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The northern sea defence of Maasvlakte 2

Hydraulic boundary conditions and design

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Graduation committee

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Preface

This study was performed within the framework of research on the land reclamation of the port of Rotterdam, Maasvlakte 2. This report deals with the design of a sea defence for the northern side of Maasvlakte 2. This research is the final requirement before receiving the Master's degree in Civil Engineering at Delft University of Technology (DUT).

For the hydraulic boundary research, I performed simulations with SWAN. During the model set-up and calibration phase I worked together with Michiel Muilwijk: An informative and humorous experience.

I would like to thank the members of my graduation committee for their critical advice and my colleagues at Ingenieursbureau Gemeentewerken Rotterdam (IGWR) for their practical support.

Furthermore I would like to use this opportunity to thank both IGWR and the section Hydraulic Engineering of DUT, for placing a work station and many more facilities at my disposal.

Daan van Rooijen
May 2005

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Summary

In 1996 the Dutch government started a national discussion about the necessity of the expansion of the port of Rotterdam. It was concluded that a lack of space would occur in the near future. Therefore, the development of a project called "Maasvlakte 2" was initiated. The total area of waterways, sea defences, infrastructure and industrial areas of Maasvlakte 2 will be 2000 ha. The sea defence of Maasvlakte 2 will consist of two parts: the western and northern sea defence. To compensate for the loss of beaches, the western sea defence will be constructed as an artificial dune. This type of sea defence is not preferred for the northern sea defence, because of its more adverse orientation to potential maximum storms (design storm), that will necessitate frequent maintenance operations. These dredging operations will cause hindrance to navigation and will result in uncertainties in the total life cycle costs.

This thesis research focuses on the design of the northern sea defence. The primary objective is to design an innovative, feasible and cost friendly sea defence. Sub objectives are firstly to find accurate hydraulic design conditions in the vicinity of Maasvlakte 2 and secondly to determine the possibility of two cost reducing methods for the northern sea dike.

The research was divided in three phases: the modelling of the incident waves with the SWAN model, a literature study of the project history, and finally the design and economic optimisation of the northern sea defence.

Combined statistics of wave height, wave period, still water level and wind speed were obtained by applying physical relations between the parameters at Europlatform and IJmuiden measuring stations. Wave propagation from relative deep water towards the location of the future sea defence was simulated with the SWAN model (Simulating Waves Nearshore). Maximum hydraulic conditions that are exceeded with a probability of 1/10,000 per year will occur at the North West corner of Maasvlakte 2. The maximum parameters are: significant wave height, 6.9m; peak period, 13.8s; and water level, NAP+4.8m. The significant wave height decreases with 15% towards the eastern end of the structure. Good estimates of the combined statistics of wind, waves and water levels were found by using physical relations. For the purpose of this thesis, the results of the simplified formulas were used as the input of the SWAN model.

Previous studies proposed large and therefore expensive designs due to the adverse orientation of the northern sea defence. This thesis studies the feasibility of two cost reducing methods for the northern sea dike. Firstly the feasibility of a low crested sea dike with discharge canal; secondly the feasibility of allowing a certain amount of damage to the structure by applying a relatively light, less expensive, armour grading. The safety in the area will not decrease because the water return level with an exceedance frequency of 10^{-4} is lower than the designed terrain height, thus flooding will be highly improbable. For both methods an optimisation of the total life cycle costs was made for two types of armour layers: Interlocking elements and quarry stone.

For the interlocking elements a comparison was made between the Accropode and the XBloc elements; this showed that the latter is most economical, mainly because of its less dense packing method. Unit costs, representing purchase and construction costs of building

materials, were used to make an efficient comparison of design alternatives. By applying the concept of a low crested sea dike with an armour layer of XBloc elements, unit costs are reduced by 20 %. No cost reductions are obtained for a sea dike with a quarry stone armour layer. The reason is that the asphalt top layer must be replaced by heavy, expensive rock to ensure the stability of the top part of the construction in the case of overtopping waves.
In the proposal of allowing damage, the application of XBloc armour units will make the construction fail in a progressive way when design conditions are exceeded, due to the interlocking of the elements. Therefore, application of XBloc elements in combination with the method of allowing damage is not possible. The non standard 3-7 ton graded armour layer leads to the lowest life cycle costs of a quarry stone sea dike. Costs are reduced by 20 % in comparison with a traditional designed armour layer.

The construction of a low crested sea dike with a crest level at NAP+14.0m and use of interlocking element armour layer is recommended. Costs for construction and building materials are €21,000 per running metre. This is €11,300 less than the optimised sea dike with a quarry stone armour layer and a crest level at NAP+13.0m.

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List of Symbols

Symbol	Description	Unit
α	Front slope angle	[-]
α	Dimensionless constant depending on the wind direction	[-]
β	Angle of incidence	[°]
C	Costs in a specific year	[€]
C	Chezy friction coefficient	[$m^{0.5}$/s]
C_D	Drag coefficient	[-]
C_r	Reflection coefficient	[-]
C_t	Transmission coefficient	[-]
D_{n50}	Nominal stone diameter	[m]
d	Depth	[m]
Δ	Relative density	[-]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all waves	[m]
H_{m0}	Spectral wave height	[m]
H_t	Distance between low water and top of toe construction	[m]
H_m	Distance between low water and bottom	[m]
IC	Indexated costs	[€]
K_D	Stability coefficient	[-]
k_r	Bottom roughness	[m]
L	Wave length	[m]
N	Number of waves	[-]
ξ	Surf similarity parameter	[-]
r	Inflation	[-]
ρ	Density	[kg/m^3]
s	Wind set-up	[m]
s_p	Wave steepness	[-]
t	Time after the year 2005	[yr]
T_p	Peak period, wave period that corresponds with the peak in the energy density spectrum (see Figure 43)	[s]
T_m	Mean wave period	[s]
u_{10}	Wind speed 10 m above sea level	[m/s]
u_*	Friction velocity	[m/s]
X	Straight line fetch	[m]
ψ	Shield stability parameter	[-]

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1 Introduction

1.1 Background

1.1.1 Maasvlakte 2 history

In 1993 twenty-three private parties signed the ROM-Rijnmond agreement. The intention of these parties was to develop the economic functioning together with quality of life in the Rijnmond area. One of the projects this group worked on was the expansion of the port of Rotterdam.

The government started in 1996 a national discussion and a research path concerning the necessity of the expansion of the port of Rotterdam. It was concluded that a lack of space would occur in the near future. These conclusions together with the fact that this situation is unfavourable for economical reasons were stated in *Verkenning Ruimtetekort Mainport Rotterdam (VERM)*.

In 1997 *Project Mainportontwikkeling Rotterdam (PMR)* was founded after project decision *Ruimtetekort in mainport Rotterdam of* the Dutch government. Its goal is to develop project activities to realize the two objectives: Coordination of series of projects designed to strengthen the 'Mainport' and to improve the quality of the living environment. The parties involved in PMR are listed below:

- Department of public transport, public works and water management
- Department of public housing, environmental issues and spatial planning
- Department of economic affairs
- Department of financial affairs
- State of Zuid-Holland
- City of Rotterdam

In 1998 the so called *Startnotitie PKB+ Mainportontwikkeling Rotterdam* was published by the initiators. The society was informed about the way project would be organised and the project's ambitions.

The cabinet published the report *PMR op Koers* in 1999 where it was concluded that the necessity of expansion of the port of Rotterdam was proved by PMR. Subsequently the cabinet sent a letter to the House of Commons with in it the opinion that the present area of the port of Rotterdam lacks space for expansion of harbour activities. Therefore, new land should be reclaimed. The ministries were going to be responsible for the *PKB+ procedure* (Planologische kernbeslissing), while the city of Rotterdam and the province of Zuid-Holland would work on the exploitation of the projects. This report is part of the latter group.

The tender phase will commence late 2005 and the construction phase will start in 2008.

1.1.2 Design history

Since 1994 numerous studies were performed and coordinated by subsequently: *Maasvlakte 2 Organisatie* and *Project Mainportontwikkeling Rotterdam*. Now, one decade later, the construction phase is getting closer.

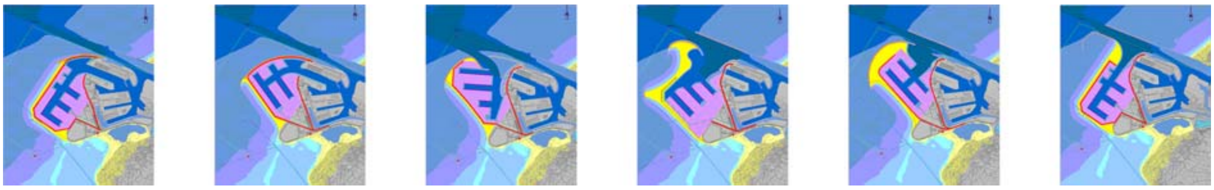

Figure 1 Lay-out alternatives, Expertisecentrum Mainportontwikkeling Rotterdam

Figure 2 shows the lay-out, as it is known at time of the publication of this report. Calculations and argumentations in this study are based on this lay-out.

The total area of waterways, sea defences, infrastructure and industrial areas is 2000 ha. The focus of this study lies at the protection of the northern side of Maasvlakte 2. The western and southern shore will be protected by an artificial dune.

Figure 2 Lay-out "doorsteek variant"

1.2 Problem analysis

1.2.1 Hydraulic boundary conditions

For a proper design the hydraulic boundary conditions at the location of Maasvlakte 2 must be known. RIKZ ran simulations with the computer model SWAN (Simulating WAves Near shore). For three reasons the results were not very reliable. Firstly these calculations did not take bathymetry transformation by the construction of Maasvlakte 2 into account, secondly no combined statistics of wind, waves and water levels were applied and thirdly the results showed strong wave growth within the model area, which is questionable.

1.2.2 Design

August 1996, Grabowsky&Poort BV, nowadays known as Arcadis BV, made an inventory of possible constructions for Maasvlakte 2. The emphasis was put on innovative constructions. Two brainstorm sessions with recognized hydraulic engineers were held [ref 12]. It was concluded that a project so great should be used to maintain the Dutch lead in the field of hydraulic engineering.

In July 1997 Project Organisation Maasvlakte 2 ordered an inventory of all common sea defence types. Except for the artificial dune alternatives, the traditional sea-dike with a quarry stone armour layer was found to be most inexpensive; approximately 135 million euros. However the enormous quantities of rock needed may cause logistic problems.

Regardless the conventional character and the logistic difficulties of a sea-dike with quarry stone, De Wilde (year unknown), Kortlever (2001) and Expertisecentrum Mainport-ontwikkeling Rotterdam made similar designs of traditional sea-dikes. At this point, the innovative intentions formulated by Grabowsky&Poort BV seemed to be faded.

1.2.3 Problem statement

The problem statement is defined as follows:

The current design of the northern sea defence is traditional and quite expensive. The enormous quantities of quarry-stone that needed may lead to logistic difficulties. Exact hydraulic conditions are not known at the location of Maasvlakte 2.

1.2.4 Research objectives

The primary objective is to design an innovative, cost friendly sea defence taking into account the availability of the building materials. Sub objectives are firstly to find accurate hydraulic design conditions in the vicinity of Maasvlakte 2 and secondly to determine the possibility of two cost reducing methods for the northern sea dike. To reduce costs firstly the feasibility of a low crested sea dike with discharge canal is studied, and secondly the feasibility of allowing a certain amount of damage to the structure by applying a relatively light, less expensive, armour grading.

1.3 Approach

Chapter 2 describes the requirements as prescribed by Maasvlakte 2 Organisation. In chapter 3 the analyses of the hydraulic boundary conditions is shown. In the end of this section the values of the design parameters are given. A literature study of previous design reports was performed and is described in section 4. In the end of this chapter the choice is made what type of sea defence will be designed in detail in chapter 5. In chapter 6 the main conclusions and recommendations are stated.

2 Design requirements

2.1 Introduction

The requirements have been subject to some changes in the past. An attempt was made to limit the amount of requirements to stimulate an innovative design process.

2.2 Boundary conditions

1. Calculations of the hydraulic conditions in the vicinity of Maasvlakte 2 will be discussed in chapter 3.
2. The outline of the land reclamations and its phasing is shown in Figure 3. The western side of Maasvlakte 2 will be constructed as a beach with artificial dune. Application of this solution on the northern side of Maasvlakte 2 involves great uncertainties with respect to sedimentation of the Maasgeul. For this reason the sea defence on the northern side must undoubtedly contain some artificial elements [ref 9].
3. The grain diameter (D_{50}) used for the land reclamation is 285μm [ref 11].
4. The design height of the land to be reclaimed is determined at NAP+6.20m [ref 9].

2.3 General requirements

5. The life span of Maasvlakte 2 is set at 100 years. When a free area is reserved for the expansion of the northern sea defence, a design lifetime of 50 years can be taken. The design life span of the western, natural sea defence will be 100 years. The first phase construction has a life span of 10 years. The Maasvlakte 2 sea defence will not be part of the primary sea defence. However, the land reclamation will be protected as if it was an area behind a primary sea defence [ref 9].
6. A minimum safety condition for the protection against flooding is set at 1/10,000 per year [ref 8]. The allowable return period for erosion will be determined in chapter 5.
7. The design of the sea defence should not exclude future expansions of the Rotterdam harbour [ref 8].

2.4 Performance requirements

8. The sea defence protects against high waters, waves and erosion of the terrain [ref 9].
9. The harbour of Rotterdam should be 'normal' accessible for navigation at a maximum wind speed of 9 Beaufort [ref 8]. The design ship which should be taken into account is the Malacca-max (L=400m, W=60m, D=21m), 18000 TEU [ref 8]. A maximum reflection coefficient of 0.25 is accepted.
10. The design value of the overtopping discharge is traditional set at 10 l/s/m [ref 9]. In case of the application of a low crested sea-dike, this requirement will be ignored.
11. Sedimentation of the Maasgeul is allowed to increase up to 100 % [ref 8].

2.5 Constructive requirements

12. Demands on building and construction materials must keep to the 'Bouwstoffen besluit' (Dutch environmental regulations regarding construction materials) [ref 9].
13. Cross sections of the hard sea defence designs must comply with the requirements from 'Leidraad Zee- en Meerdijken' [ref 9].

14. Because of the dimensions of the project, availability of building materials must be taken into account.
15. The work must be done so that it does obstruct navigation as less as possible.

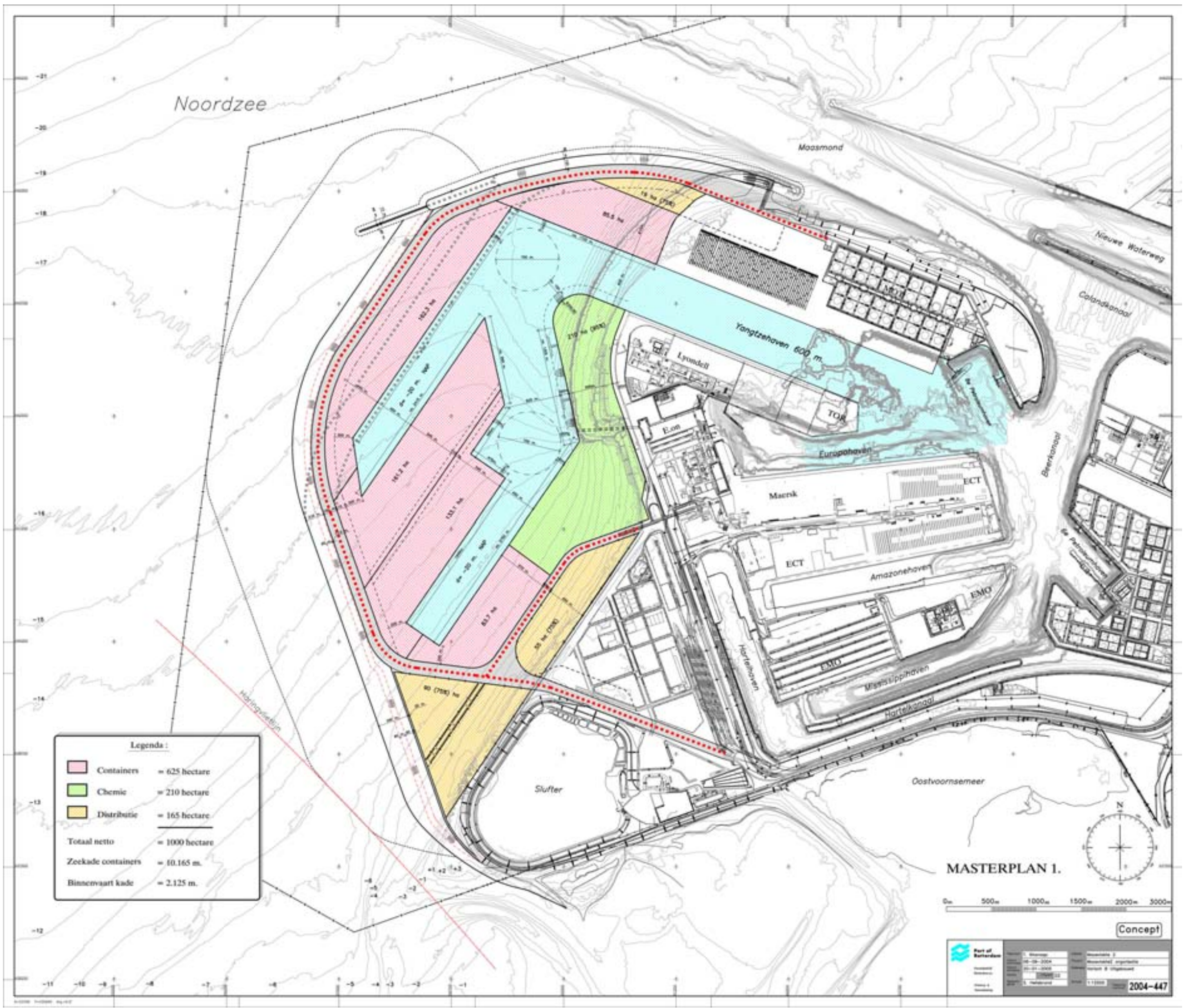

Figure 3 Phasing of Maasvlakte 2

2.6 Recreation and environmental requirements

16. The amount of space taken by the land reclamation above MHSL must be kept as small as possible [ref 8].
17. As less primary building materials as possible must be used. This means that the amount of sand to be dredged must be minimized and the use of secondary materials must be stimulated [ref 9].
18. The areas which needs frequent fore shore sand suppletions must be kept as small as possible [ref 8].

3 Hydraulic boundary conditions

3.1 Introduction

In order to make a reliable design of the northern sea defence it is important to obtain detailed wave information at the intended project location. For most cases where the sea defence is part of the primary water retaining construction, the wave information is given in *Hydraulische randvoorwaarden 2001 (HR2001)* [ref 5]. In the case of the sea defence of Maasvlakte 2 this information is not available, because this sea defence is not defined as a primary water retaining construction (yet).

Jacobse and Groos [ref 13] performed a hydraulic boundary study using the SWAN software; however the reliability of the results is questionable. Firstly these calculations did not take bathymetry transformation by the construction of Maasvlakte 2 into account, secondly no combined statistics of wind, waves and water levels were applied, which causes a great overestimation of the storm conditions and thirdly the results showed strong wave growth within the model area, which is questionable.

Apart from the fact that reliable wave information is essential to making a steady design, there are two other advantages of performing a hydraulic boundary study. Firstly the detailed output data can be used for the design of varying cross-sections along the structure, because incoming waves are likely to differ in force and orientation per location.

A second reason to run these computations is that detailed information about the wave characteristics can be used in a program like REFDIF to observe wave patterns caused by incoming and reflected waves at the location of the port entrance. This is important for ships manoeuvring into the entrance channel under heavy storm conditions.

This close to shore wave information is obtained by running simulations, giving a wave height, period and direction for varying return periods at chosen points close to the shore of the second Maasvlakte. Delft University of Technology developed two models for simulating waves near shore. The second generation model HISWA and the third generation model SWAN. Because the SWAN model is the more modern of the two, the simulations in this research were carried out with SWAN.

In the following sections the setup of the SWAN simulations is described. Every mayor parameter of the input is thoroughly discussed in the sections 3.3 to 3.6. The results are presented in section 3.7.

An overview of the adjustments and changes that were made to refine the model and while shortening the computational time can be found in section F.2. For background information regarding working of the SWAN model the reader is referred to appendix B.

Nautical convention

Wind and wave directions are defined according to the nautical convention. This means that the direction where wind and waves come from, calculated clockwise from the north (360º or 0º), is used. In the figure below the 330º area with a section width of 30 is hatched.

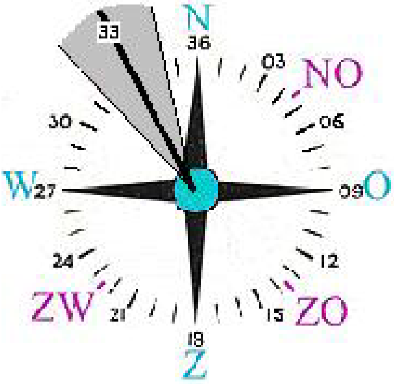

Figure 4 Wind and wave directions according to the nautical convention

3.2 Accuracy

During the calibration of the model, the differences between the cases were expressed in terms of percentages. If the output using a new model set-up only differed 0 to 5 %, the change was noted as insignificant. When the effects became more apparent, 10% or more, the changed parameter had a significant effect. These accuracy limits are coupled to the formulas that are used in the design phase. For example the mass of an armour unit can be calculated using the Hudson (1953) formula [ref 17]:

$$M_{unit} = \frac{\rho_s H_s^3}{K_D \Delta^3 \cot\alpha}$$

Where

M_{unit}	Mass armour unit
H_s	Significant wave height
ρ_s	Specific density of material
K_D	Damage coefficient
Δ	Relative density [$=(\rho_s-\rho_w)/\rho_w$]
α	Slope angle

An increase of the significant wave height with 5 % leads to an equal increase in the D_{n50} of armour rock in terms of percentages. In this stage of the research, 10 % is taken as the upper limit. The results as a whole from the SWAN computations should be validated against near shore wave measurements, but because those measurements are rare the results are judged on the basis of common sense only.

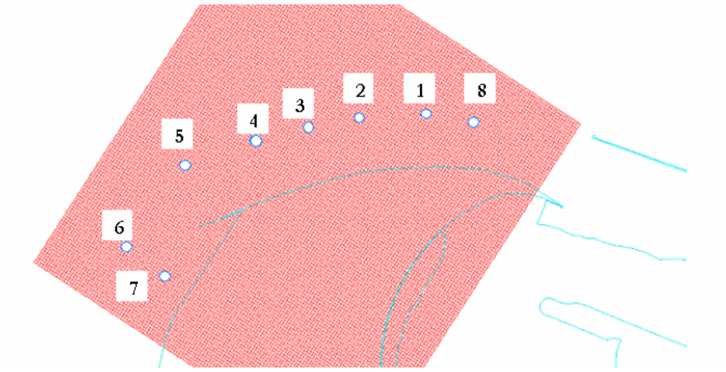

Figure 5 Location of control points

The numbers in Figure 5 indicate the output locations used for the calibration of the model.

3.3 Grids and Bathymetry

3.3.1 Grids

In this case the area of interest is located close to the future beach and dike that protects the area and the propagations start far offshore where detailed information is given by two offshore measuring installations (See Figure 6). The distance between Europlatform and IJmuiden is approximately 100 kilometres. These points form part of the first grid. When this grid is extended to the coast to include Maasvlakte 2 either computations would take very long or the area of interest would not be sufficiently described. Since both of these situations are not favourable, grid nesting is applied.

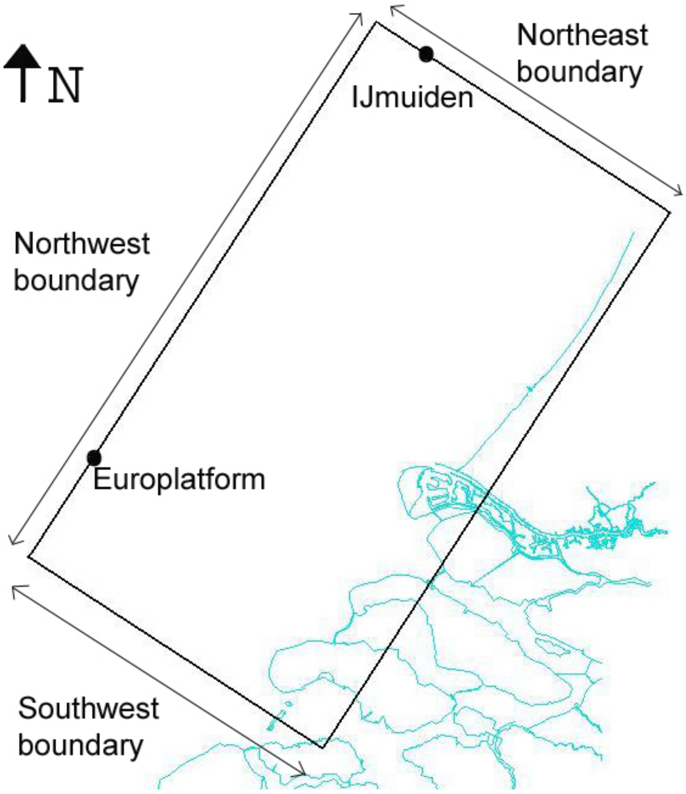

Figure 6 position of the exterior grid on the North Sea

Grid Nesting

To obtain detailed information in the area of Maasvlakte 2, a series of nested grids is introduced, starting with a large exterior grid covering the measuring locations and the Dutch North Sea shore (See Figure 6). The second grid, the intermediate grid, is nested in the Exterior grid and covers the entire area of the second Maasvlakte and an area of 10 kilometres on either side (Figure 7). The last grid is the detailed grid, and this grid covers only the part of the Maasvlakte where the wave characteristics are demanded.

The exterior grid has cells of 500 by 500 metres (Figure 7). These cell dimensions are chosen because the use of smaller grid cells over such a large area is not possible due to limited computational power.

Because the detailed must describe the bottom variations in the area in detail, the elements of this grid should be as small as possible. The detailed grid has cells of 40 by 40 metres since computations with grid cells of 25 by 25 metres did not show a significant change in the output data.

The step from the exterior grid, with elements of 500 m towards the small elements of the detailed grid has become too large to take in one grid, so an intermediate grid was put in between. The elements of this grid were chosen such that the step from exterior to intermediate grid was roughly the same as the step from the intermediate to the detailed grid. For the intermediate grid an element size of 100 by 100 m was chosen.

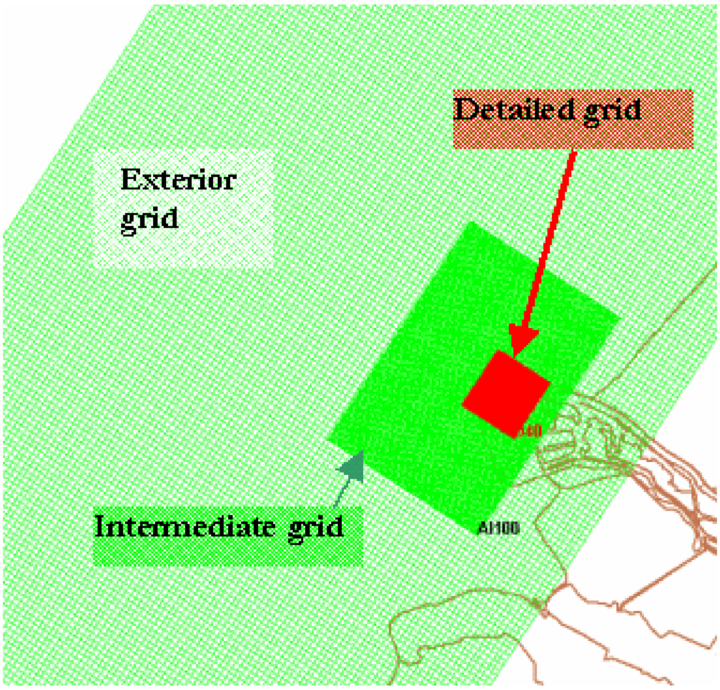

Figure 7 Nested grids give more detailed output

Table 1 Grid (cell) dimensions

	Cell size [m]	***Grid dimensions [km]***
Exterior grid	500 x 500	100 x 55
Intermediate grid	100 x 100	22 x 15
Detailed grid	40 x 40	5.6 x 5.2

3.3.2 Bathymetry

The bathymetry file that was used is constructed from a large data file obtained from the Dutch Navy (Hydrografische Dienst van de Koninklijke Nederlandse Marine). The depth points in the data file are mapped on the grids from Table 1 using the nearest point averaging method.

3.4 Boundary orientation

The exterior grid has three boundaries on the seaside and one boundary on the landside (Figure 6). The values that are imposed on the model boundaries are:

- Wave height,
- Wave direction and
- Wave period.

Other parameters that are significant for the results in the area of interest are:

- Still water level
- Wind speed

More detailed information about these five parameters is given in section 3.5.

A simplification was made by shifting the "real" values of the Europlatform and the IJmuiden station towards the ends of the boundaries, as is shown in Figure 8. The output differences between cases with and without shifted stations (respectively right and left side of Figure 8) were only 2 to 3 % and shifting of stations was therefore accepted.

