117.4	126.7	136.3	146.3	156.7	167.5
10.0	22.0	110.7	119.7	129.1	138.9	149.1	159.6	170.5
12.0	26.4	112.7	121.9	131.4	141.3	151.6	162.3	173.4
14.0	30.8	114.1	123.3	133.0	143.0	153.4	164.2	175.3
16.0	35.2	116.0	125.4	135.2	145.3	155.9	166.8	178.1
18.0	39.6	118.0	127.5	137.5	147.8	158.4	169.5	180.9

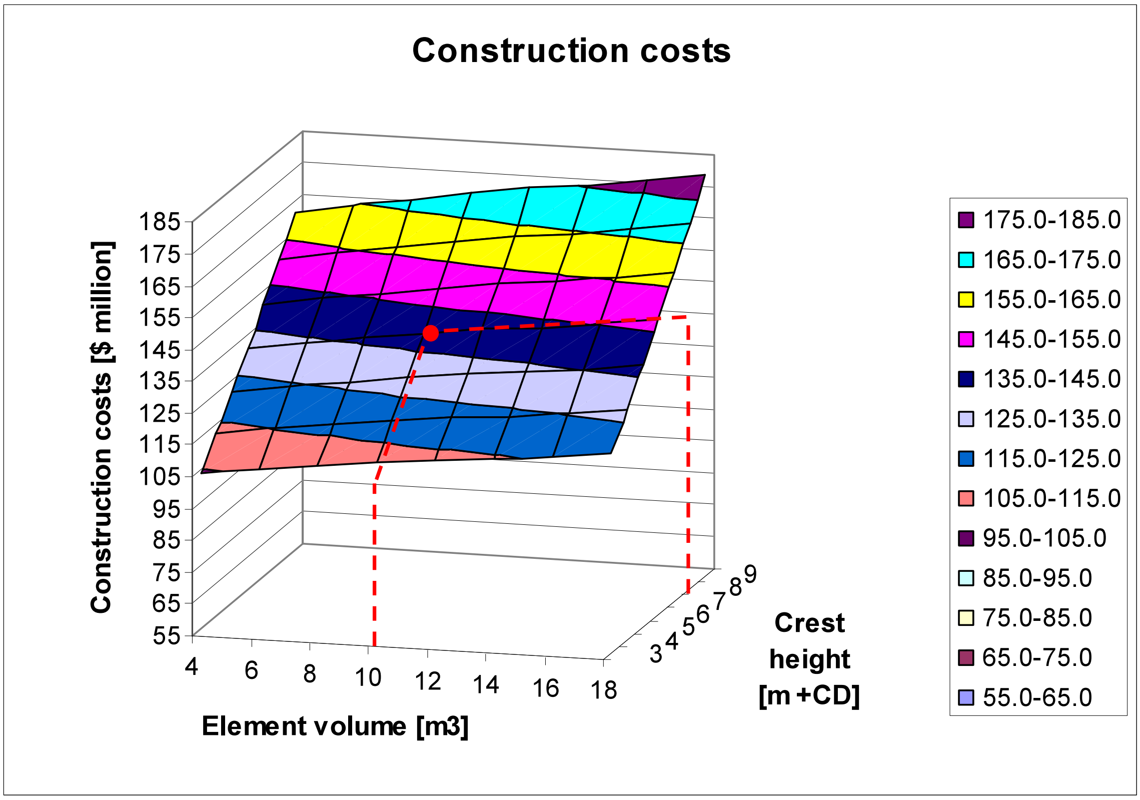

Figure 40 Construction costs of the alternatives

The alternative with the lowest considered crest height and lowest element weight is alternative with the lowest construction costs: 104.1 $ million. These costs can be considered as the initial construction costs (I_0) to be made, independent of the variation of crest height and Core-loc® 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 = $I_0(W, R_c)$.

In Vrijling (1998) these additional costs are linearised for the weight variation and the variation of crest height. Examining Figure 40, 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 Table 29.

Table 29 Derivatives of construction costs

Element Volume	Weight	Crest height [m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	I Rc
[m3]	[t]	3	4	5	6	7	8	9	[$ million/m]
4.0	8.8	104.1	112.6	121.5	130.8	140.5	150.6	161.0	9.49
6.0	13.2	106.6	115.3	124.4	133.9	143.8	154.0	164.6	9.67
8.0	17.6	108.6	117.4	126.7	136.3	146.3	156.7	167.5	9.82
10.0	22.0	110.7	119.7	129.1	138.9	149.1	159.6	170.5	9.97
12.0	26.4	112.7	121.9	131.4	141.3	151.6	162.3	173.4	10.11
14.0	30.8	114.1	123.3	133.0	143.0	153.4	164.2	175.3	10.21
16.0	35.2	116.0	125.4	135.2	145.3	155.9	166.8	178.1	10.35
18.0	39.6	118.0	127.5	137.5	147.8	158.4	169.5	180.9	10.49
Iw [$ million/t]		0.44	0.47	0.50	0.54	0.57	0.60	0.63	

The derivative for the construction costs due to the variation of the crest height is significantly influenced by the element weight and visa 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.

9.3 Maintenance

Part of the optimisation of the costs of the breakwater is the maintenance strategy. This strategy should depend on the initial construction strength and deterioration rate. The deterioration rate determines the resulting increasing probability of collapse. Optimisation of the total lifecycle costs of the breakwater should be a balance between the maintenance costs and the financial benefits of the lower probability of collapse. The maintenance costs of a breakwater are assumed to be two percent of the initial construction costs.

9.4 Downtime costs

Part of the downtime costs consists of the missed benefits that would be generated if the port would be operating normally. The missed benefits are dependent on the average throughput per day and the port dues demanded. The throughput per year of the port is assumed to increase with a percentage each year. The container throughput is known for 2001 and is 543,000 TEU per year (API, 2001b). A forecast study by APIVER (API, 2002) predicts the throughputs given in Table 30.

Table 30 Throughput forecast APIVER

Year	Throughput [*1000 TEU]
2001	543
2005	1650
2015	3410
2025	4510

However, the data of APIVER does not take into account several important issues. The first neglected issue is the rapidly increasing number of well-equipped Mexican ports. The second is the construction of a new railway between the United States and Mexico, which will bring additional competition. Additional factors, e.g. the construction of a highway from the economic centre of Mexico, Mexico-City, to the nearby competitor, the port of Tuxpan, will also influence the throughput of the Veracruz port. A thorough study to make a reliable forecast is beyond the scope of this thesis and a simplification will therefore be applied. A growth rate of 5% per year is assumed to forecast the throughput during the lifetime of the breakwater and is shown in Figure 41.

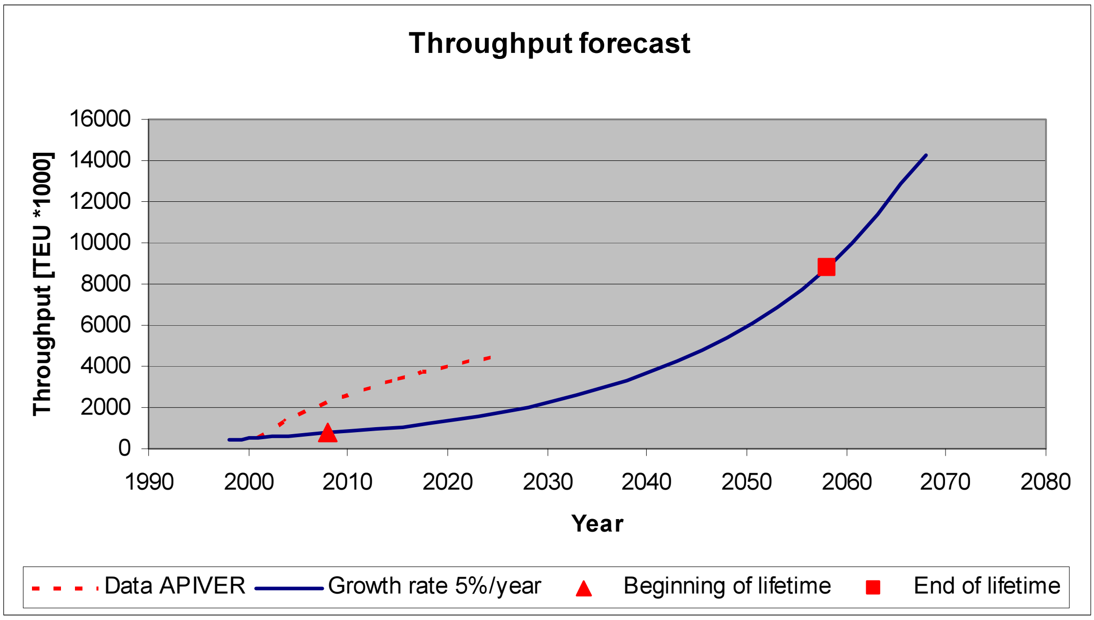

Figure 41 Throughput forecast

The start of the lifecycle of the breakwater is assumed to be in the beginning of the year 2008. The composition of the suspension cost per day, for the year 2008, is given in Table 31. The container throughput is given in TEU (Twenty feet Equivalent Unit). A throughput in TEU provides the total number of 20ft containers as the total container volume would be consisting solely of 20ft containers. All containers contribute to the total throughput according to size compared to a 20ft container.

An estimation of worldwide port dues per TEU is given by Welters (2002) and amounts a 150 $/TEU. In Ashar (2001) a comparison is made between port dues in ports in Colombia and in the Caribbean. Newly privatised ports in Colombia have an average due of 134 $/TEU and the port of Miami has a due of 111 $/TEU. The fierce competition between the recently privatised ports of Mexico necessitates a competitive pricing strategy. Therefore, a port due of 115 $/TEU is assumed for the port of Veracruz.

Part of the downtime costs are dependent on the throughput. Therefore, the downtime costs are to be calculated separately for every year in the lifecycle period due to the economic development of the port over the years. Vrijling (1998) gives a multiplier 1.5 for the indirect economic damage.

Table 31 Suspension cost per day for the year 2008

Item	Description	[$/day]
Loss of income, direct	Throughput: 764,000 [TEU/ year]	240,730
	Port dues: 115 [$/TEU]	
Loss of income, indirect	Damage to reputation/ day	140,000
	Terminals, shipping lines, other parties	50,000
	Subtotal	430,730
Indirect economic damage	Multiplier 1,5	
	Total suspension damage	646,095

9.5 Collapse cost

In case of major damage to the breakwater, the damage costs consist of the structural damage and of economic damage during the time for repair of the breakwater (Vrijling, 1998). The time necessary for repair is assumed to be one year. The structural damage is dependent on the initial construction costs of each alternative and the economic damage is dependent on the throughput development. For each year and each alternative the collapse costs are to be calculated independently. As an example the collapse costs of alternative 1 (element weight 8.8t and crest height 3.0 m +CD) in the year 2008 are provided in Table 32.

Table 32 Collapse costs for alternative 1 in the year 2008

Item	Description	[$ million]
STRUCTURAL DAMAGE		
Damage to breakwater	20% of construction cost	21
Damage to other structures in port	Terminals, slope protection, harbour lights	5
Mobilisation of contractor	Lump sum	4
	Subtotal	
ECONOMIC DAMAGE		
Loss of income, direct	Throughput: 764,000 [TEU/ year]	88
	Port dues: 115 [$/TEU]	
Loss off income, indirect	Damage to reputation	50
Loss of lives	< 10, economic damage negligible	-
Claims	Terminals, shipping lines, other parties	100
	Subtotal	247
Indirect economic damage	Multiplier 1,5	
	Total structural and economic damage	370

10 Results

10.1 Introduction

First the economic optimal design is given for the same conditions the deterministic design is calculated for. Subsequently, the probabilities of failure are calculated for several different scenarios with varying sea level rise and the application of deterioration of the Core-loc® armour. Also the effects of a maintenance strategy are included in a scenario.

Sea level rise

The sea level rise is assumed to rise linear during the lifetime from zero up to the level of 0.15m at the end of the lifetime. To evaluate the effects of the sea level rise also a calculation is made without a sea level rise.

Core-loc® deterioration

The deterioration of the strength of the Core-loc® armour, due to decrease of the stability of the Core-loc® elements, is included in the calculation of probability of collapse. To investigate the influence of this deterioration the also an optimisation without deterioration is conducted.

Maintenance strategy

Maintenance is simplified as a reset of the deteriorating Core-loc® armour strength to the initial strength at the start of the lifetime. With the assumed breakage rate 0.1% of the legs is broken in the tenth year of the lifetime. Replacement of elements can prove to be difficult, especially replacement of the elements under water level. The replacement of all broken elements is assumed to be possible at the costs of 2% of the construction costs of the breakwater. Two maintenance strategies are observed: no maintenance and maintenance with a return period of ten years.

Economic development

The development of the throughput of the port of Veracruz is uncertain. A growth rate of five percent per year is used in the design. With the determination of the optimum designs resulting from a growth rate ranging from 0% to 9% per year, the influence of the economic development on the economic optimal design of the port of Veracruz is evaluated.

Discount rate

The discount rate is dependent on the availability of assets to the port authority of the port of Veracruz. The more difficult and expensive it is to assemble assets the higher the discount rate. The discount rate will be varied from 0% to 10% to give an indication of the influence of the discount rate on the optimal design geometry.

10.2 Economic optimal design

The breakwater is optimised for the conditions used in the deterministic design: 0.15m sea level rise, the occurrence of deterioration of the Core-loc® armour strength and no maintenance.

10.2.1 Collapse due to Nortes and hurricanes

For the situation in which both the Nortes and the hurricanes can cause damage to the breakwater Figure 42 shows the total discounted costs over the lifecycle of the breakwater and in Table 33 the total discounted costs for all observed alternatives are given. The axis for the crest height is inverted compared to Figure 40.

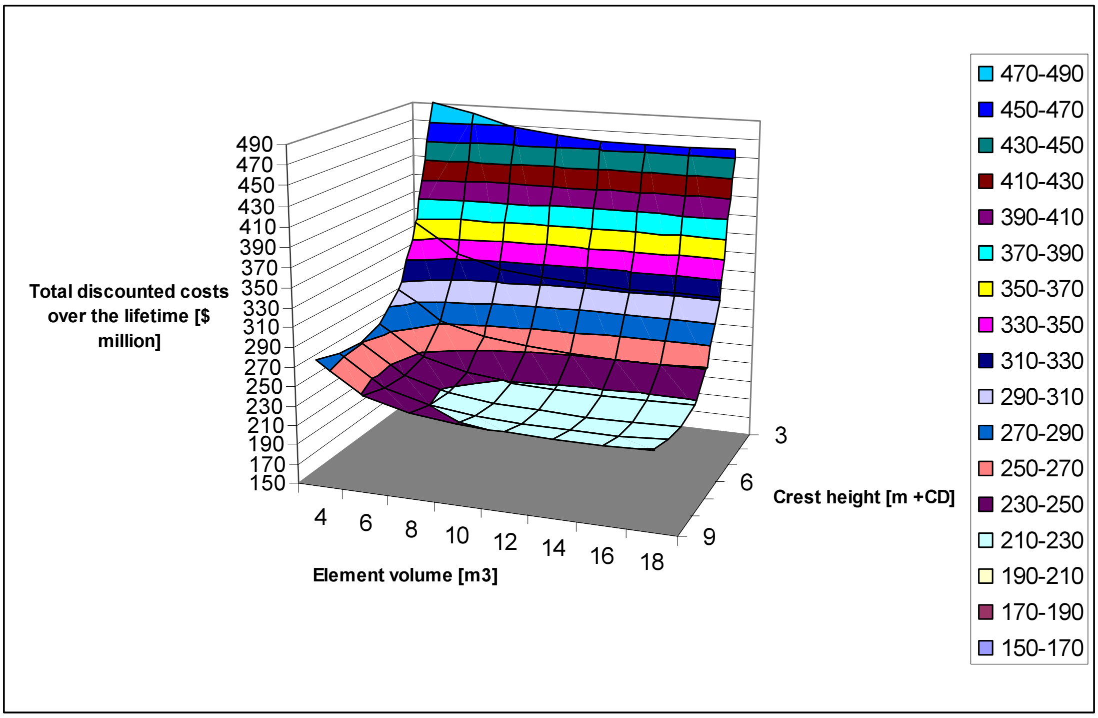

Figure 42 Total discounted costs of all alternatives with a sea level rise of 0.15m per 50 year

Table 33 Total discounted costs of all alternatives [$ million]

Volume	Crest height [m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]	[m +CD]
[m3]	3	4	5	6	7	8	9
4	511.5	366.0	302.9	277.8	270.9	272.8	279.4
6	480.8	335.4	272.3	247.3	240.4	242.4	249.0
8	467.9	322.6	259.6	234.7	227.8	229.9	236.6
10	461.9	316.7	253.8	229.0	222.3	224.5	231.3
12	459.3	314.2	251.4	226.7	220.1	222.5	229.4
14	457.7	312.7	250.0	225.4	218.9	221.3	228.3
16	457.5	312.6	250.1	225.6	219.2	221.8	228.9
18	458.2	313.4	251.1	226.7	220.5	223.2	230.5

The economic optimal design has a crest height of 7m +CD and an element volume of 14 m^3 (30.8t). The total discounted construction costs are 218.9 $ million. In Figure 43 the contributions of the costs to the total costs are provided.

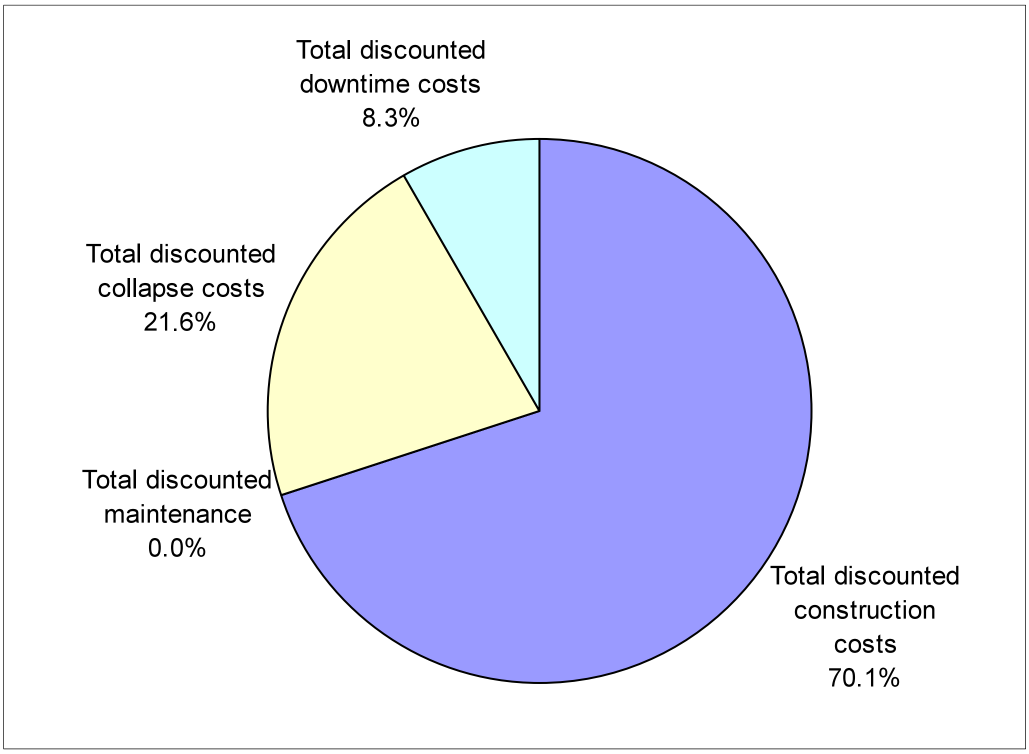

Figure 43 Distribution of the total discounted costs

10.2.2 Collapse only due to Nortes

If the probability of occurrence of hurricanes is reduced to zero, the probabilities of collapse decrease and the economic optimal geometry becomes less dependent of the element size. In Figure 44 it is clearly visible that above an element size of 8m^3, the total costs do not vary significantly. The optimal geometry still has a crest height of 7m +CD and an element volume of 14 m^3. The occurrence of hurricanes has thus little influence on the optimal geometry of the breakwater. The total discounted costs are 192.4 $ million. This is significantly lower compared to the situation including the occurrence of hurricanes, due to the lower probability of collapse.

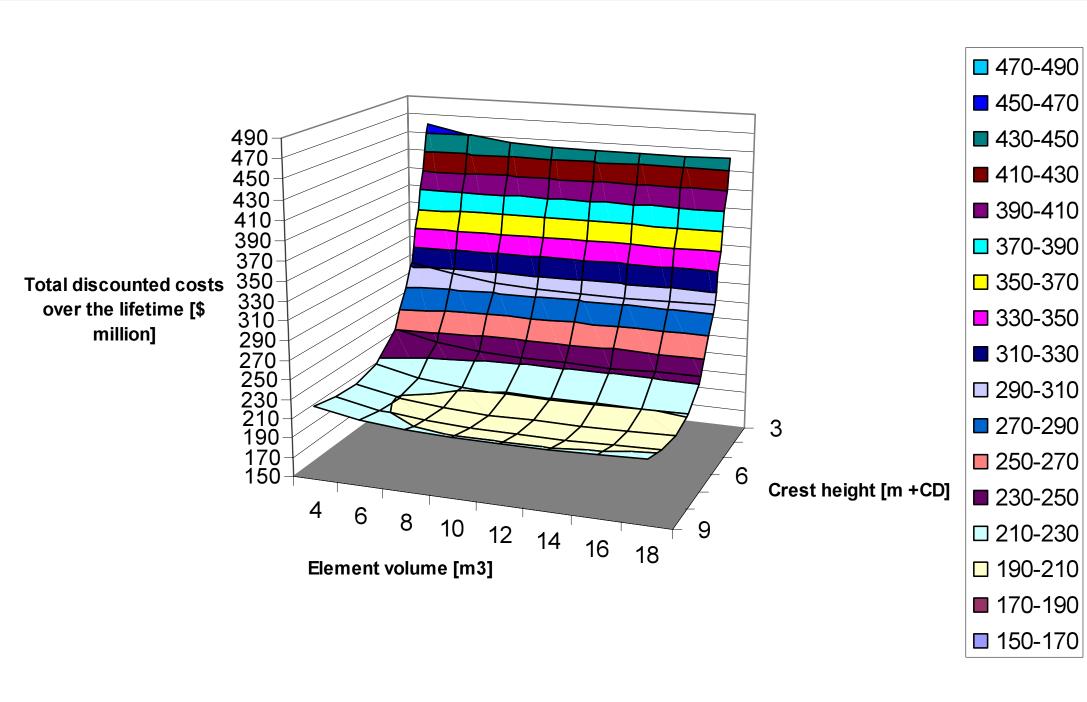

Figure 44 Total discounted costs over the lifetime without hurricanes [$ million]

10.2.3 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 7m +CD and the results are shown in Figure 45. The influence of the element volume variation is small above a size of $10m^3$. This complies with an element weight of 22t.

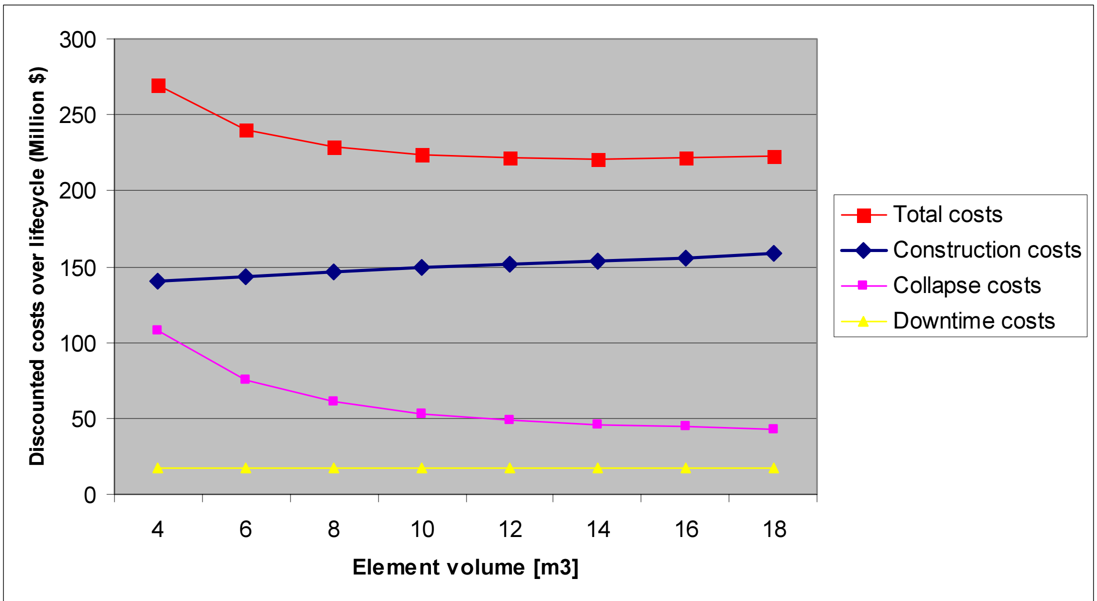

Figure 45 Costs for element volume variation for crest height 7m +CD

With the stability formula the element volume can be translated in the design wave height. The results are given in Figure 46. The design wave height of 8m gives the lowest costs over the lifetime of the breakwater. The significant design wave height of the deterministic design was 7.2m taking into account only the Nortes and 8.6m for hurricanes. The probabilistic design takes both into account. The design wave height of 8m is therefore in accordance with the results of the deterministic design.