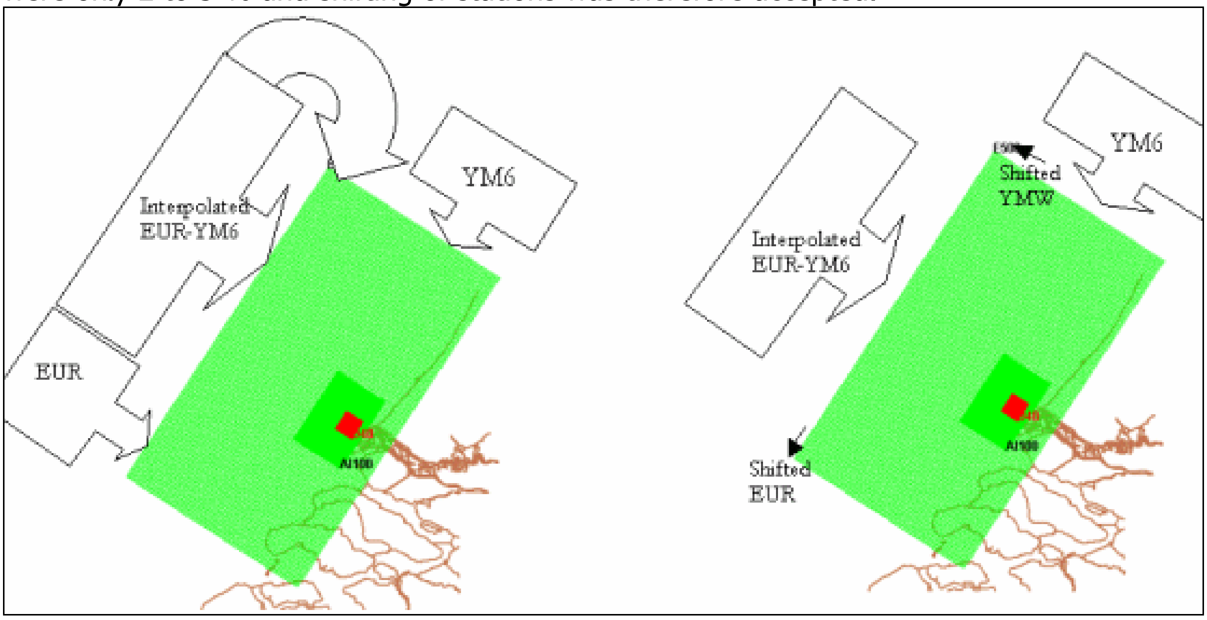

Figure 8 Interpolated boundaries

3.5 Boundary parameter reduction

When it is assumed that offshore significant wave heights, wave periods, wind speeds and near shore water levels exceed their design values simultaneously, very conservative conditions are taken. A reduction can be found by looking closely at physical relations between the parameters. More about the relations is explained in the sections below.

Roskam, Hoekema and Seijffert [ref 16], calculated the extreme value distribution of

- H_S, significant wave height
- T_P, peak period
- Still Water level (Table 4)

for small sectors of the wind direction using wind, water level and wave measurements over the period 1981-1996. The summation of the exceedance frequencies of a certain parameter over all directions equals the omni directional extreme value for this parameter. In Table 2 the corresponding parameters are shown for the Europlatform and in Table 3 for the IJmuiden Ammunition dump station.

Wave heights and wave periods show the highest values for waves coming from the north west (330º) for all measuring locations. The results by Roskam et al, also clearly show that there is a large "land wind sector" present, from 10º to 220º, where the water and wave parameters show very low values. This sector will from now on be disregarded in the current research.

The accuracy of the measured wave height at Europlatform and IJmuiden is in the order of 6 % and for the period in the order of 3 % [ref 30].

A common sector width of 30º is used.

Table 2 Extreme values for 10^{-4} exceedance frequency at Europlatform

Directional sector (width 30º)	Significant wave height (m)	Mean spectral period (s)	Peak period (s)
210º	7.33	8.38	11.44
240º	7.42	8.43	11.64
270º	7.41	8.57	11.84
300º	7.83	8.94	12.45
330º	8.15	9.23	13.00
360º	7.55	8.91	12.52
Omni directional	8.40	9.40	13.30

The storm surge levels shown in are measured at the Hook of Holland station, because this is the measuring station that is closets to the area of interest. More on storm surge levels can be found in the section about water levels.

Table 3 Extreme values for 10^{-4} exceedance frequency at IJmuiden ammunition dump

Directional sector (width 30º)	Significant wave height (m)	Mean spectral period (s)	Peak period (s)
210º	7.38	8.81	12.28
240º	8.00	9.40	13.44
270º	8.22	9.74	13.96
300º	8.49	10.29	14.89
330º	8.82	10.59	15.28
360º	8.14	10.12	14.57
Omni directional	9.10	10.80	15.70

Since the water level varies along the Dutch coast a correction should be applied on the water levels from Hook of Holland. According to Van der Hout [ref 10] a water level reduction of 10 cm between Hook of Holland and Maasvlakte 2 is realistic. In Figure 9 the spatial distribution of the water levels is shown.

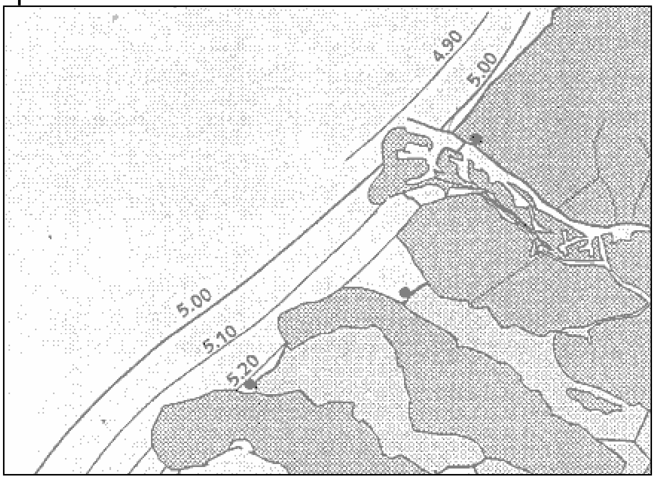

Figure 9 Spatial distributions of the water levels along the Dutch coast

Table 4 Extreme water levels for 10^{-4} frequency at Hook van Holland (HvH) and Maasvlakte 2 (MV2)

Directional sector (width 30º)	Surge level HvH (m+NAP)	Surge level MV2 (m+NAP)
210º	2.63	2.53
240º	3.23	3.13
270º	4.13	4.03
300º	4.76	4.66
330º	4.87	4.77
360º	3.81	3.71
Omni directional	5.00	4.90

According to the *3e Kustnota* [ref 15] an anticipating scenario regarding the rising of the sea level must be followed. This implies that 65 centimetres must be added to the high water level. In the overview of the model boundaries in section 3.5.8 this is done.

Overtopping calculations showed that although waves from the 360º direction come in more straight, waves with a direction of 330º cause more overtopping. Because stability of the armour layer is not influenced by the angle of incidence, also for this process, the direction with highest waves (330º) is normative.

Appendix C. shows the angles of incidence of both the 330º and 360º runs.

3.5.1 Approach

The reductions will be determined keeping the wave height (H_{m0})[1] at a fixed value and reducing the other hydraulic parameters one by one. The wave height is chosen because

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The return levels of the wave height in *Richtingsafhankelijke extreme waarden voor HW-standen, golfhoogten en golfperioden* are given as the spectral wave height (Hm0) which is only 1 or 2 percent higher than the significant wave height (H_s). Because the latter is used as input for the SWAN computations, the reductions found in this fixing one of the other parameters leads to an increase in the marginal probability of the wave height, which is off course nonsense.

Combined statistics are obtained by finding physical relations between these parameter sets:

Wave height – wave period (section 3.5.2)
Wave height – wind speed (section 3.5.3)
Wave period – wind speed (section 3.5.4)
Wind speed – wind set-up (section 3.5.5)

Because some of the parameters have a direct relation and others only have an indirect relation, not every relation is treated with the same importance.

3.5.2 Wave height and wave period

There exists a very strong correlation between the wave height and the period. The correlation between the two parameters is analyzed first by looking at the wave steepness. The wave steepness (sp) is defined as the quotient between the wave height (Hm0) and the length (L). The wavelength (L) is determined using an approximation that was derived from the CEM [ref 22].

$$sp = \frac{H_{M0}}{L}, \quad L \approx \frac{gT^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T^2 g}\right)}$$

Since the wave steepness described above does not include any depth influence a second relation for the steepness is used. The relation according to Roskam et al. is given (for Europlatform) by the following set of equations.

$$\begin{aligned}
&T_{M02} = 3.14\sqrt{H_{M0}} \\
&H_{M0}/d \leq 0.14 \qquad \rightarrow \frac{T_P}{T_{M02}} = 1.33 \\
&0.14 \leq H_{M0}/d \leq 0.35 \rightarrow \frac{T_P}{T_{M02}} = 0.5\frac{H_{M0}}{d} + 1.26
\end{aligned}$$

Comparison between the two relations indicated that for the lower regions of wave heights they are practically the same, but as waves start getting bigger the depth becomes increasingly important and the second relation gives lower values for the wave period.

section are a bit conservative. However the differences between Hs and Hm0 are so small that both parameters will be used for the wave height.

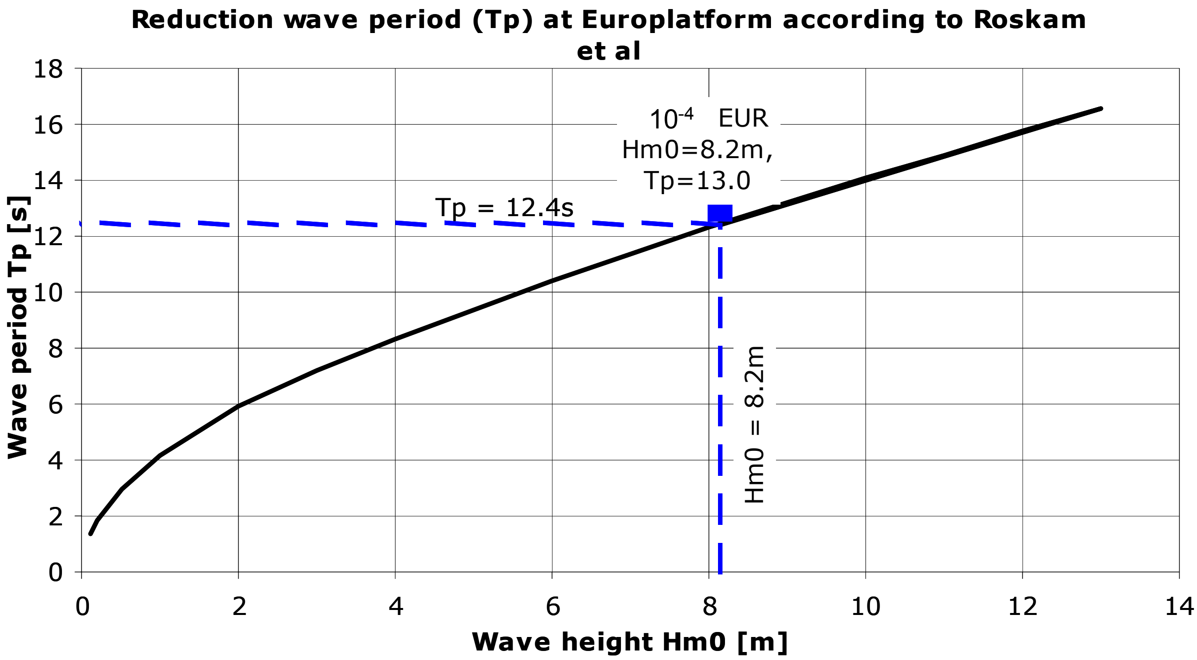


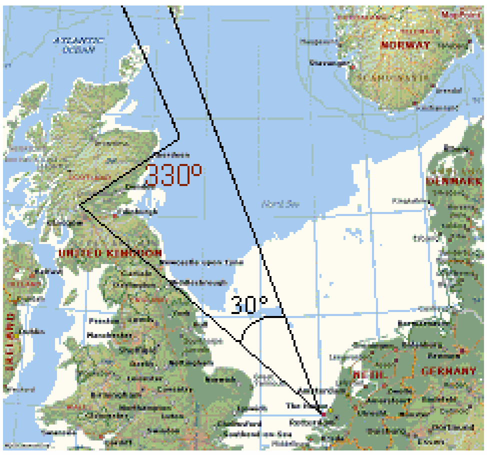
Figure 10 Wave steepness dependent of relative depth

Conclusion

Because the steepness of wave seems to become slightly less in intermediate water depths lower periods can be assumed. From Figure 10 a new value for the wave period can be found at the point where the line representing the physical relation crosses the wave height of 8.15 m. At this point the period is 12.4 s.

3.5.3 Wave height and wind speed

The wave growth by wind depends mainly on the fetch distance, the wind speed and the storm duration. The straight-line fetch distance for the 330º section varies between 650 and 2000 km (see Figure 11). Because of the great fetch distance, the storm duration becomes normative for the wave height [ref 22]. On the right-hand side of Figure 11, the dotted line shows the wind speed at *Licht Eiland Goeree* during the storm in February 1953. A wind speed over 20 m/s (40 knots) endured for 26 hours. The length of this exceptional long storm is assumed to be normative.



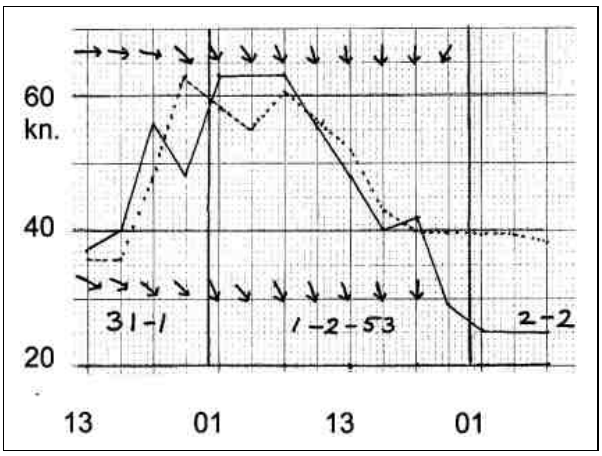


Figure 11 (left) Fetch length of 650 km and (right) storm duration of 26 hours, February 1953,* Licht Eiland Goeree*, dotted line

Brettschneider found relations of depth limited wave growth, however application of these formulas cause an increase in the marginal probability of the wind speed and are therefore found unsuitable.

The formulas by Demirbilek, Bratos and Thomson (1993) give a solution for duration limited wave growth. Equations governing wave growth with wind duration can be obtained by converting the duration into an equivalent fetch given by:

$$\frac{gX}{u_*^2} = 5.23 \times 10^{-3} \left(\frac{gt}{u_*}\right)^{\frac{3}{2}}$$

X = straight line fetch distance over which the wind blows

H_{m0} = energy-based significant wave height

C_D = drag coefficient

U_{10} = wind speed at 10 m elevation

u_* = friction velocity

See Demirbilek, Bratos, and Thompson (1993) for more details.

$$\frac{gH_{m_0}}{u_*^2} = 4.13 \times 10^{-2} * \left(\frac{gX}{u_*^2}\right)^{\frac{1}{2}}$$

and

$$\frac{gT_p}{u_*} = 0.651 \left(\frac{gX}{u_*^2}\right)^{\frac{1}{3}}$$

$$C_D = \frac{u_*^2}{U_{10}^2}$$

$$C_D = 0.001(1.1 + 0.035\, U_{10})$$

In Figure 12 the relation between wind speed and wave height is plotted for a storm with a duration of 26 hours. It can be seen that a wave height of 8.15 m corresponds with a wind speed of 21.5 m/s.

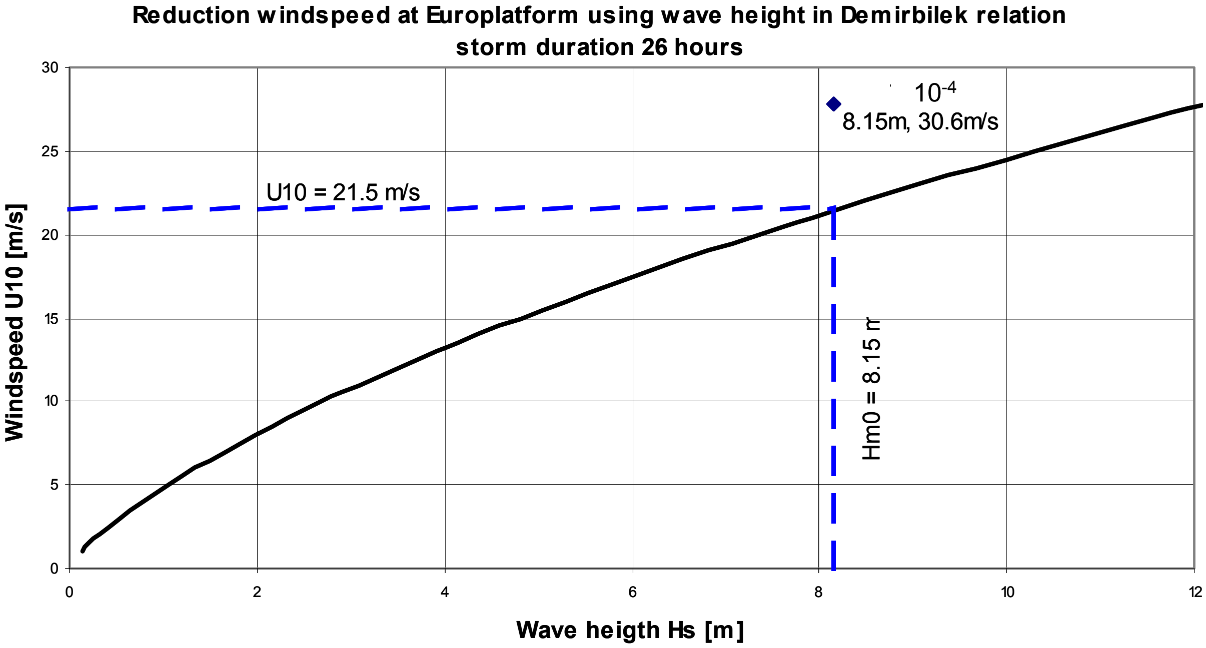


Figure 12 Wave height – wind speed relations according to Demirbilek et al.

Conclusion

Taking only the relation between wind speed and wave height into account one could conclude that the wind speed can be reduced to 21.5 m/s. Because of the number of parameters and their relations, the final reduction will be discussed in section 3.5.8.

3.5.4 Wind speed and wave period

The relation between the wind speed and the wave period is found in the same way as for the wave height. For the formulas used, the reader is referred to section 3.5.3. In Figure 13 the Demirbilek relation between wind speed and wave period is plotted for a 26-hour storm. The horizontal line in the top right corner indicates the reduction of the wave period that has been determined in section 3.5.2.

By using this wave period (12.4 s) and the Demirbilek relation the wind speed can be reduced from 30.6 m/s to 27.8 m/s.

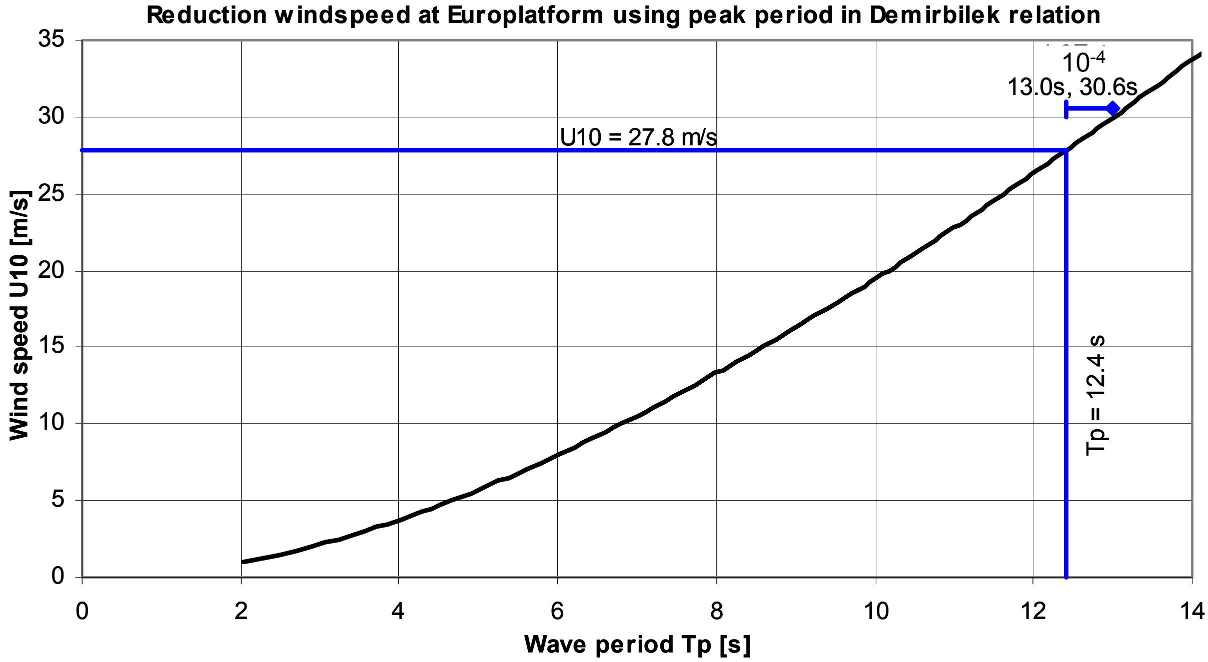


Figure 13 Wave period – wind speed relations according to Demirbilek et al.

Conclusion

While using a fixed wave height of 8.15 m reduces the wind speed down to 21.5 m/s, a wave period of 12.4 seconds causes a wind speed reduction down to 27.8 m/s. This matter will be discussed further in section 3.5.8.

3.5.5 Wind speed and wind set-up

The water level under storm conditions can be considerably higher than under normal conditions. This water level (h) can be divided into two main components, the astronomical tide (a) and the wind set-up (s). In formula: h = a + s

The wind set-up (s) in this equation has a direct relation with the wind speed according to Weenink (1958)

$$s = \frac{u^2 \alpha}{g}$$

In which

u = wind speed [m/s]

α = dimensionless constant depending of the wind direction [-]

g = gravitational constant [m/s^2]

The value of the constant α is determined by calibrating the formula using storm measurements from storms coming from a 330º direction. These storm measurements where taken from www.golfklimaat.nl. For this calibration the measurements from Europlatform were used and the value found for the constant α is 0.043 [-]. With this value the relation between the wind speed and the wind set-up can be represented as the solid line in Figure 14.

The blue point in Figure 14 represents the wind speed and the wave set-up with a return period of 10,000 years. The wind set-up equals the water level (NAP+4.77m) minus the spring tide level (NAP+1.40): NAP+3.37m.

Also in Figure 14 are the reduced wind speeds U_{10} = 21.5 m/s (reduced in relation to the wave height), and U_{10} = 27.8 m/s (reduced in relation to the wave period). The corresponding wind set-ups are 2.0 m for a wind speed of 21.5 m/s, and 3.4 m for a wind speed of 27.8 m/s.

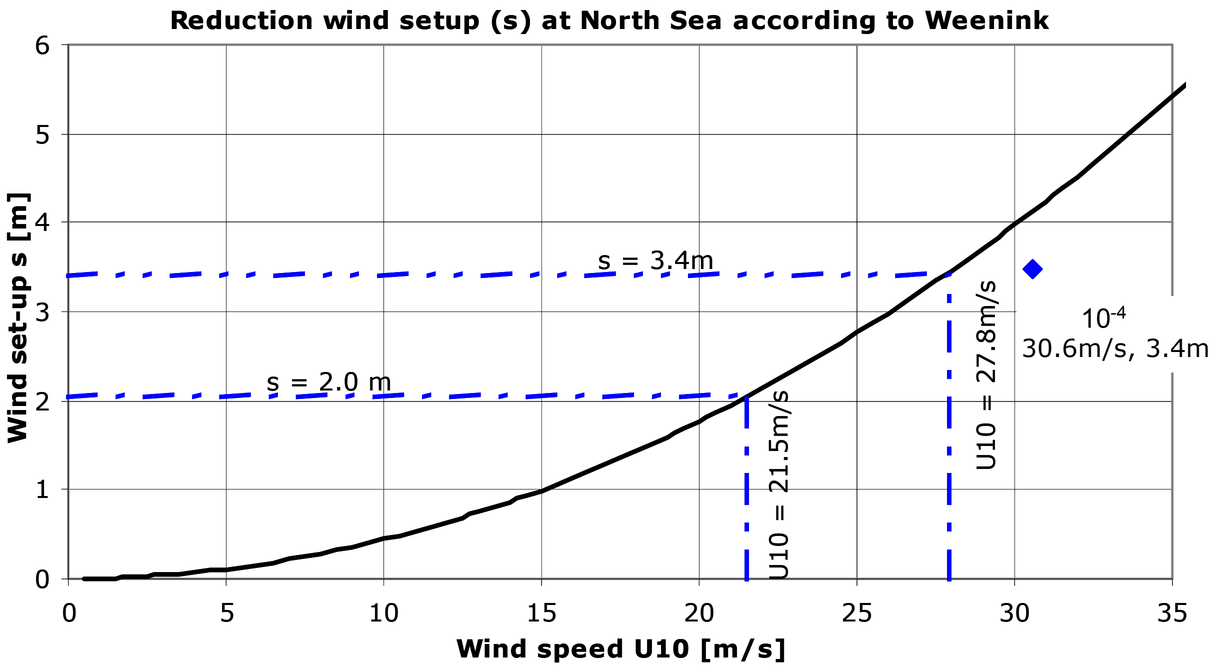


Figure 14 Wind speed and wind set-up

Conclusion

The relation between the wind speed and the wind set-up shows that when the wind speed is not reduced, the wind set-up would become higher than the 10^{-4} condition. If however a reduction in the wind speed is taken into account a reduction in the wind set-up will be the result. The final values that will be used as SWAN boundary conditions are given in section 3.5.8.

3.5.6 Wave height and wind set-up

Finding a relation between the wave height and the wind set-up is a lot less trivial than was the case for the relations above. The wind set-up is not a direct effect of an increase in wave height and their relationship is difficult to determine. One way of getting an idea of the relation between the wave height and the wind set-up is by using the wind speed as a common cause factor

Both the wave height and the wind set-up are a direct effect of the wind speed. For the wave height the previously explained relation of Demirbilek will be used. This formula calculates the wave height as a function of the wind speed. For the wind set-up the earlier used formula of Weenink is used. Like the formulas of Demirbilek with the wave height, the formula by Weenink calculates the wind set-up as a function of the wind speed. For the exact formulas the reader is referred to sections 3.5.3 and 3.5.5.

The results are shown in Figure 15, where the solid line represents the relation between the wind set-up and the wave height. Also indicated in this figure is the point representing the reduced 10^{-4} conditions for both the wave height and the wind set-up. This point is shown in Figure 15 by the blue square.

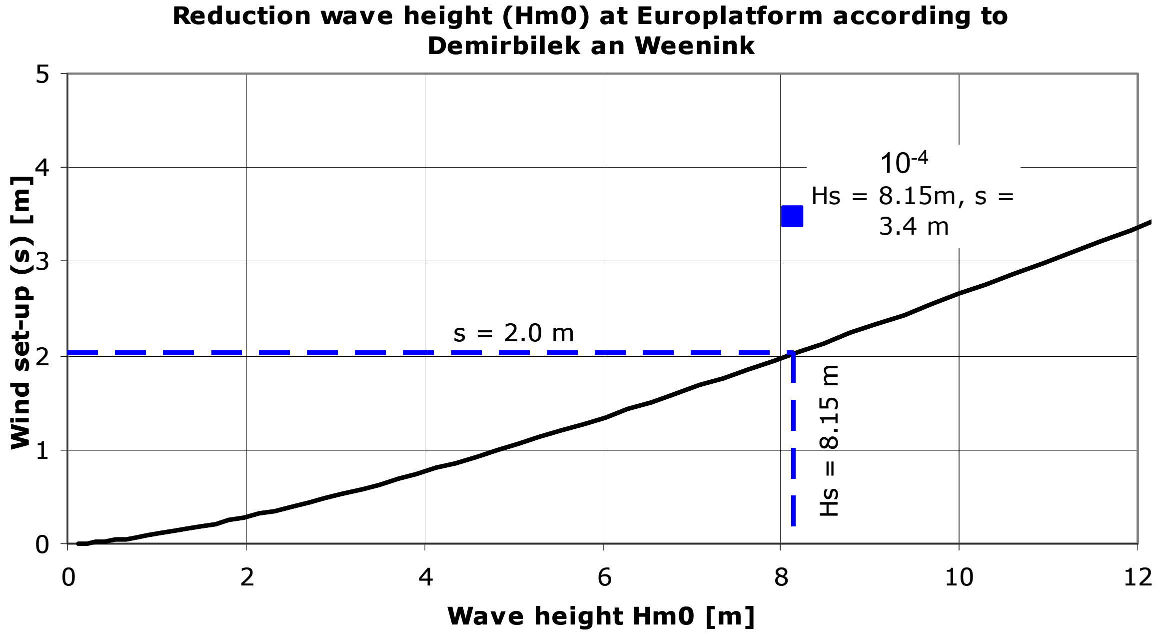


Figure 15 Relation between the wave height and the wind set-up

Conclusion

What this analysis clearly shows is the effect of the wind speed on the wind set-up. In section 3.5.3 the wind speed was reduced according to the relations of Demirbilek. This reduction can be found again in Figure 15, because this figure uses the same relation between the wind speed and the wave height. When the wave height is fixed at the 10^{-4} condition, the wind speed is automatically reduced to 21.5 m/s (instead of 30.6 m/s) and the wind set-up reduces to 2.0 m. (Blue dashed line)

3.5.7 Wave period and wind set-up

For the wave period and water level more or less the same train of thought applies as for the wave height and the water level. In previous sections it has been shown that the wave height and the wave period have a direct relation. When similar to the previous section the wind speed is used as common cause factor and the formulas of Demirbilek and Weenink are used to relate the wind speed to respectively the wave period and the wind set-up, the relation found is shown in (Figure 16). The relation between the wave period and the wind set-up is represented by the solid line. The blue square in the Figure represents the reduced 10^{-4} conditions for both the wave period and the wind set-up.

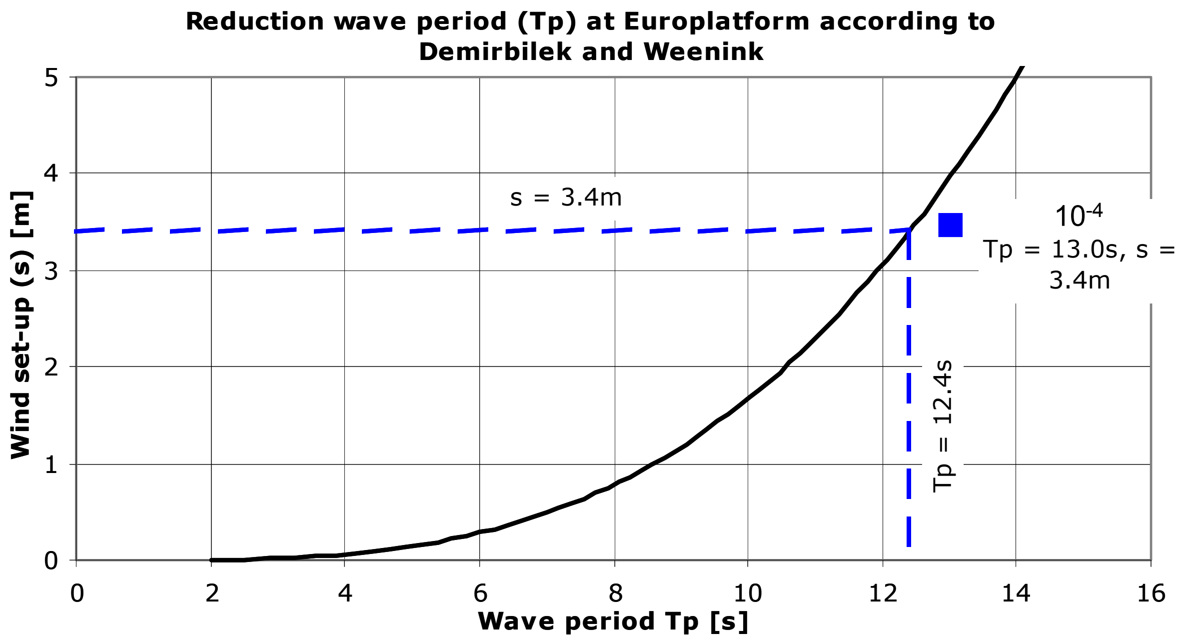


Figure 16 Relation between wave period and wind set-up

Conclusion

As was the case for the relation between the wave height and the wind set-up, the relation between the wave period and the wind set-up is a reflection of the effect of the wind speed on, in this case, the wave period. The slightly reduced wind speed that was found using the wave period leads to small decrease in the wind set-up, shown by the blue dashed line in Figure 16. This is a direct result of the formulas used and the nature of the relation.

Clearly the decrease in the wind set-up as a result of the relation to the wave period (3.4m) is a lot smaller than the decrease as a result of the relation with the wave height (2.0m).

3.5.8 Example parameter reduction

An example of the boundary parameter reduction is given for the Europlatform station with overturns period of 10,000 years. It is stressed that the reductions discussed in the text below apply to Europlatform only.

The reductions will be determined keeping the wave height H_s at a fixed value and reducing the other hydraulic parameters one by one. The wave height is chosen because fixing one of the other parameters leads to an increase in the marginal probability of the wave height, which is off course nonsense.

Because of the strong relation between wave height and wave period, a reliable reduction of the wave period (Tp) is found. The adjusted peak period for Europlatform is 12.4 s.

More difficult is the reduction of the wind speed since it can be directly derived from the wave height (U_{10}=21.5 m/s) or from the reduced wave period (U_{10}=27.8 m/s). It is chosen to use the average of both values, U_{10}=24.7 m/s.

Since the wind set-up is a direct result of the wind speed, the wind set-up is derived from the wind speed. As for the wind speed, the two values found for the wind set-up are averaged. A wind set-up of 2.7 m is derived. By adding the astronomical tide, a water level of NAP+4.1m is found.

It is stressed that the wind speed of 24.7 m/s is the expected wind speed during design conditions. Verkaik and Melger found a similar wind speed (26.8) during extreme storm conditions that are exceeded with a probability of 1/10,000 per year. A summary of this research is given in appendix D.

The wind speed imposed on the model is deduced in the previous section.

3.5.9 Conclusions boundary parameter reduction

By using physical relations instead of a pure statistic approach as used by De Haan, a very good estimate of possible reductions is found. The use of simplified formulas could have led to a deviation of the results from actual values. Because the determination of the hydraulic parameters is only a small part of the total research, no further emphasis is put on the accuracy of the found reductions.

All parameters apply to a segment width of 30º from a 330º direction.

Table 5 and Table 6 summarise the reduced significant wave height and peak period. Table 7 shows the reduced water levels, expected wind speeds and wind speeds imposed on the model.

Table 5 Significant wave height and peak period - 330º direction - Europlatform

Europlatform 330º	1/1 per year	1/10 per year	1/100 per year	1/1,000 per year	1/10,000 per year
Hs [m]	3.8	5.5	6.6	7.4	8.2
Tp [s]	8.1	9.9	11.0	11.8	12.4

Table 6 Significant wave height and peak period - 330º direction - IJmuiden

IJmuiden 330º	1/1 per year	1/10 per year	1/100 per year	1/1,000 per year	1/10,000 per year
Hs [m]	4.0	5.9	7.1	8.0	8.8
Tp [s]	9.1	11.3	12.9	14.2	15.0

The wind speed (U_{10}) and the normative high water level (MHW) are taken constant over the simulation area in the SWAN model. The adjusted values of the significant wave height and the peak period in IJmuiden (Table 6) do not lead to realistic values of the wind speed and the high water level. Therefore, these values are derived from the Europlatform values alone. As discussed earlier, 65 centimetres is added for future rising of the sea level.

Table 7 Water levels and wind speeds constant over the model area

Model area 330º	1/1 per year	1/10 per year	1/100 per year	1/1,000 per year	1/10,000 per year
MHW (NAP+m)	2.9	3.6	3.7	4.4	4.8
$U_{10\text{-expected}}$ (m/s)	12.8	18.0	20.5	22.8	24.7
$U_{10\text{-model}}$ (m/s)	13.0	16.0	17.5	19.0	20.0

3.6 Flow

In proceedings of the research executed by Jacobse and Groos [ref 13], Vledder [ref 25] investigated the influence of flow on waves. By running various SWAN computations he concluded that on the north-west corner of Maasvlakte 2 the relative influence from flow on waves is greatest (20%) with waves and wind from the direction 210ºN. However, the significant wave height and mean wave period are much greater with wind and waves from the direction 315ºN. (See Appendices G. and H.).

Furthermore as the significant wave height, with wind and waves from 315ºN, is increasing with a maximum of 4% along the output points, the mean wave period is decreasing with 2.5%. So the wave load (H_s*T_{m01}) as shown at the bottom of Appendix B, due to flow increases with only 3.8%. Since these peak values are only true for less than half an hour, it is decided not to take the influence of flow into account.

3.7 SWAN results

In Figure 17 the wave field for phase 2 is shown for hydraulic design conditions with a return period of 10,000 years. Appendix J. shows the wave field for phase 1 of Maasvlakte 2. Regarding the wave field no abnormalities are observed in the SWAN results. After refracting towards the Dutch coast, the waves refract towards Maasvlakte 2. The reason the waves do not come in perpendicular to the construction at all locations is due to the steep slope of the constructions.

The left subplot of Figure 19 shows an increase of the wave height from the northeast of Maasvlakte 2 towards the southwest side. It is obvious that the channel has a great influence on the wave field. Diverging waves at the southern border of the channel result in lower wave energy density in the immediate vicinity of this channel, such as the northeast side of Maasvlakte 2.

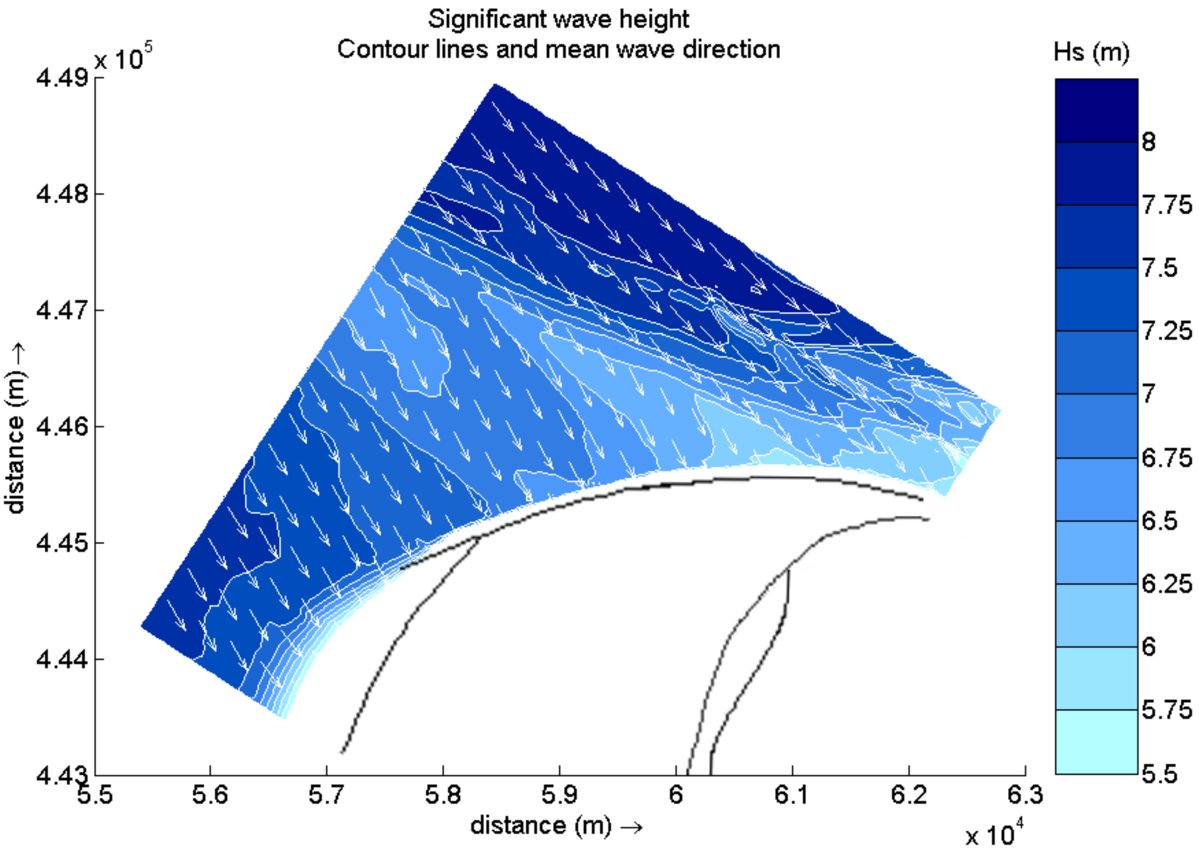


Figure 17 Wave field 10,000 year return period, 330º, MV2 phase 2

A presentation of the results is given in Figure 19 where the significant wave height and mean wave period are plotted along the NAP-17m depth contour, which is close to the coastline of the first and second phase of Maasvlakte 2. The path of the contour lines for the second phase can be seen in Figure 18.

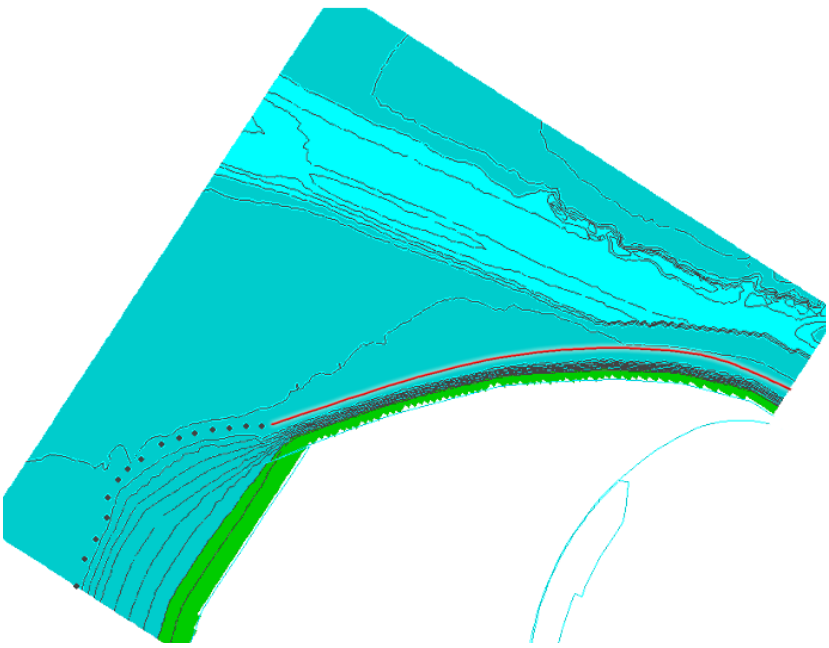


Figure 18 The red line indicates a bottom level of NAP-17.0m for which output is given in Figure 19

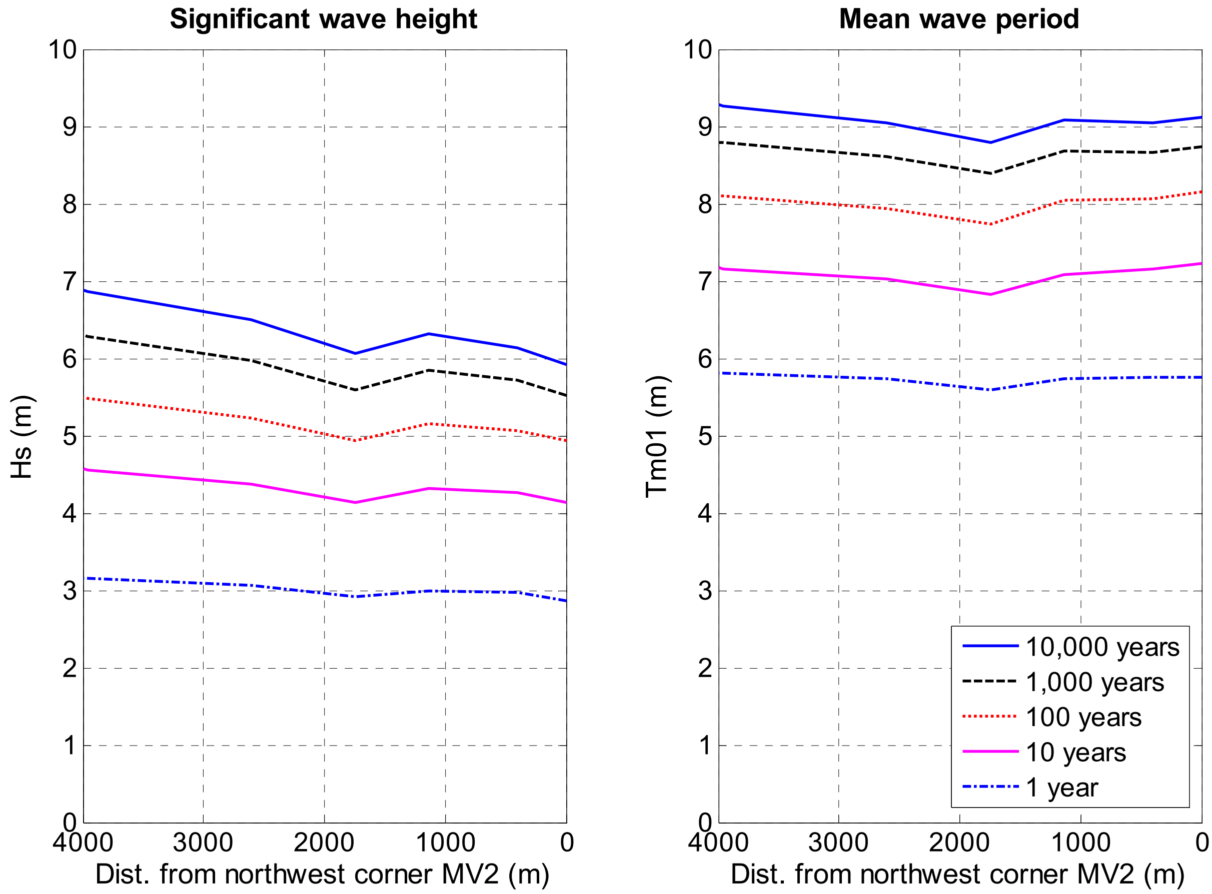


Figure 19 Wave conditions along the NAP-17m contour line at the toe of the shore protection

Appendix J. shows the wave conditions along the NAP-17m contour line for phase 1 of Maasvlakte 2.

From the simulation output the maximum values are subtracted and listed in Table 8. In chapter 5 more information is given about the possibilities of varying crest height and armour unit diameter along the sea defence.

Table 8 Calculation values boundary conditions Maasvlakte 2, hard sea defence

Model area 330º	1/1 per year	1/10 per year	1/100 per year	1/1,000 per year	1/10,000 per year
MHW (NAP+m)	2.9	3.6	3.7	4.4	4.8
H_s (m)	3.2	4.5	5.5	6.3	6.9
T_m (s)	5.8	7.2	8.2	8.8	9.3
T_p (s)	8.3	10.7	12.1	13.8	13.8
U_{10} (m/s)	12.8	18.0	20.5	22.8	24.7

4 Alternatives northern shore protection

4.1 Introduction

In the last decade different studies investigated solutions for the sea defence on the northern side of Maasvlakte 2. This chapter's goal is to collect the results and determine which alternatives are to be examined more closely in chapter 5. Through a close examination of relevant literature the most promising sea defence structures are being compared. The reader is referred to appendices K. to L. for an extended summary of the used literature.

During the literature study a distinction is made between the type of structure, section 4.2 and the armour layer, section 4.3.

It is stressed that the costs found in different studies and given in this section are to be used as an indication only. Since boundary conditions and calculation methods differ, a comparison of these costs would lead to incorrect conclusions.

4.2 Types of structures

Eversdijk, Kleef, Kruithof, Plate and De Gijt [ref 7] compared different types of structures in 1997 and came up with a few well founded design alternatives of the northern shore protection. As can be seen in Table 9, the artificial dune alternatives are relatively inexpensive. Only the traditional sea dike can compete.

Eversdijk states that with respect to the dune alternatives large uncertainties exist with regard to the maintenance costs because of erosion of the dune and accretion of the Maasgeul. Meetings with engineers of Port of Rotterdam supported this theory; as a result a shore protection using an artificial dune at the northern side of Maasvlakte 2 is not further investigated.

Table 9 Cost alternatives Eversdijk, P.J. & Kleef, M.J. & Kruithof, T. & Plate, S.E. & Gijt, de,J.G.[ref 7]

Costs (x 10^6 €) *Alternative*	*Building materials per 4 km, 2005* [2]
Artificial dune	
▪ Unprotected	119
▪ Zuiderdam alternative	249
▪ Hydraulic fill dams	144
▪ Quarry stone breakwater	119
Sea dike	168
Caisson	354
Block wall	484
Retaining wall	807

Construction and design of the retaining wall alternative is problematic. Furthermore costs are so high that this alternative is not recommendable. As for the retaining wall construction, the caisson and block wall constructions are so expensive that application is not reasonable.

[2] Cost are indexated to the year 2005 from the original price level of 1997 using a 4% inflation rate.

Special attention should be paid to scour protection within the design of the sea-dike alternative. It is recommended to distinguish different parts of the sea defence. Furthermore the orientation can be optimized.

The most important recommendations by Eversdijk et al. concerning the northern shore protection are stated below.

- "It is recommended to make a design of both the sea dike and the artificial dune in more detail.
- In the next phase of the design process, all the steps taken should be reconsidered taking into account the adjusted requirements. Special attention should be paid to possible alternatives and design assumptions."

Expertisecentrum PMR [ref 9] also came up with a preliminary design of a traditional sea dike. The main difference with earlier studies lies within the adjusted requirements, especially concerning the used safety levels. The final cross-section of this research can be seen below.

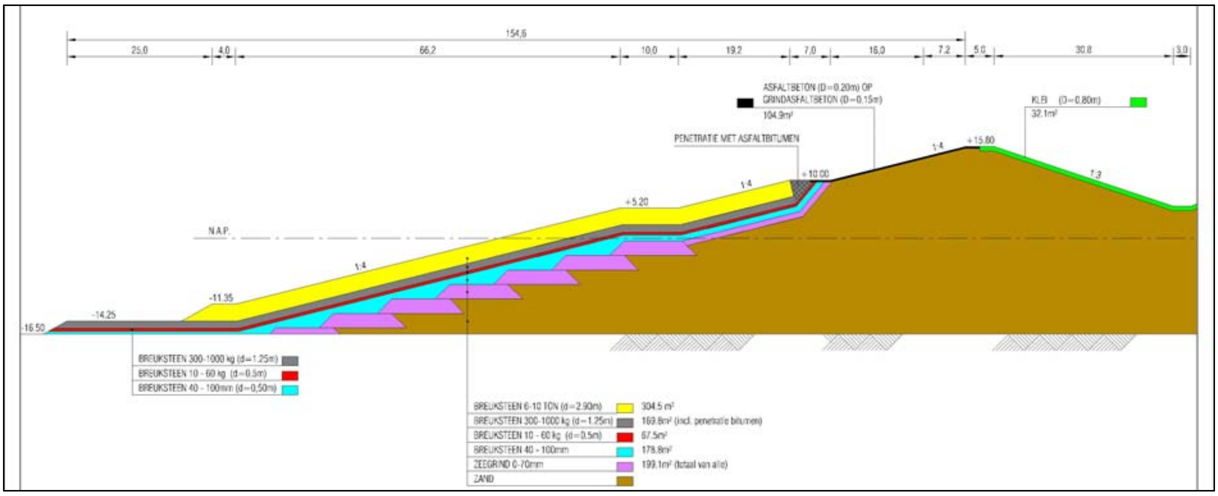


Figure 20 Cross-section northern sea dike [ref 9]

Most of the feasible concept designs found in literature are rather conservative. This does not correspond with the conclusions from the brainstorm sessions, organised by *Project Organisatie Maasvlakte 2* in June and August 1996, which said: "The Maasvlakte 2 Project is unique and prestigious. … The design phase should be innovative …" [ref 12].

Eversdijk mentioned the possibility of a low crested sea dike, but than dropped the idea "because there is not enough knowledge about its use available in the Netherlands". The general concept of an overtopping dike is to save building materials by decreasing the crest height, allowing more water to flow over the structure than is the case with a traditional sea-dike. Off course measures have to be taken to drain off the seawater behind the structure (Figure 21).

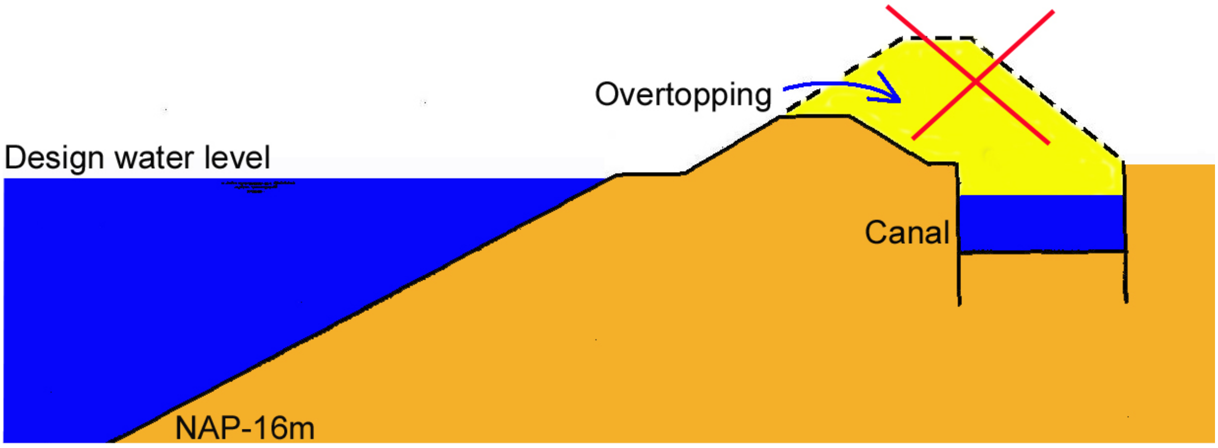


Figure 21 Reduction of building materials by allowing overtopping

One could argument that the intrusion of salt water in the ecosystem behind the sea defence is unfavourable; however environmental laws state that the soil quality may not decrease due to ones actions. Since the land is reclaimed with saline sand from the sea, the quantity of salt in the system will not increase due to the construction of a low crested sea-dike instead of a traditional sea dike which allows very little overtopping. Therefore, the quality of the soil does not decrease but stays the same.

Calculations made clear that decreasing of the crest height is most cost effective on dikes with an expensive top part of the construction. This is the case for a sea-dike with a top layer of artificial armour units.

4.3 Armour layer

4.3.1 Armour units

The choice in favour of a traditional sea dike with quarry-stone armour layer is explained as follows by Eversdijk: "usage of placed concrete elements is not possible because of the local wave climate. Furthermore it is assumed that dumped concrete elements on the chosen slope (1:4) is less profitable than application of quarry-stone".

This reasoning is thought to be rather poor, since firstly most concrete elements are effectively used on slopes varying from 1:1 to 1:2, which will cause a great reduction in costs, and secondly it is not proved that for instance a single layer of cubes is unstable under design conditions.

Figure 22 XBloc developed by Delta Marine Consultants

Figure 23 shows different types of concrete armour units. Because the development of the XBloc (Figure 22) started only in 2001 and is not used anywhere, it is not printed in the Coastal Engineering Manual.

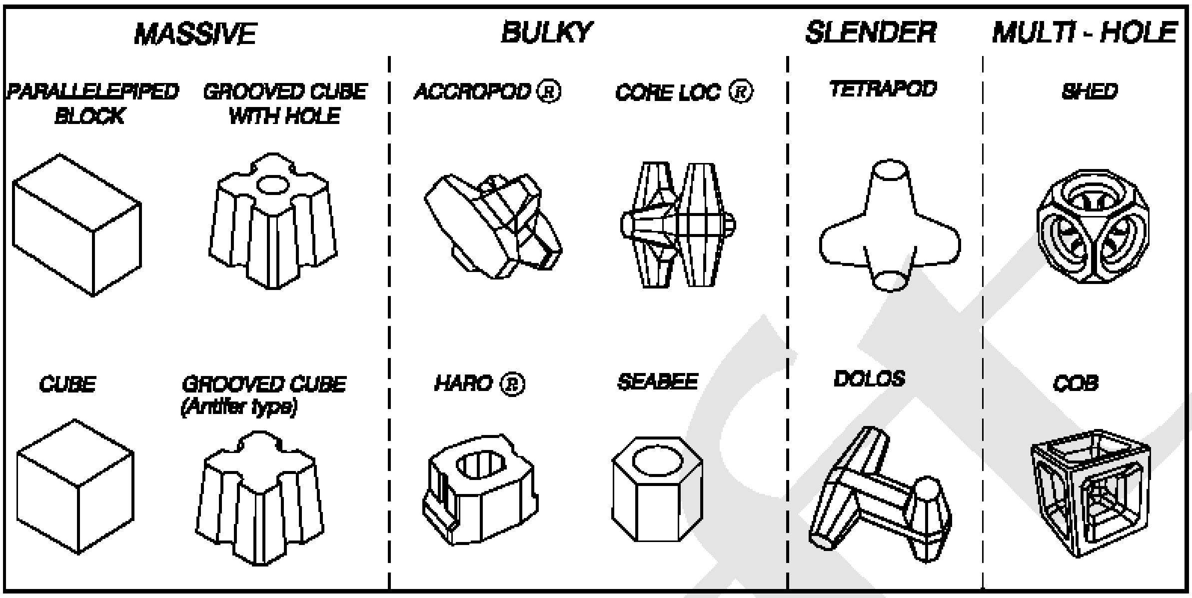


Figure 23 Examples of concrete armour units [ref 22]

Existing insight within the Maasvlakte 2 organisation learned that with the current used boundary conditions both sea dikes with armour layers of interlocking elements and quarry stone have equal costs. An armour layer consisting of a double layer of cubes seemed very expensive.

Table 10 Specifications for different armour units and quarry stone [ref 22]

Armour unit	Number of layers	Maximum steepness	Rougness coefficient, γ_r	Stability coefficient, K_D	Porosity, n
Quarry stone	2	1:4	0.55	2	0.4
Cube	2	1 : 1.5	0.55	6.5	0.4
XBloc	1	1 : 1.33	0.55	16	0.6
Accropode	1	1 : 1.33	0.55	12	0.5

4.3.2 Single layer of cubes

D.N. Klazinga [ref 14] found cost for a single and a double layer of cubes to be almost identical and very expensive. Furthermore a single layer of cubes, applied with a slope 1:1.5 has a very smooth surface, which causes great wave reflection. It is thought that the maximum reflection coefficient of 25% will be exceeded. At last great uncertainty regarding stability of a single layer of cubes exists. Taking into account the reasons above, a sea dike with a single cube armour layer is not favourable.

4.3.3 Artificial concrete quarry stone

The enormous quantities of quarry-stone needed are thought to lead to logistic difficulties. If necessary the production of quarry stone with concrete could be an option. However calculations show that replacement of the quarry stone armour layer by artificial concrete armour rock causes an increase of building material costs with 60%[3]. Therefore, application of fabricated quarry rock is not likely.

[3] Only costs for building materials are taken into account. For a traditional sea dike the armour layer is approximately 25% of the costs. Assumpitons are: concrete: €110,-/m3, porosity stones: 0.4, quarry stone costs:

4.3.4 Allowing damage

Since the terrain is designed at a height of NAP+6.20m and the water level which is exceeded with a probability of 1/10,000 per year is NAP+4.80m, one can ask themselves if there is any danger in case the structure fails. In the worst case there will be tens of meters of coastal erosion but certainly no danger to inundation exists. From this point of view, the calculation of the size of the armour elements depends not on the safety level, but on finding an economic optimum.

The allowable damage level, S for quarry stone or N_{od} for elements, could be increased, which causes a decrease in the applicable element size. Because damage will occur a few times during the life cycle of the structure, calculations have to make clear if this is favourable in an economic sense. Application of placed elements in combination with the method of allowing damage is not possible since small element displacements tend to set ongoing damage in motion.

4.4 Conclusions

The artificial dune alternative is according to Eversdijk et al. the most economic solution, but great uncertainties exist concerning the amount of maintenance that will be necessary. The expected frequent dredging operations cause vagueness in the total life cycle costs, which is not favourable. Furthermore dredging operations will hinder the navigation. For these two reasons the artificial dune alternatives whether or not in combination with hard construction elements are not further investigated.

Because preliminary calculations showed that the overtopping concept is promising, it is decided to make a detailed design of this type of structure in the coming sections.

Decreasing of the crest height is most cost effective on dikes with an expensive top part of the construction. This is the case with a sea-dike with interlocking armour elements. A choice between the Accropode and XBloc unit will be made in chapter 5. For comparison reasons the concept of a low crested dike with a quarry stone armour layer is designed in more detail as well.

The application of a sea dike with a single layer of cubes is not preferred for two reasons. Firstly the smooth surface causes too much reflection and secondly costs are according to Klazinga [ref 14] in the order of magnitude of a double layer of cubes, which is relatively expensive.

Application of concrete interlocking elements in combination with the method of allowing damage is not possible because due to those interlocking capacities the construction fails in a progressive way, when design conditions are exceeded.

A solution for the logistic problems due to enormous quantities of quarry stone needed may be found by replacing the top stones with artificial concrete stones. A great disadvantage is the fact that cost rise by 60%. Therefore application of this concept is not recommended.