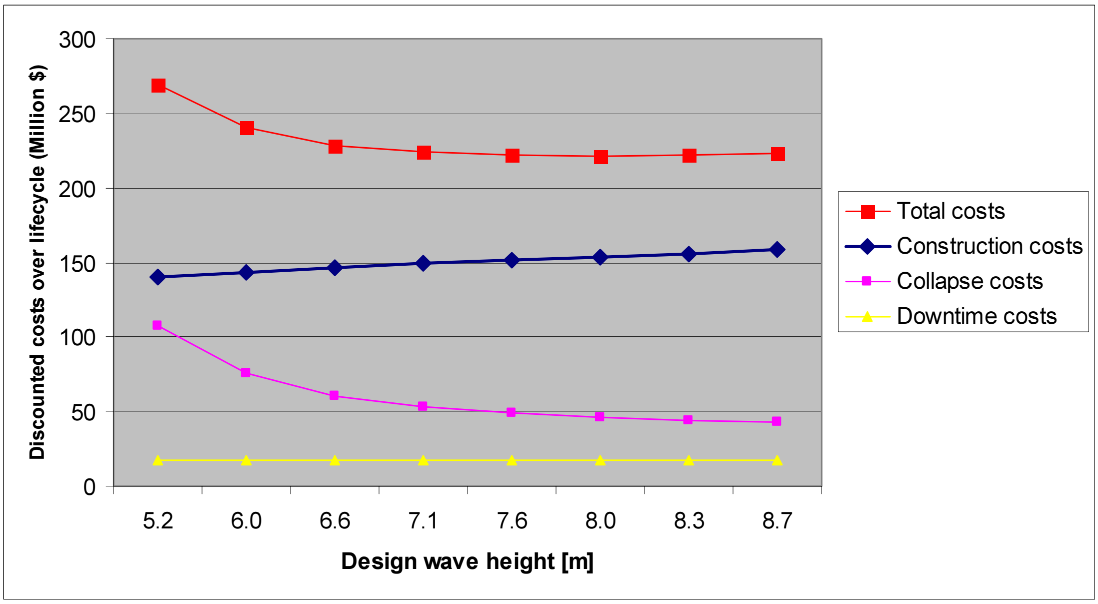

Figure 46 Costs for significant design wave height variation for crest height 7m +CD

10.2.4 Crest height

The crest height evaluated for the optimum element volume of $14m^3$ and the variations of the costs are depicted in Figure 47. The downtime costs show initially a steep decrease with the an increasing crest height. The optimum is found at 7m +CD. However, a higher crest height of 8m +CD increases the total costs not significantly. The availability or absence of financial means over time will determine the preferred alternative. If the port authority has a weak financial position at the construction time the alternative with the cheapest construction will be favourable.

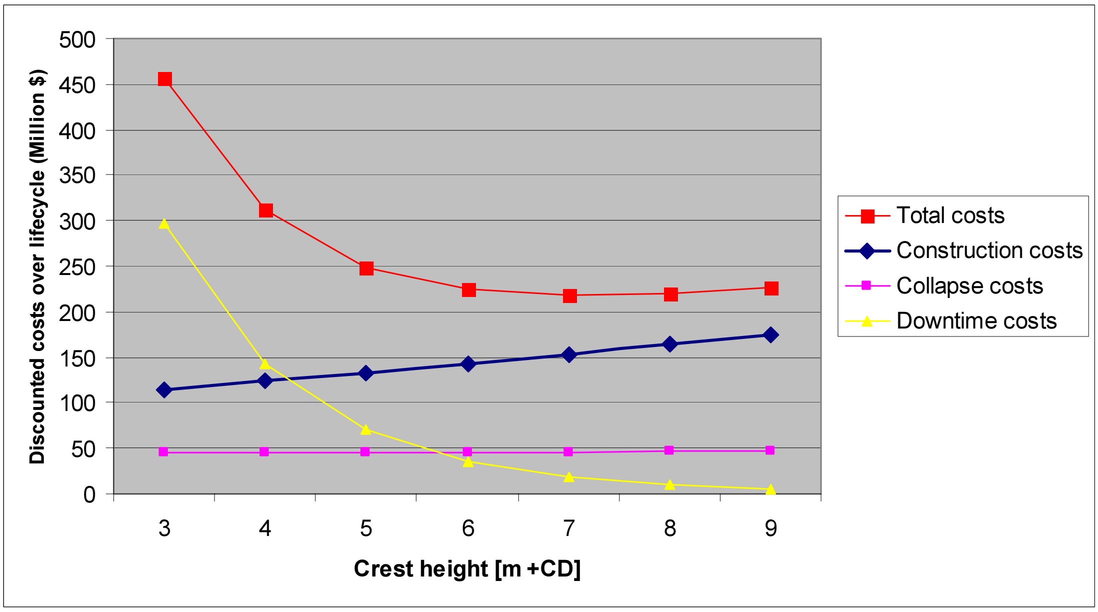

Figure 47 Costs for crest height variation for an element weight of $14m^3$

In Figure 48 the downtime development over the lifetime for the economic optimal geometry of 7m +CD is given. The downtime increases during the lifetime due to the sea level rise. The downtime at the end of the lifetime is approximately 10% more than at the start of the lifetime. The downtime is on average 0.215% with the economic optimal crest height. This is very little compared to the demand of 5% by APIVER. This results in very high downtime costs for the deterministic design of 3m +CD based on the 5% criterion.

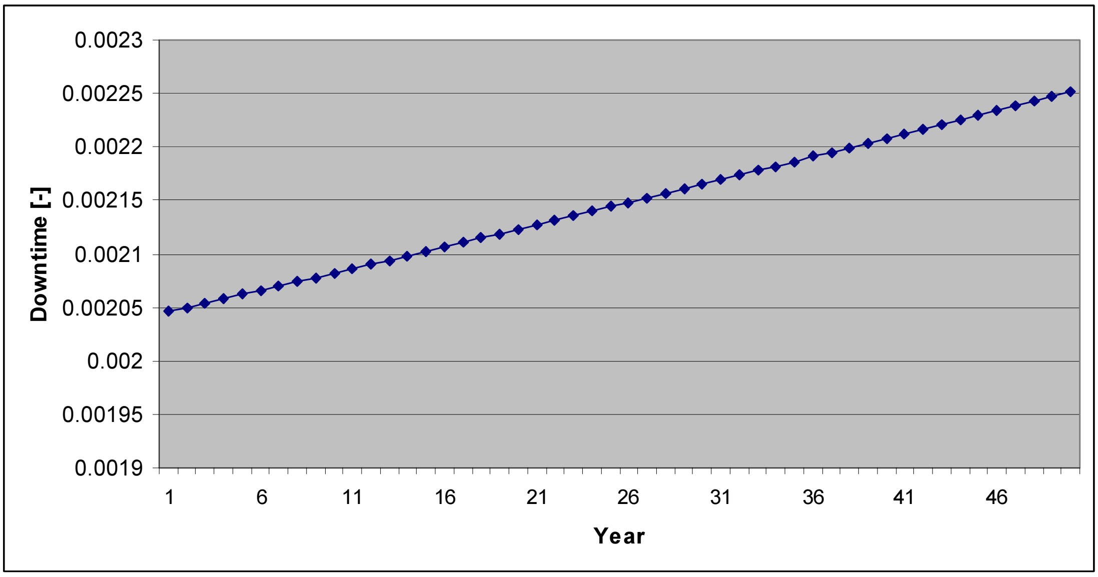

Figure 48 Development of downtime for a crest height of 7m +CD

10.3 Sea level rise

With no sea level rise, the downtime and downtime costs decrease compared to a scenario with sea level rise. The total costs of a scenario without sea level rise are given in Figure 49. The optimal geometry is again a crest height of 7m +CD and an element volume of 14m^3 and the total costs amount to 217.9 $ million. This 1 $ million lower than in the scenario with a sea level rise of 0.15m.

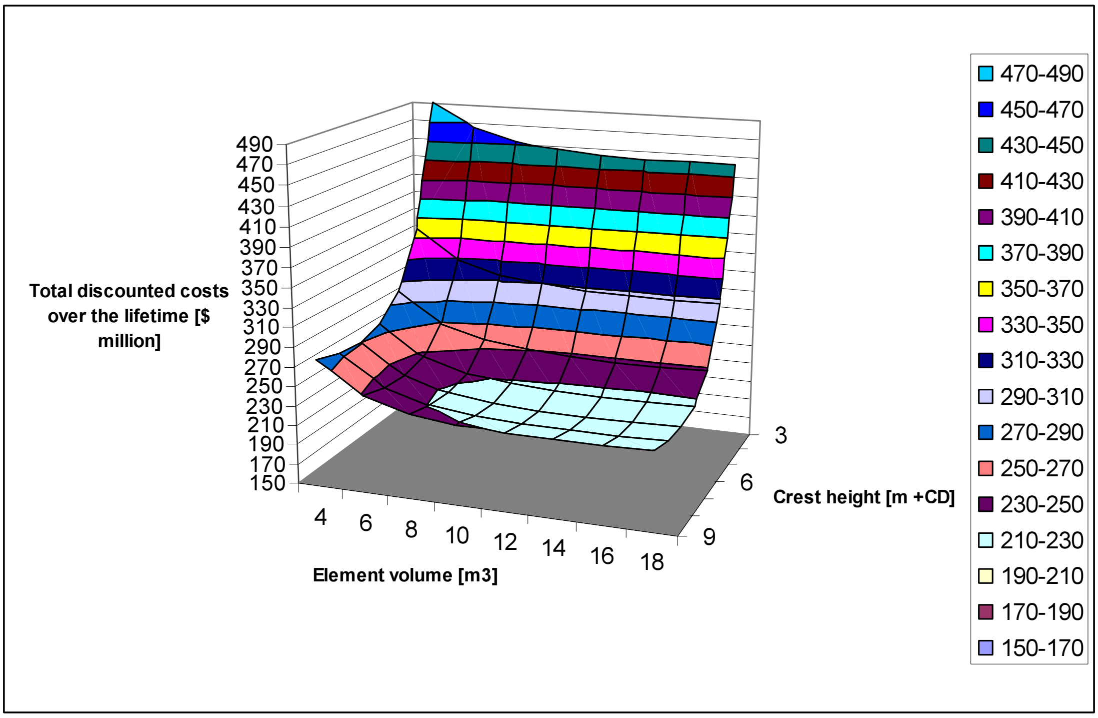

Figure 49 Total discounted costs of all alternatives with no sea level rise

10.4 Core-loc® deterioration

The optimum breakwater geometries are also calculated for the scenario with no deterioration of the strength of the Core-loc® armour and no sea level rise. The optimum crest height and element volume do not change. The total costs are 1.2 $ million lower than for the scenario with deterioration.

The probability of collapse increases up to 10% at the 50th year of the lifetime if deterioration is accounted for. The collapse costs only increase with 2.7%. If the lifetime of the breakwater would have been longer the probability of collapse would increase substantial in the years after the 50th year. The evaluated deterioration is not of importance for Core-loc® armour with a lifetime of 50 years or shorter.

10.5 Maintenance strategy

The economic optimal design with the application of the maintenance strategy with a ten year return period has the same geometry as the design without maintenance. The additional costs for maintenance do not compensate for the reduction in collapse costs: the total costs amount to 221.8 $ million. This is 2.9 $ million more expensive than without maintenance. The maintenance strategy with the assumed maintenance costs and return period does not provide lower total costs over the lifetime. If the breakwater would be optimised for a longer lifetime the progressive deterioration would become more severe and maintenance strategies will become economical.

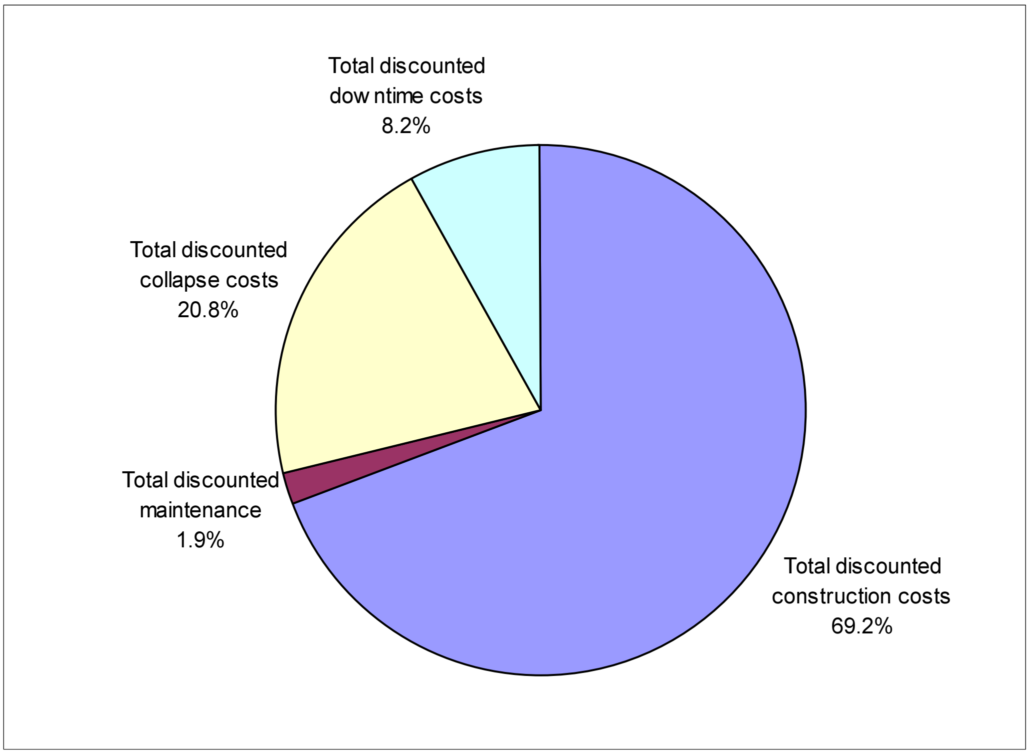

Figure 50 Distribution of the total discounted costs with maintenance

10.6 Economic development

The development of the port is taken into account by the growth rate of the throughput. The rate is varied in a range of no growth up to a growth of 8% to evaluate the influence of the development of the port on the optimal geometry parameters. A larger throughput in the port increases the consequences of downtime due to collapse of the breakwater or due to excessive wave transmission. The downtime costs increase and a stronger and higher breakwater could provide lower total costs over the lifetime.

No development

Without development of the port the throughput will be constant on the level of the initial year of the lifetime. The financial consequences of collapse and downtime decrease and the optimal geometry shifts down to a crest height of 6m +CD and an element weight of 12m^3. The total costs over the lifetime amount to 185.0 $ million. The costs over the lifetime of the original optimal geometry parameters, crest height 7m +CD and element volume 14m^3, are, with no development of the port throughput, 188.5 $ million.

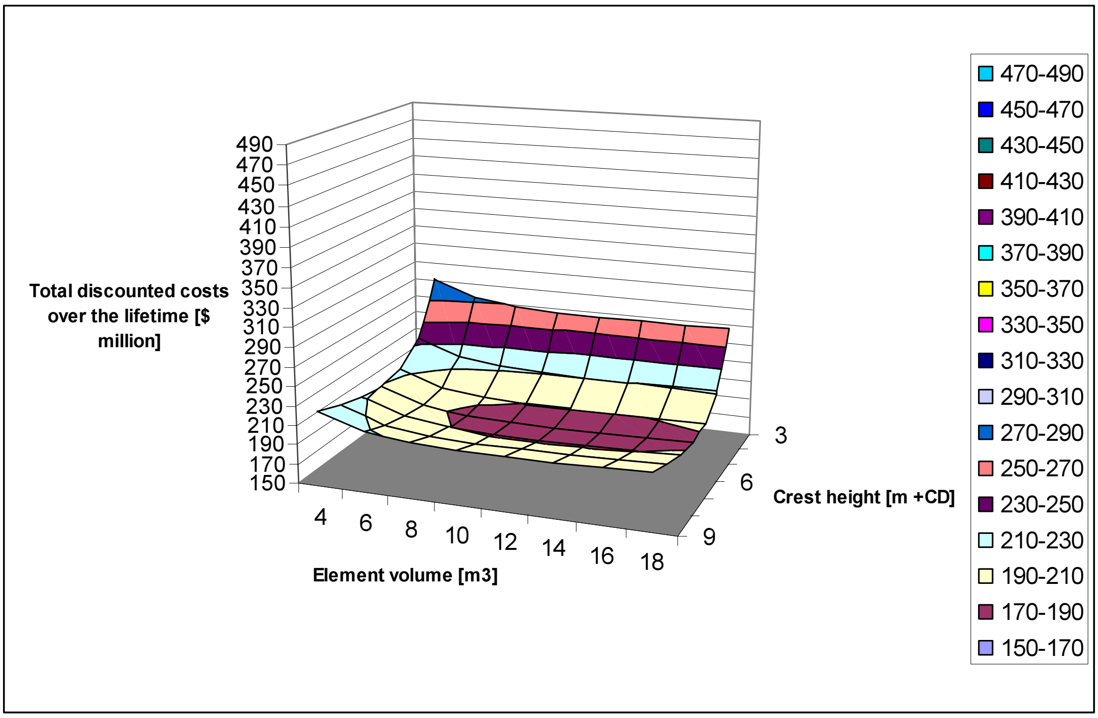

Figure 51 discounted costs over the lifetime without throughput growth rate [$ million]

Three percent growth per year

The optimum geometry does not vary and stays 7m +CD and $14m^3$. However, the total costs of the lower breakwater crest height alternatives still drop drastically, compared to the 5% growth with 100 $ million, as shown in Figure 52. The total costs of the alternative show smaller differences as indicated in

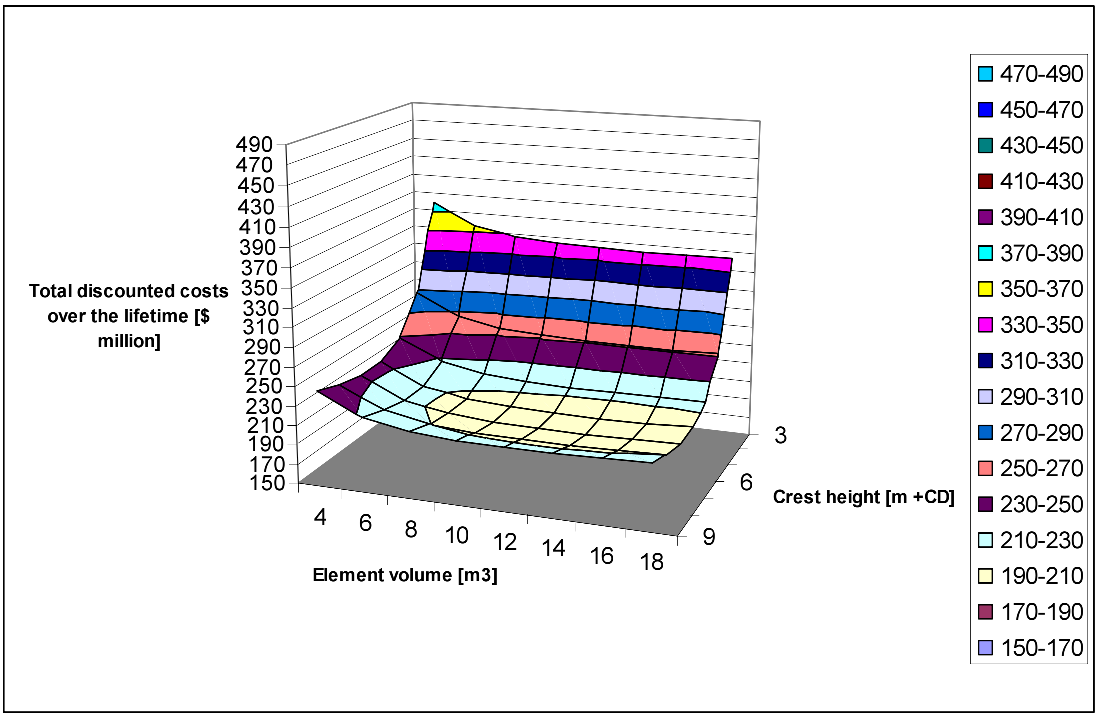

Figure 52 Total discounted costs over the lifetime with a 3% throughput growth rate [$ million]

Seven percent growth per year

An increased economic development of the port of seven percent per year shifts the optimum geometry to an element volume of 16m^3 and a crest height of 8m +CD. The total costs are 252.4 $ million, while the total costs for the original geometry are 257.0 $ million. Over the 50 year lifetime this is not a very large difference. But the initial construction costs increase significantly from 153.4 $ million to 166.7 $ million.

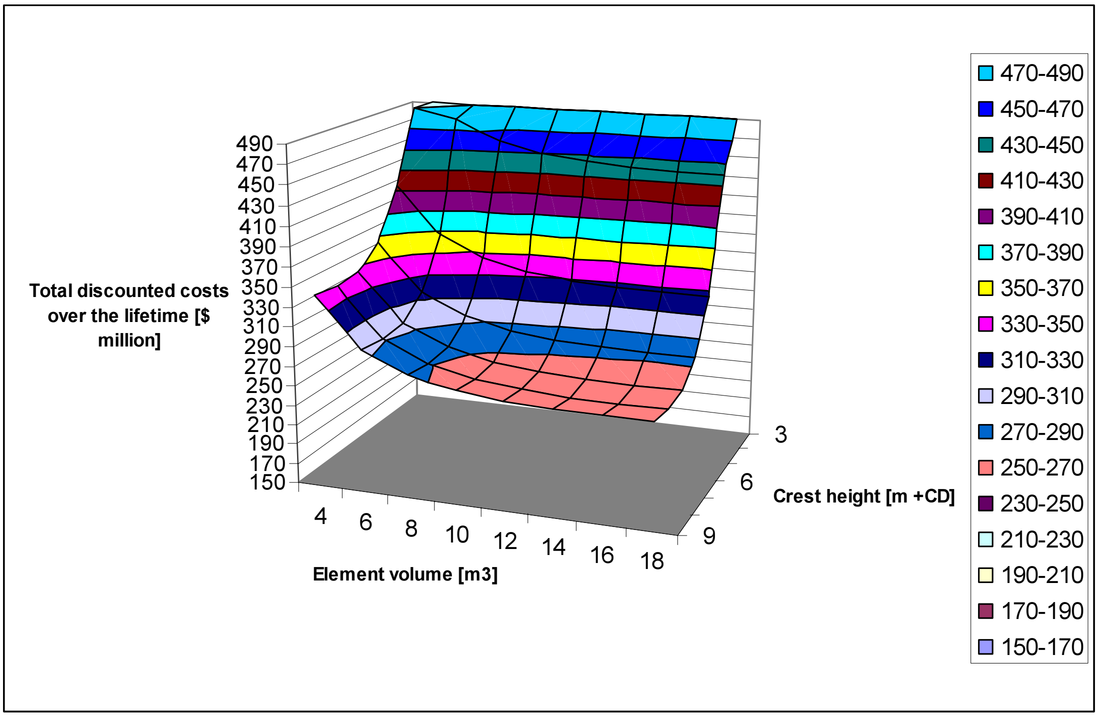

Figure 53 Total discounted costs over the lifetime with a 7% throughput growth rate [$ million]

Nine percent growth per year

A growth rate of the throughput of the port per year of 9% increases the consequences of collapse and downtime significantly as can be seen in Figure 54 by the steep slopes and higher total costs over the lifetime. The optimum geometry has an element volume of $18m^3$ and a crest height of 9m +CD. If a growth rate of 9% could be realistic, larger element sizes and higher crest heights should also be investigated.