€18.5/m3. The costs for artificial concrete stones are 3.5 times the costs for quarry stone (110*(1-0.6)/18.5), subsequently costs for a sea dike with an armour layer of artificial concrete rocks are 1.6 times the costs for a traditional quarry stone sea dike (3.5*25% + 1.0*75%=1.6).

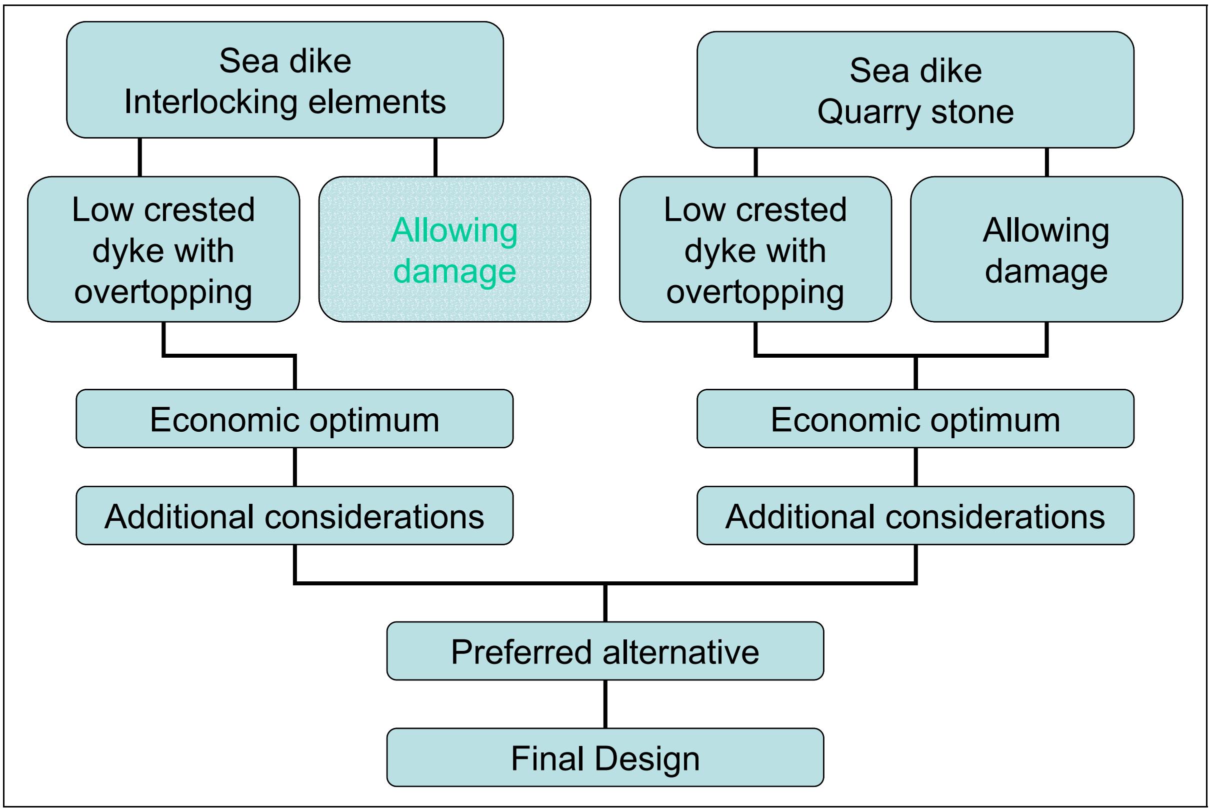


Figure 24 Schematisation of design phase based on preliminary calculations and literature study

5 Design

5.1 Introduction

The conclusions of the literature study indicate that the concept of a low crested sea-dike is promising. Section 5.2 describes the design considerations and calculations of this specific subject. Section 5.3 treats the dimensioning of the armour layer and thereby the underlying layers. The remaining construction aspects are described in sections 5.4. Section 5.5 describes the construction method and the costs of the final design.

5.2 Low crested sea-dike

The concept of a low crested sea-dike is to reduce costs by lowering the crest height.
Facilities have to be made to take care of the water behind the dike.
The aspects that lead to higher cost in comparison with traditional sea-dikes are mentioned in the left side of Figure 25. On the right hand side the profits are mentioned.

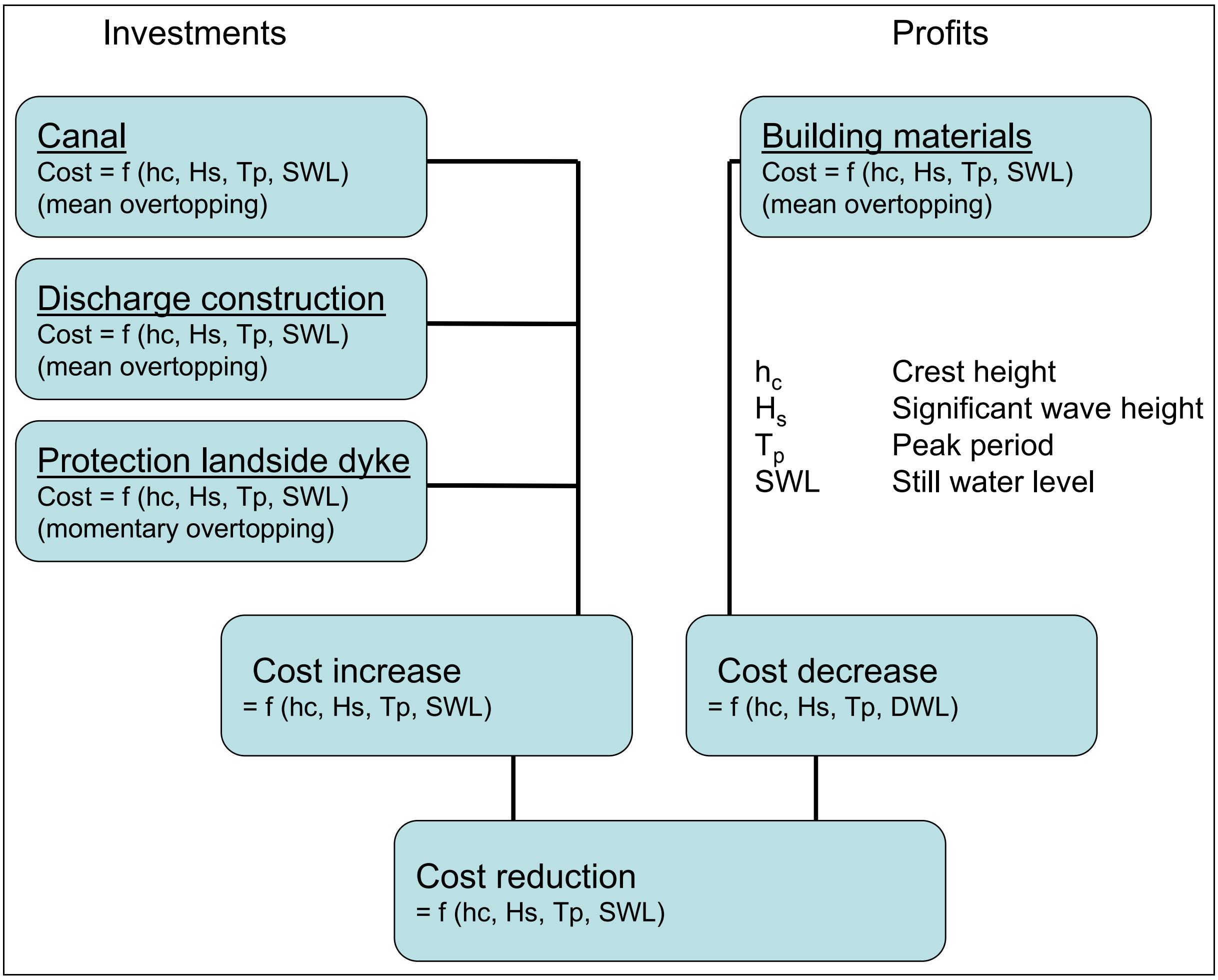


Figure 25 Relations cost reduction due to decreasing crest height

5.2.1 Canal and discharge construction

Figure 27 shows four different methods to discharge the water of the overtopping waves. Depending on the crest height, the overtopping discharge can be hundreds of litres per meter dike per second; momentary even ten times more. The choice is based mostly on the estimated mutual difference in construction costs and amount of industrial hindrance that is expected during storm conditions. Other considerations are the uncertainty with respect to the frequency and intensity of maintenance and the hindrance for recreation in the area.

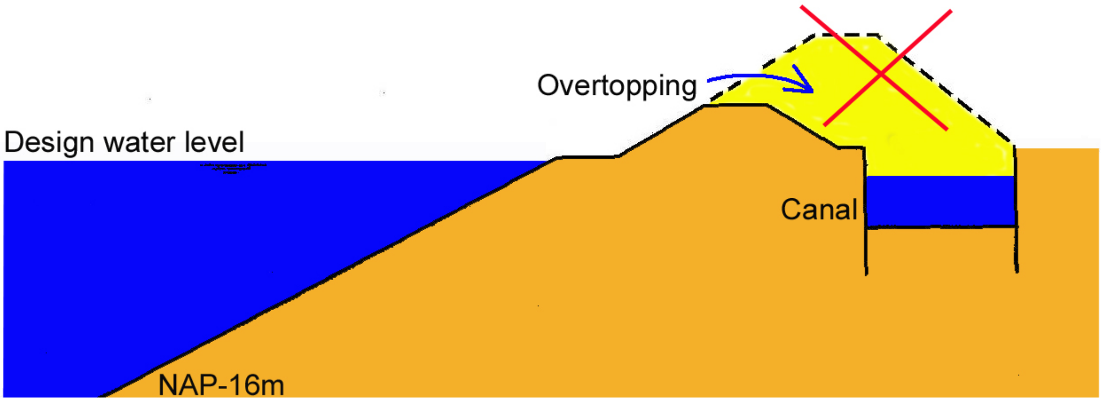


Figure 26 Concept of low crested sea-dike

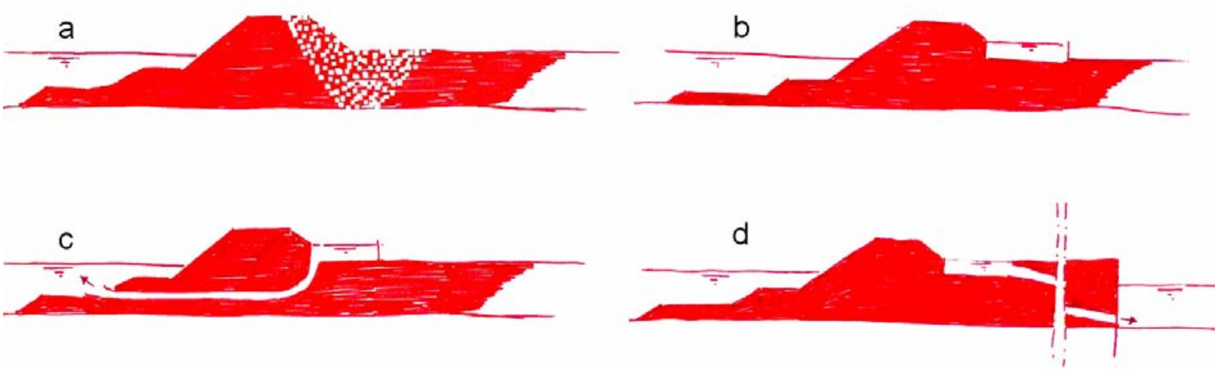


Figure 27 Discharge alternatives: porous dike (a), canal (b), discharge through dike (c) and discharge to harbour (d)

Via a multi criteria analyses the most favourable discharge construction for the northern sea defence is determined. The arguments of the scores given in Table 11 are stated below.

Despite the fact that for the porous dike alternative no canal has to be constructed, it is estimated to be the most expensive solution. Because the design ground water level is NAP+4.8m, the application of porous material instead of sand must be done over a great area. High material costs and the usage of a large area on reclaimed land make this a costly alternative. The water outlet through the dike (c) is expected to be more expensive than alternatives b and d, especially because all three alternatives need a canal construction to collect the overtopped water.

Since there is only few experience in the application of alternatives a and c, a negative score is given for maintenance uncertainty. The water outlet in the port basin will cause some flow, which could hinder berthing ships. Therefore, alternative d is judged negative at *economical hindrance*.

The water discharge at both ends of the northern sea defence will locally cause a flow. Surfing conditions at the western end of the construction may decrease by this phenomenon.

Furthermore the discharge construction on the beach decreases the quality of the landscape. Therefore, alternative b is given a negative evaluation with respect to *recreational hindrance*.

Table 11 Multi criteria analyses discharge method for low crested sea-dike concept

Criteria	*Weighing factor*	*Porous dike (a)*	*Discharge at ends (b)*	*Discharge through dike (c)*	*Discharge to port (d)*
Construction costs	3	-	+	0	+
Maintenance uncertainty	2	-	0	-	0
Economical hindrance	3	+	+	+	-
Recreational hindrance	1	+	-	+	+
Score		**1 -**	**5 +**	**2 +**	**1 +**

Based on the multi criteria analyses a canal construction is chosen that collects and discharges the water to both the eastern and western end of the sea defence (alternative b). The cross-section dimensions of the canal are determined in the overtopping section below.

Overtopping

The minimum safety level for the protection against flooding is determined by the government and set at 1/10,000 per year. Due to the high ground level of the reclaimed area (NAP+6.2m), flooding of Maasvlakte 2 has an occurrence frequency far smaller than 1/10,000 per year. Therefore, this requirement is interpreted that there may be a maximum probability of 1/10,000 per year that infrastructure situated directly behind the sea defence will suffer some damage. As a result overtopping calculations are performed with conditions that are exceeded with a probability of 1/10,000 per year.

Overtopping calculations are performed with the software PC Overslag. Figure 28 shows the results for a sea-dike with a quarry stone armour layer as well as an armour layer that consists of concrete elements. No crest construction is applies. Information about the used theory of overtopping waves is stated in Appendix M.

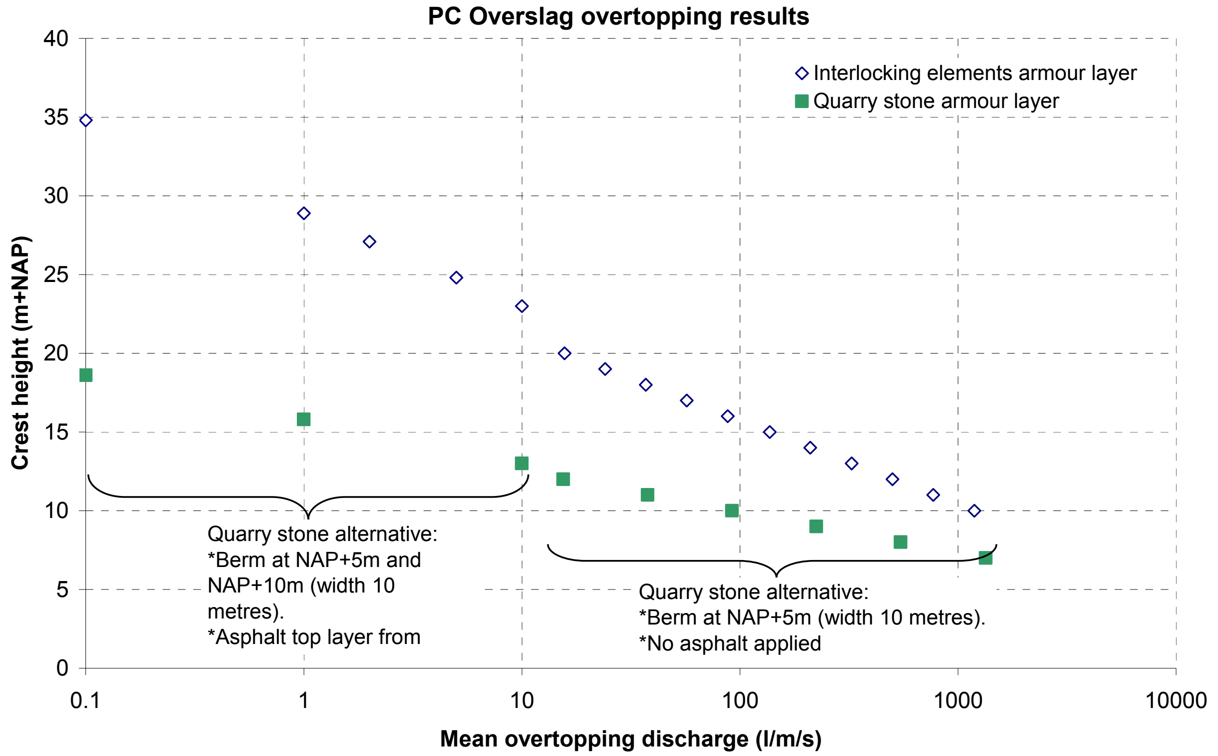


Figure 28 Mean wave overtopping discharge calculated at the exterior crest line for 10^{-4} conditions

Table 12 Description PC Overslag input

Armour layer	Slope	Berm	Roughness
Interlocking elements	3:4	No	0.55
Quarry stone	1:4	NAP+5m, width: 10m	0.55
Quarry stone, Asphalt above NAP+10m	1:4	NAP+5m, width: 10m NAP+10m, width: 10m	0.55 1.00

The results obtained by using PC Overslag are used for dimensioning the discharge canal. It is assumed that all the water overtopping the crest is collected in the canal. This is slightly conservative because part of the wave will discharge through the crest elements and filter layers and therefore needs no discharge through the canal.

By creating discharge constructions at both ends, the maximum discharge through the canal can be divided by 2 (see Figure 29). The maximum discharge is situated at both ends and is determined by the average overtopping discharge and the length of the sea dike and can be calculated with:

$$Q_{Canal\_End} = \frac{1}{2} Q_{Overtopping} L_{Dyke}$$

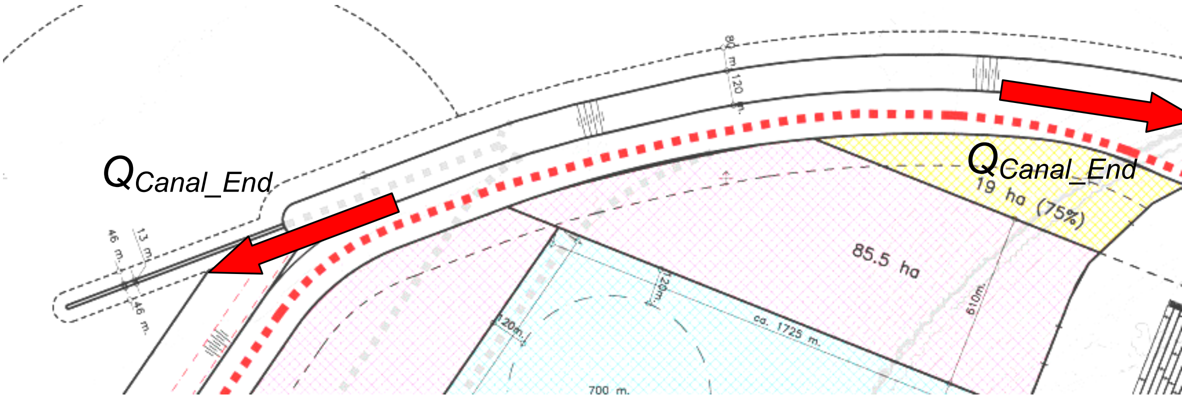


Figure 29 Discharge constructions situated at both ends of the sea dike

The cross-sections are determined with the formula for open flow:

$$Q_{canal} = WhC\sqrt{Ri}$$

where

W = canal width
h = water level in the canal
C = Chezy friction factor (50 $m^{0.5}/s$)
i = slope (0.001)

For more details regarding the formula above the reader is referred to appendix O.

Since the outflow can reach speeds up to 4 m/s a construction is needed to slow down the flow. The costs for the construction are estimated at € 0.5 million per side.

5.2.2 Protection rear slope

By situating the canal directly behind the crest, space is used more effectively and thus costs are reduced. Another advantage is that there is no rear slope that needs extra protection against overtopping waves. Therefore no extra costs are taken into account for protection of the rear slope.

5.2.3 XBloc, Accropode and quarry stone

As usual the dimensioning of artificial elements is determined by the company that patented the unit.

Delta Marine Consultants (DMC) gives the following basic rules for a concept design (see also N.):

XBloc unit height: $D \geq \dfrac{H_s}{1.92\Delta}$ $\qquad \Delta = \dfrac{\rho_{concrete} - \rho_{seawater}}{\rho_{seawater}}$

XBloc unit weight: $W_{XBlock} = \rho_{concrete} \dfrac{D^3}{3}$

Slope: 3V 4H
K_D value trunk: 16

Sogreah consultants recommend K_D values of 12/15 (breaking waves / non breaking waves) for the design of Accropode armour layers. The Sogreah recommendations appear conservative with respect to experimental results (Van der Meer, 1988). Because DMC recommends a significant higher K_D coefficient (16) than Sogreah consultants the XBloc unit weight is about 0.75 times the weight of the Accropode element. It may be clear that the smaller XBloc units cause a reduction in costs due to concrete saving. The design tool on the website www.xbloc.com, recommends the use of 15 tons XBloc units.

Further designs with interlocking elements will be made using the 15 ton XBloc unit.

Quarry stone

The quarry stone armour layer is designed with the Van der Meer formula for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\frac{1}{\sqrt{\xi_m}}$$

Basic assumptions that are made for the stability of the top layer are:

- density stone: 2650 kg/m^3
- density sea water: 1030 kg/m^3
- storm duration: 6 hours
- Permeability factor 0.1

When a damage level of 10 is accepted with a return period of 10,000 years a grading of 8-12 tons is stable. This rock grading will be used for comparison methods between the interlocking element and the quarry stone alternatives. Further optimisation of the quarry stone top layer is done in the end of this section.

5.2.4 Cross-section

In order to optimise the concept of a low crested sea dike, cross-sections are designed for both the alternative with a quarry stone (8-12 tons) and an XBloc (15 ton) armour layer. Two examples are shown in Figure 30; the reader is referred to appendices P. to T. for an overview of all low crested dike alternatives.

Table 13 Overview design parameters for cost comparison

	Quarry stone alternatives	*XBloc alternatives*
Crest level	Varying	Varying
Crest width	10 m	10 m
Armour layer	3.1 m; 8-12 ton quarry stone	2.5 m of 15 ton XBloc
First filter layer	1.8 m; 1-3 ton	1.8 m; 1-3 ton
Second filter layer	1.0 m; 60-300 kg	1.0 m; 60-300 kg
Third filter layer	1.0 m; 5-40 kg	1.0 m; 5-40 kg
Core material	Sand	Sand
Slope	1:4	3:4
Toe level	NAP-9.0m	NAP-9.0m
Bed protection	25 m; 60-300 kg and 5-40 kg	25 m; 60-300 kg and 5-40 kg
Canal dimensions	Depending on crest height	Depending on crest height

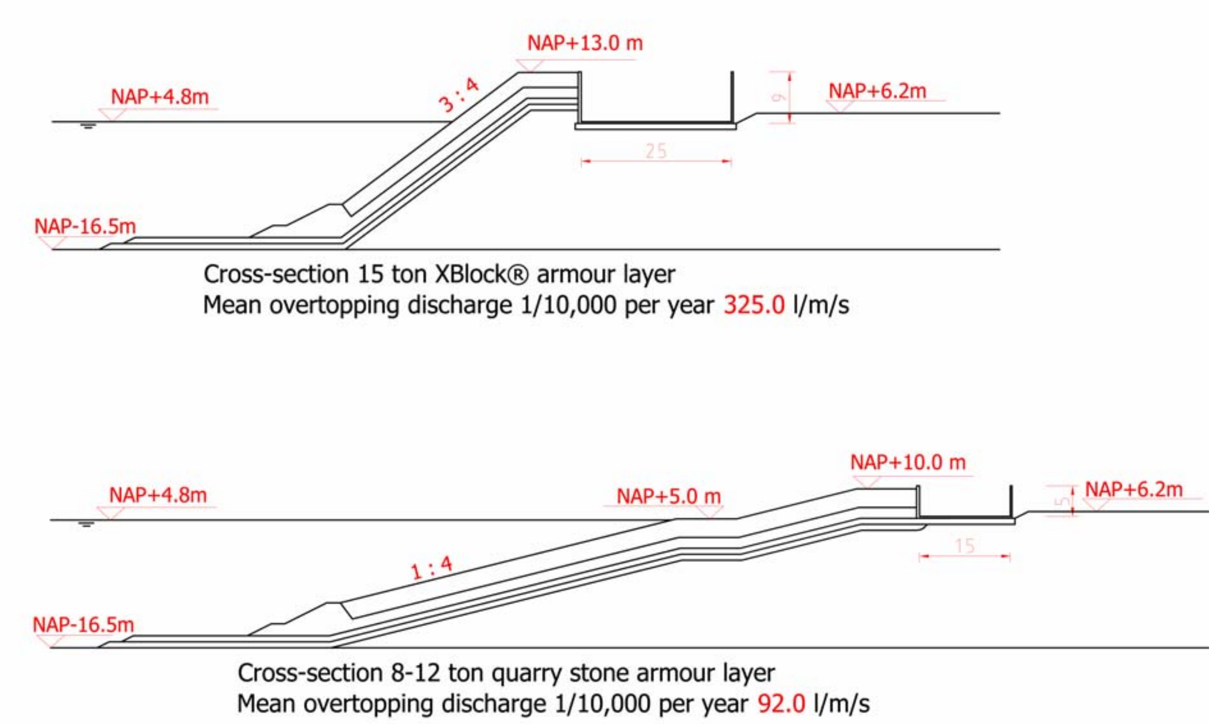


Figure 30 Low crested sea dike alternative, 8-12 ton quarry stone armour layer

5.2.5 Multi functional use of land

In order to economize the sea dike the possibility to locate one or more carriageways in the discharge canal is investigated (see Figure 31). In the case that waves are overtopping the dike, the two carriageways situated in the canal will be closed and temporary only two lanes will be available.

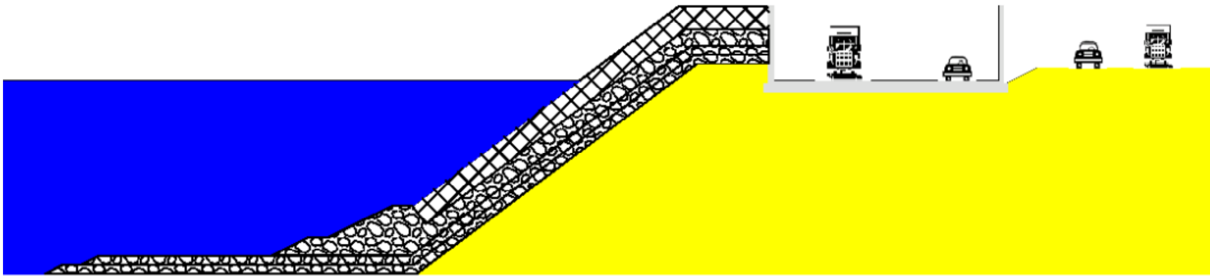


Figure 31 carriageways inside the discharge canal

Because this situation is highly unfavourable the crest height must be so high that the event of closing off the carriageways may only occur in the case that sea conditions do not allow vessels to enter the port of Rotterdam for more than 3 days. The reason a period of 3 days is chosen is the fact that the average waiting time of a container is approximately 3 days. Wind speeds higher than 10 Beaufort have a exceedance frequency of 1/100 per year (Appendix W.).

Subsequently the probability of closing off the carriageways may be no more than 1/100 per year. Combining this information with the rough-and-ready-rule that safe driving is still possible behind a sea dike with an average overtopping discharge of 0.01 l/m/s, it can be reasoned that in the case of situating carriageways in the discharge canal, the crest height must be high enough to ensure a maximum average overtopping discharge of 0.01 l/m/s with a exceedance frequency of 1/100 per year.

Calculations with PC Overslag show that the minimum crest level for quarry stone and a XBloc alternative allowing a maximum overtopping discharge of 0.01 l/m/s is respectively NAP+15.2m and NAP+28.5m (see Table 14). In the cost section the economic considerations are made.

Table 14 Minimum crest levels to ensure average overtopping criteria of 0.01l/m/s

Exceedance frequency	*Quarry stone armour layer*	*XBloc armour layer*
1/1 per year	NAP+8.8m	NAP+15.8m
1/10 per year	NAP+12.3m	NAP+23.2m
1/100 per year	NAP+15.2m	NAP+28.5m

A dike with two carriage ways inside the discharge canal must have a minimum crest level of NAP+28.5m. Instead of reducing costs, costs will rise by doing so.

5.2.6 Cost

Unit prices

In order to make an economical comparison of the low crested dike alternatives, unit prices for different building materials must determined. Due to the differences in handling costs between light and heavy graded rocks these unit prices differ significantly.

Conversations with experts taught that rocks with a weight up to 3 tons can be handled relatively easy with a grab or dumped with a side stone dumping vessel. Heavier rocks must be prepared with lifting hooks and can only be lifted with large cranes. Percentages varying from 5 to 50% are added to the building material prices in order to obtain a unit price that includes buying and placing of the materials. Figure 32 shows the results.

The price for production and placement of the XBloc units is calculated by adding €11,- per ton (= €27,50 per m3) to the price for concrete. The unit price for XBloc elements with a weight of 15 tons becomes €100,- + €27,50 = €127,50 per m^3.

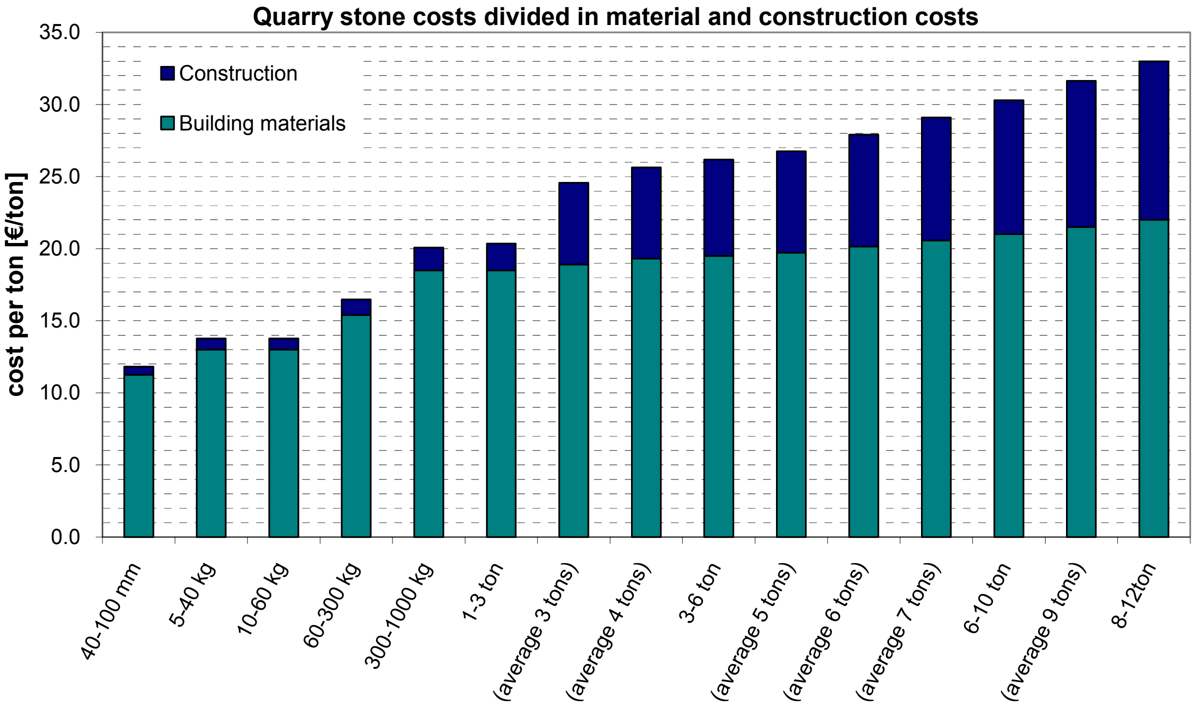


Figure 32 Unit prices based on material and handling costs

The canal will be constructed with prefab concrete slabs. Costs for construction and placement are estimated to be €110,- per m^3. The floor and landside wall are designed with a thickness of 0.3m. Because the sea side wall supports the crest partially, the thickness is 0.5m.

Low crested sea-dike

From Figure 33 it becomes clear that quarry stone alternatives are far more expensive than the XBloc alternatives. The most right point on the green line represents a traditional sea dike with a quarry stone top layer from NAP-16m to NAP+10m and an asphalt layer from NAP+10m to NAP+13m. Because the asphalt top layer in the design must be replaced by an 8-12 ton armour layer in case of a low crested sea dike, no cost savings are obtained.

As discussed earlier in this section, a dike with two carriage ways inside the discharge canal must have a minimum crest level of NAP+28.5m. Instead of reducing costs, costs will rise by doing so.

Table 15 Unit prices of additional construction material

Building material/unit	*Unit prices (production and construction costs)*	*unit*
XBloc	127,50	€/m^3
Sand	4,-	€/m^3
Canal concrete	110,-	€/m^3

The economic optimum for a low crested sea dike lies on the XBloc-line at a crest level of NAP+14.0m (€20,800 per meter). The right end of the blue line represents a traditional sea dike with a crest level of NAP+21m which allows a maximum overtopping of 10 l/m/s with a return period of 10,000 years.

The quantities of building materials are taken directly from the cross-sections in Appendices P. to T. For thorough cost calculations the reader is referred to appendix U.

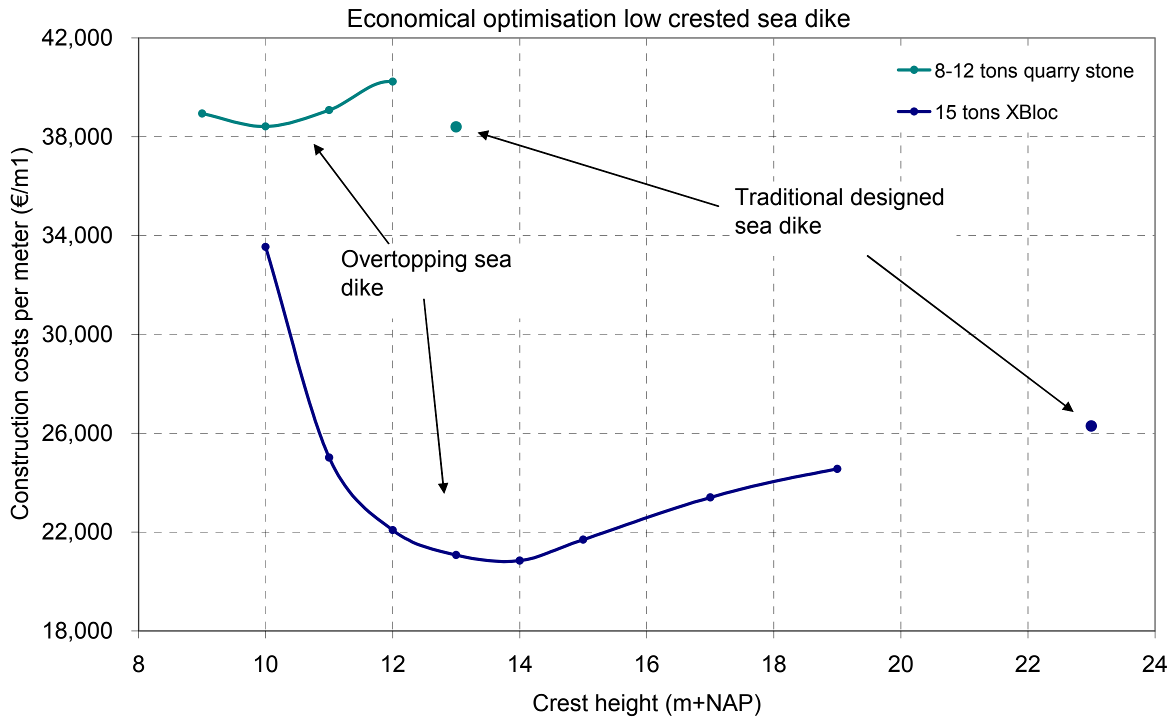


Figure 33 Economic optimisation for a low crested sea-dike[4]

5.2.7 Conclusions low crested sea-dike

A sea-dike with a crest level of NAP+14.0m and two carriageways in the canal is most inexpensive (€19,800 per meter)[5]. However the disadvantages of industrial traffic delay by closing of two lanes weighs heavily. A construction with a crest level of NAP+14.0m and the infrastructure situated behind the canal is preferred.

The economic optimum for a low crested sea dike is found for a low crested dike with XBloc armour units and a crest level NAP+14.0m. Costs are €20,800 per meter[6]. Costs for an end construction are included; maintenance costs are excluded.

[4] Costs are based on unit prices from Figure 32 and Table 15.
[5] Costs are based on unit prices from Figure 32 and Table 15.
[6] Costs are based on unit prices from Figure 32 and Table 15.

5.3 Allowing damage

Since the terrain is designed at a height of NAP+6.20m and the water level which is exceeded with a probability of 1/10,000 per year is NAP+4.80m, one can ask themselves if there is any danger in case the structure fails. In the worst case there will be tens of meters of coastal erosion but certainly no danger to inundation exists. From this point of view, the calculation of the size of the armour elements depends not on the safety level, but on finding an economic optimum.

The allowable damage level, S for quarry stone could be increased, which causes a decrease in the applicable rock grading. Because damage will occur a few times during the life cycle of the structure, calculations have to make clear if this is favourable in an economic sense.

Application of concrete elements in combination with the method of allowing damage is not possible because due to the interlocking capacities of the XBloc unit, which makes the construction fail in a progressive way.

In this section the application of a smaller rock grading will be investigated. The top and bottom part of Figure 34 show cross-sections with respectively a heavy and a relative light grading.

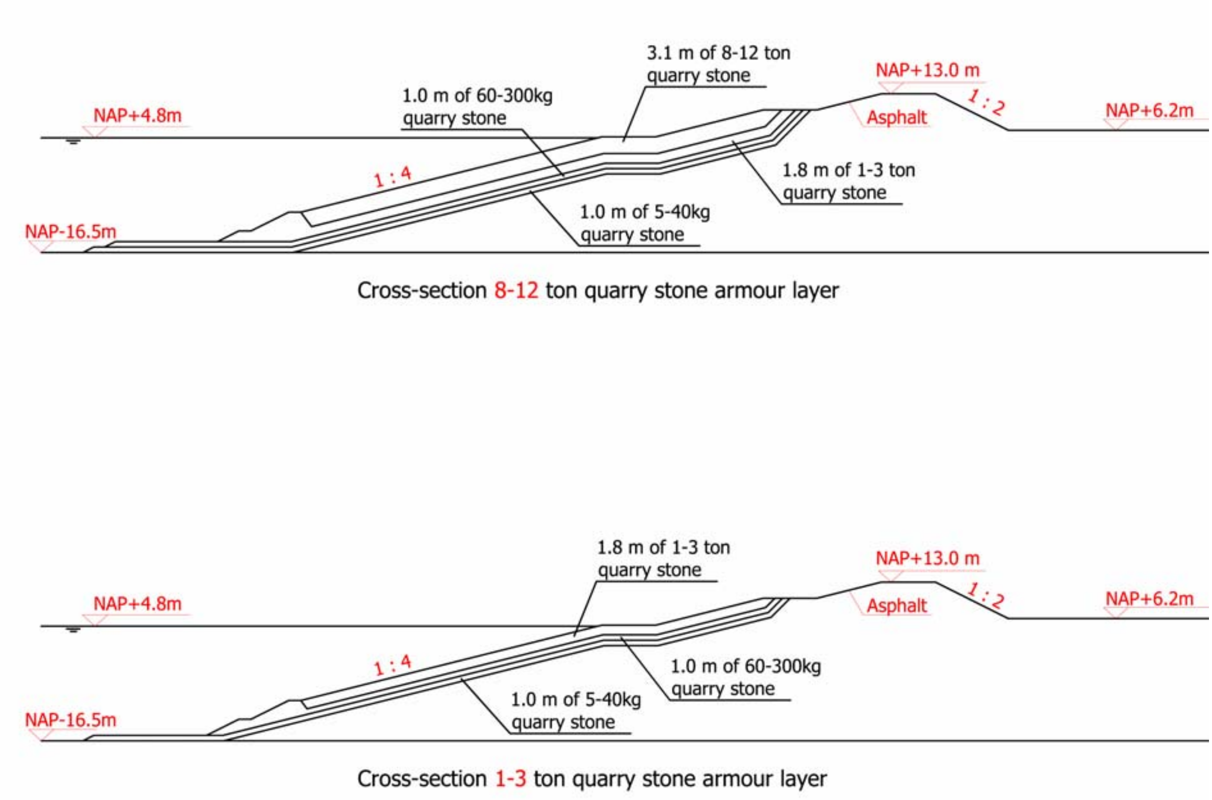


Figure 34 Cross-sections for concept of allowing damage

5.3.1 Approach

The goal of this section is to determine if cost savings can be obtained for the quarry stone alternative by lowering the initial construction costs and accepting the fact that damage of the structure has to be repaired with a certain frequency. Figure 35 shows an increase in construction costs while expected repair costs (risk) decrease.

By calculating the expected damage levels of different armour layers for varying storm conditions and multiplied by the expected quantity of those storms, the expectation values of the repair costs are determined. Converted to the price level of 2005 assuming 4% inflation, the repair costs and initial construction costs together form the expectation costs of the structures with different armour layers.

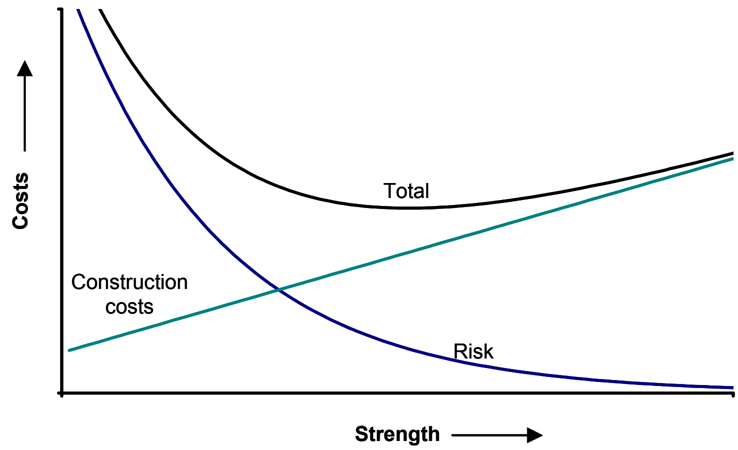


Figure 35 Risk and construction costs for increasing strength of a structure

5.3.2 Damage

For the rock gradings in Table 16 damage levels are determined with the formula developed by Van der Meer for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \frac{1}{\sqrt{\xi_m}}$$

where:

$$\xi_m = \frac{\tan \alpha}{\sqrt{s}} = \frac{\tan \alpha}{\sqrt{H_s / L_0}}$$

Basic assumptions that are made for the stability calculations of the top layers are:

- Density stone: 2650 kg/m^3
- Density sea water: 1030 kg/m^3
- Storm duration: 6 hours
- Inflation: 4% per year

Table 16 Construction costs of sea dikes with different armour gradings[7]

Rock grading	W50 [ton]	Dn50 [m]	Construction costs [€/m1]
1-3 ton	2	0.88	22,844
No standard grading	3	1.04	24,761
No standard grading	4	1.10	26,678
3-6 ton	4.5	1.18	27,637
No standard grading	5	1.24	28,595
No standard grading	6	1.31	30,512
No standard grading	7	1.38	32,429
6-10 ton	8	1.43	34,346
No standard grading	9	1.50	36,263
8-12ton	10	1.56	38,180

Table 17 Parameters for stability calculations with Hudson's formula

Exceedance frequency	10^{-4}	10^{-3}	10^{-2}	10^{-1}	10^{0}
Slope angle, tan alpha	0.25	idem	idem	idem	Idem
Significant wave height, H_s	6.9	6.3	5.5	4.5	3.2
Mean wave period, T_m	9.3	8.8	8.2	7.2	5.8
Wave steepness, s	0.05	0.05	0.05	0.06	0.06
Surf similarity parameter, ξ_m	1.11	1.10	1.09	1.06	1.01
Relative density, Δ	1.57	idem	idem	idem	Idem
Number of waves, N	2323	2455	2634	3000	3724
Permeability, P	0.1	idem	idem	idem	Idem

From Figure 36 it can be concluded that storm conditions which are expected hundred times during the life cycle of the structure (100 years) do not cause serious damage to any of the alternatives except for the 1-3 ton armour layer where 3 or 4 stones per meter are displaced. A storm which is expected to occur 0.01 times during the lifecycle (storm conditions 10^{-4}) cause failure of all of the alternatives except for the ones with a armour rock grading of 8-12 tons and the non standard grading with W_{50}=9 ton. Exact values can be found in appendix X.3.

[7] The construction costs of the alternatives are interpolated linear between the calculated costs of the 1-3 ton alternative and the 8-12 ton alternative Figure 34. Costs are based on unit prices from Figure 32 and Table 15. Standard rock gradings are adopted from the Coastal Engineering manual.

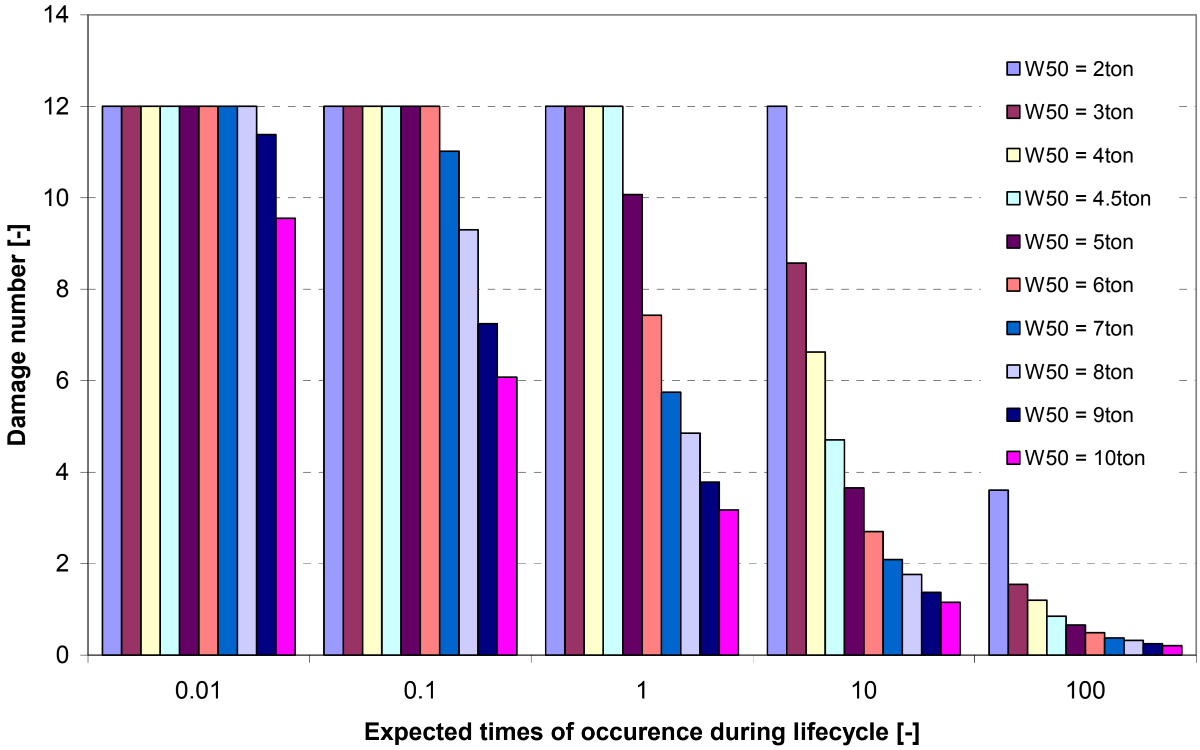


Figure 36 Number of stones displaced for different armour layers. S>12 means failure

5.3.3 Costs

Since it is quite costly to collect displaced stones at a water depth of 17 metres, it is assumed that new rock will be used for the repair of the damaged armour and possible filter layers.

Four phases of damage are distinguished:

- $S \leq 6$: The filter layer is not exposed; new rocks have to be placed. Because no filter layers are damaged, repair activities are relatively simple.
- $6 < S \leq 9$: Part of the filter layer is exposed. These parts together with the armour rock have to be repaired.
- $9 < S \leq 12$: The under layers are exposed. Intensive repair is necessary.
- $S > 12$: The construction failed and has to be rebuilt.

The reader is referred to appendix X.2 for exact repair costs of the phases described above.

Damage that is expected to occur 100 times during the lifecycle is assumed to occur one time every year.

Maintenance costs are indexated to the year 2005 with:

$$IC = C\left(\frac{1}{1+r}\right)^t$$

Where

IC = Indexated costs, 2005 [€]

C = Repair costs in the year (2005 + t) [€]
r = Yearly inflation [-]
t = Time after 2005 [year]

Figure 37 shows the economic optimum for a quarry stone sea-dike for an armour layer rock grading of 3-7 tons (€ 32,300 /m1).

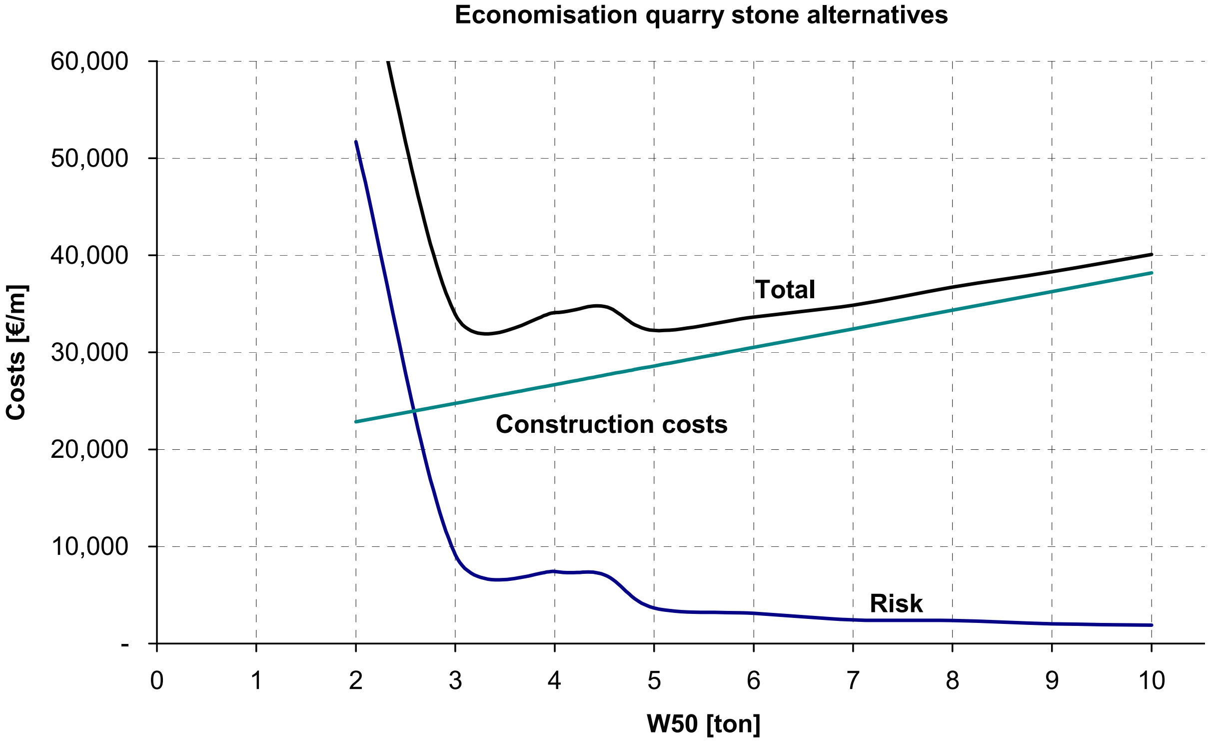


Figure 37 Economical optimisation quarry stone sea dike[8]

5.3.4 Conclusions allowing damage

The concept of allowing damage can not be used with XBloc or other interlocking elements because these structures fail in a progressive way.

Construction costs[9] of a quarry stone sea-dike can be reduced from 40,000 €/m1 (8-12 tons) to 32,300 €/m1 (grading W_{50}=5 tons) by allowing damage. Nevertheless a sea dike with a quarry stone armour layer can not compete with an interlocking element armour layer which is approximately 10,000 €/m1 less expensive. If for any reason a quarry stone sea dike is preferred, an armour layer with a nominal diameter, D_{n50}=1.24 is recommended.

A great disadvantage of designing a sea dike with a higher damage probability is the fact that it might influence the image of the Port of Rotterdam in a negative way. One can imagine that a construction that already needs repair after a couple of years does not look very well in the news.

5.4 Design cross-section

Now that the crest level (NAP+14.0m) and the type of armour layer is determined (XBloc 15 ton), dimensioning of additional cross-section characteristics is treated in this section.

[8] Costs are based on unit prices from Figure 32 and Table 15.
[9] Costs are based on unit prices from Figure 32 and Table 15.

5.4.1 Filter layers

DMC recommends a first filter layer of 1-3 ton graded rock with a thickness of 1.8 meters. The second filter layer consists of 10-60 kg graded rock. To create a slope of 3:4 bunds can be constructed with sea gravel. A more innovative and probably less expensive method is the use of geocontainers or geotubes. Because the latter method is designed to fill and place the geotube in a continuous way it is well qualified for great distances as is the case for Maasvlakte 2. Van Zijl (2004) calculated costs for pontoon and machinery at € 50.000 per week (84 hours). Assuming a work speed of 300 meters geotubes per hour the construction costs for 1 metre of geotubes become 2 €/m1 (=50.000/(84*300)). Material costs of a geotubes with a perimeter of 5 m are € 4.8/m1. The unit costs of 1 metre geotubes becomes € 6.80/m1.

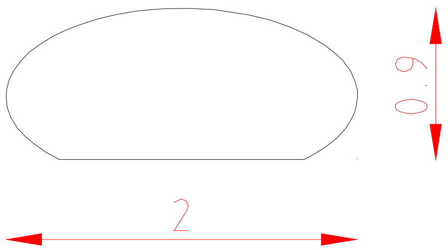


Figure 38 Geotube: geotextile 400gr/m2, unit price € 6.80/m1

5.4.2 Toe construction

Since the toe's main function is to support the XBloc armour layer and thereby the whole structure, it is designed to withstand 10^{-4} conditions (Table 18). A rule of thumb is that the weight of individual stones in the toe construction should be approximately W_{toe}= $0.1W_{XBloc}$=1.5 tons [ref 22]. To be stable under design conditions a toe with 1-3 ton rock grading should be constructed relatively deep.

Table 18 Design parameters for toe design

Return period	10,000 years
Hs	6.9 m
DWL minimum	NAP-2.2m[10]

A relation between the stability parameter and the relative toe depth is given (CUR/CIRIA, 1991):

$$\frac{H_s}{\Delta D_{n50}} = 8.7\left(\frac{h_t}{h_m}\right)^{1.4} \quad \text{for } \frac{h_t}{h_m} \geq 0.4$$

Usage of the formula above will lead to a damage level of approximately 2 under design conditions.

[10] Landaanwinning Ontwerp rapport Terrein, Zeewering en Havendam [ref 9]

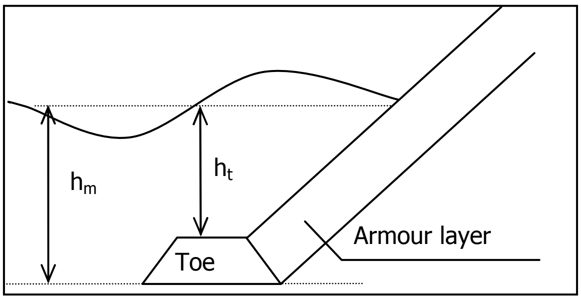


Figure 39 Relative toe depth

It follows that a toe with rock grading 1-3 ton is stable at a maximum toe level of NAP-12.1, therefore the top of the toe is situated at a level of NAP-12.5m.

5.4.3 Wave reflection

The reflection of head-on waves is calculated with (CEM):

$$C_r = \frac{H_{s\_reflected}}{H_{s\_incident}} = \frac{a\xi^2}{\left(b+\xi^2\right)}$$

Where:

a = 0.48 and b = 9.62 [11]

$$\xi = \frac{\tan\alpha}{\sqrt{H_s / L_{op}}}$$

α = slope angle

L_{op} = deep water wave length $= \dfrac{gT^2}{2\pi}\tanh\left(\dfrac{2\pi d}{L_{op}}\right)$

This leads to a reflection coefficient, C_r=0.34 using the peak period and significant wave height with a return period of 1 year. These reflection calculations assume that wave energy is either reflected or absorbed; no wave overtopping is taken into account.

For conditions with a return period of 1 year, a transmission coefficient, C_t of 0.06 can be deduced from Figure 40. This leads to a new reflection coefficient C_r=0.28. Allsop used a slope of 1:2 in his experiments which leads to a smaller transmission than the 3:4 slopes used in the XBloc design, therefore a transmission coefficient higher than 0.06 might be justified.

Since the maximum reflection coefficient is set at 0.25 (requirement 9, chapter 2), the XBloc alternative with a slope of 3:4 can only be applied in the case that overtopping is allowed. Measure that can be taken to reduce the reflection coefficient are:

11 Since no wave reflection data is available for XBloc® elements, fitted Tetrapode coefficients are used with a 3:4 slope

- Increase of the first filter layer thickness. By creating a more open structure, more wave energy is absorbed.
- Reducing the slope from 3:4 to 1:1.5. Economic optimisation in appendix Y. shows that with a slope of 1:1.5 the optimum cross-section is found with a crest level of NAP+12.5 (€24,000/m1).
- Decreasing the crest level. By lowering the crest level, more wave energy is transmitted and thus not reflected. Figure 33 shows that further lowering of the crest height is expensive.

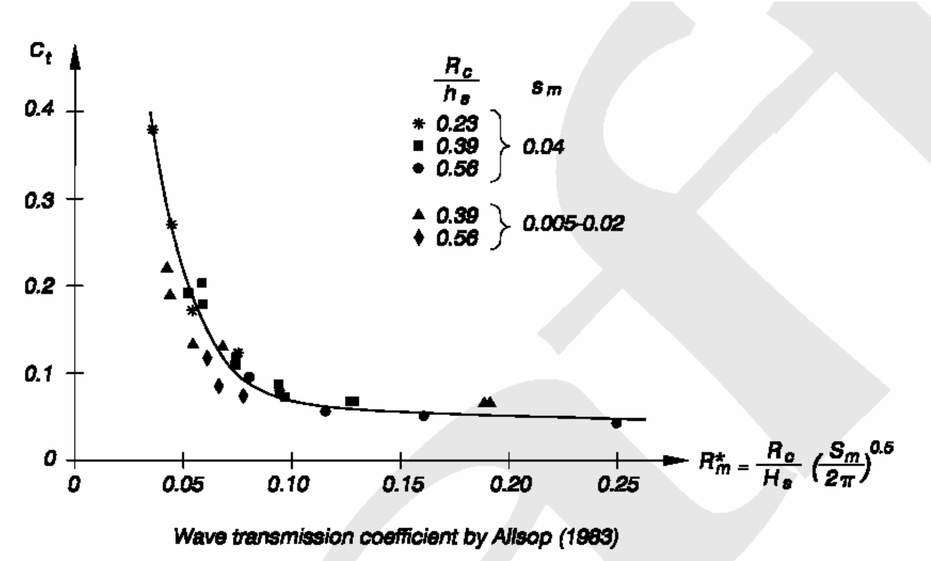


Figure 40 Wave transmission coefficients by Allsop (1983) [ref 22]

5.4.4 Scour protection

To prevent the toe from sliding in the scour hole in front of the structure scour protection is applied. The flow which causes scour at the bottom level is induced by:

- Incident waves,
- Reflected waves and
- Tidal flow.

Maximum scour protection is needed at locations were incident and reflected waves coincide.