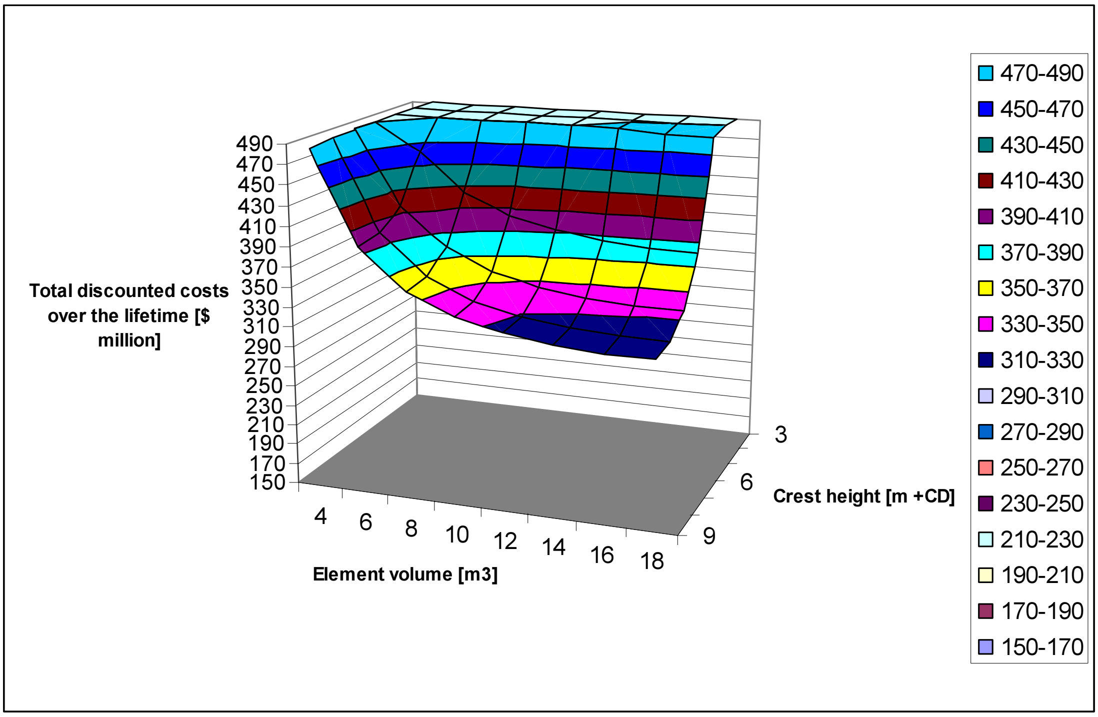

Figure 54 Total discounted costs over the lifetime with a 9% throughput growth rate [$ million]

Influence economic development

The influence of the growth of the throughput on the construction costs for the most economic design over the lifetime is given in Table 34. The optimal design with 3% growth is equal to 5% but the difference in total costs with the other alternatives is much smaller. The impact of a lower growth rate than 5% on the initial construction costs is much smaller than the impact of a higher development.

Table 34 Influence economic development

Growth rate [-]	Construction costs [$ million]
0%	141.3
3%	153.4
5%	153.4
7%	166.7
9%	180.9

10.7 Discount rate

The discount rate was assumed constant at a level of 5% over the lifetime. The effects of a discount rate ranging 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. This is shown in Figure 55. The most economic alternative is the strongest and highest breakwater alternative. This alternative decreases the collapse and downtime costs most. Alternatives with larger elements and higher crests are possibly even more economic.

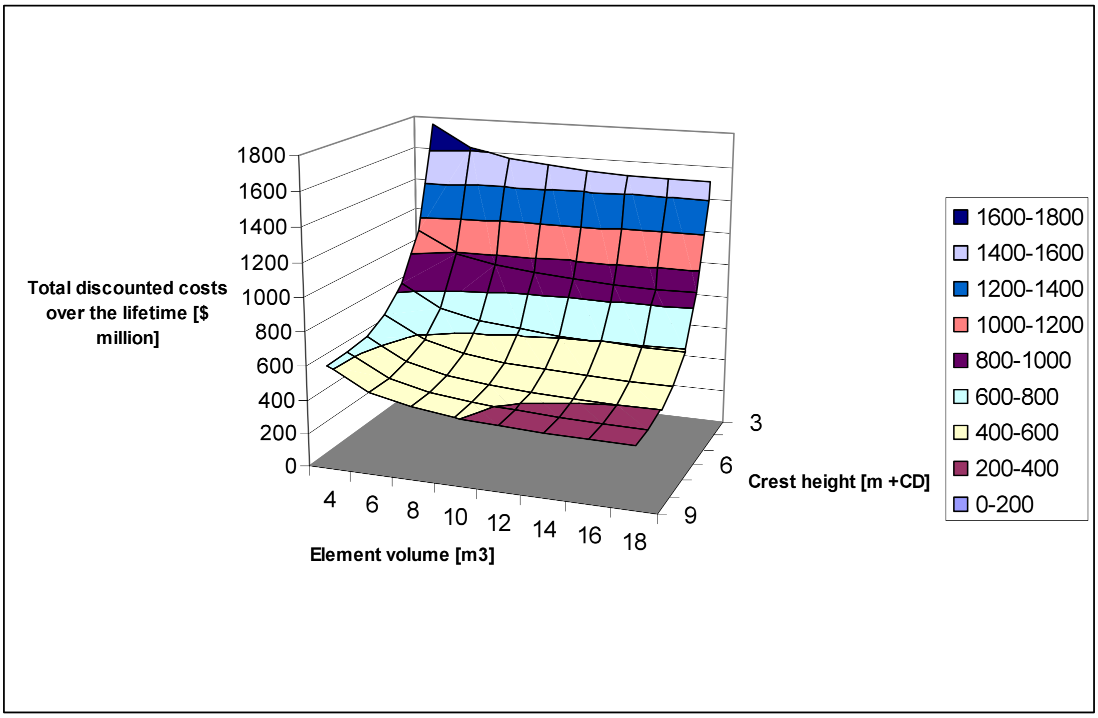

Figure 55 Total discounted costs with a discount rate of 0%

Discount rate 10%

The discount rate of 10% decreases the present value of the discounted collapse and downtime costs. This is shown in Figure 56. A breakwater with a crest height of 6m +CD and an element weight of 10m^3 is the economic optimal design. The decreased failure costs allow for a lower and weaker breakwater.

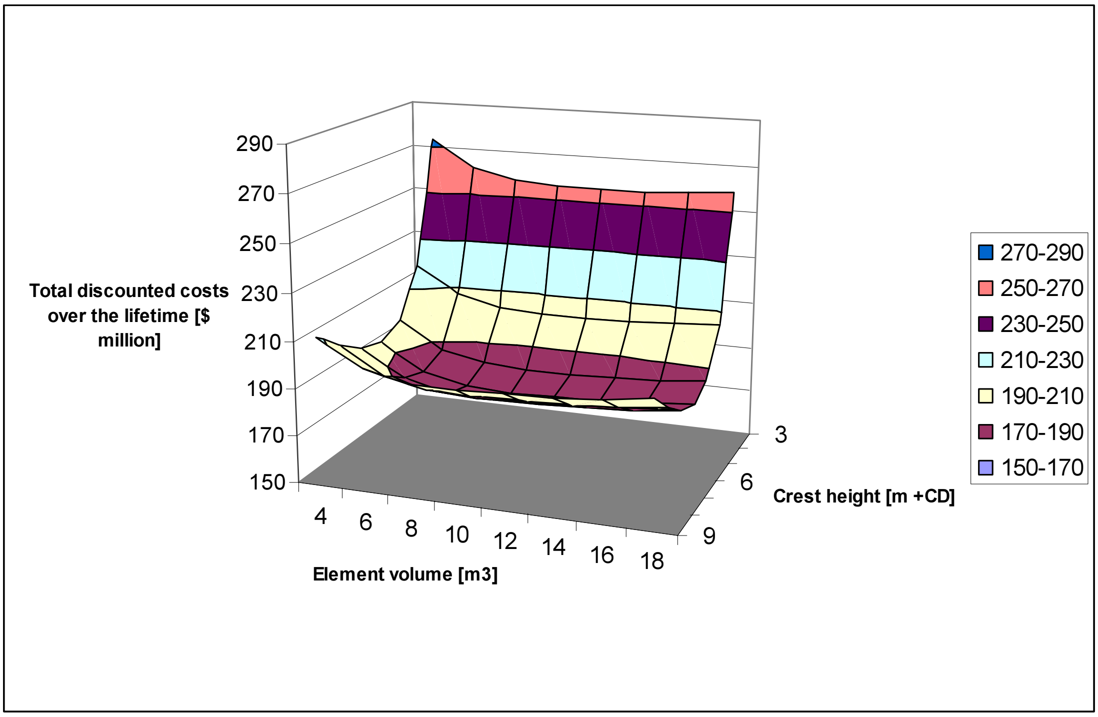

Figure 56 Total discounted costs with a discount rate of 10%

Influence discount rate on initial construction costs

The influence of the discount rate on the initial construction costs is given in Figure 57. The higher the discount rate, the lower the construction costs for the most economic design. The discount rate depends on the availability of assets to the port authorities of the port of Veracruz. The optimum breakwater dimensions are thus also dependent on the availability of assets of the port authorities of the port of Veracruz.

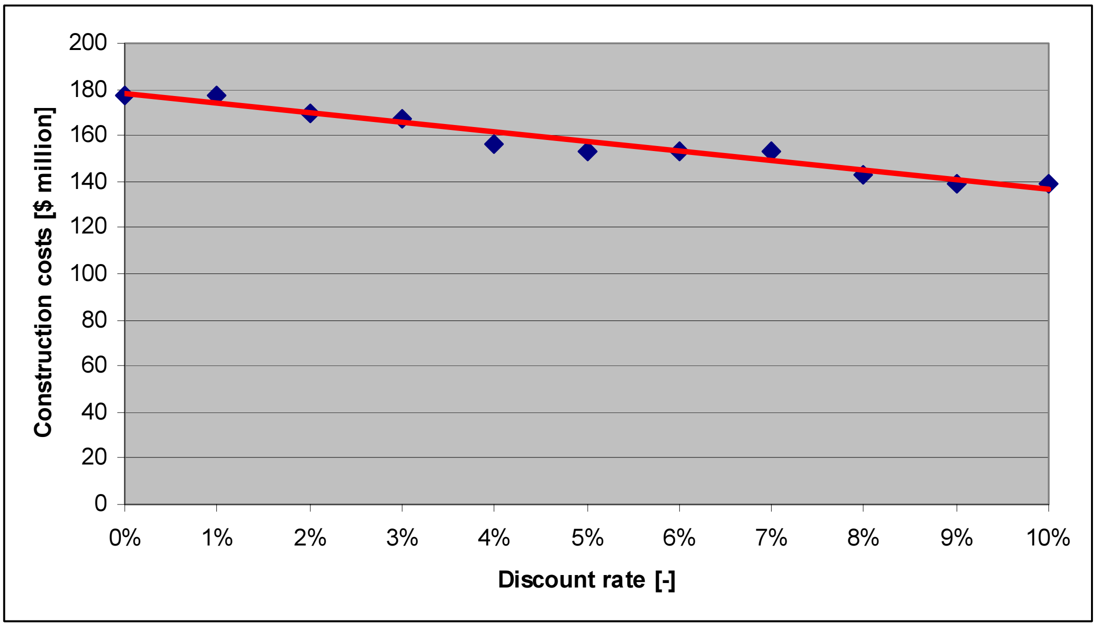

Figure 57 Influence discount rate on construction costs

11 Conclusions

Method of construction

The comparison of land-based and water-based construction provides considerable lower construction costs for the water-based construction method, even if the increased downtime during construction is taken into account.

Probability of collapse

The probabilities of collapse for the Core-loc® armour and the toe structure remain approximalety constant for Core-loc® elements larger than 10m^3 (22t). The effects of the deterioration of the Core-loc® armour strength are only of importance after the 30th year of the lifetime of the breakwater. Therefore the return period of the maintenance strategy should be taken longer than the applied 10 years.

The probabilities of collapse due to the Nortes and hurricanes are approximately equal. If the occurrence of hurricanes is overestimated the real probability of collapse of the breakwater could be up to half of the calculated probability of 0.004 per year for elements equal to or larger than 10m^3 (22t).

A sea level rise of 0.15m per 50 year has no significant influence on the probabilities of collapse of both the Core-loc® armour and toe structure.

The probability of collapse in one year is of a different order of magnitude than the probability of collapse in 50 years. The uncertainty in the strength of the Core-loc® armour and toe structure is not high enough to make the subsequent years dependent. On the contrary, the high uncertainty in the wave height makes the subsequent years almost completely independent. Therefore, the probability of collapse is say p per year then the collapse failure is approximately N.p per during the lifetime of N years (Vrijling and Van Gelder, 1998). According to Vrijling and Van Gelder (1998) the correlation in the reliability in two subsequent years i and i+1 is:

$$\rho(Z_i, Z_{i+1}) = \frac{\sigma_R^2}{\sigma_R^2 + \sigma_S^2}.$$

The relative large variation of the wave load, compared to the variation of the strength, gives a very small correlation.

Probability of functional failure

An increase of the crest height up to 6m +CD causes a large reduction of the downtime. Above this level the additional reduction decreases rapidly with increasing crest levels.

For low crest heights the sea level rise has a relative larger increasing effect on the downtime than for high crest heights.

For a crest height of 7m +CD the downtime is more likely to be caused by collapse of the breakwater than by transmission of wave energy with the breakwater intact.

Optimum breakwater geometry

The economic optimal design of the breakwater has a crest height of 7m +CD and an element volume of $14m^3$ (30.8t). The total discounted costs over the lifetime of the breakwater are 218.9 $ million of which 153.4 $ million are the initial construction costs.

Varying the element size and crest height around the optimal geometry does not give a substantial increase of the total costs. The 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 collapse failure is not dependent on the element size. The influence on the probability of collapse due to Core-loc® armour failure is therefore limited. The intrusion of wave energy throught the entrance is independent of the crest height and reduces the influence of the crest height on the downtime. The decrease of the failure probabilities are thus not substantial. The resulting changes in construction costs and collapse and downtime costs are threfore not significant.

Hurricanes

If the occurrence of hurricanes is neglected, the economic optimal geometry of the breakwater does not change. Due to the lower probability of collapse, the collapse cost decrease with 26.5 $ million. Because the downtime costs and construction costs do not vary, the total costs over the lifetime also decrease with 26.5 $ million to 192.4 $ million.

Element weight

The economic optimal Core-loc® element volume of $14m^3$ corresponds with an element weight of 30.8t. Using the Hudson formula, the deterministic significant design wave height is determined at 8m. The significant design wave height of the deterministic design was 7.2m taking into account the Nortes and 8.6m taking into account hurricanes. The probabilistic design takes both into account. The design wave height of 8m is therefore in accordance with the results of the deterministic design. In Figure 58 the implications of both deterministic design wave heights are given. The dashed line indicates the Nortes design wave height and the dashed-dotted line indicates the hurricane design wave height.

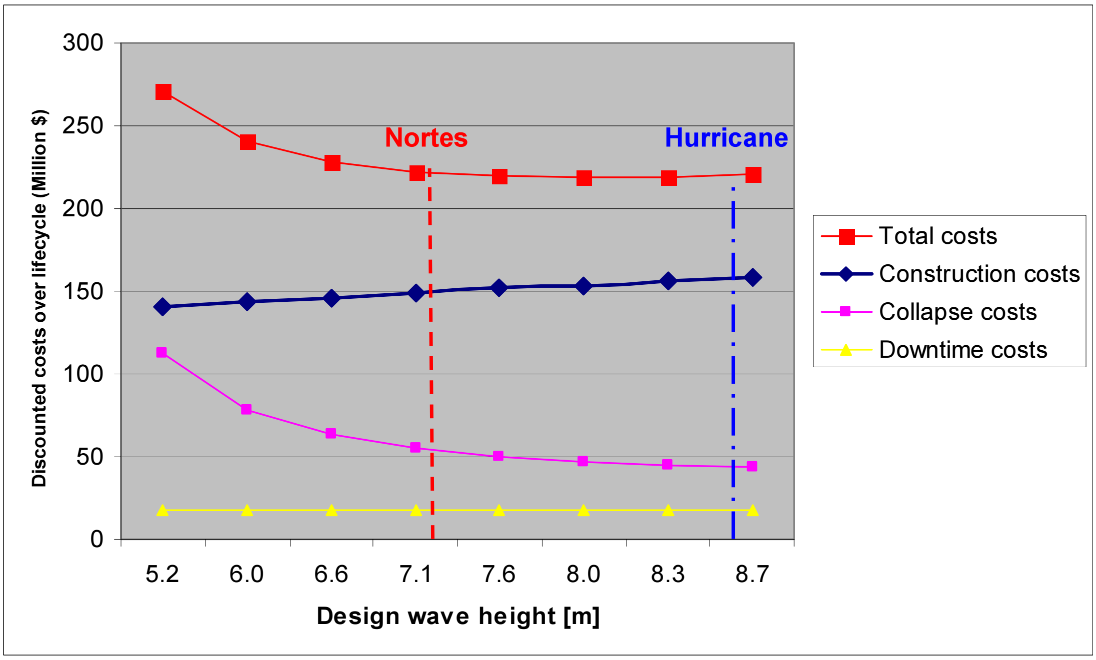

Figure 58 Discounted costs for the significant design wave height variation

The variation of the element weight does not cause severe variations in the total costs. Only relative light elements result in substantially more expensive breakwater alternatives.

Crest height

The consequences of downtime are more severe than APIVER anticipates. Their maximum downtime limit of 5% results in a breakwater crest height, that is too low. The downtime costs are almost 300 $ million higher than the economic optimum design. The optimal crest height has an average downtime over the lifetime of approximately 0.22%. In Figure 59 the decreasing effect of an increasing crest height on the downtime and total costs is visible. For crest height variations around the optimal crest height, the consequences for the total cost are not significant.

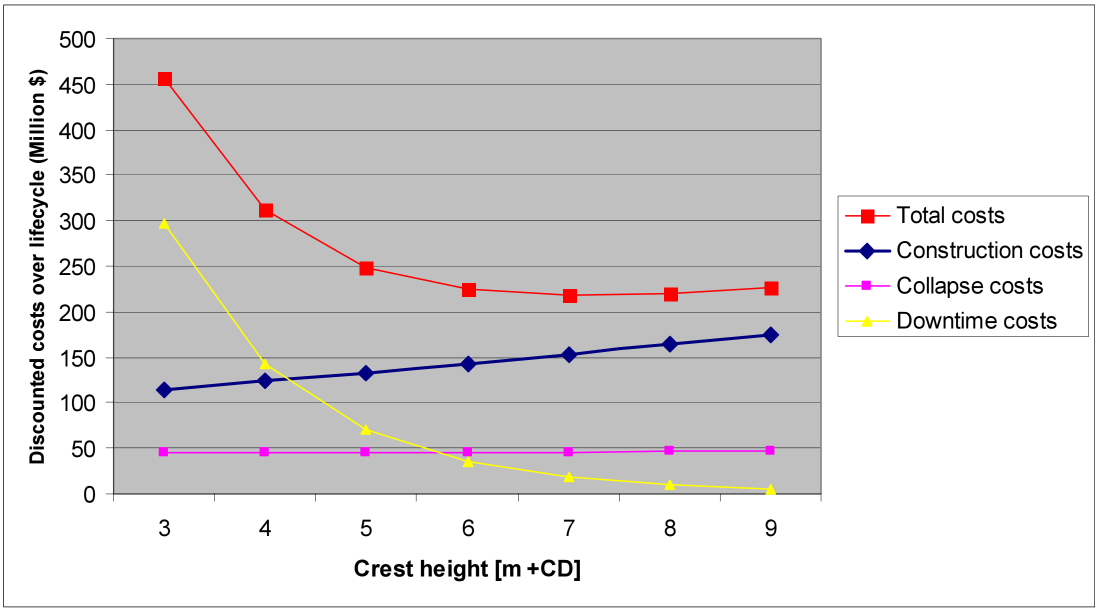

Figure 59 Discounted costs for the crest height variation

Availability of funds

The small increase of the total costs due to a lower crest height or a smaller Core-loc® element size makes the availability of funds by the port authority of interest. If sufficient funds are not available during the time of construction to construct the most economic alternative, based on the total costs over the lifetime, a cheaper construction alternative can be the best alternative possible. But the total costs will be higher due to the increasing collapse and downtime costs.

If the port authority decides to borrow additional financial means to pay for the breakwater construction the interest rate will influence the discount rate. For the Veracruz situation the discount rate shows a linear relation with the construction costs of the most economic alternative. If the port authority is confronted with high interest rates, resulting in a high discount rate of 10%, a breakwater with a crest height of 6m +CD and a Core-loc® element volume of 10m^3 (22t) will be the most economic alternative.

Sea level rise

The absence of a sea level rise decreases the downtime costs, especially for relative low crest heights. A sea level rise of 0.15m per 50 year only increases the total costs with 1 $ million.

Core-loc® armour deterioration

The time dependent deterioration of the Core-loc® armour increase the collapse costs over the lifetime by only 2.7%. Breakwaters designed for lifetimes longer than 50 years are not observed in this study, but the development of the probability of collapse indicates that armour deterioration will increase the collapse costs significantly.

Economic development

The downtime and collapse costs are based on the forecasted development of the throughput of the port of Veracruz of 5%. The optimal design geometry with 3% growth is equal to the scenario with 5% growth, but the difference in total costs with the other alternatives is much smaller. The impact of a lower growth rate than 5% on the initial construction costs is much smaller than the impact of a higher economic development rate. If the economic development is underestimated the total costs will increase rapid.

12 Recommendations

Construction method

The economic optimal crest height is significantly higher than the crest height used for the water-based construction in the deterministic design. The lower costs for the water-based construction in comparison with the land-based construction method are caused by the decreases of quantities. Due to the increase of the crest height with 4m a new comparison should be made to determine the cheapest construction method.

Shallow water waves

The translation of the deep water wave height to shallow water wave height is based on a 2-dimensional simplification. The 3-dimensional reality could provide different results. Further investigation is recommended.

Core-loc® breakage rate

The time dependent progressive deterioration of the Core-loc® armour is included in the calculation as well. The collapse probabilities are consequently higher at the end of the breakwater lifetime. In that period the economic downtime consequences are higher as well. Because of these increased financial consequences not taking into account the effects of deterioration is not recommended. The rate of the deterioration should be further investigated. Wave loads on existing Core-loc® breakwaters and the resulting deterioration of the armour should be monitored. Model test using ‘breakable’ Core-loc® elements should provide additional information on the relation between exceeded wave loads and deterioration of the Core-loc® armour.

Element size

The Core-loc® strength can be dependent on the size of the elements. This relation can result in a lower ‘strength per ton’ for the larger elements. This results in weight or size dependent deterioration rate. The effects of increasing element size on concrete stresses should be investigated.

Maintenance

Because the probability of collapse becomes of importance after the 30th year of the lifetime, the maintenance strategy should be investigated for breakwaters with a longer lifetime. The progressively increasing collapse costs due to deterioration of the Core-loc® armour can be compensated with a maintenance strategy to provide lower total costs.

Economic development

The downtime and collapse costs are based on the forecasted development of the throughput of the port of Veracruz. The estimated rate of growth of 5% should be further investigated. The increasing competition of other ports should be analysed thoroughly before any realistic forecast can be made.

Quarry optimisation

For quarry optimisation better knowledge of the other components of the port extension is eminent. Also concrete secondary armour units can be considered as a possible cheaper option. This is especially recommended if large Core-loc® elements are applied.

Fixed toe elements

Various methods exist to fix the Core-loc® toe. Most are expensive, but seem to improve the stability of the Core-loc® layer to a great extent in model tests. As hardly any experience exists outside a laboratory with the long term behaviour and stability of Core-loc® elements due to toe fixation, the benefits are uncertain. The effects of toe instability for Core-loc® armour should be taken into account if the Core-loc® breakwater design is further optimised.

Wave period

The influence of the wave period is not taken into account in this study. Because the wave period determines the number of wave loads exceeded on the breakwater during a single storm, during the storm season and during the lifetime of the Core-loc® armour, this should be further investigated.

Porosity of Core-loc® layer

The distribution of the stability of the Core-loc® over the two components weight and interlocking is arbitrary. Additional components are possible, such as the porosity of armour layer. The high porosity of the Core-loc® armouring contributes to the stability of the armour elements. The ability to dissipate large amounts of energy between the elements reduces wave forces and run-up of waves.

Hurricane

The hurricane storm surge is approximated with a normal distribution. In reality the probabilities of occurrence of the high storm surges are lower than the probabilities of occurrence of low storm surges. A better distribution should be applied to represent the storm surge occurrence.

Failure mechanisms

The failure due to collapse was simplified to only two failure mechanisms. Other mechanisms should be evaluated as well to determine a more accurate failure probability. The calculated probability of collapse is an underestimation of the real probability of collapse. Especially the influence of sea level rise has significant consequences on other failure mechanisms, e.g. the damage on the lee-side of the breakwater.

Lifetime

The breakwater is designed for a lifetime of 50 years. Consequences of a longer design lifetime should also be investigated, because the deterioration effects increase progressively over time and the sea level rise will increase as well.