The maximum horizontal particle velocity on the bottom by waves is given by(Appendix A.):

$$U_{hor\_bottom\_waves} = \frac{gT}{L}\frac{H}{2}$$

A combination of incident and reflected waves lead to a horizontal flow at the bottom of 2.8 m/s. Since the maximum tidal flow, $U_{hor\_bottom\_tide}$= 1.4 (Appendix Z.) is directed perpendicular to the flow, the maximum bottom velocity is given by:

$$U_{bottom\_\max} = \sqrt{\left(U_{hor\_bottom\_waves}\right)^2 + \left(U_{hor\_bottom\_tide}\right)^2}$$

$U_{bottom\_\max}$ = 3.6 m/s

Izbash found the following relation for the nominal diameter of the scour protection:

$$D_{n50} = \frac{U^2}{\psi \Delta C^2}$$

Where:

$$C = 18\log\left(\frac{12h}{k_r}\right)$$
$$k_r = 2D_{n50}$$
$$\psi = 0.03$$

From these calculations a D_{n50} of 0.1 m follows (50 /150mm graded rock). Because scour can induce instability of the toe and thereby instability of the whole armour layer, scour protection is designed in a conservative way. Instead of 50/150mm graded rock, 10 -60 kg graded rock is applied with a minimum layer thickness of 4 stones (=1.0m) and width of 25 meters. A filter layer between the sea bed and the falling apron must be applied. This layer consist of 40/100 mm gravel and has a thickness of 1.0 m.

5.4.5 Economic optimisation XBloc sea dike

The length profile of the XBloc sea dike can be optimised in an economic sense by varying

- XBloc dimensions per location
- canal dimensions per location
- crest height per location

Varying XBloc dimensions per location

By distinguishing three parts of the sea dike, three XBloc dimensions are calculated using the stability number recommended for design by Delta Marine Consultants:

$$\frac{H_s}{\Delta D_n} = 2.8 .$$

Figure 41 shows the design wave height for the three distinguished sections. In comparison with applying the largest XBloc over the whole area a cost reduction of €0.5 million is obtained. Because the total number of XBloc units is so great (≈ 30,000), it is assumed that moulds are specially produced for the Maasvlakte 2 project. Subsequently there are no extra costs for producing three types of moulds instead of producing only one type.

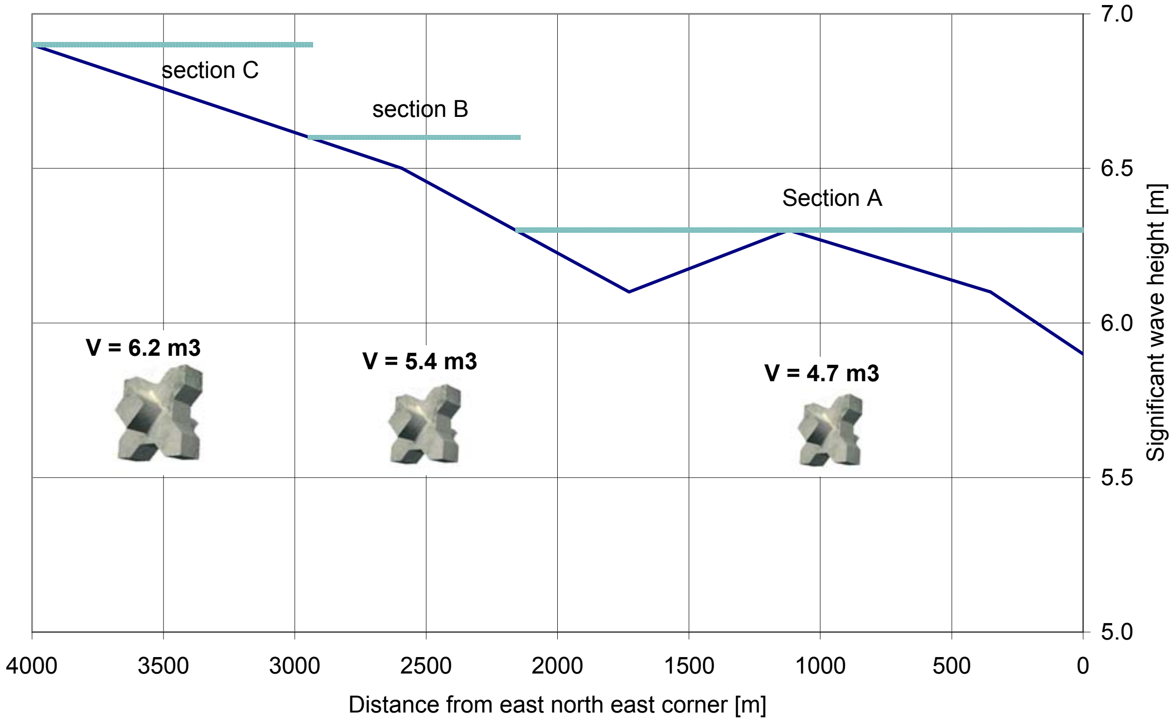


Figure 41 Three sections with varying XBloc dimensions

Canal optimisation

In theory the canal width in the middle of the sea dike needs to have a capacity large enough to collect the water from one overtopping wave. The end dimensions are designed to discharge all the water from the overtopping waves of the rest of the structure, which results in a canal width of 21 metres on either end side of the sea dike.

However the infrastructure behind the dike should be free of overtopping water during design conditions (1/10,000 per year). Under design conditions, a wind speed of 24.7 m/s could carry waves tens of metres through the air. Therefore it is save to have the canal as a wide buffer zone behind the crest at all locations.

In other words the canal is best constructed with a width of 21 metres along the whole structure.

5.5 Final Design

5.5.1 Cross-sections

Detailed cross-sections of the final design are shown in appendix AA.1.

5.5.2 Construction method

The land reclamation Maasvlakte 2 will be constructed in 2 phases as shown in Figure 3. The construction method for both phases is equal and can be divided in 3 sub phases:

Sub phase 1

Geotubes are being placed and meanwhile sand is pumped in the created reservoir with pipes and dumped by trailing suction hopper dredgers. The sand on the top part of the dike construction will be placed by trucks and cranes.

Sub phase 2

The quarry stone is placed both with land and water based equipment. For the rocks split barges and side stone dumping vessels can be used.

Sub phase 3

Placing of the XBloc armour units must be done very precisely, therefore it is best done with land base equipment. The maximum horizontal distance from the outer crest line to the end of the XBloc layer is approximately 35 meters. The use of a crane with a reach of 35 metres is for accuracy reasons preferred before a smaller crane on a pontoon. Placement of the crest elements and the concrete canal slabs is done simultaneously.

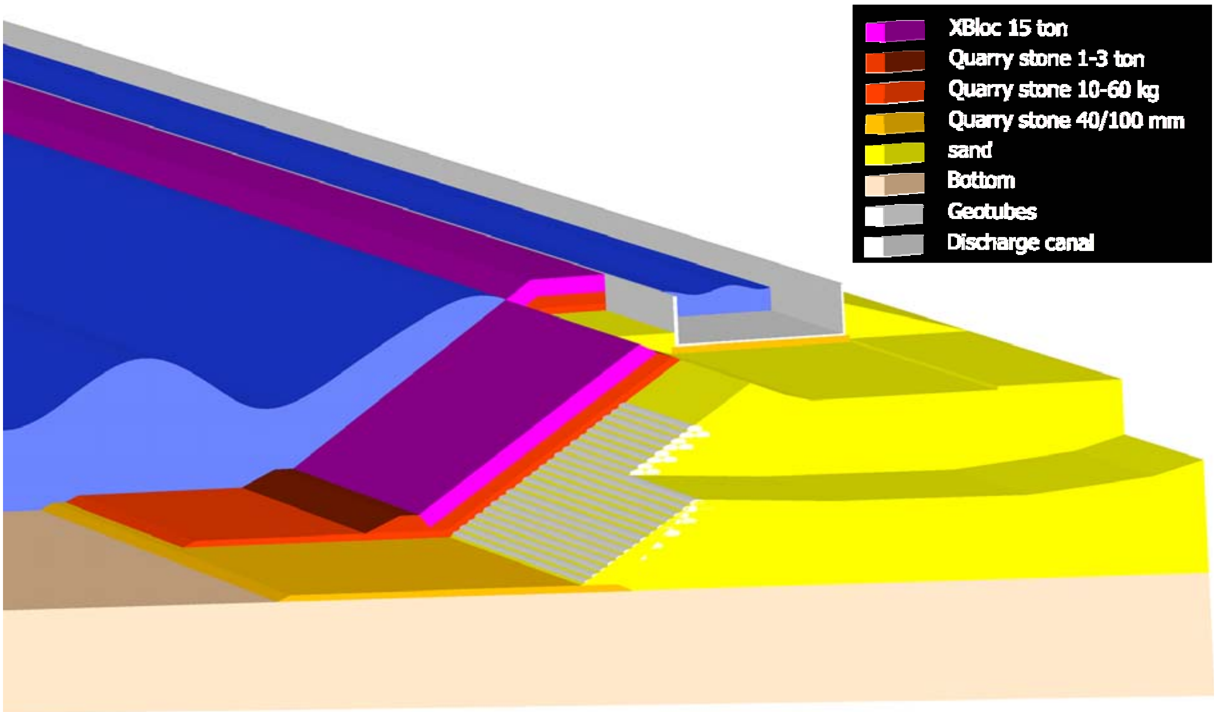


Figure 42 Sub phases of sea dike construction

5.5.3 Costs

Construction costs[12] of the low crested dike as shown in Figure 42 are €20.600 per running metre. Costs for 4500 metres of sea defence are approximately € 93 million.

The costs are based on the unit prices shown in Figure 32 and Table 15. This means no costs for taxes, engineer, insurance, profit, etc. are taken into account. For a more detailed cost calculation the reader is referred to appendix AA.2.

[12] Costs are based on unit prices from Figure 32 and Table 15.

5.6 Conclusions design phase

For the interlocking elements a comparison was made between the Accropode and the XBloc elements; this showed that the latter is most economical, mainly because of its less dense packing method.

By applying the concept of a low crested sea dike with an armour layer of XBloc elements, unit costs are reduced by 23%. No cost reductions are obtained for a sea dike with a quarry stone armour layer. The reason is that the asphalt top layer must be replaced by heavy, expensive rock to ensure the stability of the top part of the construction in the case of overtopping waves.

Based on the multi criteria analyses a canal construction is chosen that collects and discharges the water to both the eastern and western end of the sea dike. Main decision criteria are the low constructions costs and the simplicity of the structure. The canal dimensions are reduced by discharging the overtopping water to both ends instead of only one end.

Costs are not reduced by constructing carriageways inside the canal. The main reason is that industrial traffic delay by closure of carriageways must be prevented from happening at all times. A dike with a crest level of NAP+28.5m satisfies this demand but is very expensive.

In the proposal of allowing damage, the application of XBloc armour units will make the construction fail in a progressive way when design conditions are exceeded, due to the interlocking of the elements. Therefore, application of XBloc elements in combination with the method of allowing damage is not possible.

The non standard 3-7 ton graded armour layer leads to the lowest life cycle costs of a sea dike with a quarry stone armour layer. Costs are reduced by 20 % in comparison with a sea dike with a traditional designed armour layer (8-12 ton).

6 Conclusions and recommendations

6.1 Conclusions

The primary objective was to design an innovative, cost friendly sea defence. This objective is met by designing a low crested sea dike with discharge canal. The first sub objective was to find accurate hydraulic design conditions in the vicinity of Maasvlakte 2. Maximum hydraulic conditions that are exceeded with a probability of 1/10,000 per year will occur at the North West corner of Maasvlakte 2. The maximum parameters are: significant wave height, 6.9m; peak period, 13.8s; and water level, NAP+4.8m. The significant wave height decreases with 15% towards the eastern end of the structure.

The second sub objective was to determine the possibility of two cost reducing methods for the northern sea dike. To reduce costs firstly the feasibility of a low crested sea dike with discharge canal was studied, and secondly the feasibility of allowing a certain amount of damage to the structure by applying a relatively light, less expensive, armour grading.

For the interlocking elements a comparison was made between the Accropode and the XBloc elements; this showed that the latter is most economical, mainly because of its less dense packing method.

By applying the concept of a low crested sea dike with an armour layer of XBloc elements, unit costs are reduced by 20 %. No cost reductions are obtained for a sea dike with a quarry stone armour layer. The reason is that the asphalt top layer must be replaced by heavy, expensive rock to ensure the stability of the top part of the construction in the case of overtopping waves.

The concept of allowing a certain amount of damage to the structure does not lead to a cost reduction for a sea dike with XBloc armour units because this type of structure fails in a progressive way. However lifecycle costs of a sea dike with a traditional designed armour layer (8-12 ton), can be reduced by 20% by applying an armour layer with a W_{50} of 5 tons.

For the northern sea defence costs are optimised by applying a low crested sea dike with a crest level at NAP+14.0m and the use of an interlocking elements armour layer. Costs for construction and building materials are €21,000 per running meter. This is €11,300 less than the optimised sea dike with a quarry stone armour layer and a crest level at NAP+13.0m.

6.2 Recommendations

Good estimates of the combined statistics of wind, waves and water levels were found by using physical relations. For the purpose of this thesis, the results of the simplified formulas were used as the input of the SWAN model. To give insight in the accuracy of these results a comparison with results from a statistic approach could be made.

The construction of a low crested dike with an interlocking element armour layer with a crest level at NAP+14.0m is recommended.

Instead of varying the size of the XBloc armour units along the structure, high density concrete could be considered. A more detailed study must prove which method is more profitable.

The outer wall of the discharge canal is assumed to be 0.5m thick. Because wave forces act on this wall, it should be dimensioned in more detail.

There is only little known about the stability of geotubes and the forces on the geotextile during construction. Scale tests have to be performed to clear these uncertainties.

Model tests should be performed to prove that overtopping waves are collected in the canal and discharged to both ends of the construction

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8 Appendices

A. Theory and definitions

Table 19 Summary of used linear wave theory formulas

	Shallow water	Transitional water	Deep water
Relative depth	$\frac{d}{L} < \frac{1}{25}$	$\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	$\frac{d}{L} > \frac{1}{2}$
Wave celerity	$C = \frac{L}{T} = \sqrt{g \cdot d}$	$C = \frac{L}{T} = \frac{gT}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
Wavelength	$L = T\sqrt{g \cdot d} = C$	$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$
Horizontal particle velocity	$u = \frac{H}{2}\sqrt{\frac{g}{d}} \cos\theta$	$u = \frac{H}{2} \frac{gT}{L} \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \cos\theta$	$u = \frac{\pi H}{T} e^{\left(\frac{2\pi z}{L}\right)} \cos\theta$

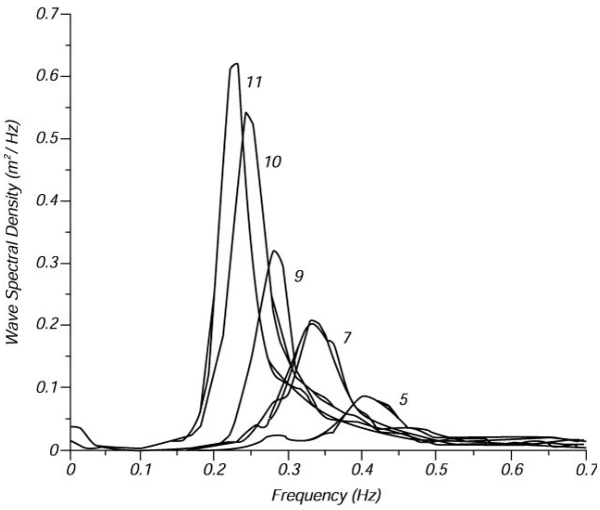


Figure 43 Wave spectra for varying wind conditions

B. Simulating Waves Near shore (SWAN)

B.1. Model

The model SWAN is a third-generation (phase-averaged), wave model for the simulation of waves in waters of deep, intermediate and finite depth. It is the successor of the stationary second-generation Hiswa model (Holthuijsen et al., 1989).
Although SWAN is a non-stationary model, it can work in the stationary mode. In this research SWAN version 4.31 is used.

The stationary assumption is considered acceptable for most coastal applications because the travel time of the waves from the seaward boundary to the coast is relatively small compared to the time scale of variations in incoming wave field, the wind or the tide.

The SWAN model was developed at Delft University of Technology, Delft (the Netherlands). WL | Delft Hydraulics has integrated the SWAN model in several models like Delft3D.

Basic equation

The SWAN model is based on the discrete spectral action balance equation. The Action density with respect to the energy density is given in the equation $N(\sigma,\theta)=\frac{E(\sigma,\theta)}{\sigma}$, where σ = relative frequency (as observed in a frame of reference moving with current velocity) and θ = wave direction (normal to wave crest of each spectral component).

The action density balance equation reads:

$$\frac{\delta}{\delta t}N+\frac{\delta}{\delta x}c_X N+\frac{\delta}{\delta y}c_Y N+\frac{\delta}{\delta \sigma}c_\sigma N+\frac{\delta}{\delta \theta}c_\theta N=\frac{S}{\sigma} \quad \text{(basic equation)}$$

The first term on the left-hand side is the <u>local rate of change of action density in time</u>. The second and third terms are the <u>propagation of action in geographical space</u>, with propagation velocities c_X and c_Y.
The fourth term represents the <u>shifting of the relative frequency</u> due to variations in the depth and currents. The last term on the left-hand side represents the <u>depth and current induced refraction</u>. The term on the right-hand side is a <u>source term for generation, dissipation and nonlinear wave-wave interactions</u>.

Boundary conditions

In SWAN both in geographical space and spectral space, the boundary conditions are fully absorbing

SWAN simulates the following physical phenomena:

- Wave propagation in time and space
- shoaling
- refraction due to current and depth
- frequency shifting due to currents and non stationary depth
- Wave generation by wind
- Nonlinear wave-wave interactions (both quadruplets and triads)
- Whitecapping,
- bottom friction

- depth-induced breaking
- Blocking of waves by current

Diffraction and reflections are not explicitly modelled in SWAN but diffraction effects can be simulated by applying directional spreading of the waves.

B.2. Description of input

SWAN was ran inside the Delft3D model suite. A SWAN simulation needs a number of input parameters and files; the primary input files are the command file, bathymetry file and the bathymetry grids. The input sections are given below and discussed briefly in this section.

- Flow
- Grids
- Time Frame and Water Level
- Boundaries
- Obstacles
- Physical parameters
- Numerical parameters
- Output

Flow
SWAN offers the opportunity to enter flow velocities en water levels obtained from previous flow runs. These velocities and water levels can have some effect on the wave height and direction and are directly read from the output files of Delft 3-D Flow. In the section "Time frame" a water level and velocity can also be defined but under the flow option the water level and the velocities can vary in space only. Further on in this paragraph will be explained why in this stage of the research these effects on the waves is neglected.

Grids
The bathymetry grid file is the file that describes the grid on which the bathymetry is based, i.e. the begin point of the grid, the size and the number of elements and the orientation. It gives the coordinates of every point in this grid. The bathymetry depth file gives for each location the depth with respect to NAP. These files form the basis on which the computations are run. The computations can in their turn be carried out on the same grid or on a different grid. When the computational grid is the same as the bottom grid the computations are the most accurate because no accuracy is lost in interpolations between the grid points of both grids.

Time frame and water level
The second important input is the Time Frame where the user can define at which point in time the calculations should be done, but in this case more important, the user can also enter hydrodynamic data in the form of: water level, X velocity and Y velocity. These parameters will be assumed the same in every point of the grid

Boundaries
The boundaries that can be imposed on the grid are very important for the calculations. Any inaccuracy on the boundaries proceeds into the area and can lead to inaccuracies in the area of interest. It is therefore important to define these boundaries as accurate and detailed as possible.

Obstacles
Obstacles blocking partial or completely the wave transmission can be entered, but in this stage of the research this function has not yet been used.

Physical Parameters
In Delft 3-D the physical parameters can be divided into constants, wind, processes and various. The process of wind growth in the area is rather important because in this case the area is quite large, the fetch can be up to 100 kilometres. The wind velocity and direction is the only physical parameter that is changed in this research. The other parameters were kept at constant values, which are as follows.

Constants

▪ Gravity	9.81 m/s^2
▪ Water density	1025 kg/m^3
▪ Minimum Depth	0.05 m
▪ Forces:	Wave energy dissipation rate

Processes

▪ Formulation	third generation
▪ Bottom friction	JONSWAP (Coefficient 0.067)
▪ Depth induced breaking	B&J model (α = 1, γ = 0.73)
▪ Non linear triad interactions	LTA (α = 0.1, β = 2.2)

Various

- Wind growth, White capping, Quadruplets, Refraction and Frequency shift activated

Numerical Parameters
The numerical parameters were kept at constant values like most of the physical parameters. The only exception is the number of iterations that was for most of the runs limited to five iterations to save time. A sensitivity analyses showed that the results converge when the number of iterations is set on 15. From now on this is the amount of iterations that will be used for the simulations. In the next phase of this research the amount of iterations will be taken as high as possible. The settings of the rest of the numerical parameters are shown in Figure 44.

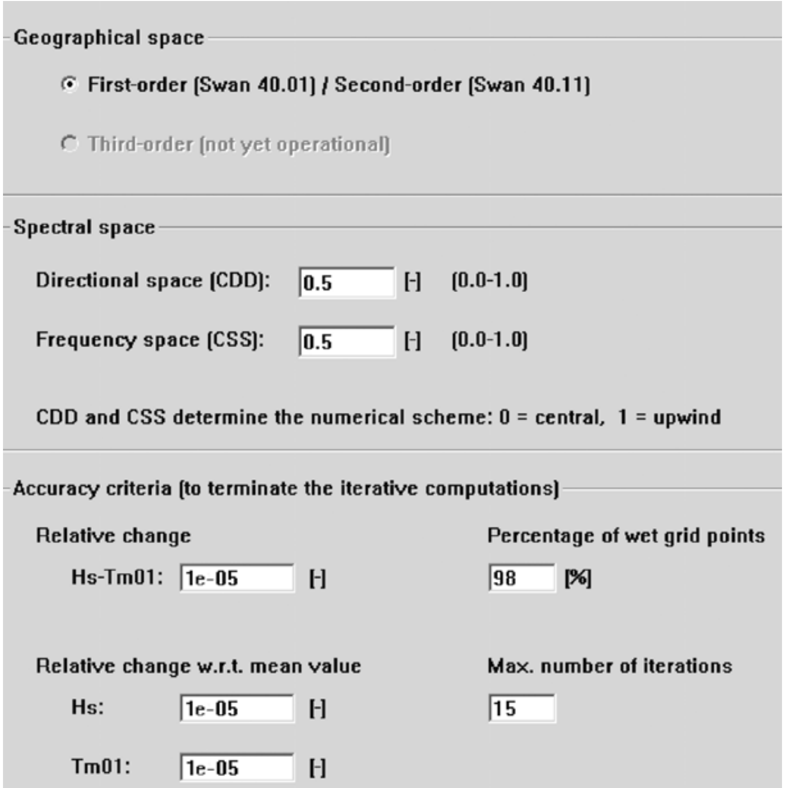
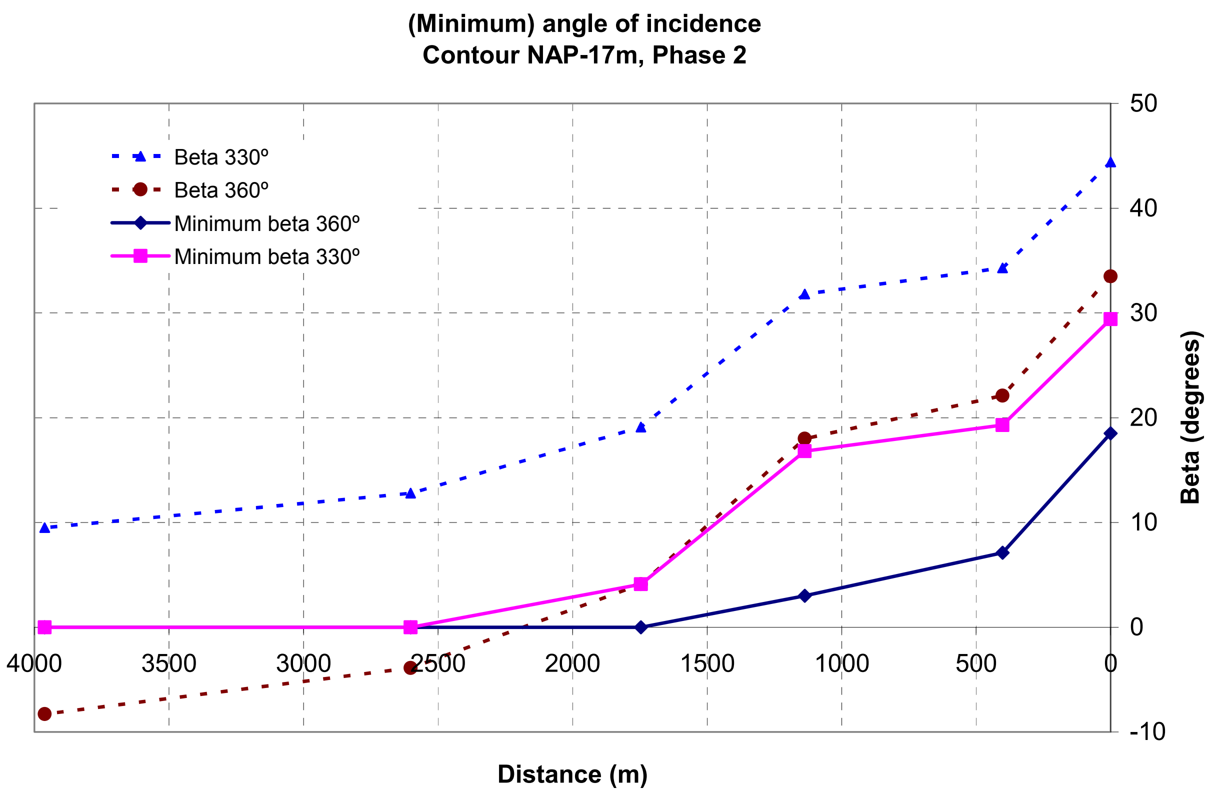


Figure 44 Numerical parameters

By setting the accuracy criteria to a value of 1e-05, one can determine the exact number of iterations by hand. Calculations showed that after 15 iterations the results are diverged to a constant value.

Output
A last input parameter in SWAN concerns the output. SWAN can show the results of the simulations graphically over a previously specified grid, this grid can be the same as the computational grid, but SWAN can also write the wave characteristics into a table for a number of user specified control points. In these points the effect of the change of one parameter can be compared numerically, giving a good idea of the deviations in terms of percentages.

C. Angles of incidence for 10^{-4} conditions (representative for other return frequencies)



D. Reduction of wind speed - literature

In this paragraph the directional wind speed return levels are found by extensive examining of relevant literature. First a wind speed reduction will be done by applying the results of Verkaik, Smits and Ettema [ref 24], subsequently by using results of De Valk and Melger [ref 23] the wind speed can be reduced even more. Also an explanation is given for the use of data from measuring station Europlatform only.

Statistical wind speed reduction by Verkaik et al

Earlier studies used wind data from Wieringa and Rijkoort [ref 27] to obtain return levels for different return periods. Verkaik et al concluded that the method developed by Rijkoort, the so-called Rijkoort-Weibull model (RW-model), contained some severe weaknesses, concerning the persistence, tail correction, and the effect of low wind speeds on the extreme return levels.

In his analyses, Verkaik used the peak-over-threshold values (POT-values) instead of analyzing all hourly wind speeds. In this way the non-interesting, low wind speeds are excluded from the analysis. A comparison between the RW-model and the POT-model shows that, averaged over the Netherlands, the 10,000-year return levels of the POT-model are about 10% lower (≈32 m/s) than those of the RW-model (≈35 m/s).

Since the heaviest wave attack is expected from a Western direction and the Europlatform is located due West from the Maasvlakte, wind data from Europlatform is used on the total model area. No spatial interpolation methods are necessary since the large-scale roughness is identical in the complete area of interest.

Statistical wind speed reduction by De Valk and Melger [ref 23]

In their study Jacobse and Groos [ref 13] assumed that offshore significant wave heights, wave periods, wind speeds and near shore water levels exceed their design values simultaneously. By doing so very conservative values of near shore wave heights and periods were found. De Valk and Melger found a further reduction of wind speed return levels by another statistical analysis. This reduction is based on expected wind speeds at given wave periods.

Depending on the different methods, different wind speed reductions are obtained. In the first and second method, wind speeds are calculated from omni-directional and wind direction dependent distribution functions. The results are based respectively on RIKZ/KNMI marginal exceedance frequencies and on estimated wind direction dependence. The third method calculates wind speeds from omni-directional and wave direction dependent distribution functions. In the latter method, the results are based on estimated wave direction dependence.

Table 20 Wind speed reduction based on different methods [ref 13]

Results based on:	RIKZ/KNMI wind marginal exceedance frequencies	estimated wind directional	estimated wave directional
Wave period [s]	Reduction [m/s]	Reduction [m/s]	Reduction [m/s]
7	0.8	4	8.4
8	2.7	3.3	5.6
9	5.3	3.3	5.2
10	7.7	3.5	5.1
11	9.8	3.7	5.1
12	12.1	3.9	5.1

In Table 20 the wind speed reductions obtained by the different methods are shown. Although the wind speed reduction differs per method, its values are in the same order of magnitude. Since the smallest peak period used for the extreme value simulations in this research is 11.44 seconds [ref 16] and the goal is to come up with a solid design, a relative conservative wind speed reduction of 3.8 m/s is chosen.

Discussion wind reduction

Table 21 shows the wind speed return levels and reductions at Europlatform for different return periods and directions. Verkaik et al noted that the reduction of 10% is an average for all measuring stations. He also stated that large differences between stations can be found but did not quantify the error.

Tables on the HYDRA website [ref 29] show that by using the data from Europlatform over the total area instead of combining data from Europlatform, IJmuiden and Goeree, an error of plus and minus 1 m/s is made.

According to the research done by De Valk and Melger, it can only be concluded that the reductions are all in the same order of magnitude and substantial.

Table 21 Wind speed return levels with a return period of 10.000 years

Wind speeds	Wieringa and Rijkoort	After 10% reduction by Verkaik et al	After reduction of 3.8 m/s by De Valk and Melger
Wind direction [ºN]	10^{-4} conditions [m/s]	10^{-4} conditions [m/s]	Combined wind and wave conditions 10^{-4} [m/s]
210	35.0	31.5	27.7
240	38.0	34.2	30.4
270	42.0	37.8	34.0
300	38.0	34.2	30.4
330	34.0	30.6	**26.8**
360	28.0	25.2	21.4

E. Wind speed - stationary runs with SWAN

The relation used for the reduction of the wind speed by Demirbilek, is based on a duration-limited situation. When this wind speed is used in the stationary SWAN model, the wind conditions are exaggerated, because a stationary model assumes unlimited time. The extra heavy conditions lead to initial wave growth at the boundary of the model, which lead to unrealistic high wave conditions in the area of interest.

To solve this problem, a lower wind speed has to be found. As can be seen in I.1 to I.5, too small wind speeds cause a second peak in the wave spectrum. This second peak is the result of the low wind speed, which creates waves with a higher frequency than the ones imposed at the model boundary.

To make the best fit for the wind and wave conditions, the wind condition that will be imposed on the model needs to have a smooth spectrum with only one peak and must have as little initial wave growth as possible. The wind speeds that meet those demands are listed in the table below. The results in section 3.7 are taken from these runs.

Table 22 wind speeds that meet the criteria of wind growth and wave spectra

Exceedance frequency	*Wind speed imposed on the model (m/s) ($U_{10\text{-}model}$)*
10^{0}	13.0
10^{-1}	16.0
10^{-2}	17.5
10^{-3}	19.0
10^{-4}	20.0

F. Description of model adjustments and sensitivity analyses of input parameters

F.1. Approach

All the adjustments and tests that were carried out to set the parameters to their optimum values are described in the following section. Appendix F. gives an overview of the important runs. The runs use several conditions each with a specific direction and originating from one of the measuring points. The four different conditions that are used are shown in Table 23.

Table 23 Four conditions used for refining the model

Condition	*Surge level*	*Wave height*	*Wave period (T_P)*	*Direction*
North_EUR	3.81 m + NAP	7.55 m	12.5 s	0º
North_YM6	3.81 m + NAP	8.14 m	14.6 s	0º
South_EUR	0 m + NAP	7.2 m	11.5 s	210º
West_EUR	0 m + NAP	8.1 m	12.3 s	300º

Besides these directional conditions a few other conditions were used. For the first few runs with a Northern direction a condition similar to the West_EUR was used with a wave height of 8.2 m and a period of 13.6 with a direction of 0º, this condition will be named North_0m.
In total eight different sets of runs were executed changing the following parameters.

1. The first runs were to check and improve the grids. The conditions that were used were South_EUR, West_EUR and the condition described in the above.
2. The second set was run to check the effect of the wind on the results. Wind conditions between 38 m/s and 0 m/s. The wave conditions were either West_EUR or zero.
3. In the third set the boundaries were investigated for sensitivity when described in sections. This was done using the North_EUR condition and shoaling the boundary sections.
4. A wind file was created describing in every point of the grid a wind velocity between 28 and 0 m/s. The wave conditions were North_EUR for the Northwest boundary and North_YM6 for the Northeast boundary. The results are not reliable because of some mistakes in the wind file but it is included in F.2 for completeness.
5. In the previously used bottom files the sloping bathymetry just in front of the second Maasvlakte was not entered yet. Investigated was what the effect of this changed bathymetry was under both the South_EUR condition and the North_EUR condition.
6. A simple comparison was made using North_EUR on the Northwest and North_east boundary and using North_YM6 also on the Northwest and North_east boundary.
7. A further simplification on the boundary conditions can be achieved by shifting the EUR conditions and the YM6 conditions towards the ends of the Northwest boundary. This is done for both the South_EUR condition and the North_YM6 condition.
8. Finally the effect of scour holes in front of the Maasvlakte on the waves is investigated. To do this new control points had to be placed just behind the scour holes. These runs were done using the North_EUR and North_YM6 for the Northwest respectively the Northeast boundary and for a new case with waves coming from 330º. Wave height is 8.2 m, period is 13.0 s and the water level is 4.87 m above NAP.

The results and the differences in the results that followed from these adjustments are given in Appendix F.2. In this table there is a reference case defined for each of the 8 sets of test runs and the adjustments and deviations within this set is described.

F.2. Adjustments and resulting changes in output parameters

Series name	Reference case		
	Grids and Bathymetry	Boundaries	Wind
Grids	GI100 GD40	North_0m West_EUR South_EUR	28 m/s 38m/s 28 m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Run_04	**North_0m**	AI100 D40	2 %
Det and Int klein 210 gr	**South_EUR**	AI100 D40	2 %
DET and Int klein 300gr	**West_EUR**	AI100 e D40	3 %

Series name	Reference case (Run_05)		
	Grids and Bathymetry	Boundaries	Wind
Wind&waves	AI100 D40	West_EUR	38m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Run_06		Only wind 38 m/s	-16 %
Run_07	**West_EUR**	No wind	- 36 %
Run_08		Medium wind 20 m/s	- 21 %

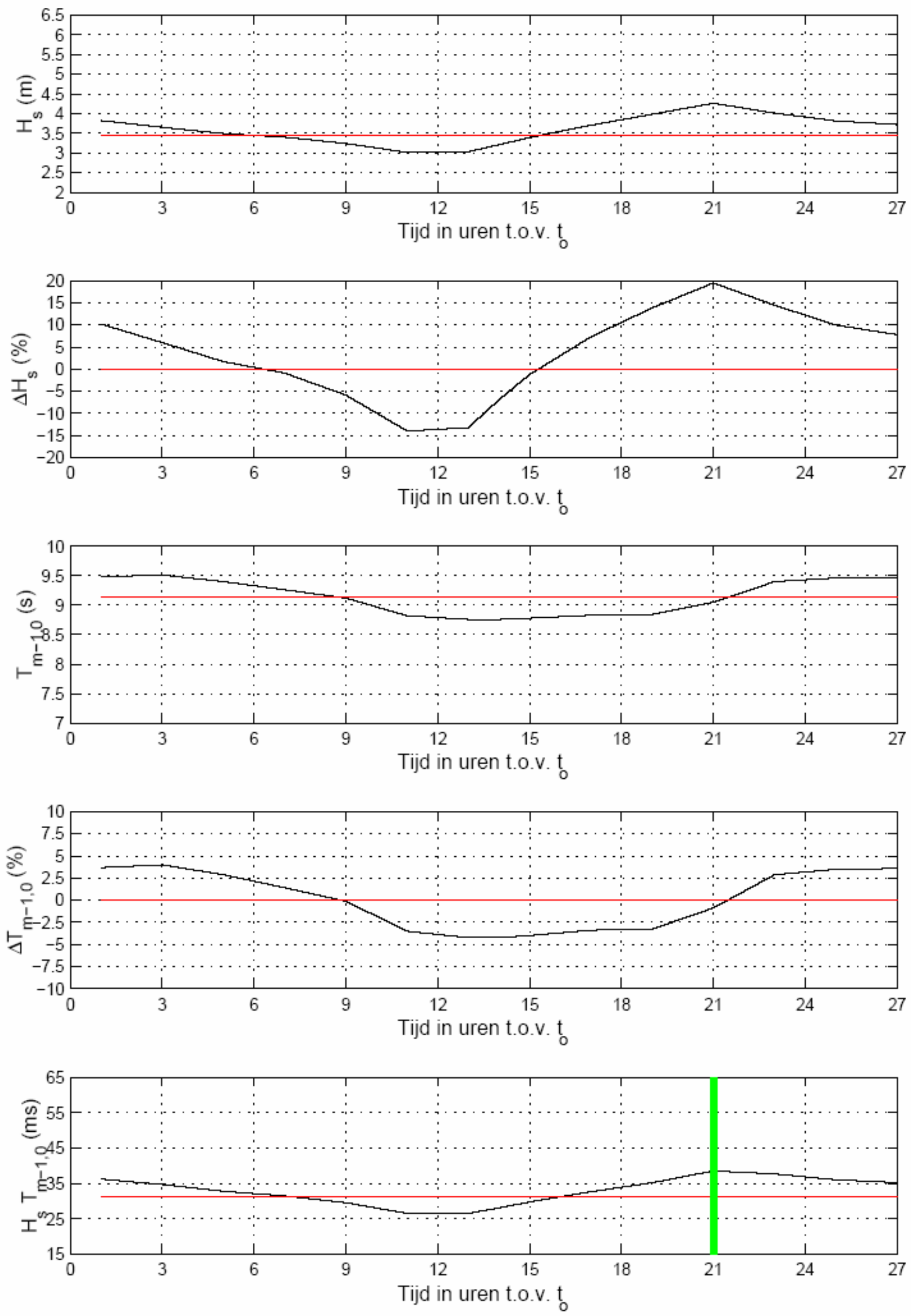
Series name	Reference case (Run_09)		
	Grids and Bathymetry	Boundaries	Wind
Sections	AI100 D40	NW&NE: North_EUR	28m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Run_10		NE in 10 sections	< 1 %
Run_11	**North_EUR**	NE in 2 sections	< 1 %
Run_12		NE sections Counter clockwise	12 % (counterclockwise is the wrong direction)
1 op 10000 NWshoaled en NO	**North_EUR**	10 sections NW	< 1 %
1 op 10000 NWshoaled en NO 7.55		10 new sections NW	- 1 %

Series name	Reference case (Run_09)		
	Grids and Bathymetry	Boundaries	Wind
Windfile	AI100 D40	NW: North_EUR NE: North_YM6	28m/s
Runs	Condition	Changed Parameter	Resulting difference in %
wind file	As reference	wind file	- 40 %

Series name	Reference case (Run_09)		
MV2 in bottom	Grids and Bathymetry	Boundaries	Wind

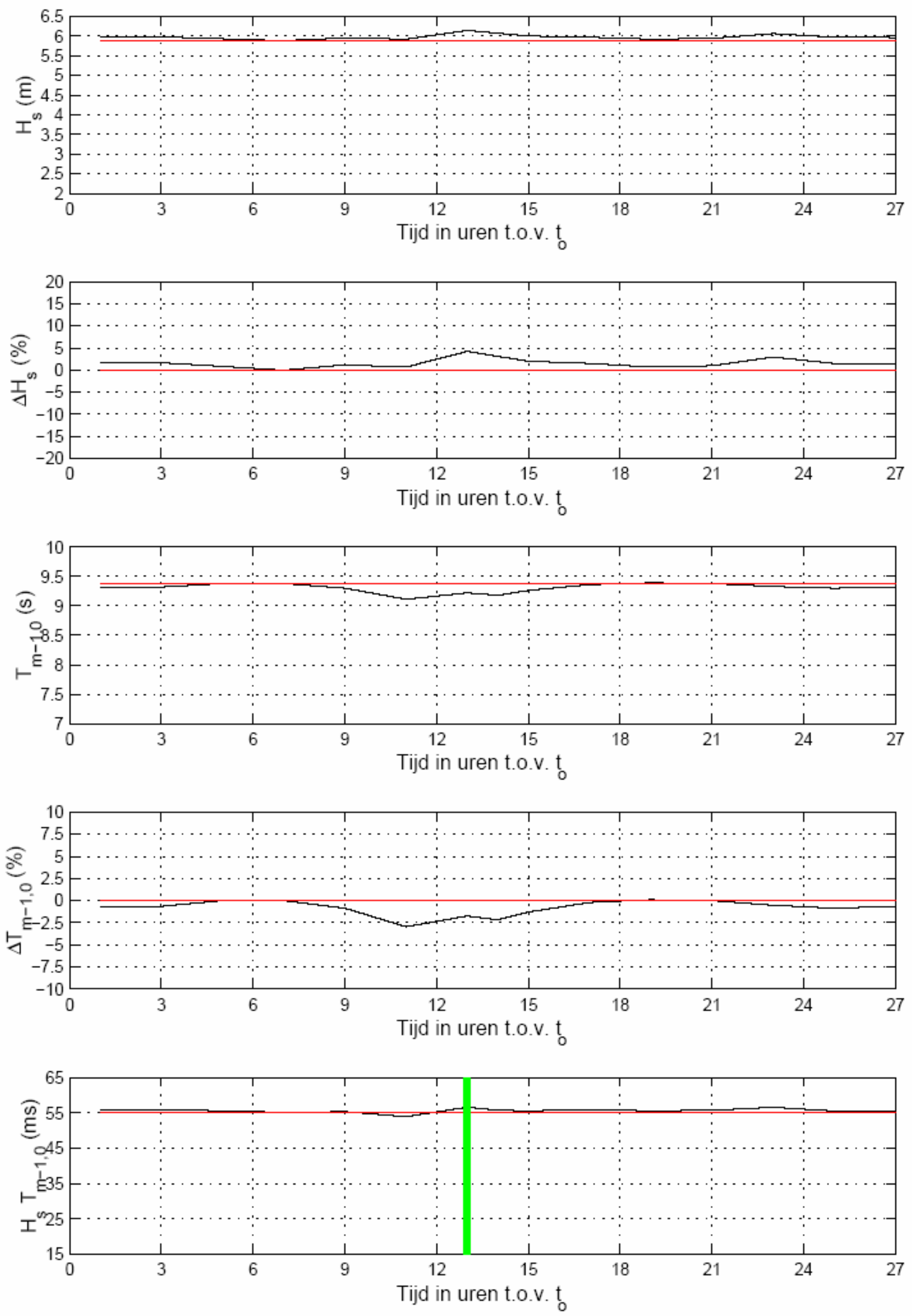
	AI100 D40	NW: North_EUR NE: North_YM6	28m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Run newMV2 360gr	**North_EUR**	MV2 in det grid	10 %
Run_20		MV2 in det and int grid	10 %
Run_19	**South_EUR**	MV2 in det grid	10 %
Run newMV2 in det and int		MV2 in det and int grid	10 %
Series name	Reference case (Run_09)		
All EUR and All YM6	Grids and Bathymetry	Boundaries	Wind
	AI100 D40	NW: North_EUR NE: North_YM6	28m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Run_15		NE&NW all IJmuiden	6 %
Run_17		Both bound in sections	5%
Series name	Reference case (Run_09)		
MV2 hoeken	Grids and Bathymetry	Boundaries	Wind
	AI100 D40	NW: North_EUR& NE: North_YM6 SW: South_EUR	28m/s 35 m/s
Runs	Condition	Changed Parameter	Resulting difference in %
RunMV2hoeken EUR&YMW 210gr	Southwest	NW 2sections	No reference
RunMV2 Hoeken EUR&YMW 360gr	North_YM6	NW 2sections	5 %
	Reference case (Run_09)		
Scour	Grids and Bathymetry	Boundaries	Wind
	AI100 D40	NW: North_EUR& NE: North_YM6 7.84 8.2 13 330gr NW	28m/s 34 m/s
Runs	Condition	Changed Parameter	Resulting difference in %
Scour 360gr	As reference	With Scour	- 7 % tot + 2 %
Scour 330gr	7.84 8.2 13 330gr NW 34 m/s	With Scour	- 4 % tot + 2 %

G. Influence of flow on waves, 225º section



Tijdvariatie golfhoogte, golfperiode en belastingfunctie Windrichting 225 º voor uitvoerpunt 1 Tijdstip van maximale belasting is aangegeven met grijze streep		
Onderzoek onzekerheid extreme golfbelastingen A901	Alkyon	Fig. 2.14.1

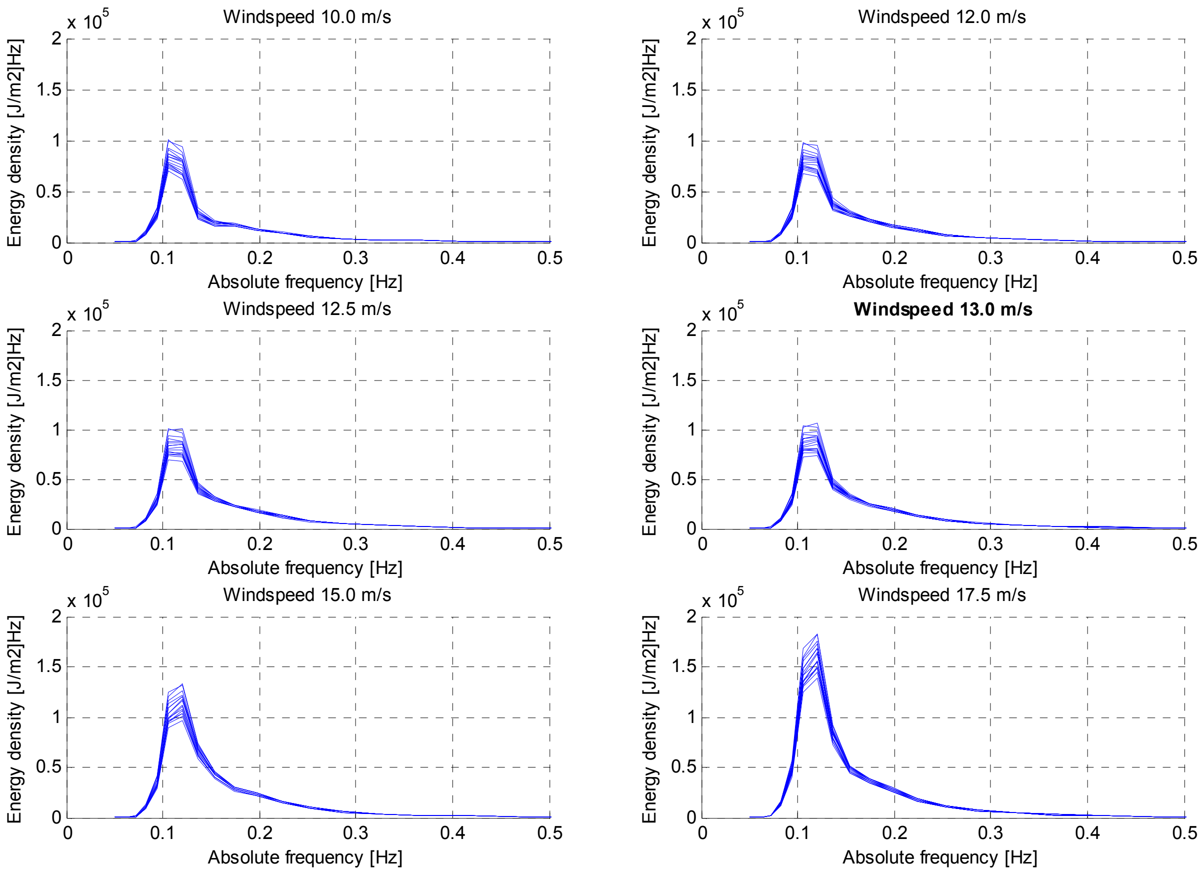
H. Influence of flow on waves, 315º section



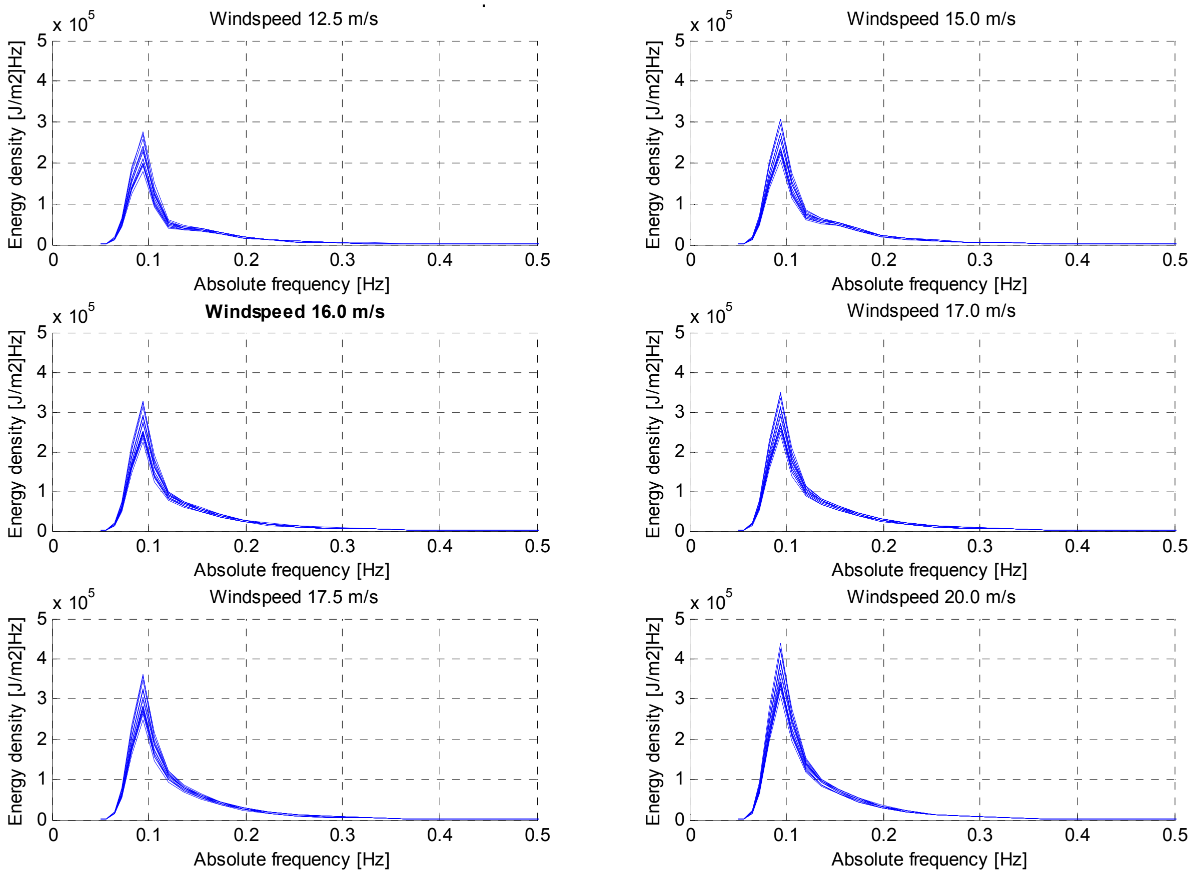
Tijdvariatie golfhoogte, golfperiode en belastingfunctie Windrichting 315 º voor uitvoerpunt 4 Tijdstip van maximale belasting is aangegeven met grijze streep		
Onderzoek onzekerheid extreme golfbelastingen A901	Alkyon	Fig. 2.15.4

I. Influence wind speed on output

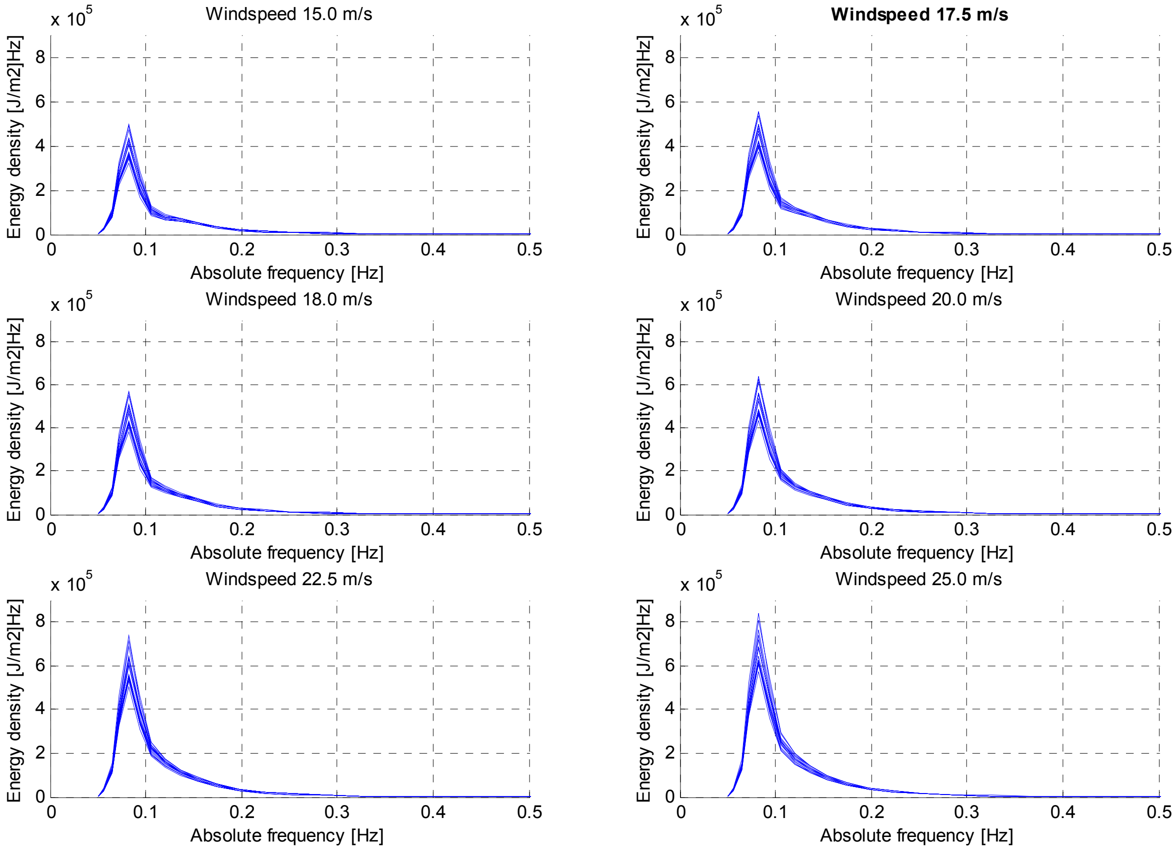
I.1. Wave spectra for varying wind speeds, 1/1 per year



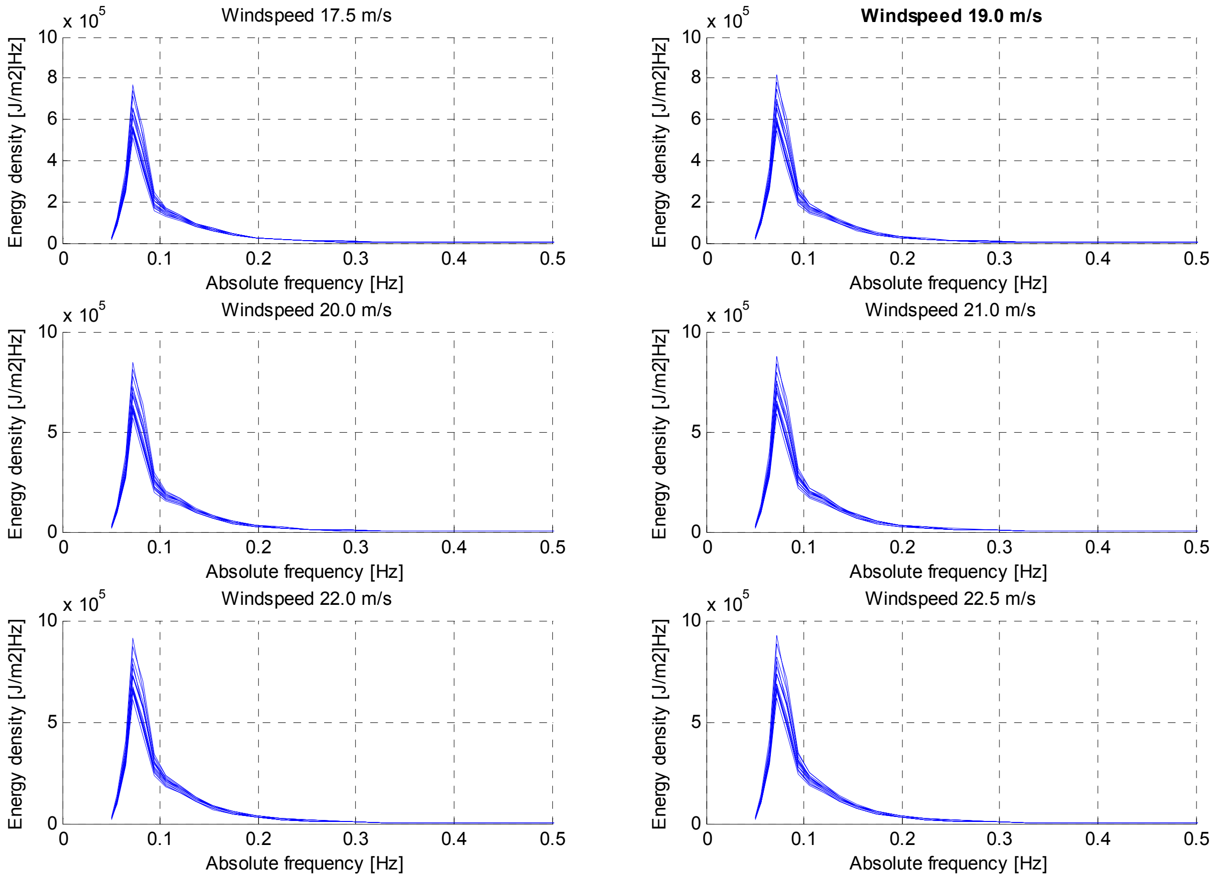
I.2. *Wave spectra for varying wind speeds, 1/10 per year*



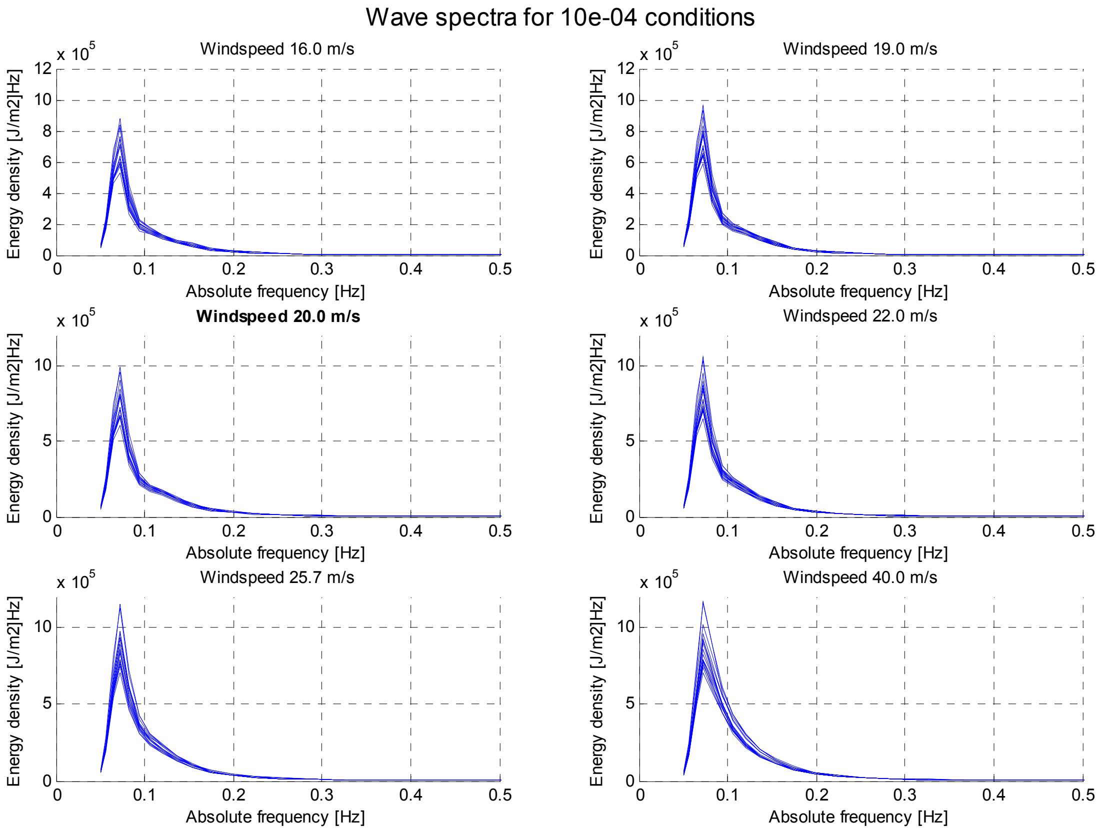
I.3. Wave spectra for varying wind speeds, 1/100 per year