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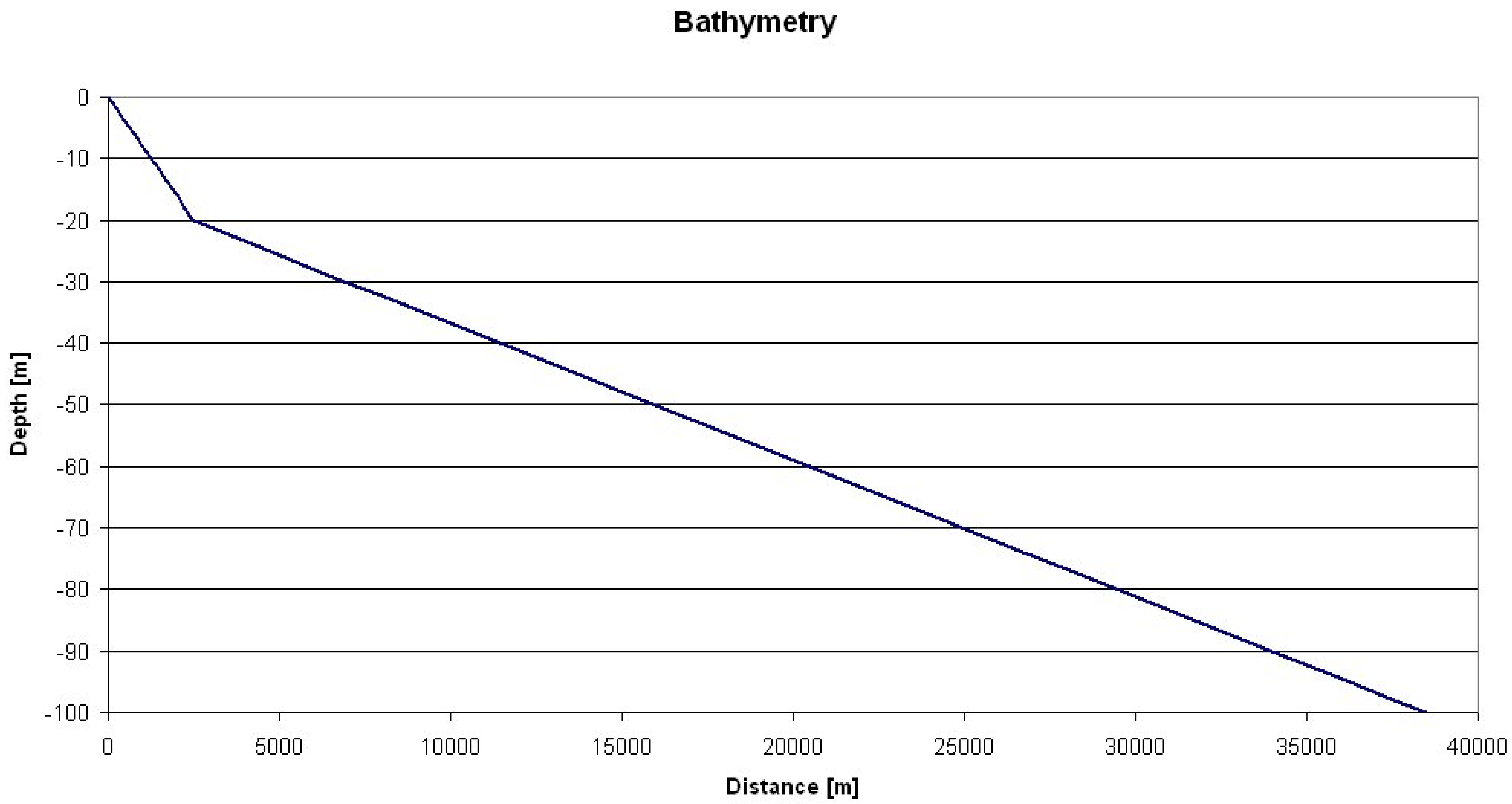
Appendix I: Bathymetry

API (2001) schematises the foreshore at Veracruz in two parts. The deepest part, from 38500 up to 2500 m in front of the shore, having a slope of 1/450 and the steeper, shallower part from 2500 to the shore having a slope of 1/125. The breakwater is located 2000 m from the shore at a depth of –16 m +CD. This simplified, two-dimensional, representation of the bathymetry is used for the preliminary calculations. However, the 3-D effects should be evaluated in further design and model tests. In the following table the depth-distance combinations are provided. On the next page the depth is given.

Distance to shore	Depth
[m]	[m +CD]
38500	-100
2500	-20
2375	-19
2250	-18
2125	-17
2000	**-16**
1875	-15
1750	-14
1625	-13
1500	-12
1375	-11
1250	-10
1125	-9
1000	-8
875	-7
750	-6
625	-5
500	-4
375	-3
250	-2
125	-1
0	0


Bathymetry
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
Depth [m]
0
5000
10000
15000
20000
25000
30000
35000
40000
Distance [m]

Appendix II: Tide

The tide data is derived from the website http://tbone.biol.sc.edu/tide of the University of South Carolina. Their data gives the following mathematical bounds for the water levels at Veracruz:

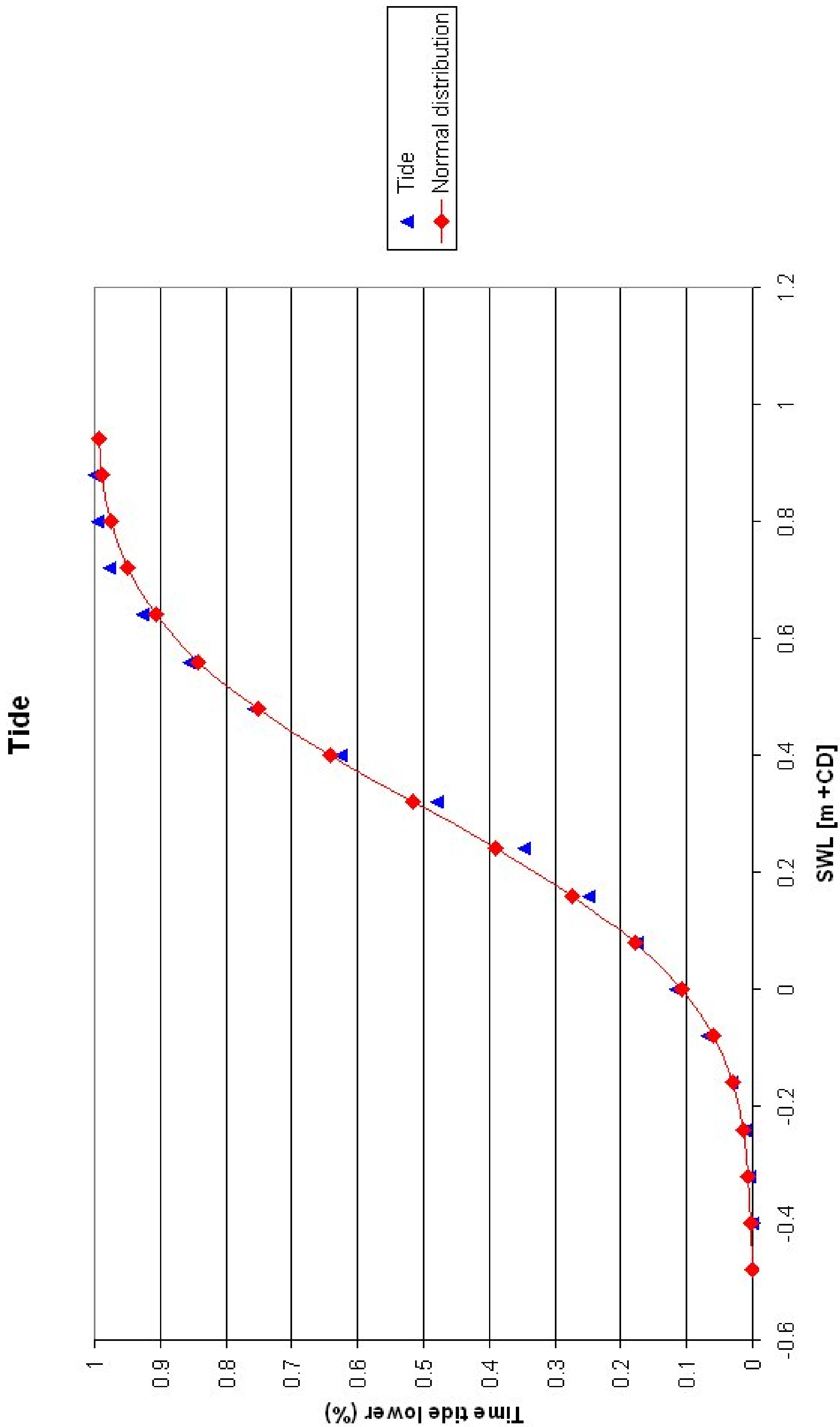
- Mathematical upper bound: 1.23 meter
- Mathematical lower bound: -0.61 meter
- Mean Tide Level: 0.31 meter

To the water level occurrence data a normal distribution is fitted, with a mean (μ) of 0.31 m +CD and a standard deviation (σ) of 0.25 m. Especially for the lower water levels the normal distribution gives a good fit. This is important as breakwater toe stability is greatly determined by the water level. Both the actual and the normal distributed curve are given at the following page.

Numerically the expected values of several low and high water level occurrences are determined and are provided in the following tables.

Tide lower than	Occurrence	Expected water level	Tide lower than	Occurrence	Expected water level
[m +CD]	[-]	[m +CD]	[m +CD]	[-]	[m +CD]
-0.33	0.005	-0.41	0.00	0.11	-0.12
-0.27	0.01	-0.36	0.02	0.12	-0.11
-0.20	0.02	-0.30	0.03	0.13	-0.10
-0.16	0.03	-0.26	0.04	0.14	-0.09
-0.13	0.04	-0.23	0.05	0.15	-0.08
-0.10	0.05	-0.21	0.06	0.16	-0.07
-0.08	0.06	-0.19	0.07	0.17	-0.06
-0.06	0.07	-0.17	0.08	0.18	-0.05
-0.04	0.08	-0.15	0.09	0.19	-0.05
-0.03	0.09	-0.14	0.10	0.2	-0.04
-0.01	0.1	-0.13	0.31	0.5	0.11

Tide higher than	Occurrence	Expected water level	Tide higher than	Occurrence	Expected water level
[m +CD]	[-]	[m +CD]	[m +CD]	[-]	[m +CD]
0.95	0.005	1.03	0.62	0.11	0.74
0.89	0.01	0.98	0.60	0.12	0.73
0.82	0.02	0.92	0.59	0.13	0.72
0.78	0.03	0.88	0.58	0.14	0.71
0.75	0.04	0.85	0.57	0.15	0.70
0.72	0.05	0.83	0.56	0.16	0.69
0.70	0.06	0.81	0.55	0.17	0.68
0.68	0.07	0.79	0.54	0.18	0.67
0.66	0.08	0.77	0.53	0.19	0.67
0.65	0.09	0.76	0.52	0.2	0.66
0.63	0.1	0.75	0.31	0.5	0.51


Tide
Tide
Normal distribution
Time tide lower (%)
SWL [m +CD]
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.6
-0.4
-0.2
0
0.2
0.4
0.6
0.8
1
1.2

Appendix III: Storm surge

To evaluate the effects of a storm surge the maximum wind velocity is combined with extreme storm conditions in the Veracruz area.

All necessary information as bathymetry and representative storm conditions are used as input as listed the following table.

Parameter	Input
Wave height deep water	10.0 m
Peak period	8.45 s
Maximum wind speed	26 m/s
Density seawater	1025 kg/m^3
Friction coefficient	0.01

The wave propagating and set-up 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 set-up and wave height difference at the breakwater location proved to be minimal and in the order of centimetres. The wind had also little effect and hardly increased the wave height at the breakwater as given in the table below. These results justify the neglecting of storm surge effects.

Distance to shore	Depth	Wind 26m/s Hs	Wind 0m/s Hs
[m]	[m]	[m]	[m]
38500	-100	10.2	10.2
2500	-20	5.54	5.51
2375	-19	5.51	5.48
2250	-18	5.48	5.45
2125	-17	5.44	5.41
2000	**-16**	**5.38**	**5.36**
1875	-15	5.31	5.29
1750	-14	5.22	5.2
1625	-13	5.1	5.08
1500	-12	4.94	4.93
1375	-11	4.75	4.74
1250	-10	4.52	4.51
1125	-9	4.25	4.24
1000	-8	3.94	3.94
875	-7	3.61	3.6
750	-6	3.24	3.24
625	-5	2.84	2.84

* Battjes, J., Janssen (1984) Delft Hydraulics, Report M1882, 1984

Appendix IV: Argoss

Actual and reliable wave records for the Veracruz area are not available in sufficient numbers. However, to provide realistic wave loads for the breakwater calculations use is made from satellite data available on the internet at the wave data site Argoss. On this site worldwide wind and wave data, measured with satellites, is offered. These data source has been validated with waverider buoys. The period of measurements of the satellites is 15 years. Every year this source will be updated. www.waveclimate.com is the internet site from the ARGOSS organisation. This means: Advisory and Research Group on Geo Observation Systems and Services. On the Internet site www.waveclimate.com it is possible to download this wind- and wave data from the whole world. This data is based on satellite observations. Subscription to the site is needed to download data/information from this site. Hydronamic has a subscription to this internet site.

The site's system, which is used to download the needed information, is named CLAMS: CLimatic AssessMent System. (This manual is meant to be a guide for using CLAMS in relation with workability.) Quote from the Argoss WebPages:

"Accurate estimates of wind and wave climate.

The CLAMS system allows users to make accurate estimates of the wind and wave climate in all coastal areas around the globe. The climate estimates are based on satellite observations acquired over the past 15 years.

The online CLAMS system enables users to analyse these observations in many different ways: by using histograms, joint distribution plots, time series, or by estimating the return period of extreme conditions.

For many applications, such as the assessment of the response of vessels and structures to incoming waves, spectral wave information is essential. The CLAMS system uses a unique set of spectral wave observations acquired with the ERS- ½ Synthetic Aperture Radar (SAR). The SAR observations complement the significant wave height measurements from radar altimeters of the Geosat, ERS- ½ and Topex/Poseidon missions. Statistics on the wind is based on ERS- ½ Scatterometer data."

Appendix V: Deep water wave height

The data provided by Argoss has to be translated into a probability distribution for extreme wave heights for the Ultimate Limit State (ULS) conditions and into a occurrence distribution for normal wave heights for the Serviceability Limit State (SLS).

ULS distribution

For the ULS significant wave distribution only the wave data containing waves higher than 1.5 m is used. The reason of introducing a threshold is to avoid that small variations in wave height during long, calm periods have significant influence on the final result. Basically, one should place the threshold as high as possible, as long as the base for statistics contains sufficient data for analysis.

A Gumbel and a Weibull distribution are fitted to the data by regression analysis.

The Gumbel and Weibull distributions are as follows:

$$Weibull\ \ probability\ of\ \ exceedence = Q = \exp\left[-\left(\frac{H_s-\gamma}{\beta}\right)^{\alpha}\right]$$

$$Gumbel\ \ probability\ of\ \ exceedence = Q = 1-\exp\left[-\exp\left(-\frac{H_s-\gamma}{\beta}\right)\right]$$

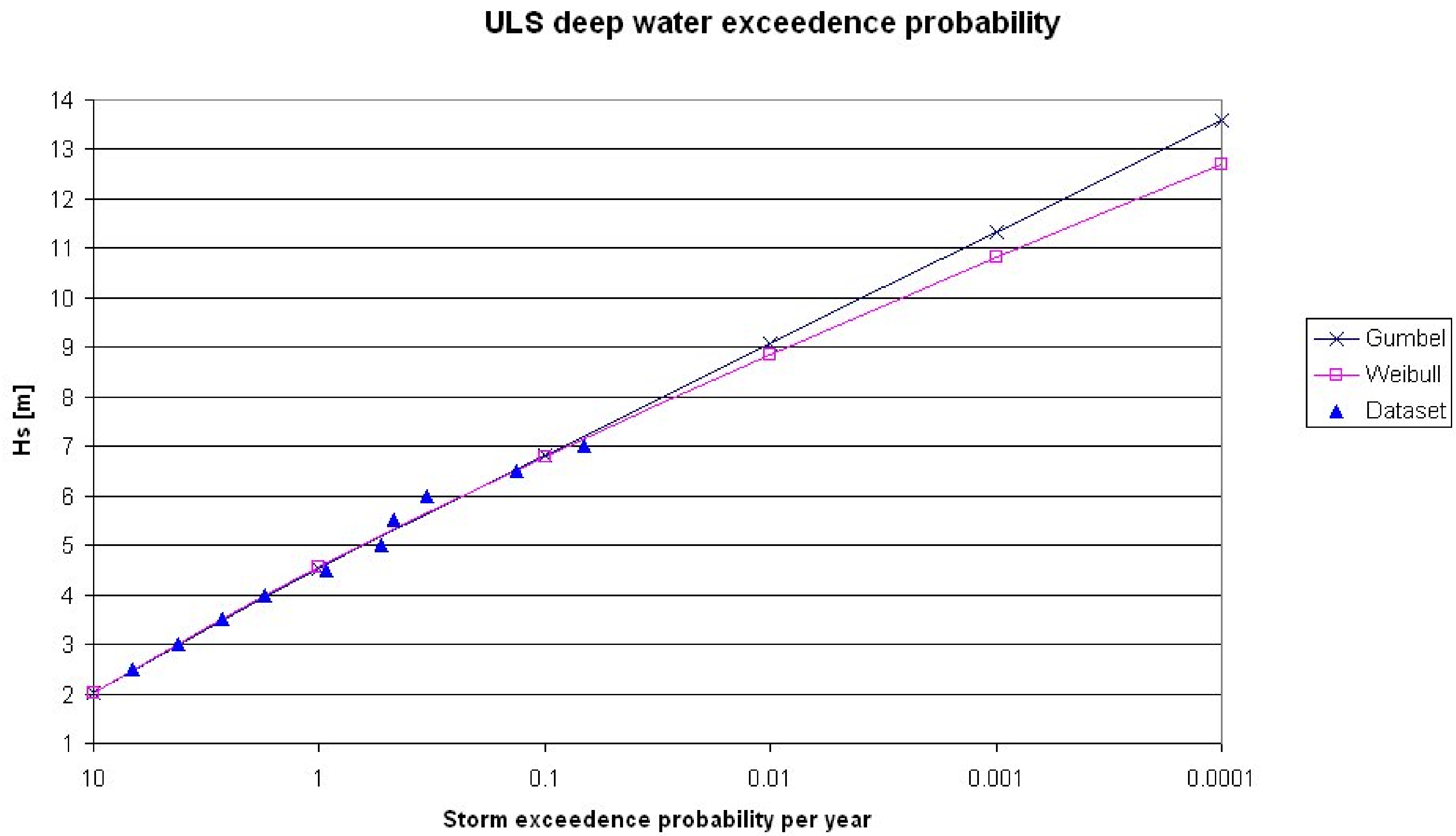
The Gumbel and Weibull exceedence probability and return period are provided in the following table and in the figures at the following pages.

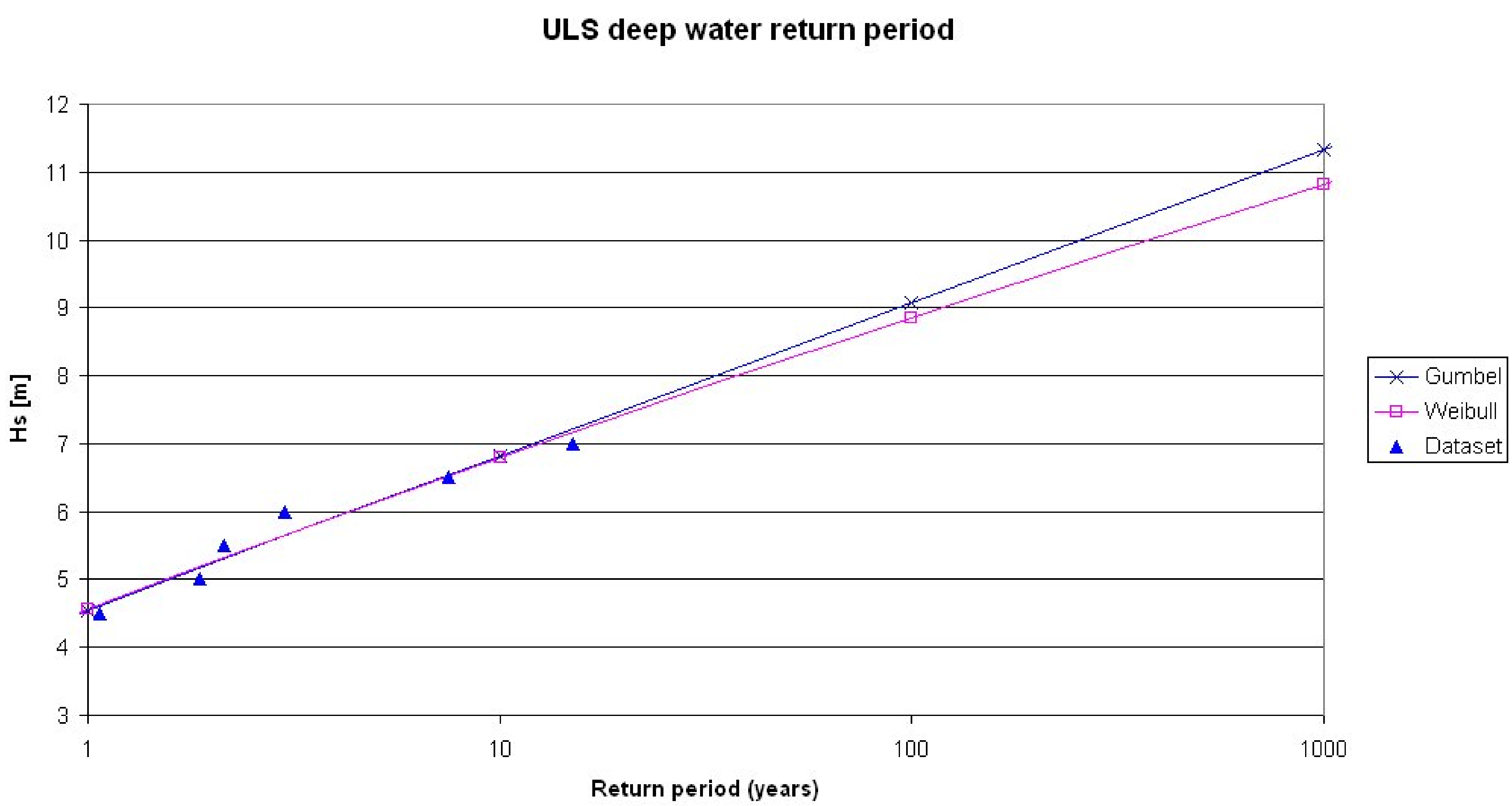
	Gumbel	Weibull
Correlation with dataset	0.995	0.996
Alpha	-	1.2000
Beta	0.97937	1.4691
Gamma	1.54491	0.8130

Both the Gumbel and Weibull distribution fit the data well. Analysing the figures with the Weibull and Gumbel distribution on the following pages, both distributions fit the data well.

The following figures are provided at the following pages:

- ULS significant deep water exceedence probability
- ULS significant deep water return period


ULS deep water exceedence probability
Hs [m]
14
13
12
11
10
9
8
7
6
5
4
3
2
1
10
1
0.1
0.01
0.001
0.0001
Storm exceedence probability per year
Gumbel
Weibull
Dataset


ULS deep water return period
Hs [m]
12
11
10
9
8
7
6
5
4
3
1
10
100
1000
Return period (years)
Gumbel
Weibull
Dataset

SLS distribution

The SLS distribution is used for several goals. Data is necessary to predict the average yearly wave load on the breakwater to forecast the transmission. However, for the construction of the breakwater also a seasonal distribution of the waves during the year is important. Therefore, the significant wave height distribution is provided for the whole year as well as for different characteristic seasons.

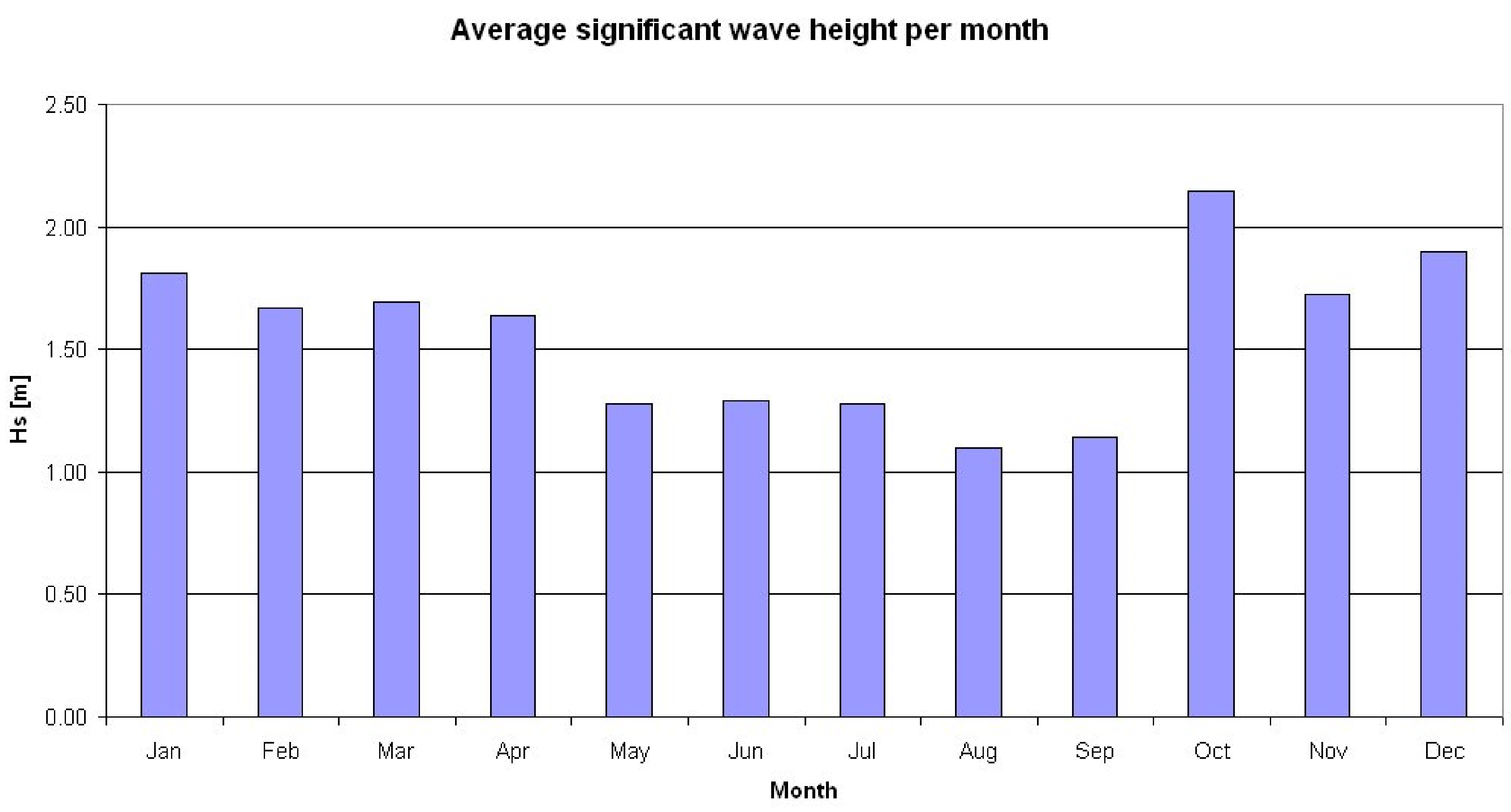
First the seasons were determined. The monthly distributions and the division in four wave seasons are provided at the following pages. Subsequently, a Gumbel and a Weibull distribution are fitted to the data by regression analysis for all four datasets. The results are provided in the following tables.

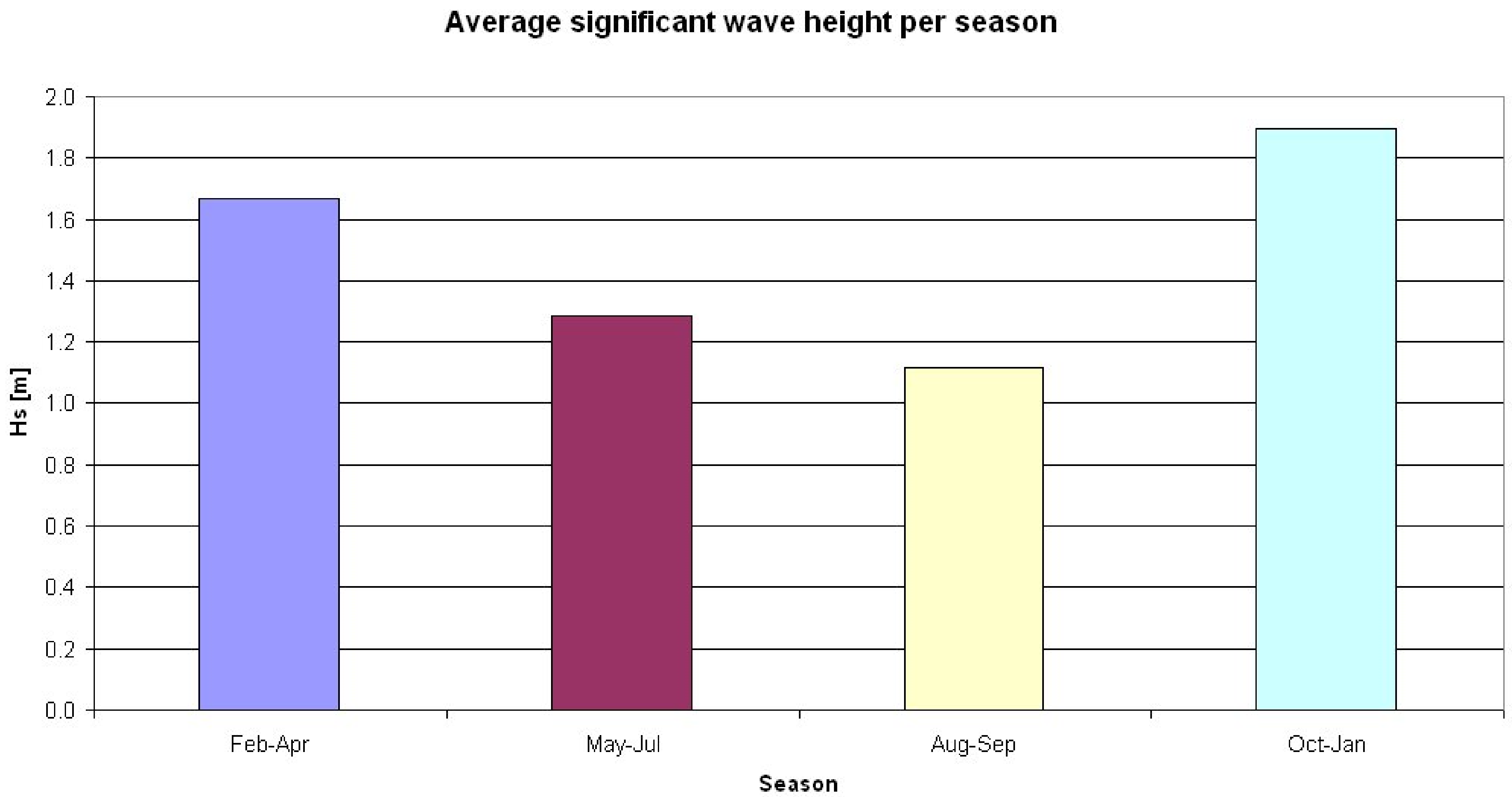
Yearly conditions	Weibull
Correlation with dataset	0.997
Alpha	0.9382
Beta	0.8512
Gamma	0.3948

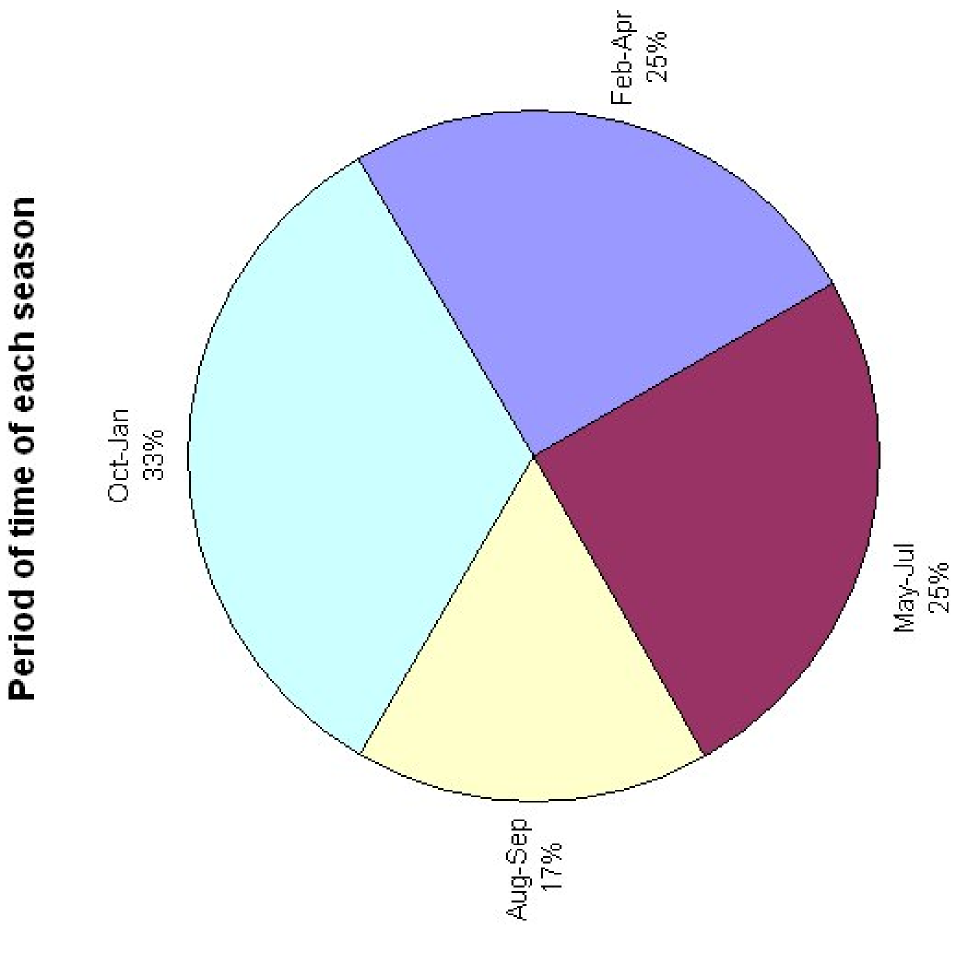
Season	%time seasonal occurrence	Average Hs [m]	Distribution	Beta	Gamma	Alpha
Feb-Apr	25%	1.67	Gumbel	0.6274	1.0624	
May-Jul	25%	1.28	Weibull	1.2111	-0.0458	2.3590
Aug-Sep	17%	1.12	Gumbel	0.3488	0.6546	
Oct-Jan	33%	1.90	Weibull	1.5028	0.2123	1.1561

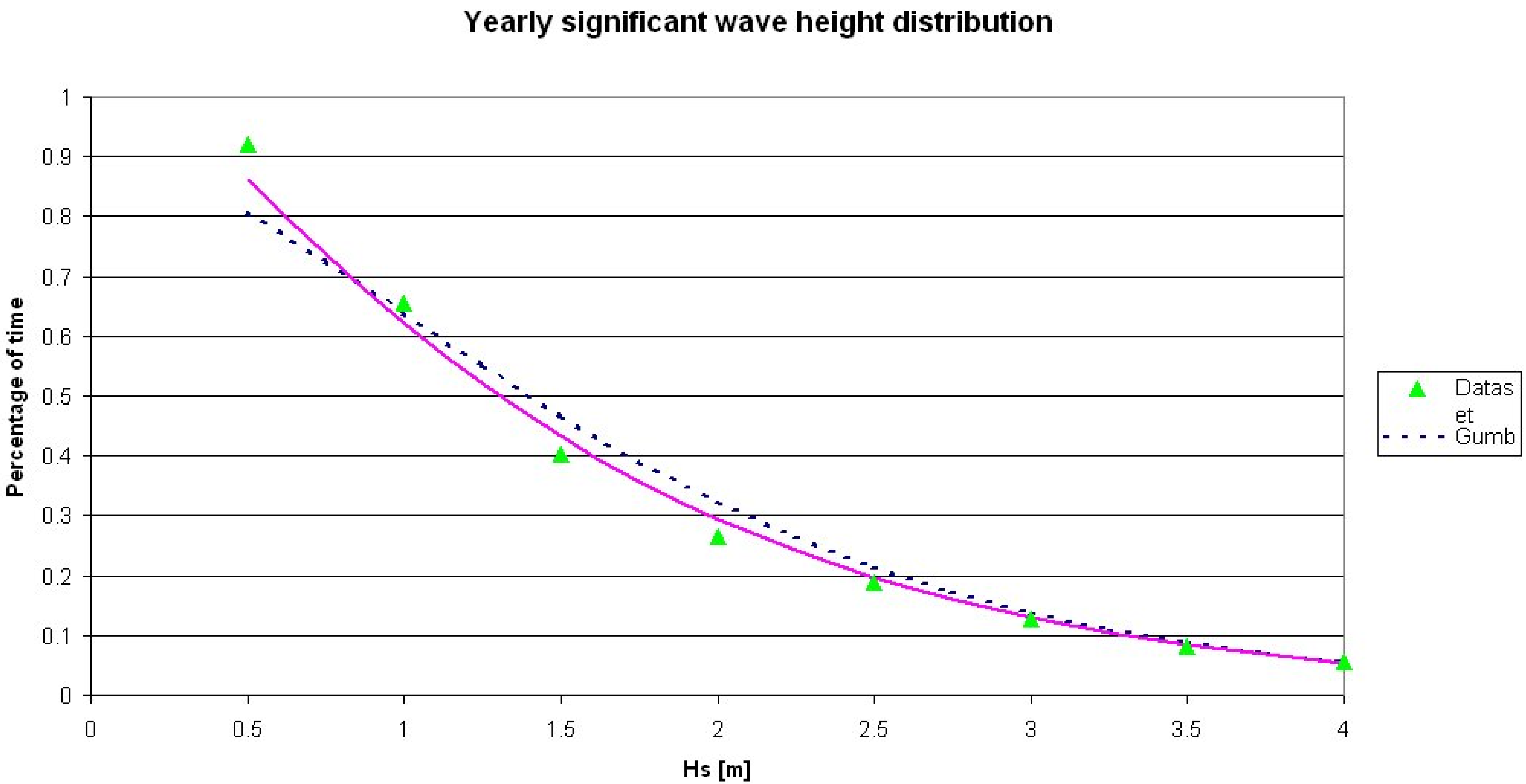
The following figures are provided at the following pages:

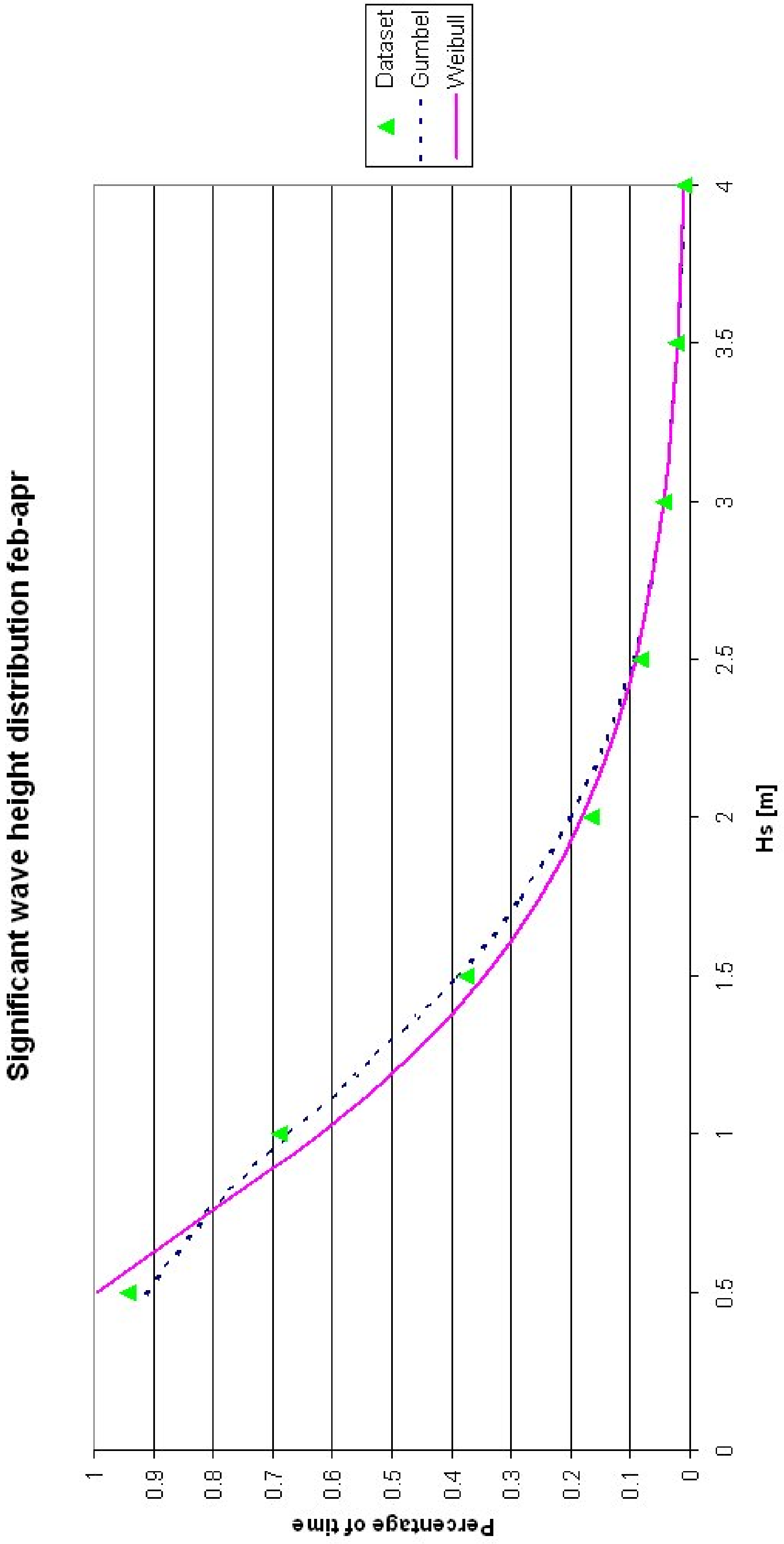
- Monthly distribution of the significant wave height
- Seasonally distribution of the significant wave height
- Period of time each season occurs
- Yearly wave conditions as a percentage of time
- Feb-Apr wave conditions as a percentage of time
- May-Jul wave conditions as a percentage of time
- Aug-Sep wave conditions as a percentage of time
- Oct-Jan wave conditions as a percentage of time


Average significant wave height per month
Hs [m]
2.50
2.00
1.50
1.00
0.50
0.00
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Month

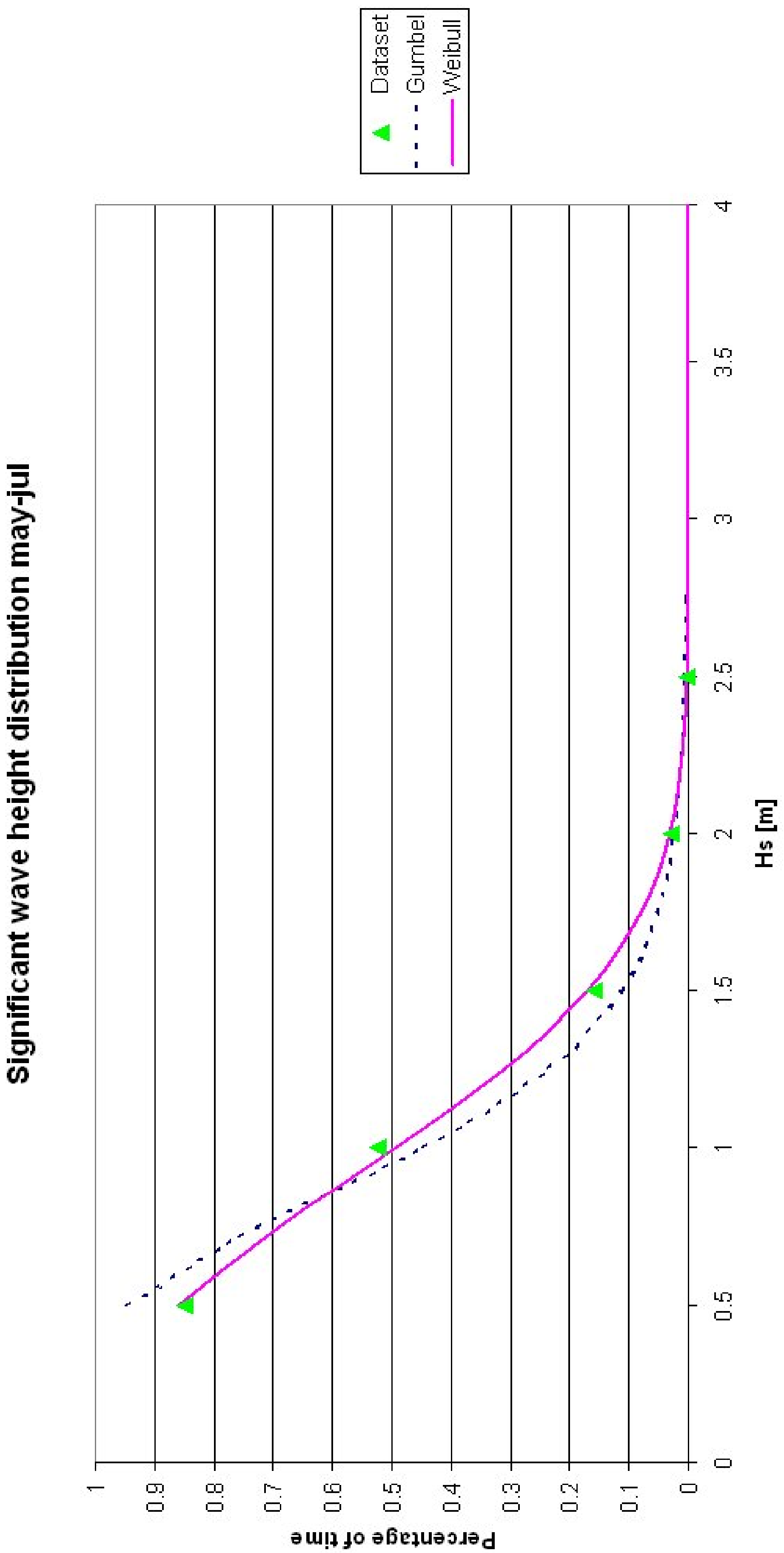

Average significant wave height per season
Hs [m]
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Feb-Apr
May-Jul
Aug-Sep
Oct-Jan
Season

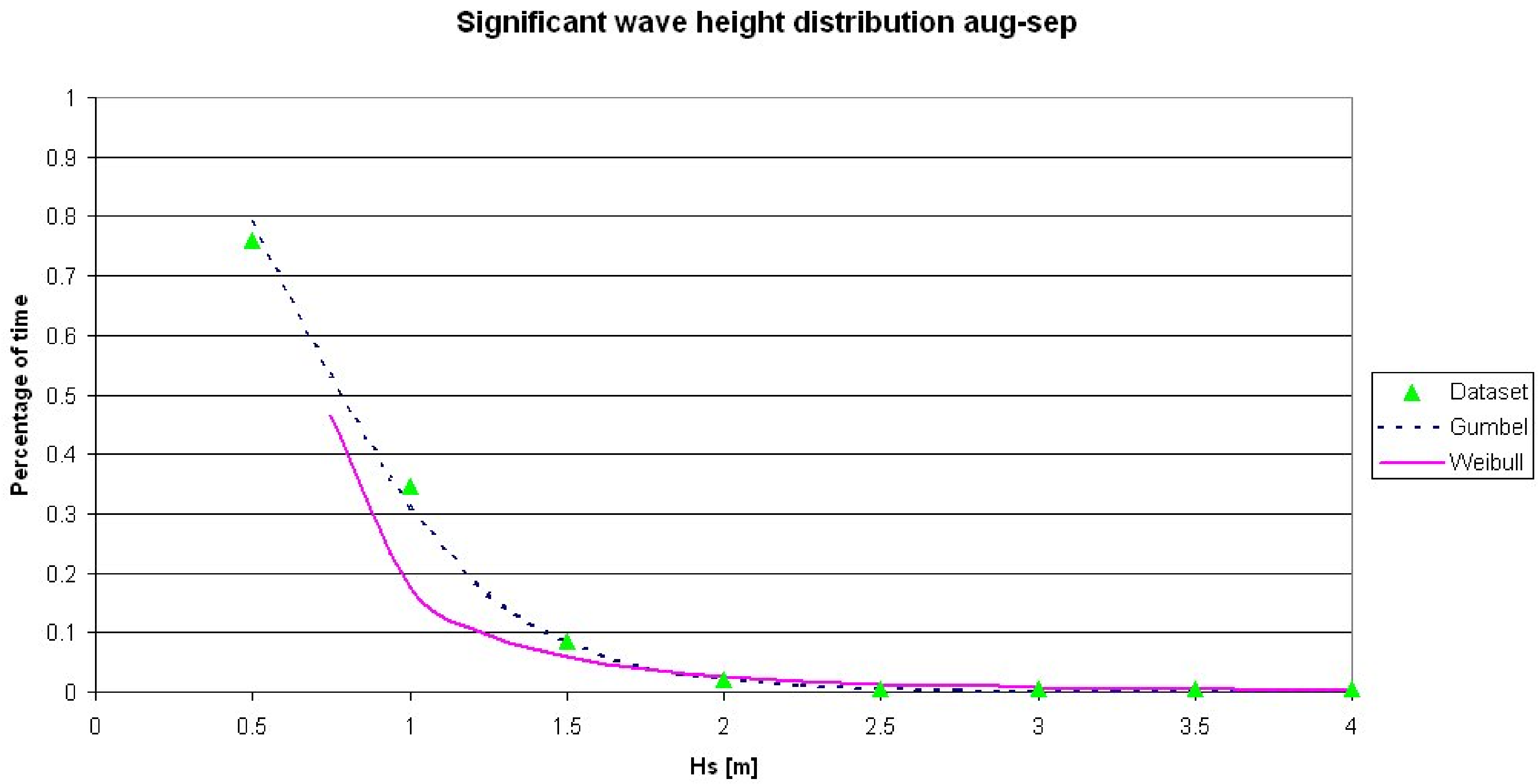

Period of time of each season
Oct-Jan
33%
Feb-Apr
25%
May-Jul
25%
Aug-Sep
17%

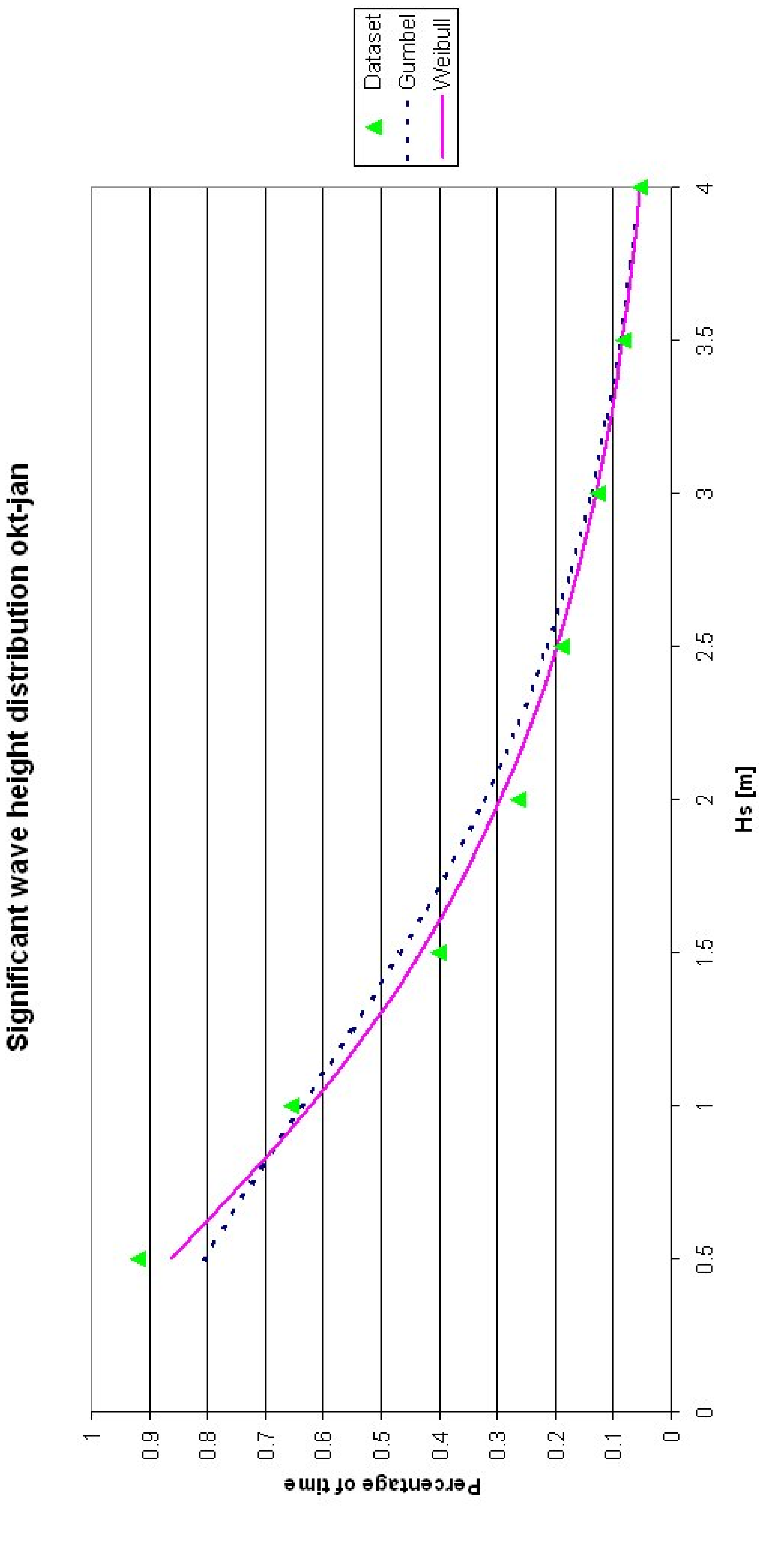

Yearly significant wave height distribution
Percentage of time
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
3
3.5
4
Hs [m]
Datas
et
Gumb



Significant wave height distribution feb-apr
Percentage of time
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
3
3.5
4
Hs [m]
Dataset
Gumbel
Weibull


Significant wave height distribution may-jul
Percentage of time
Hs [m]
Dataset
Gumbel
Weibull
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0
0.5
1
1.5
2
2.5
3
3.5
4


Significant wave height distribution aug-sep
Percentage of time
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
3
3.5
4
Hs [m]
Dataset
Gumbel
Weibull


Significant wave height distribution okt-jan
Percentage of time
Hs [m]
Dataset
Gumbel
Weibull
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.5
1.5
2
2.5
3
3.5
4

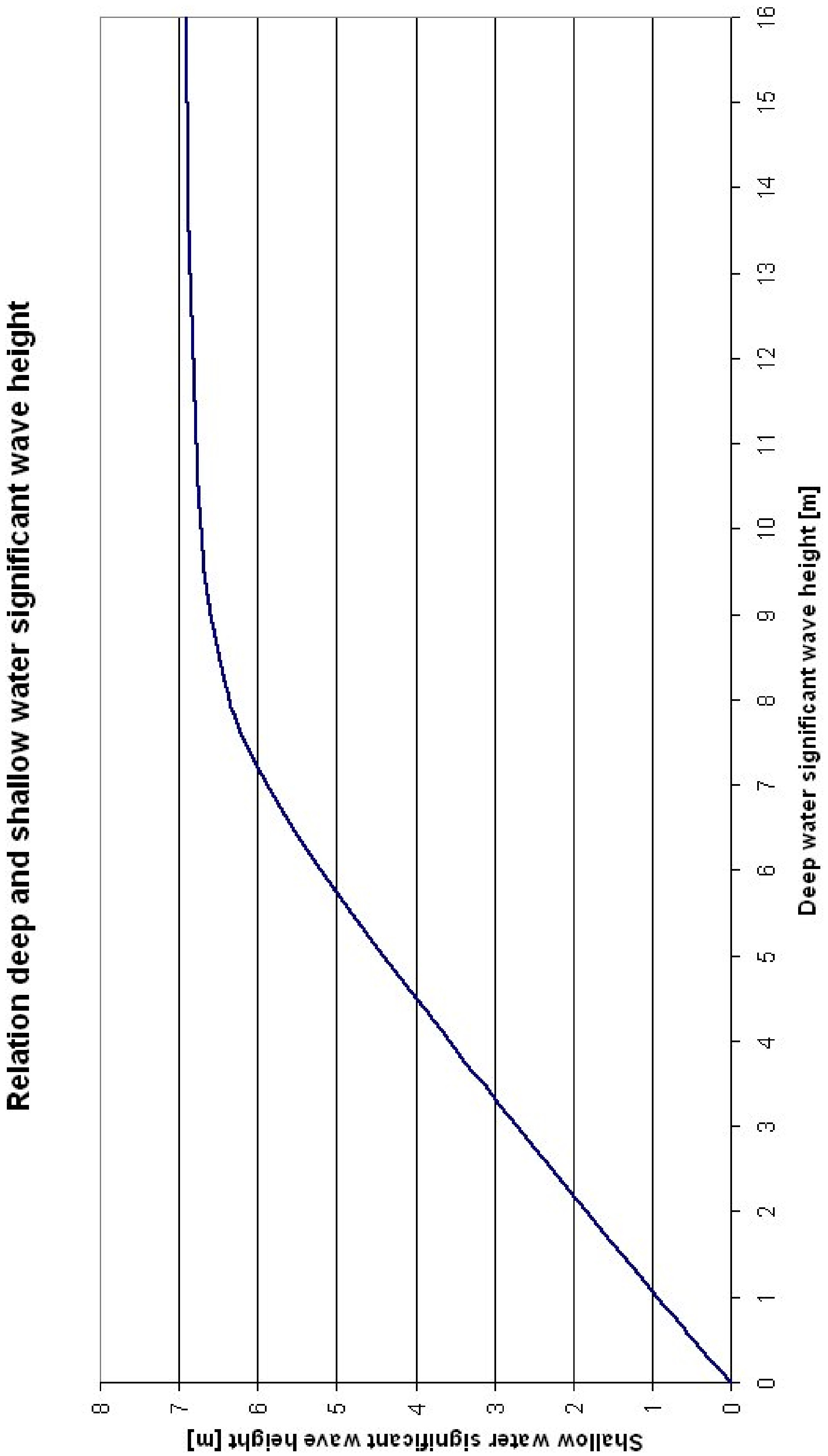
Appendix VI: Translation deep to shallow water wave height

The deepwater wave climate characteristics are transformed to the shallow water conditions at the breakwater, taking into account possible breaking of waves and bed friction. 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. For all calculations a still water level of 0.3 m +CD is assumed.

The shallow water wave height at the breakwater location is calculated with the bathymetry provided in Appendix I. At the breakwater location the water depth is 16.3 m. Thus the shallow water significant wave height is the significant wave height at that depth. For several deep water significant wave heights the resulting shallow water significant wave heights are shown in the table below.

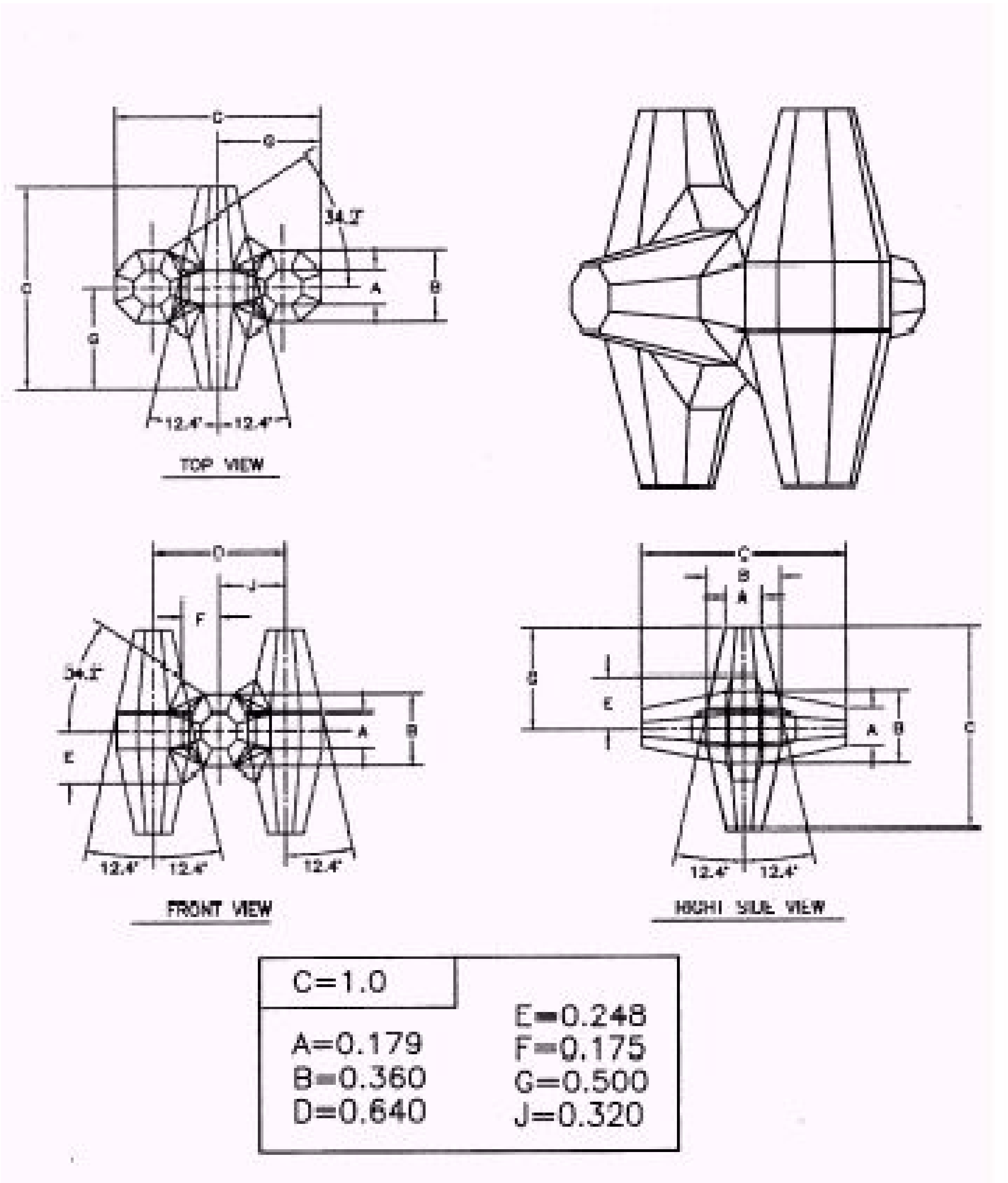
	CRESS		CRESS
Hs, 0	Hs, shore	Hs, 0	Hs, shore
[m]	[m]	[m]	[m]
0	0	4.5	3.99
0.5	0.47	5	4.4
0.75	0.7	5.5	4.81
1	0.93	6	5.2
1.25	1.16	6.5	5.57
1.5	1.38	7	5.89
1.75	1.61	7.5	6.16
2	1.83	8	6.36
2.25	2.06	9	6.6
2.5	2.27	10	6.72
2.75	2.5	11	6.79
3	2.71	12	6.83
3.25	2.94	13	6.86
3.5	3.14	14	6.89
3.75	3.37	15	6.91
4	3.57	16	6.92

The schematisation is also shown in the figure on the following page.



Relation deep and shallow water significant wave height
Shallow water significant wave height [m]
Deep water significant wave height [m]
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

Appendix VII: Core-loc® element dimensions



Appendix VIII: Balzapote quarry

For the breakwater a large amount of rock will be required for the construction of the core and filter layers and as concrete aggregate. According to a preliminary quarry analysis conducted by Boskalis (Boskalis 2002) the Balzapote quarry appeared to be the best location to acquire the demanded rock grading and is located at a reasonable distance close to the coast.

The physical properties of the rock at the Balzapote quarry are given in the following table.

Physical properties	Dimension	Value
Density	[t/m^3]	3.09
Water absorption	[%]	0.94
Los Angeles Abrasion	[%loss]	11.8
Uniaxial Compressive Strength	[MPa]	225

A preliminary yield curve is also provided. This quarry yield curve can be approximated with a mathematical description, the Rosin-Rammler equation:

$$y = 1 - e^{-\left(\frac{x}{x_c}\right)^n}$$

in which:

y: cumulative weight in % finer than x [-]

x: particle size (block size) [m]

x_c: characteristic particle size (approximately 63% smaller than x_c) [-]

n: index of uniformity [-]

With n = 0.75 and x_c = 2.68 m a close fit is achieved.

From the Rosin-Rammler equation the characteristics of the standard rock weight classes can be derived and are shown in the table underneath.

Weight class	% of yield	Dn10	Dn15	Dn50	Dn60	Dn85	Weight	D60/D10
[t]	[-]	[m]	[m]	[m]	[m]	[m]	[kg]	[-]
<0.001	30.2%							
0.01-0.06	16.0%	0.16	0.16	0.20	0.21	0.25	25	1.35
0.001-1	56.6%	0.09	0.10	0.23	0.28	0.47	36	3.05
0.06-0.3	14.4%	0.28	0.29	0.35	0.37	0.42	132	1.30
0.3-1	9.1%	0.48	0.49	0.56	0.58	0.64	530	1.21
1-3	6.2%	0.71	0.72	0.81	0.84	0.93	1649	1.19
3-6	2.8%	1.01	1.02	1.10	1.13	1.20	4124	1.12
>6	4.2%							
4-7	2.0%	1.11	1.12	1.19	1.21	1.27	5186	1.09

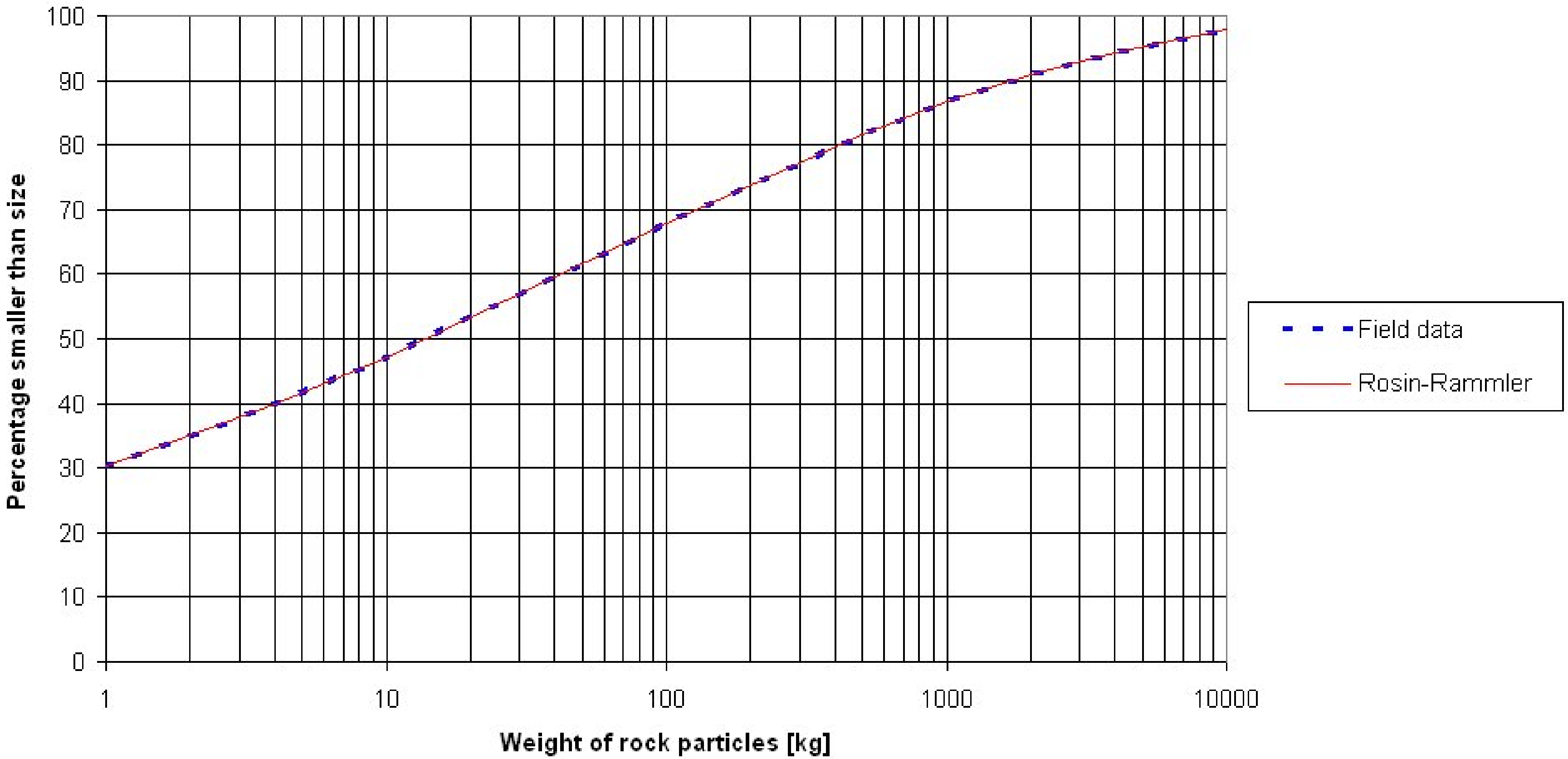
The internal stability is checked with the following rule:

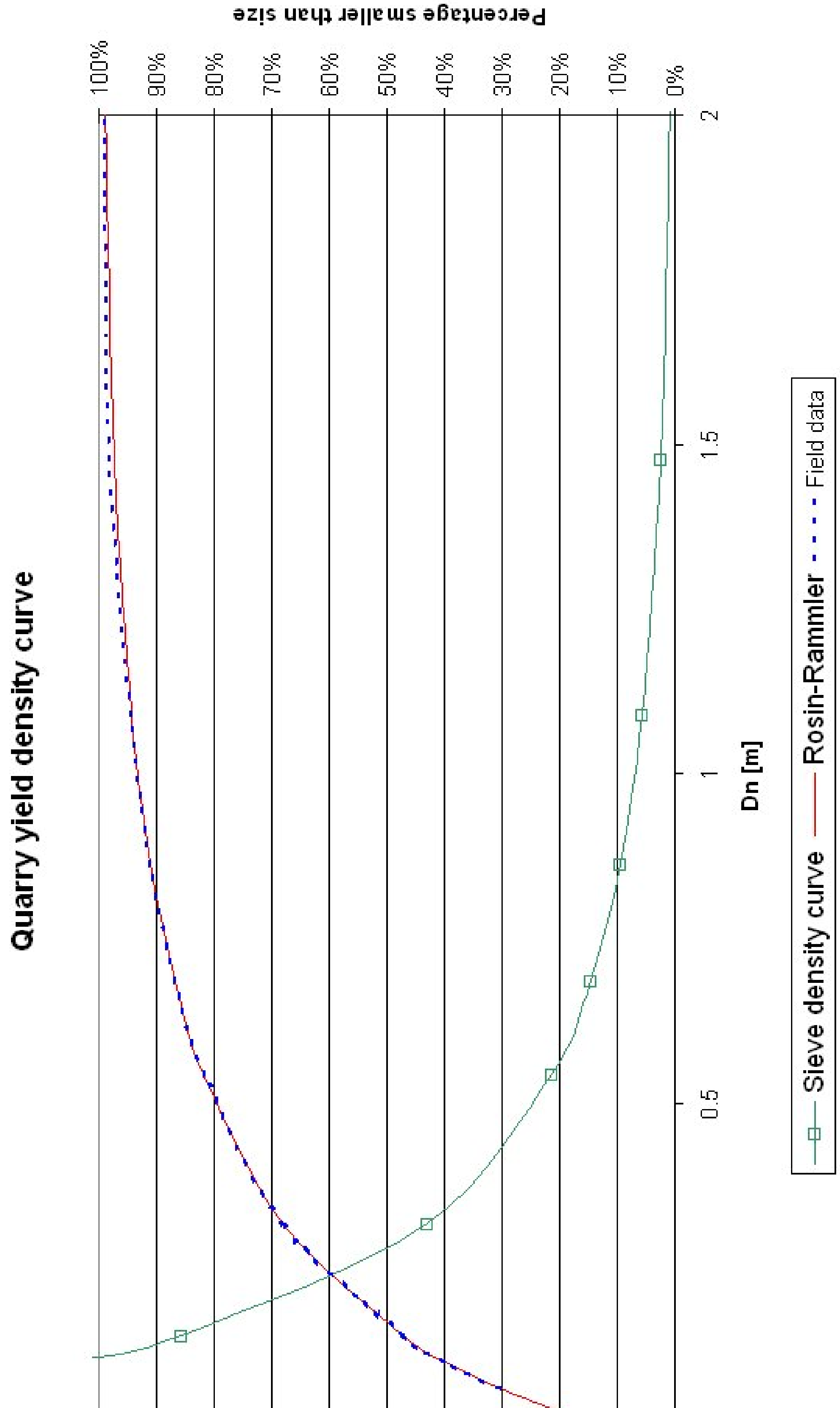
$$\frac{d_{60}}{d_{10}} < 10.$$

The ratios of all standard weight classes are satisfactory.

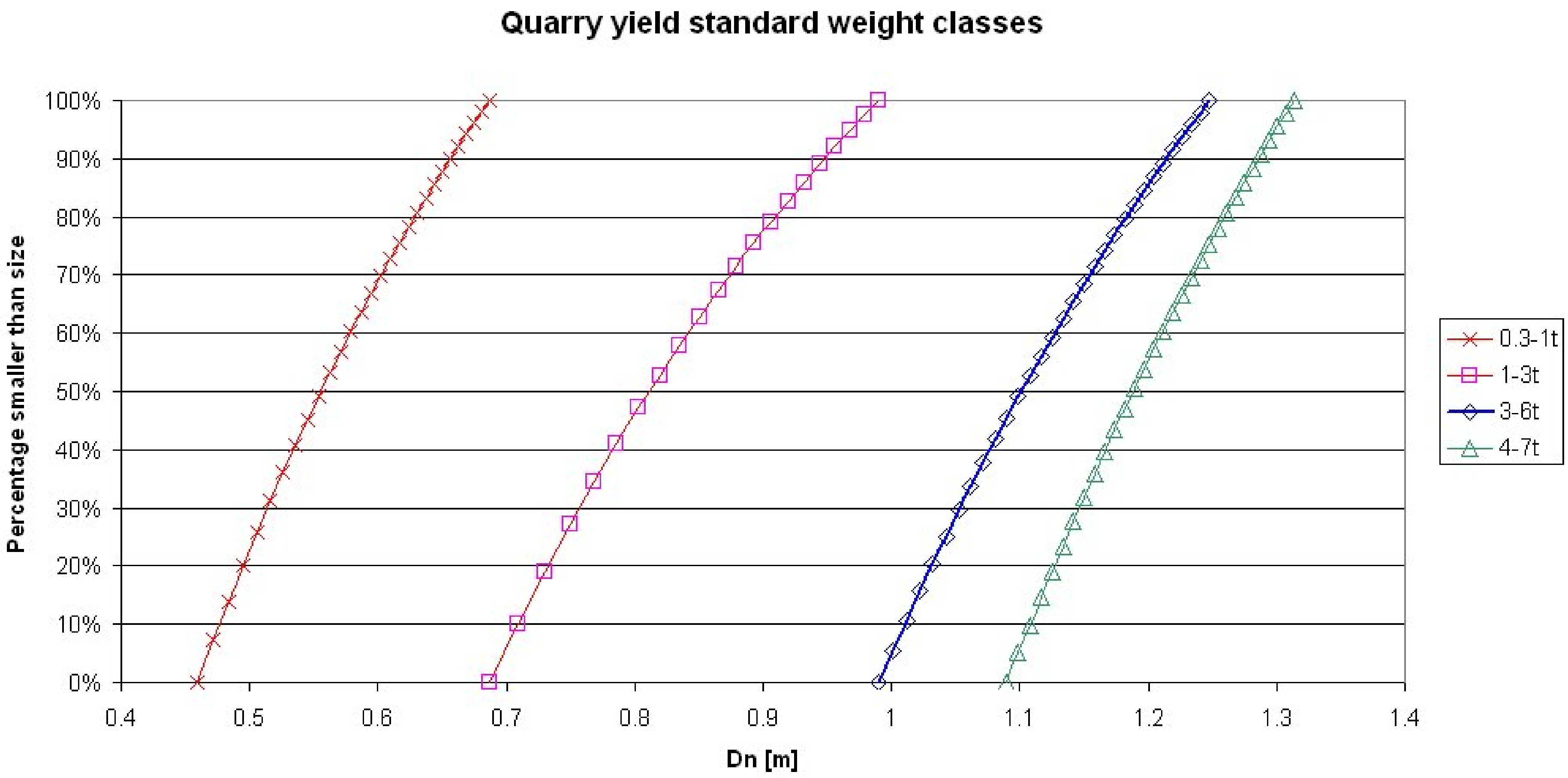
In the next figures the field data with the expected quarry yield curve (with the fitted Rosin-Rammler curve), the yield density curve and the sieve curves (based on the Rosin-Rammler curve) of several standard weight classes are shown.

Expected quarry yield curve for Balzapote quarry





Quarry yield density curve
Percentage smaller than size
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Yield density curve
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.5
1.5
Dn [m]
Sieve density curve
Rosin-Rammler
Field data


Quarry yield standard weight classes
Percentage smaller than size
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
1.3
1.4
Dn [m]
0.3-1t
1-3t
3-6t
4-7t

Appendix IX: Failure deterministic design

For the Ultimate Limit State of a deterministic design a sufficiently high wave height has to be chosen to represents the accepted failure of the breakwater. This is possible, if all other influences are assumed to be deterministic values, which have no stochastic nature.

The parameters that determine the design wave height are:

$p_{failure}$: accepted probability of failure during the lifetime of the breakwater

T : lifetime of the breakwater

f : frequency of the design wave height

RP : return period of the design wave height (=1/f)

The probability of failure can be approximated with the Poisson distribution:

$$p_{failure} = 1 - \exp(- f \cdot T)$$

This approximates the exact approximation:

$$p_{failure} = 1 - (1 - f)^T .$$

Rewritten this leads to:

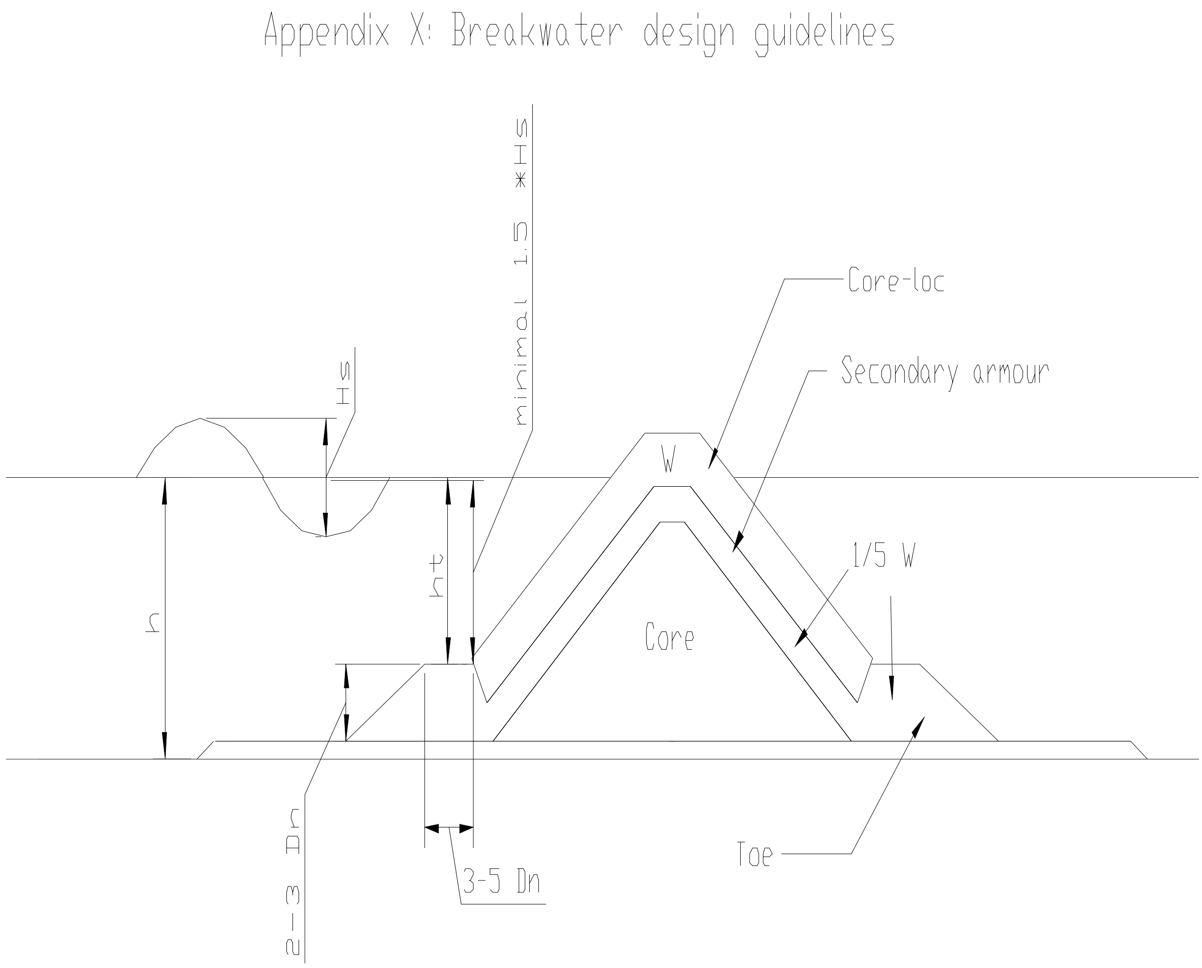
$$RP = \frac{1}{f} = \frac{1}{\frac{1}{T}\ln(1 - p_{failure})}$$

For several values of the probability of failure the results for a lifetime of 50 years are given in the following table.

Probability of failure	Return period	Return frequency
[-]	[years]	[1/years]
0.01	4975	0.0002
0.05	975	0.0010
0.10	475	0.0021
0.20	224	0.0045
0.22	200	0.0050
0.50	72	0.0139
0.64	50	0.0200
0.60	55	0.0183
0.99	11	0.0921

A probability of failure of 0.60 over the lifetime of the breakwater is chosen which leads to a return period of approximately 55 years. The return period is in this case almost equal to the lifetime of the breakwater. According to PIANC (1992) this is a realistic design return period.

Appendix X: Breakwater design guidelines



Appendix XI: Stability formulas for several breakwater armour types

The stability formulas for Accropodes, Tetrapods and Cubes in a single and double layer are given by Van der Meer (2002) for the start of damage ($N_{od} = 0$).

Accropodes

$$\frac{H_s}{\Delta \cdot D_n} = C_{Accropodes} = 3.7 .$$

Tetrapods

$$\frac{H_s}{\Delta \cdot D_n} = C_{Tetrapods} = 0.85 \cdot s_{0m}^{-0.2} .$$

In which s_{0m} is the wave steepness at deep water based on the deep water wave height and mean deep water wave length.

Cubes double layer

$$\frac{H_s}{\Delta \cdot D_n} = C_{Cubes;\, double\ layer} = 1.0 \cdot s_{0m}^{-0.1} .$$

Cubes single layer

$$\frac{H_s}{\Delta \cdot D_n} = C_{Cubes;\, \sin gle\ layer} = 3.0 .$$

Rubble mound

The following formulae provided by Van der Meer (1987a) are used to determine the stability of the rubble mound:

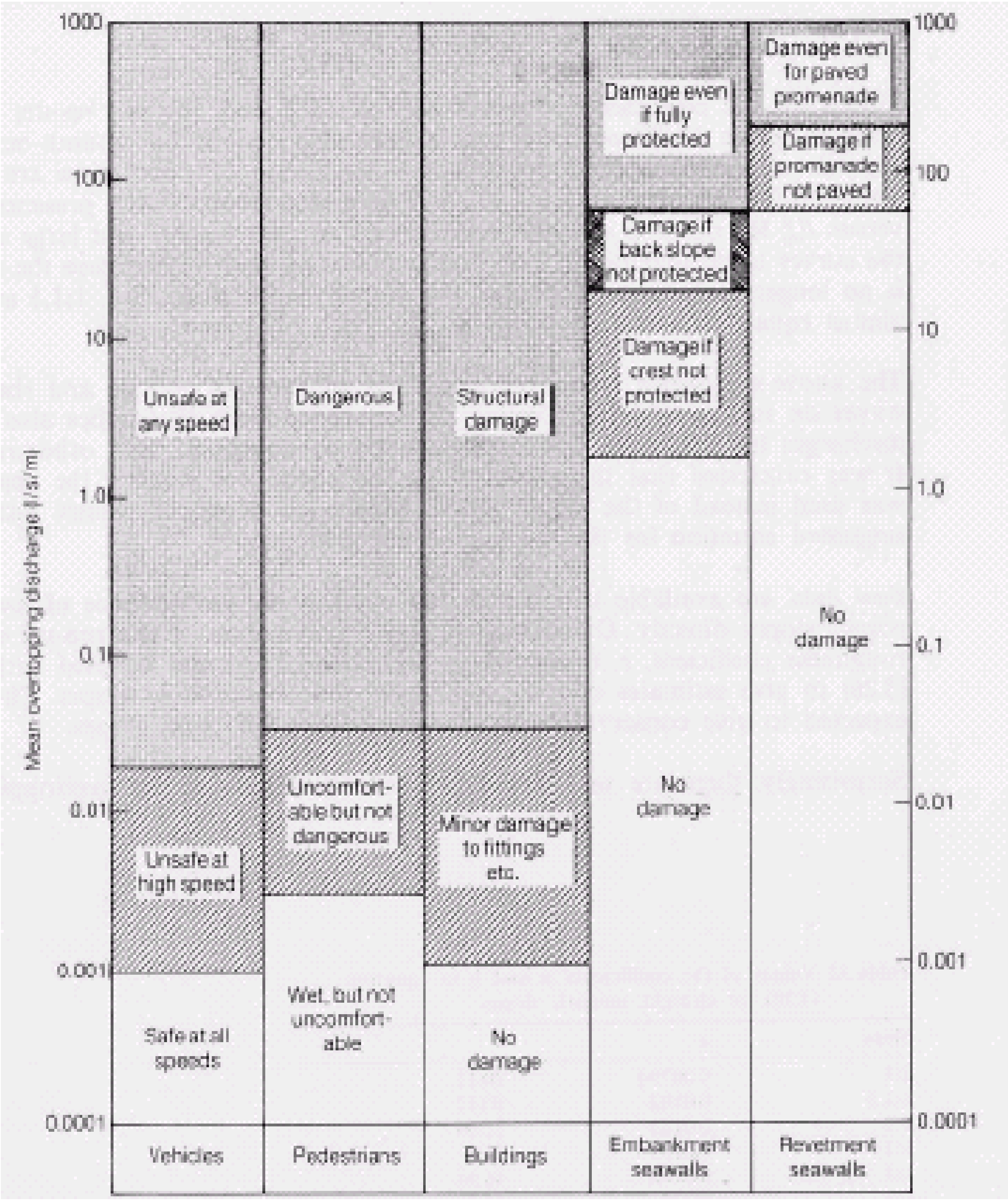
For plunging waves: $\dfrac{H_s}{\Delta \cdot D_{n50}} = 6.2 \cdot P^{0.18} \cdot \left(\dfrac{S}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5}$.

For surging waves: $\dfrac{H_s}{\Delta \cdot D_{n50}} = 1.0 \cdot P^{0.13} \cdot \left(\dfrac{S}{\sqrt{N}}\right)^{0.2} \cdot \sqrt{\cot\alpha} \cdot \xi_m^{P}$.

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

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

Appendix XII: Critical overtopping discharges



Critical overtopping discharges (CIRIA, 1991)

Appendix XIII: Quarry optimisation

The quarry output is described by the density distribution curve in Appendix VIII. From the distribution curve the output in percentage of the total rock output for the different rock sizes and classes can be determined. This data is already provided in Appendix VIII and repeated in the following table.

Weight class	% of yield
[t]	[-]
0.01-0.06	16.0%
0.001-1	56.6%
3-6	2.8%

For the breakwater alternatives the following quantities are approximated. The 10-60kg stone class overlaps with the 1-1000kg stone class and both are summarised in the column ‘Overlap’.

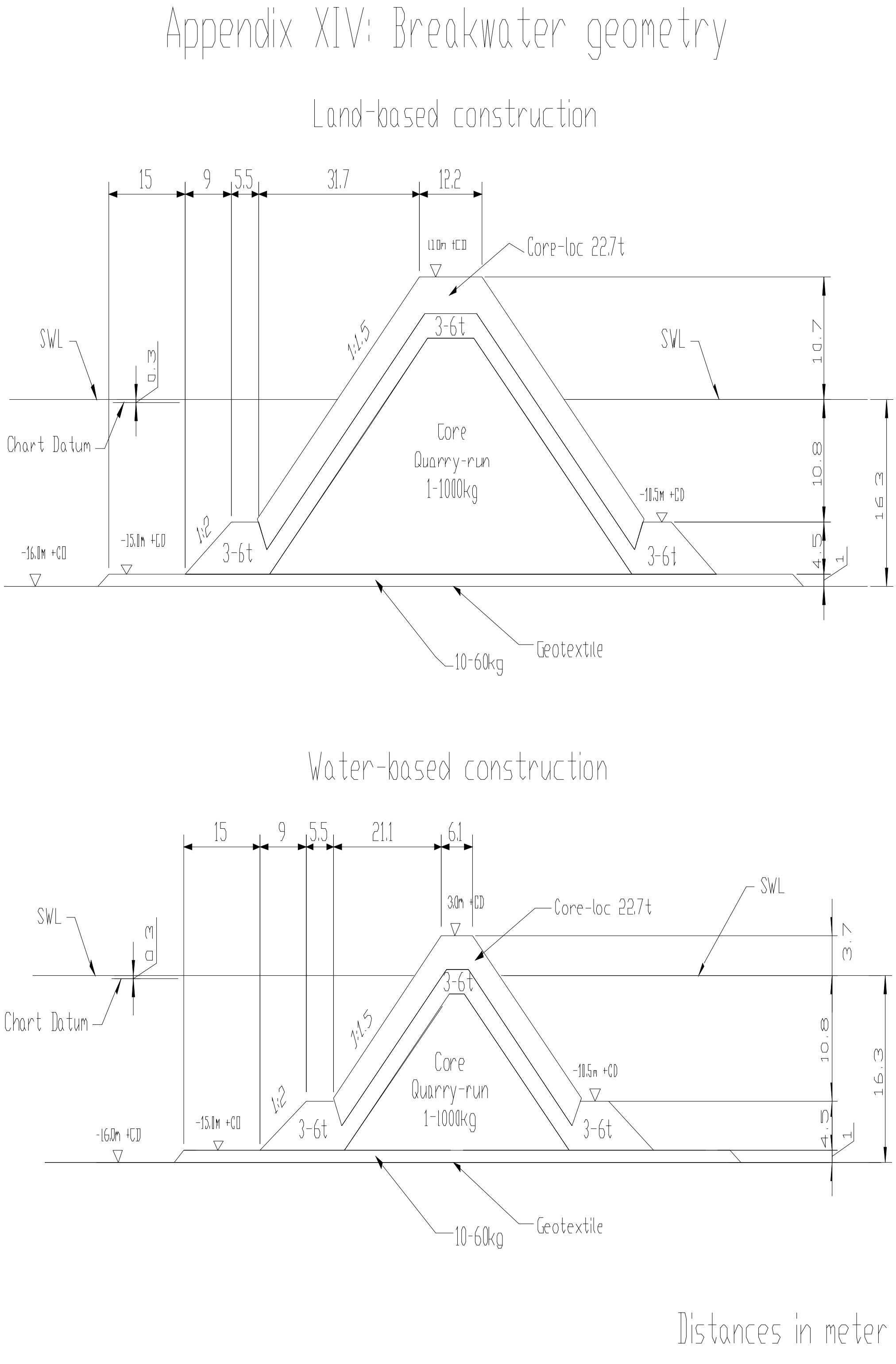
Water-based	Per 1500 m		
	Total	Distribution	Overlap
Rip-rap	[t]	[-]	[-]
1-1000kg	940,000	50%	67%
3-6t	620,000	33%	33%
10-60kg	310,000	17%	-
Total	*1,870,000*	*100%*	*100%*
Land-based	**Per 1500 m**		
	Total	Distribution	Overlap
Rip-rap	[t]	[-]	[-]
1-1000kg	2,780,000	69%	79%
3-6t	840,000	21%	21%
10-60kg	400,000	10%	-
Total	*3,920,000*	*100%*	*100%*

The data is combined in the following table and the yield/ demand ratio is determined.

Weight class	Yield	Demand water-based	Ratio yield/demand	Demand land-based	Ratio yield/demand
[t]	[-]	[-]	[-]	[-]	[-]
0.001-1	57%	67%	1.2	79%	1.4
3-6	3%	33%	11.8	21%	7.5
Other	40%	-		-	

For the water-based and land-based breakwater respectively 12 and 7.5 times more rock has to be produced compared to a perfect demand fitting quarry output distribution. The quarry yield and breakwater demand are far from matching. A large amount of surplus rock will have to be quarried. Unless this material can be used elsewhere this amount will contribute considerably to the total costs of the breakwater. In that case the use of concrete elements for the toe and the secondary layer could be more economic.

Appendix XIV: Breakwater geometry



Appendix XV: Wave energy

The total average wave energy per square unit is expressed by the following equation:

$$E = \tfrac{1}{8} \cdot \rho \cdot g \cdot H^2 .$$

In which,

E = total wave energy per square unit [J/m^2]

ρ = specific density of water [kg/m^3]

g = acceleration of gravity [m/s^2]

H = wave height [m]

The wave energy transmitted into the port basin is composed of the transmitted energy through the entrance and the energy transmitted through and over the breakwater:

$$E_{port\ ba\sin} = E_{entrance} + E_{transmission} = \tfrac{1}{8} \cdot \rho \cdot g \cdot H^2_{port\ ba\sin} = \tfrac{1}{8} \cdot \rho \cdot g \cdot H^2_{entrance} + \tfrac{1}{8} \cdot \rho \cdot g \cdot H^2_{transmission}$$

In which,

$E_{port\ basin}$ = wave energy transmitted into the port basin [J/m^2]

$E_{entrance}$ = wave energy transmitted through the entrance [J/m^2]

$E_{transmission}$ = wave energy transmitted through and over the breakwater [J/m^2]

$H_{port\ basin}$ = wave height transmitted into the port basin [m]

$H_{entrance}$ = wave height transmitted through the entrance [m]

$H_{transmission}$ = wave height transmitted through and over the breakwater [m]

The equation can be simplified to:

$$H^2_{port\ ba\sin} = H^2_{entrance} + H^2_{transmission} .$$

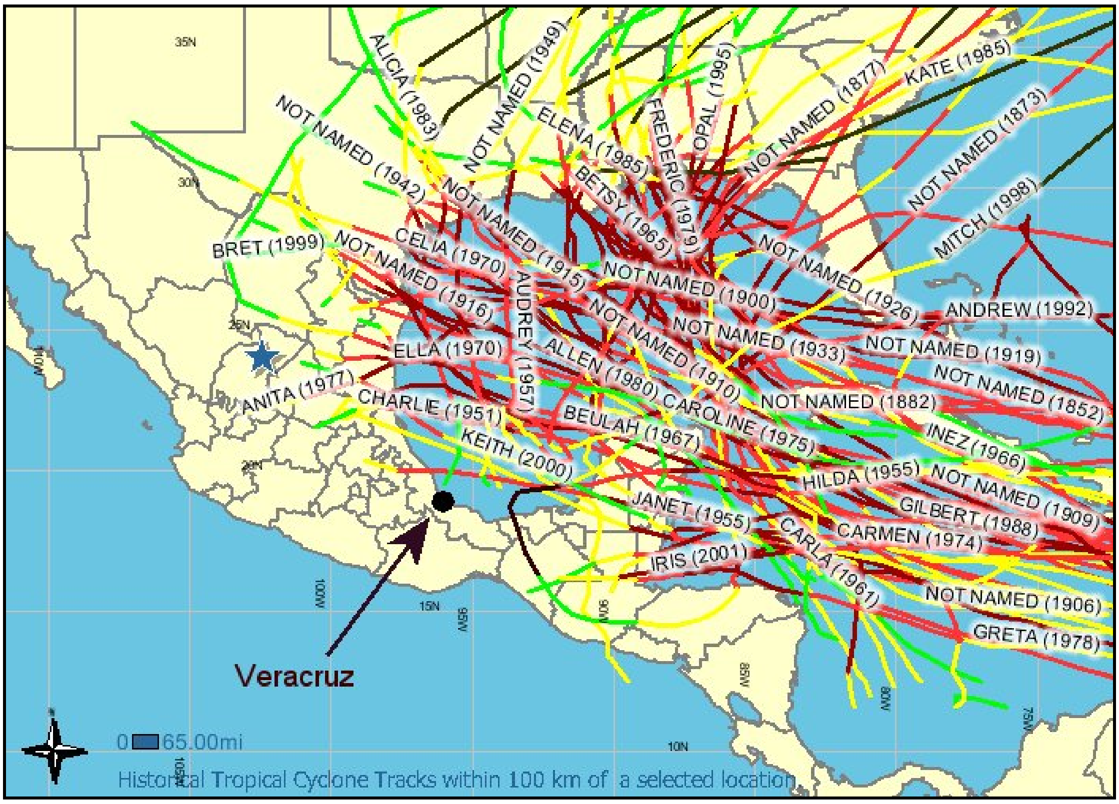
Appendix: XVI Hurricanes

Hurricanes are severe tropical storms that form in the Gulf of Mexico. If a hurricane passes the breakwater location an extreme wave load will be exerted on the protective armour layer. In the following table the Saffir-Simpson scale classification of hurricanes is provided.

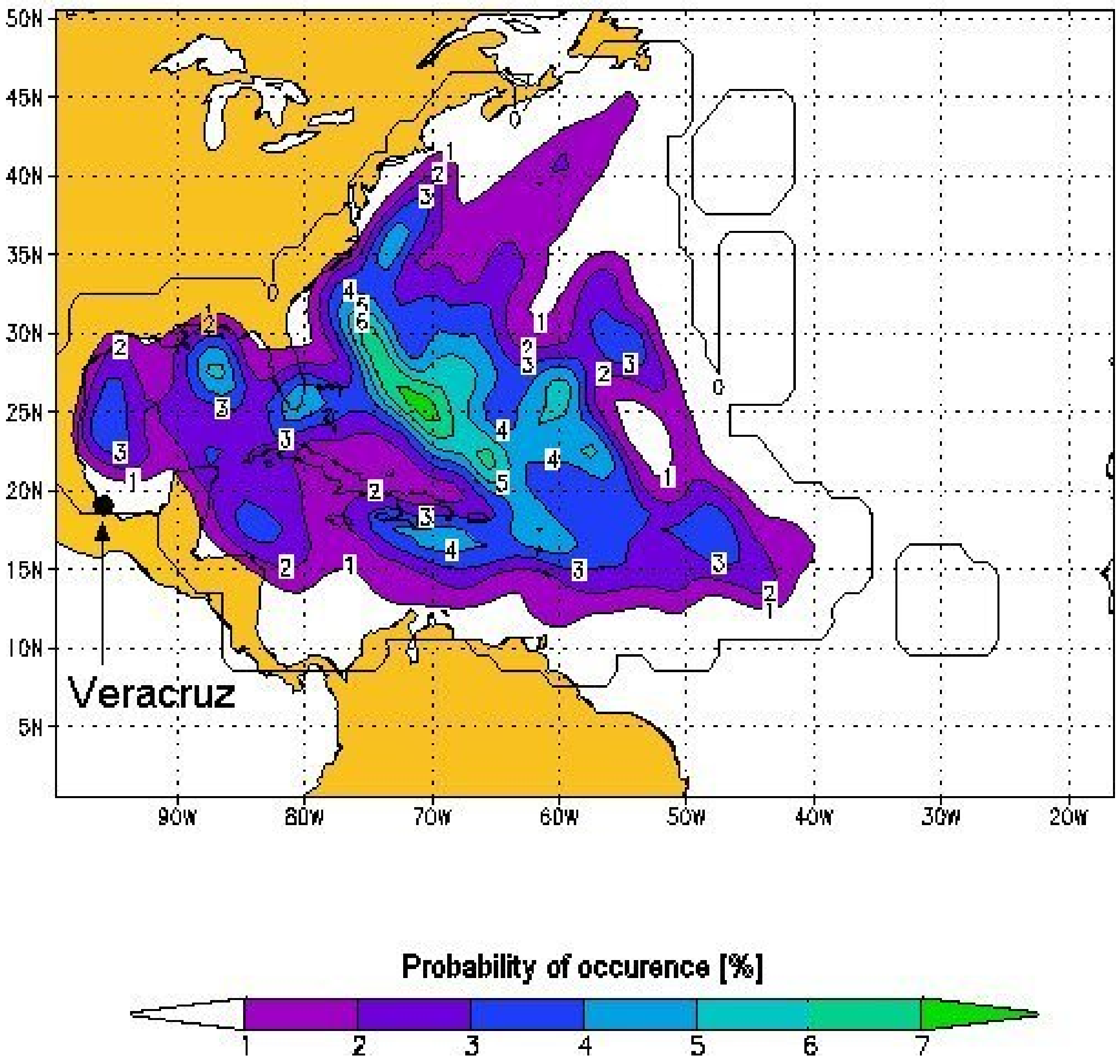
Type	Category	Pressure [mb]	Wind [knots]	Storm surge [m]
Depression	TD	-	< 34	-
Tropical storm	TS	-	34-63	-
Hurricane	1	> 980	64-82	~ 1.5
Hurricane	2	965-980	83-95	~ 2.0-2.5
Hurricane	3	945-965	96-113	~ 2.5-4.0
Hurricane	4	920-945	114-135	~ 4.0-5.5
Hurricane	5	< 920	>135	> 5.5

The Saffir-Simpson scale categorizes hurricanes on a scale from 1 to 5. Category 1 hurricanes are the weakest and 5 is the most intense. Hurricanes strong enough to be considered intense start at category 3.

The severe tropical storms have been monitored for the last 100 years (Cf. Jarvinen et al., 1984). The paths of the recorded hurricanes are given in the following figure.



Further processing of the hurricane data (Cf. Jarvinen et al, 1984) resulted in the probability of occurrence of an intense hurricane in the Gulf of Mexico indicated in the following figure.



The probability of occurrence of an intense hurricane is very small for Veracruz. From the data in the figure the probability of occurrence during the hurricane season is determined at 0.005 per year.

The wave height distribution of the hurricanes is unknown. However, since the wave height at the breakwater location is depth limited for very high waves, the storm surge level will determine the maximum wave height if a hurricane occurs. The table with the Saffir-Simpson scale provides an indication of the storm surges that can be expected. A storm surge of 3 m is assumed if a hurricane occurs. For probabilistic calculations a standard deviation of 1 m is additionally assumed to take into account the large uncertainty of the assumed storm surge.

Appendix XVII: Program listing ColVer

```
Program ColVer

Use Prob

Implicit none

! *** Program constants ***
Integer,parameter:: Nvar = 21    ! Number of random variables (X)
Integer,parameter:: Ndes = 2     ! Number of design variables (P)
Integer,parameter:: Npar = 3     ! Maximum number of distribution parameters
Integer,parameter:: Nlim = 4     ! Number of limit states
Integer,parameter:: Ncut = 4     ! Number of cut sets
! *** End constants ***

! *** Begin variables ***
Real     X(Nvar,Npar)     ! Basic variables
Real     P(Ndes)                 ! Parameters
Integer  Isens                   ! Switch for sensitivity analysis
Real     beta(Nlim)              ! Reliability index
Real     Pf(Nlim)                ! Failure probability
Real     alpha(Nvar,Nlim)        ! Influence factors
Real     Xdes(Nvar,Nlim)         ! Design point in physical space
Real     Udes(Nvar,Nlim)         ! Design point in standard-normal space
Real     tol                     ! Break-off criterion
Real     maxit                   ! Maximum number of iterations
Integer err                      ! Error code

Integer logmat(Nlim,Ncut)        ! Logical matrix (fault tree)
Real     rho(Nvar,Nlim,Nlim)     ! Correlation matrix
Real     betasys                 ! Reliability index of system
Real     Pfsys                   ! Failure probability of system
Real     Betacut(Ncut)           ! Reliability index by cut set
Real     Pfcut(Ncut)             ! Failure probability by cut set
Real     alphacut(Nvar,Ncut)     ! Influence factors by cut set
Real     alpsys_comp(Nlim)       ! Influence factor quantifying influence of individual
failure modes
Real     alpsys_var(Nvar)        ! Influence factors of system
Real     Usys(Nvar)              ! Design point of system

Real     PfC(2)                  ! Cornell system bounds
Real     PfD(2)                  ! Ditlevsen system bounds

Real     Nmin                    ! Minimum number of simulations

Integer i,j,k,l                  ! Counters

 ! Variables of main program
Real     paramCL
Real     paramYrs
Real     V(8)                    ! Variation parameter Dn

! *** End variables ***
```

```
! *** Begin program ***

! *** Open files for output ***
Open(9000,file='IIIcol000yr50.prn',action='write')

! *** Initialise all variables that serve as input ***

  ! Basic variables
!  X(1,1:3) = (/4.0,  1.544,   0.979/)                 !Ho 1yr
  X(1,1:3) = (/4.0,  5.374,   0.979/)                  !Ho 50yr
  X(2,1:3) = (/1.0,    0.0,     1.0/)                  !CclSD
  X(3,1:3) = (/1.0,  2.200,   0.050/)                  !rhoc
  X(4,1:2) = (/0.0,  1.025/)                           !rhow
  X(5,1:3) = (/1.0,   0.31,    0.25/)                  !hmax
  X(6,1:3) = (/1.0, -10.50,    0.30/)                  !ztoe
  X(7,1:3) = (/1.0, -16.00,    0.50/)                  !zbed
  X(8,1:2) = (/0.0,    0.5/)                           !Nod
  X(9,1:3) = (/1.0,   1.10,    0.05/)                  !Dn50
  X(10,1:3)= (/1.0,   0.45,    0.02/)                  !break
  X(11,1:3)= (/1.0,    1.0,    0.13/)                  !fHs1
  X(12,1:3)= (/1.0,    0.0,    0.15/)                  !fHs2
  X(13,1:3)= (/1.0,  3.090,   0.300/)                  !rhor
  X(14,1:2)= (/0.0,    0.0/)                           !zsurge
  X(15,1:2)= (/0.0,  0.001/)                           !a
  X(16,1:2)= (/0.0, 0.0691/)                           !b
  X(17,1:2)= (/0.0,   0.00/)                           !maxSLR
  X(18,1:3)= (/1.0,    3.0,     1.0/)                  !zsurgeH
  X(19,1:3)= (/1.0,    0.0,     1.0/)                  !randomH
  X(20,1:2)= (/1.0,   10.0/)                           !Tm
!  X(21,1:2)= (/0.0,  -0.33396/)                       !prob-0.333958628643304
  X(21,1:2)= (/0.0,   0.11796/)                        !prob 0.117962467457401

  ! Parameters
  P(1) = 0e0                          !start value
  P(2) = 0e0

 ! Waardes V
  V(1) = 1.5874 !Dn
  V(2) = 1.8171
  V(3) = 2.0000
  V(4) = 2.1544
  V(5) = 2.2894
  V(6) = 2.4101
  V(7) = 2.5198
  V(8) = 2.6207

  ! Sensitivity analysis (yes/no)
  Isens = 0  ! no

  ! Design point by failure mode (starting point)
  Udes = 0e0     ! All elements of vector equal to zero

  ! Logical matrix
  Logmat(:,1) = (/1,0,0,0/)          ! Cut set 1: mode 1 active
  Logmat(:,2) = (/0,1,0,0/)          ! Cut set 2: mode 2 active
  Logmat(:,3) = (/0,0,1,0/)          ! Cut set 3: mode 3 active
  Logmat(:,4) = (/0,0,0,1/)          ! Cut set 4: mode 4 active
```

```
  ! Correlation matrix
  rho = 1e0

  ! Minimum number of simulations
  Nmin = 0        ! Determine number of simulations on the basis of achieved accuracy

  ! Break-off criteria
  tol = 1e-3      ! Recommended value
  maxit = 200     ! Recommended value

!         Write(*,'(A80)')    '  Yr    V      Pfcl    Pftoe PfclH PftoeH PfC(1)  PfD(1) Pfsys
PfD(2)  PfC(2)  '
!         Write(9000,'(A80)') 'Yr V Pfcl Pftoe PfclH PftoeH PfC(1) PfD(1) Pfsys PfD(2)
PfC(2)'
          Write(*,'(A80)')    'Yr    V      Pfcl    Pftoe PfclH PftoeH  Pfsys'
          Write(9000,'(A80)') 'Yr V Pfcl Pftoe PfclH PftoeH Pfsys'

  ! *** Main: reliability calculations for a number of CL-weights for 50 years***

  ! *** Loop: 50 lifetime years
 Do paramYrs = 1,50

  ! *** Loop: 8 Core-loc® weights
  Do paramCL = 1,8

!  Do B=5e0,25e0,1e0 ! Loop over B

          ! Store value of Dncl in appropriate element of P
          P(1) = V(paramCL)
          ! Store value of year in appropriate element of P
          P(2) = paramYrs

          ! Calculate reliability by failure mode and show result on screen
          Do i=1,Nlim
           Call MCsys(X,P,logmat,Nmin,Pfsys,betasys,Pfcut,betacut,Pf,beta)
!           Call FORM(X,P,i,Isens,beta(i),Pf(i),alpha(:,i),Xdes(:,i),Udes(:,i),tol,maxit,err)
          Enddo ! i

          ! Analyse fundamental bounds and Ditlevsen bounds
!II       PfC = Cbound(beta)
!II       PfD = Dbound(beta,alpha,rho,err)

          ! Perform system reliability analysis by Hohenbichler/Rackwitz
!II       Callrelsys(logmat,beta,alpha,Isens,rho,betasys,Pfsys,Betacut,Pfcut,
          alphacut,alpsys_comp,alpsys_var,Usys,err)

          ! Write results to file and screen
          ! Width, Pf by mode (vector), lower bounds, Hohenbichler/Rackwitz,
          Upper bounds
          Write(9000,'(F8.1,F8.0,5ES12.3)') P(2),P(1)**3,Pf(1),Pf(2),Pf(3),Pf(4),Pfsys
          Write(*,'(F8.1,F8.0,5ES12.3)') P(2),P(1)**3,Pf(1),Pf(2),Pf(3),Pf(4),Pfsys

   Enddo          ! paramCL
  ! *** End loop Yrs
  Enddo! paramYrs
  ! *** End loop CL
```

```
  ! *** End main ***

! *** Close files ***
  Close(9000)

  Pause 'Calculation finished, press a key'

! *** End program ***

End program ColVer


Module Compon
! Used by probmod

Implicit none

! Module containing limit state equation for a specific case

! Delft University of Technology
! Hydraulic and Offshore Engineering Section
! Probabilistic Methods

Contains
! ************* Start of module subroutines **********************
! General interfacing limit state function
Function limit(X,P,I,newiter)

! Interface variables
Real limit
Real,Intent(in):: X(:)
Real,Intent(inout):: P(:)
Integer,Intent(in):: I
Logical,Intent(in):: newiter

! Common variables
! (void)

! Internal variables
Real Hshore
Real H0
Real Ccl
Real CclSD
Real rhoc
Real rhow
Real rhor
Real Dncl
Real H0SD
Real SLR
Real Year
Real a
Real b
Real Tm

Real hmax
Real ztoe
Real zbed
Real Nod
```

```
Real Dn50
Real break              !breaking wave/depth ratio
Real fHs1               !wave dependent uncertainty
Real fHs2               !wave independent uncertainty
Real zsurge

Real deltaC
Real deltaR
Real ht
Real h

Real N
Real Cw
Real Cil
Real maxSLR

Real randomH
Real HshoreH
Real hH
Real htH
Real zsurgeH

Real YearM
Real prob

! Constants
! (void)

! ** Start function **

! *** Begin exchange variables ***
H0          =X(1)
CclSD       =X(2)
rhoc        =X(3)
rhow        =X(4)
hmax        =X(5)
ztoe        =X(6)
zbed        =X(7)
Nod         =X(8)
Dn50        =X(9)
break       =X(10)
fHs1        =X(11)
fHs2        =X(12)
rhor        =X(13)
zsurge      =X(14)
a           =X(15)
b           =X(16)
maxSLR      =X(17)
zsurgeH     =X(18)
randomH     =X(19)
Tm          =X(20)
prob        =X(21)

Dncl        =P(1)
Year        =P(2)

! *** End exchange variables ***
```

```
! *** pre processing ***

!General pre processing
deltaC  = rhoc/rhow-1                    !relative density concrete
deltaR  = rhor/rhow-1                    !relative density rock
SLR     = maxSLR*(Year/50)               !sea level rise


!Maintenance pre processing
 If (Year/Tm<=1) then
        YearM=Year
 else
        If (Year/Tm<=2) then
          YearM=Year-Tm
        else
                If (Year/Tm<=3) then
                  YearM=Year-2*Tm
                else
                        If (Year/Tm<=4) then
                          YearM=Year-3*Tm
                        else
                          YearM=Year-4*Tm
                        endif
                endif
        endif
 endif

YearM  = Year                                   !bypass maintenance


N               = a*EXP(b*YearM)-a              !leg breakage
Cw              = 1.2*(1-0.6*N)                 !weight stability
Cil             = 3.1*(1-2*N)                   !interlocking stability
Ccl             = Cw+Cil                                !Core-loc® stability

!Gumbel waves pre processing
ht              = hmax-ztoe+SLR+zsurge          !toe depth
h               = hmax-zbed+SLR+zsurge          !water depth
H0SD   = H0*fHs1+fHs2                           !uncertainty wave height

 If (H0SD*0.86<=break*h) then   !Translation deep to shallow water
   Hshore=H0SD*0.86
 else
        Hshore=break*h
 endif


!Hurricane waves pre processing
htH             = hmax-ztoe+SLR+zsurgeH         !toe depth
hH              = hmax-zbed+SLR+zsurgeH         !water depth

        HshoreH=0
 If (hmax < prob) then
 !If (randomH>=0.227807166) then                !50yr
        HshoreH=break*hH
 !else
 endif
```

```
! *** End pre processing ***

Select case (I)  ! Select limit state by number

Case(1) ! Primary armour
  Limit = (Ccl+CclSD)*deltaC*Dncl-Hshore

Case(2) ! Toe stability
  Limit = (2+6.2*(ht/h)**2.7)*Nod**0.15*deltaR*Dn50-Hshore

Case(3)             ! Primary armour Hurricane
  Limit = (Ccl+CclSD)*deltaC*Dncl-HshoreH

Case(4)             ! Toe stability Hurricane
  Limit = (2+6.2*(htH/hH)**2.7)*Nod**0.15*deltaR*Dn50-HshoreH

Case default
  Write(*,*) 'Limit state function undefined'

End select

End function limit

End module Compon
```

Appendix XVIII: Program listing TransVer

```
Program TransVer

Use Prob

Implicit none

! *** Program constants ***
Integer,parameter:: Nvar = 11    ! Number of random variables (X)
Integer,parameter:: Ndes = 2     ! Number of design variables (P)
Integer,parameter:: Npar = 4     ! Maximum number of distribution parameters
Integer,parameter:: Nlim = 2     ! Number of limit states
Integer,parameter:: Ncut = 2     ! Number of cut sets
! *** End constants ***

! *** Begin variables ***
Real    X(Nvar,Npar)    ! Basic variables
Real    P(Ndes)                 ! Parameters
Integer Isens                   ! Switch for sensitivity analysis
Real    beta(Nlim)              ! Reliability index
Real    Pf(Nlim)                ! Failure probability
Real    alpha(Nvar,Nlim)! Influence factors
Real    Xdes(Nvar,Nlim)         ! Design point in physical space
Real    Udes(Nvar,Nlim)         ! Design point in standard-normal space
Real    tol                     ! Break-off criterion
Real    maxit                   ! Maximum number of iterations
Integer err                     ! Error code

Integer logmat(Nlim,Ncut)       ! Logical matrix (fault tree)
Real    rho(Nvar,Nlim,Nlim)     ! Correlation matrix
Real    betasys                 ! Reliability index of system
Real    Pfsys                   ! Failure probability of system
Real    Betacut(Ncut)           ! Reliability index by cut set
Real    Pfcut(Ncut)             ! Failure probability by cut set
Real    alphacut(Nvar,Ncut)     ! Influence factors by cut set
Real    alpsys_comp(Nlim)       ! Influence factor quantifying influence of individual
failure modes
Real    alpsys_var(Nvar)        ! Influence factors of system
Real    Usys(Nvar)              ! Design point of system

Real    PfC(2)                  ! Cornell system bounds
Real    PfD(2)                  ! Ditlevsen system bounds

Real    Nmin                    ! Minimum number of simulations

Integer i,j,k,l                 ! Counters

 ! Variables of main program
Real    paramRc
Real    paramYrs
Real    R(7)                    ! variation parameter crest height
!Real   Nsim                    ! simulation counter
```

```
! *** End variables ***

! *** Begin program ***

! *** Open files for output ***
Open(9000,file='IIItrans050.prn',action='write')

! *** Initialise all variables that serve as input ***

  ! Basic variables
  X(1,1:4) = (/3.0, 0.3948,  0.8512,  0.9382/)              !H0SLS
!  X(1,1:3) = (/4.0,  1.554,   0.979/)                      !H0ULS!!!!
  X(2,1:3) = (/1.0,   0.08,    0.02/)                       !Kentrance
  X(3,1:3) = (/1.0,   0.00,    0.14/)                       !fKovertopping
  X(4,1:3) = (/1.0,   0.31,    0.25/)                       !hmax
  X(5,1:2) = (/0.0,   0.00/)                                !fHshore
  X(6,1:2) = (/0.0,   0.75/)                                !Hallowed
  X(7,1:3) = (/1.0,   0.45,    0.02/)                       !break
  X(8,1:3) = (/1.0,    1.0,    0.13/)                       !fHs1
  X(9,1:3) = (/1.0,    0.0,    0.15/)                       !fHs2
  X(10,1:2)= (/0.0,   0.50/)                                !maxSLR
  X(11,1:3)= (/1.0, -16.00,    0.50/)                       !zbed

  ! Parameters
  P(1) = 0e0                                 !start value

  ! Values R
  R(1) = 3          !Rc
  R(2) = 4
  R(3) = 5
  R(4) = 6
  R(5) = 7
  R(6) = 8
  R(7) = 9

  ! Sensitivity analysis (yes/no)
  Isens = 0  ! no

  ! Design point by failure mode (starting point)
  Udes = 0e0     ! All elements of vector equal to zero

  ! Logical matrix
  Logmat(:,1) = (/1,0/)     ! Cut set 1: mode 1 active
  Logmat(:,2) = (/0,1/) ! Cut set 2: mode 2 active

  ! Correlation matrix
  rho = 1e0

    ! Minimum number of simulations
  Nmin = 0         ! Determine number of simulations on the basis of achieved accuracy

  ! Break-off criteria
  tol = 1e-3        ! Recommended value
  maxit = 200     ! Recommended value

          Write(*,'(A80)')    '   Yr       Rc     Pfsys'
          Write(9000,'(A80)') 'Yr Rc Pfsys'
```

```
 ! *** Main loop: calculations for a number of crest heights ***

 ! *** Loop: 50 lifetime years
Do paramYrs = 1,50

 ! *** Loop: 7 crest heights
Do paramRc = 1,7

            ! Store value of crest height in appropriate element of P
            P(1) = R(paramRc)
            ! Store value of year in appropriate element of P
            P(2) = paramYrs

            ! Calculate reliability by failure mode and show result on screen
            Do i=1,Nlim
             Call MCsys(X,P,logmat,Nmin,Pfsys,betasys,Pfcut,betacut,Pf,beta)
!             Call FORM(X,P,i,Isens,beta(i),Pf(i),alpha(:,i),Xdes(:,i),Udes(:,i),tol,maxit,err)
            Enddo ! i

            ! Write results to file and screen
!           Write(*,'(F8.1,F8.0,ES12.3)') P(2),P(1),Pf(1)
!           Write(9000,'(F8.1,F8.0,ES12.3)') P(2),P(1),Pf(1)

            ! Analyse fundamental bounds and Ditlevsen bounds
!           PfC = Cbound(beta)
!           PfD = Dbound(beta,alpha,rho,err)

            ! Perform system reliability analysis by Hohenbichler/Rackwitz
!           Call relsys(logmat,beta,alpha,Isens,rho,betasys,Pfsys,Betacut,Pfcut,
            alphacut,alpsys_comp,alpsys_var,Usys,err)

            ! Write results to file and screen
            ! Width, Pf by mode (vector), lower bounds, Hohenbichler/Rackwitz, Upper
            bounds
            Write(9000,'(F8.1,F8.0,2ES12.3)') P(2),P(1),Pf(1),Pfsys
            Write(*,'(F8.1,F8.0,2ES12.3)') P(2),P(1),Pf(1),Pfsys

 Enddo! paramYrs
 ! *** End loop paramYrs

 Enddo ! paramRc
 ! *** End loop paramRc

 ! *** End main

! *** Close files ***
 Close(9000)

 Pause 'Calculation finished, press a key'

! *** End program ***

End program TransVer


Module Compon
! Used by probmod
```

```
Implicit none

! Module containing limit state equation for a specific case

! Delft University of Technology
! Hydraulic and Offshore Engineering Section
! Probabilistic Methods

Contains
! ************* Start of module subroutines **********************
! General interfacing limit state function
Function limit(X,P,I,newiter)

! Interface variables
Real limit
Real,Intent(in):: X(:)
Real,Intent(inout):: P(:)
Integer,Intent(in):: I
Logical,Intent(in):: newiter

! Common variables
! (void)

! Internal variables
Real H0SLS
Real Kentrance
Real fKovertopping
Real hmax
Real fHshore            !superfluous
Real Hallowed
Real break
Real fHs1
Real fHs2
Real maxSLR
Real zbed

Real Rtop
Real year

Real SLR
Real h
Real H0SD
Real Rc
Real RelRc
Real HshoreSLS
Real Kovertopping

! Constants
! (void)

! ** Start function **

! *** Begin exchange variables ***
H0SLS               =X(1)
Kentrance           =X(2)
fKovertopping       =X(3)
hmax                =X(4)
fHshore             =X(5)               !superfluous
```

```
Hallowed                =X(6)
break                   =X(7)
fHs1                    =X(8)
fHs2                    =X(9)
maxSLR                  =X(10)
zbed                    =X(11)

Rtop                    =P(1)
Year                    =P(2)

! *** End exchange variables ***

! *** pre processing ***
SLR                     = maxSLR*(Year/50)          !sea level rise
h                       = hmax-zbed+SLR             !water depth
H0SD                    = H0SLS*fHs1+fHs2           !uncertainty wave height
Rc                      = Rtop-hmax-SLR             !crest height above SWL

!HshoreSLS=0.86*H0SD
 If (H0SD*0.86<=break*h) then             !translation deep to shallow water
   HshoreSLS=H0SD*0.86
 else
        HshoreSLS=break*h
 endif

RelRc           = Rc/(HshoreSLS)

! *** End pre processing ***

Select case (I)  ! Select limit state by number

Case(1) ! Downtime

 If (RelRc>=1.3) then
   Kovertopping=0.05
 elseif (RelRc<=-1.0) then
        Kovertopping=0.95
 else
        Kovertopping=0.56-0.39*RelRc
 endif

  Limit = Hallowed-
(HshoreSLS)*SQRT(Kentrance**2+(Kovertopping+fKovertopping)**2)

Case(2) ! Optional
  Limit = 50000-3

Case default
  Write(*,*) 'Limit state function undefined'

End select

End function limit

End module Compon
```