I.4. Wave spectra for varying wind speeds, 1/1,000 per year

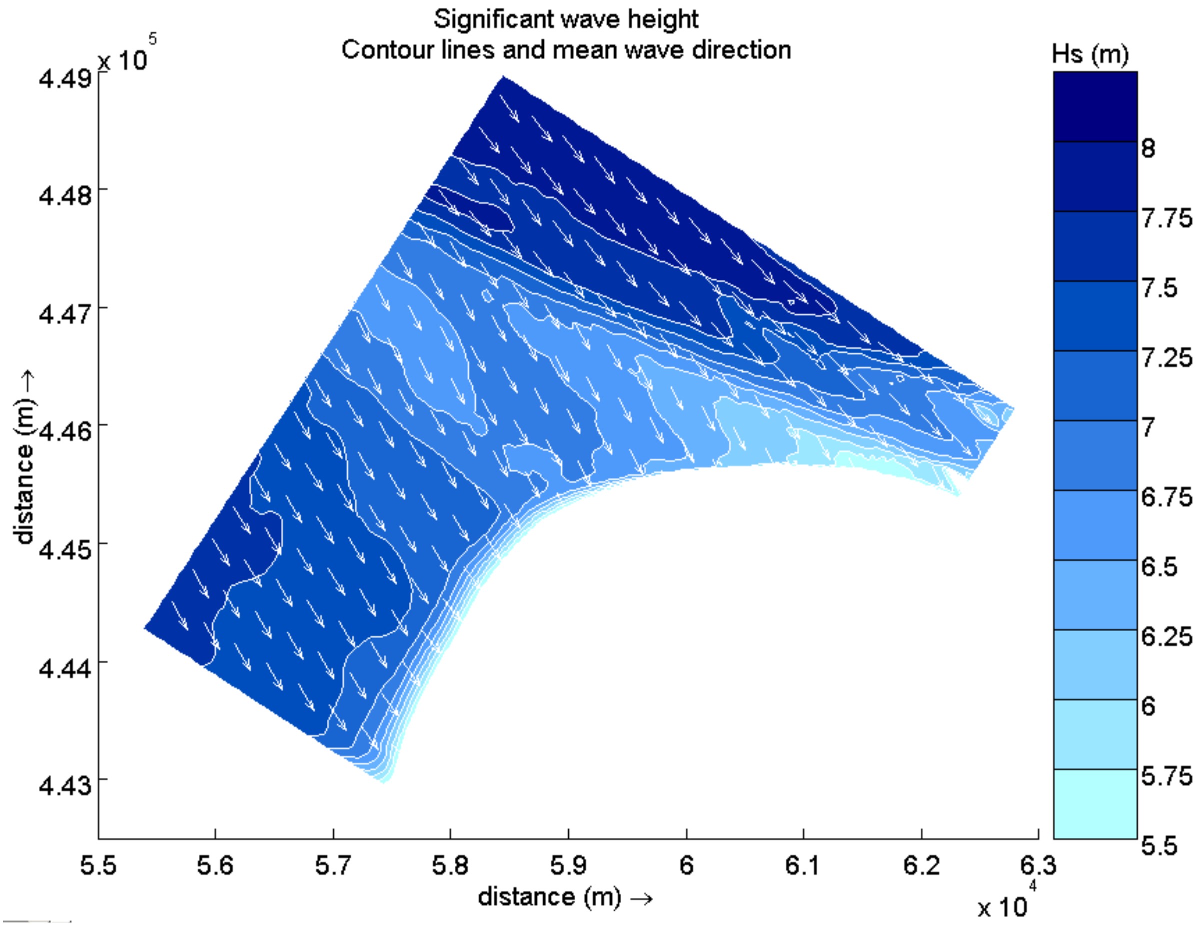


I.5. Wave spectra for varying wind speeds, 1/10,000 per year

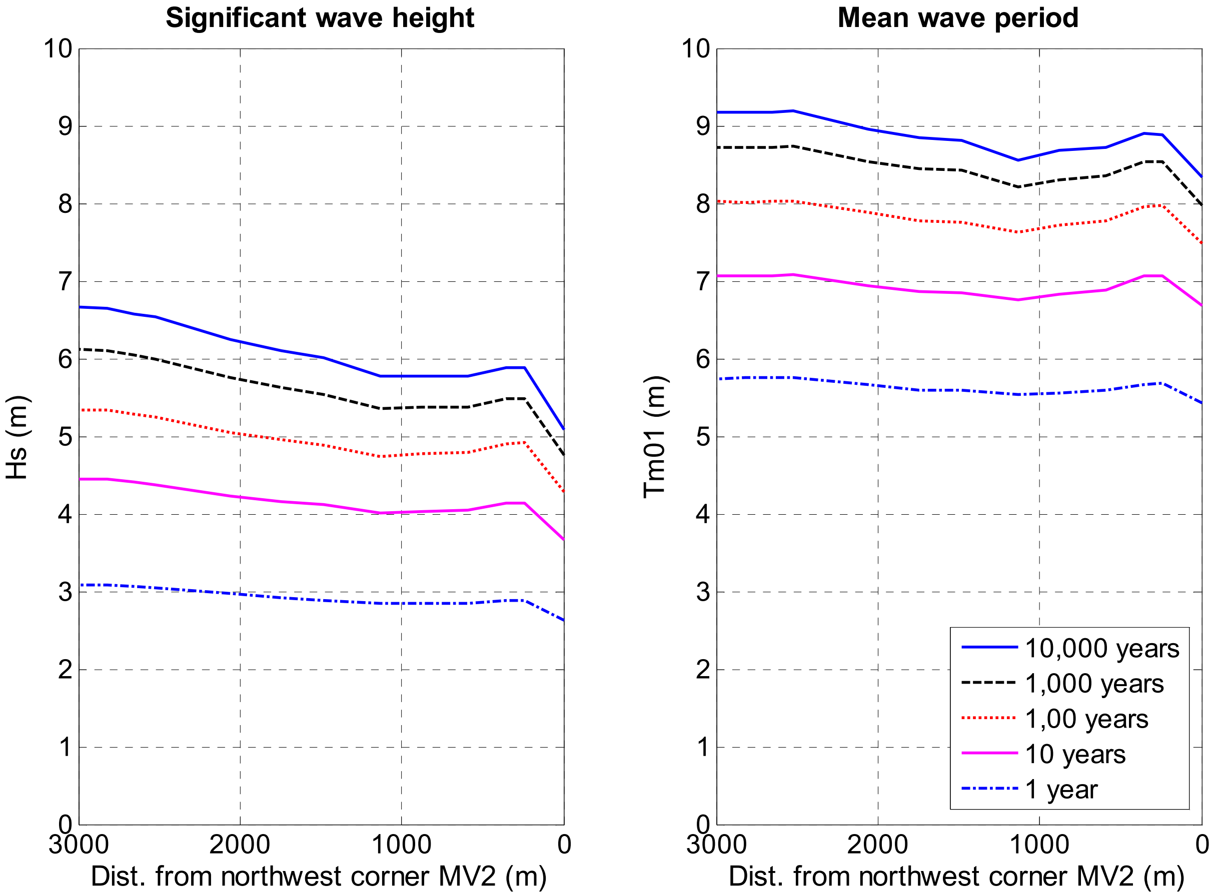


J. SWAN results

J.1. Wave field 10,000 year return period, 330º, MV2 phase 1



J.2. Wave conditions along the NAP-17m contour line at the toe of the shore protection, Phase 1



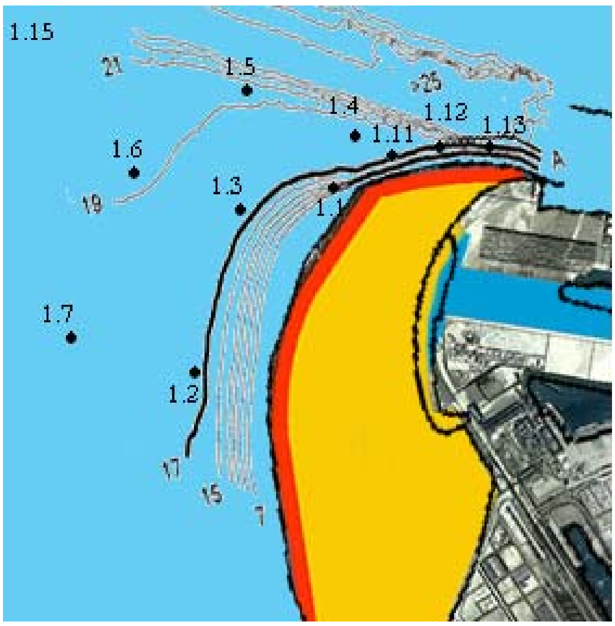


Figure 45 The white line indicates the locations with a bottom level of NAP-17.0m

K. Literature - Bouwstenen "Terrein, Zeewering en Golfbreker" voor Maasvlakte 2

K.1. Introduction

Via a thorough investigation Eversdijk et al [ref 7] came up with a few well founded design alternatives of the northern and south-western shore protection. In this appendix only the used alternatives, boundary conditions and basic assumptions for the northern side are mentioned. Eversdijk et al made some clear recommendations on what alternatives should be considered in the next design phase.

Below the main groups of coastal sea defences, which are taken into account, are listed.

- Artificial dune
- Artificial dune in combination with hard construction
- Sea dike
- Caisson
- Block wall
- Retaining wall

From every main group mentioned in the beginning of this section, one variant is chosen. In section K.3 the most important motives for these decisions are stated and the cross section parameters are given. Further more the costs will be discussed. First the program of demands as used by Eversdijk et al is given.

K.2. Requirements

General requirements

- The life span of the construction is 100 years;
- Wave and water level conditions with a return period of 100 years must be withstanded by the construction;
- The construction protects the land behind against inconvenience by waves, inundation because of high water levels and erosion by waves and flow;

Hydraulic design conditions

- High sea level (1/100 per year): NAP +3.70 m;
- High sea level (1/1000 per year): NAP +4.25 m;
- Low sea level (1/100 per year): NAP -1.90 m;
- Sea level rise (per 100 year): +0.50 m;
- Deep water wave height (1/100 per year): H_{s0}=7.25 m;
- Wave height with low sea level (1/100 per year): H_s=3.00 m;
- Peak period (1/100 per year): T_p=11.5 s;
- Tidal flow long shore (depth averaged): v_{gem}=1.5 m/s.

Basic assumptions

- Grain diameter (D_{50}) 250μm;
- Costs are converted from the price level of 1997 to 2005, assuming an average. An accuracy of plus and minus 30% is strived for. V.A.T., engineering costs, environment research costs are excluded;
- Construction costs are calculated for coastal section of 4 km;

- For the cross section designs a bottom level of NAP-15.0m is assumed;

K.3. Alternatives

Artificial dune

For the area above NAP-5.0m a cross section as found in the 'Nieuw Holland' research is used. The slopes below this level vary between 1:75 and 1:150 (Figure 46).

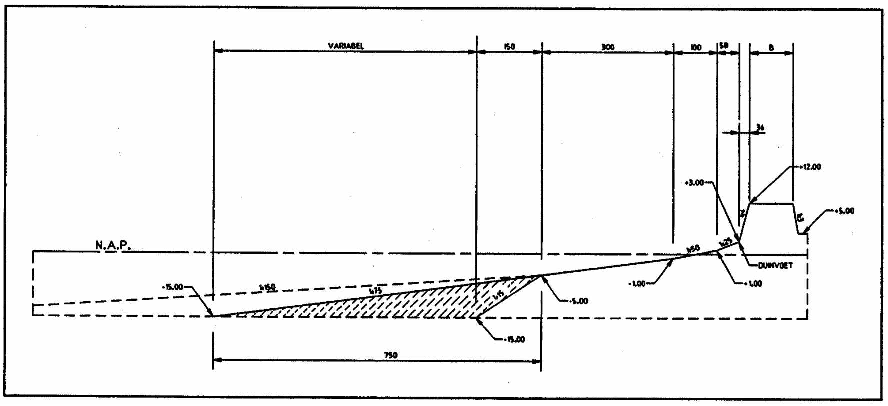


Figure 46 Basic design of artificial dune

Eversdijk et al used the software DUINAF to calculate eroding dunes at design storm conditions. Great uncertainties exist concerning the equilibrium profile and the expected amount of erosion. For this reason a positive as well as a negative scenario is given.

The optimistic scenario consists of a 1:75 slope beneath the NAP-5.0 level and sand suppletion of 0.5 million m^3 per year. In 100 years the maintenance quantity is 17 million m^3. Including a construction sum of 63 million m^3, the total sand costs add up to € 99 million.

A slop of 1:150 is assumed for the pessimistic scenario. Every third year 1.0 million m^3 has to be supplemented. For maintenance and construction a sum of 94 million m^3 is needed. The total costs are € 138 million.

For comparison reasons the average costs of both the optimistic and pessimistic scenarios, being € 119 million, is used from now on.

Artificial dune – Zuiderdam alternative

This alternative resembles the solutions as constructed at the current Maasvlakte. The dam will be constructed on the present bottom with a quarry stone core and an armour layer of concrete elements. From the toe of the construction towards MSL a 1:15 slope is formed. Because of the protection by the dam no beach nourishments are necessary.

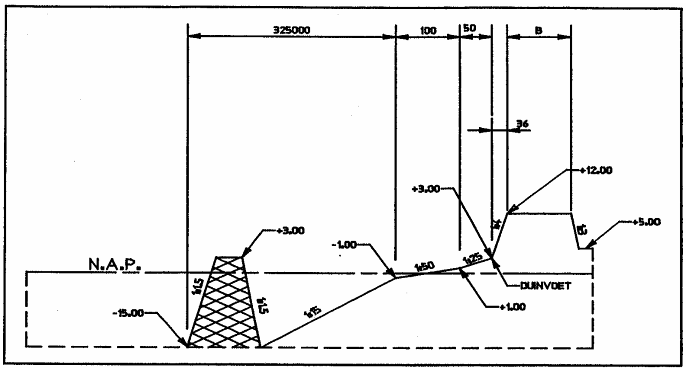


Figure 47 Artificial dune – Zuiderdam alternative, rather expensive

The costs for construction of the dam and dredged sand are respectively € 215 million en € 34 million. Total costs for the Zuiderdam-alternative add up to € 249 million.

Artificial dune with hydraulic fill dams

The alternative as discussed above is quite expensive. Large reductions on the costs can be realized by using hydraulic fill dams. The crest of the sand body is located at a depth of 5 meters below NAP. Since little wave influence is expected at this level, light bottom protection will be sufficient.

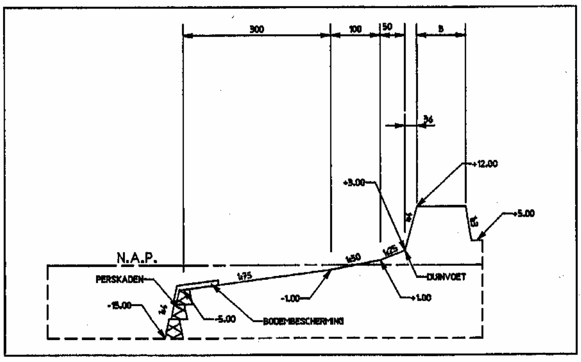


Figure 48 Artificial dune with hydraulic fill dams

As the artificial dune, the costs are calculated for an optimistic and a pessimistic scenario. Because loss of sand at the seaside is expected, an extra 0.2 million m^3 per suppletion is added in comparison with the artificial dune alternative. For the optimistic and pessimistic scenario a maintenance of respectively 0.7 million and 1.2 million m^3 every third year is presumed.

Total costs for construction and maintenance including hydraulic fill dams are € 134 million or € 150 million depending on the scenario. For comparison reasons the average costs of both the optimistic and pessimistic scenarios, being € 143 million, is used from now on.

Artificial dune – quarry stone breakwater

The rock quantity used in the 'Zuiderdam alternative' is quite large. Reductions on the costs can be realized be reducing the quantity of rock in the construction. This can be accomplished by building the dam on top of a sand body. Since erosion is expected at the toes of the breakwater, bottom protection over a wide area is needed.

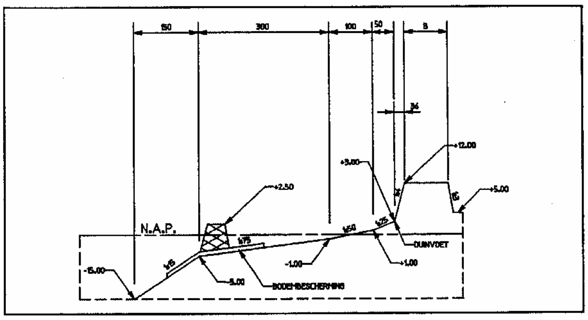


Figure 49 Artificial dune – quarry stone breakwater

Great uncertainties with respect to the reduction of structural erosion by long shore transport by the construction of a parallel exist. A reduction with respect to an unprotected artificial dune of 50% is assumed. Because of the uncertainties mentioned above, the assumption was made that an extra 0.2 million m^3 per three year has to be nourished.

For both the optimistic and pessimistic scenario maintenance costs are calculated to be respectively € 111 million and € 120 million. For comparison reasons the average costs of both the optimistic and pessimistic scenarios, being € 116 million, is used from now on.

Traditional dike

In Figure 50 initial concepts of a traditional and overtopping sea-dike are given. A more detailed design of the traditional sea-dike is made due to the argumentation below:

- More knowledge and experience is present with this type of dike;
- A relatively high dike will make people feel more safe;
- No discharge problems with the overtopping water exists;
- Maintenance costs are lower for a traditional sea-dike especially on the crest and the inner slope.

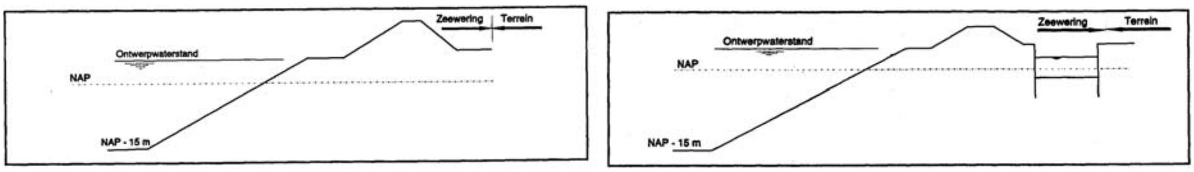


Figure 50 Traditional dike (left) and an overtopping dike (right)

The application of quarry-stone is argumented as follows: usage of placed concrete elements is not possible because of local wave climate. Furthermore it is assumed that dumped concrete elements on the chosen slope (1:4) is less profitable than application of quarry-stone.

The most important parameters of the initial sea-dike design are given below:

- Design crest level NAP + 9.05m;
- Outside slope 1:4;
- Berm level NAP + 3.70 m, berm width 10 m, berm slope 1:15;
- Inside slope 1:3;
- Armour layer quarry stone 3-6 tons;
- Hydraulic fill dams consist of stony material.

This design is made reckoning relatively large damage possibility. Therefore maintenance costs are calculated to be 5% of the construction costs. The total cost for a traditional sea dike with a length of 4 kilometres is estimated to be € 168 million.

Caisson

Different caisson concepts are shown in Figure 51. Since the rectangular caisson is thought to be the most simple and less expensive one this variant is further drawn up. A granular filter is placed under the construction.

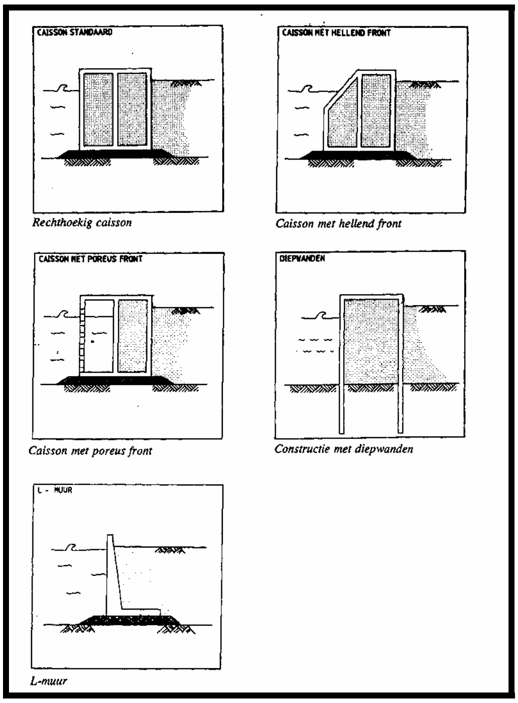


Figure 51 Five common caisson concepts

The most important design parameters are given below. In Figure 52 the resulting design is shown.

- Height NAP + 15.0 m (standing wave);
- Filter and foundation layer 4 m (NAP – 15.0 m to NAP – 11.0 m);
- Caisson height 16 m (NAP – 11.0 m to NAP + 5.0m);
- Crest construction 10 m (NAP + 5.0 m tot NAP + 15.0 m);
- Caisson width 22.5 m;
- Caisson length 100 m;
- Sea side bed protection 6-10 ton, width ca. 40 m;

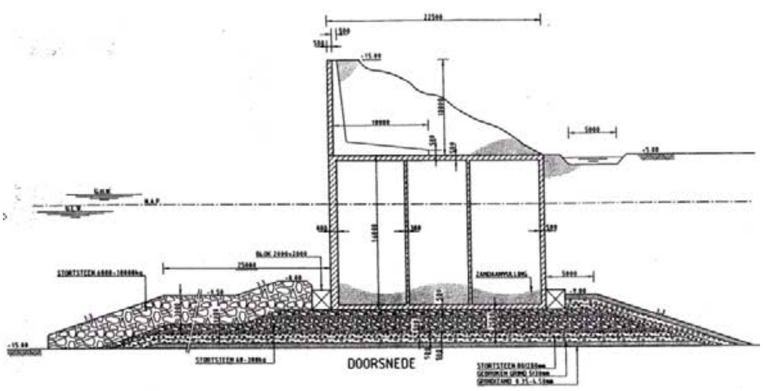


Figure 52 Design cross-section caisson construction

Total costs are estimated at € 355 million.

Block wall

The differences between a caisson and a block wall construction lay mainly in the construction phase. Where the blocks in a block wall construction are relatively small, the caissons are very large and are transported over water. A pre design of the simplest concept with sand filled blocks is shown in Figure 53. The blocks are placed staggered on top of each other.

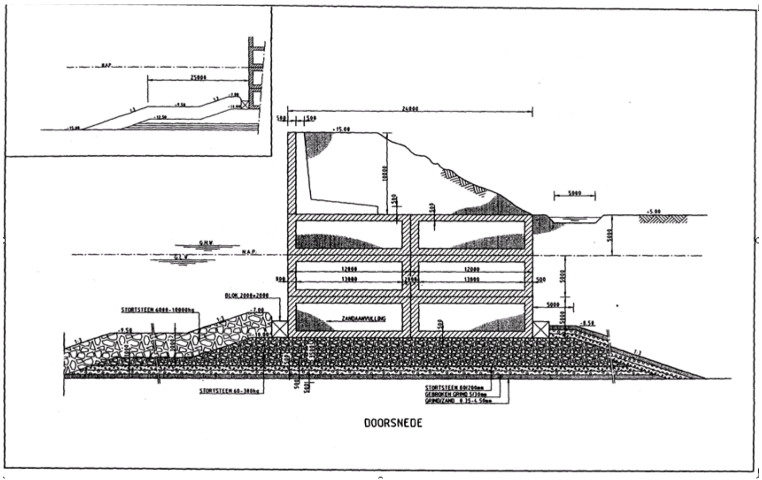


Figure 53 Design cross-section block wall construction

The most important design parameters are given below.

- Height NAP + 15.0 m (standing wave);
- Filter and foundation layer 5 m (NAP – 15.0 m to NAP – 10.0 m);
- Total height of 3 blocks 15 m (NAP – 10.0 m to NAP + 5.0m);
- Crest construction 10 m (NAP + 5.0 m tot NAP + 15.0 m);
- Block dimensions (l x w x h) 12 m x 12 m x 12 m, sand filled 1580 tons;
- Sea side bed protection 6-10 tons, width ca. 40 m;

Total cost for construction and maintenance are estimated at € 485 million.

Retaining wall with releaving floor

Figure 54 shows a few retaining wall constructions. For economical reasons a sheet pile wall construction was chosen. Bases on orientating calculations this solution did not seem realizable for design and construction reasons. The retaining wall construction with releaving floor is thought to be the only feasible alternative (see Figure 55).

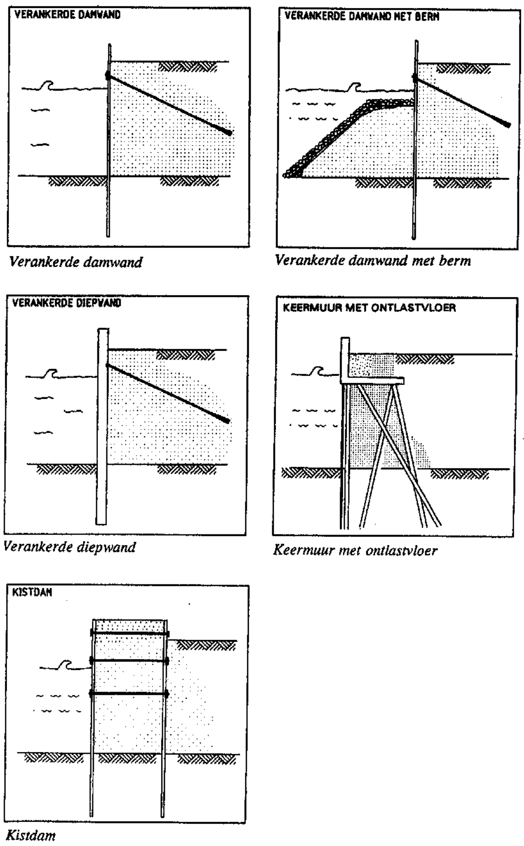


Figure 54 Five common retaining wall concepts

Since construction under water will influence the accuracy of connection parts, construction will take place in the dry.

The construction consists of:

- Sheet piles down to NAP – 35.0 m;
- Anchoring with MV-piles, directed 45 degrees backward;
- Foundation consists of 2 sets of piles and 1 single pile down to NAP – 28.0 m;
- Seaside bed protection 6-10 tons, width ca. 40 m;

Figure 55 Cross-section of retaining wall construction with releaving floor

Total cost for construction and maintenance are calculated to be € 808 million.

K.4. Comparison of alternatives

In Table 24 the effect overview is given. Since the alternatives are designed following design requirements formulated for this research only, the effects should only be interpreted as a comparison of the designs within *Bouwstenen "Terrein, Zeewering en Golfbreker" voor Maasvlakte 2.*

Table 24 Overview effects northern sea defence (price level 1997)

Aspect	Criterium	Meet-schaal	Alternatieven: Kunstmatig duin: Onbeschermd	Kunstmatig duin: Zuiderdam-variant	Kunstmatig duin: Hangend strand	Kunstmatig duin: Breukstenen golfbreker	Zeedijk	Caissons	Blokken-muur	Keermuur
Economie	• aanlegkosten	*f* miljoen	190	400	230	190	270	680	780	1.300
Uitvoering	• uitvoerbaarheid	+/0/-	+	+	+	+	+	0/-	0	0
	• aanlegtijd	+/0/-	+	+	+	+	+	0	0	-
	• kans op schade	+/0/-	0	0	0	0	-	-	-	0
	• scheepvaarthinder	+/0/-	+	+	+	+	+	0	0	+
Flexibiliteit	• gebruik als aanlegkade	+/0/-	-	-	-	-	-	+	+/0	+
	• hergebruik/verplaatsbaarheid	+/0/-	+	0	0	0	-	0	0	-
Ruimtebeslag	• aanlegbreedte[1]	+/0/-	-	0/-	0/-	0/-	0	+	+	+
Recreatie/ natuur/milieu	• mogelijkheden inpassing/gebruik	+/0/-	+	0	+	+/0	0	-	-	-
Duurzaamheid	• gebruik reststoffen, restwaarde	+/0/-	+	0	0	0	+/0	+	+	0
Morfologie	• mate van plaatselijke verstoring	+/0/-	+	0	0	0	0	-	-	-
Procedures	• aanlegvergunning	+/0/-	0	0	0	0	+	0	+/0	+

[1] breedte van de constructie inclusief een eventuele bodembeschermingsconstructie

Economy

Construction and maintenance costs are given in Table 25.

Table 25 Overview costs alternatives for a shore protection of 4 km[13]

Costs (x 10^6 €), 2005 Alternative	Bottom level [meters below NAP]			
	5	***10***	***15***	***20***
Artificial dune	56	81	119	181
▪ ***Zuiderdam alternative***	-	-	249	-
▪ ***Hydraulic fill dams***	-	-	144	-
▪ ***Quarry stone breakwater***	-	-	119	-
Sea dike	-	137	168	198
Caisson	-	292	354	404
Block wall	-	453	484	534
Retaining wall	-	546	807	1118

Construction

- Feasibility
  The construction of the artificial dune and the sea dike can take place in the traditional way. On the other hand finding a location for building the caissons can be critical because of number and the dimensions of the caissons. Placing of the caissons and block elements requires good weather conditions.

- Construction duration
  Construction of the artificial dune and the sea dike takes approximately 1 to 1.5 years. The caisson and the block wall can be built in 3 years and construction of the retaining wall takes 7 years.

[13] Costs are indexated from the 1997 price level to the year 2005. Costs include maint. An accuracy of plus and minus 30% is strived for. V.A.T., engineering costs, environment research costs are excluded;

L. Literature - Landaanwinning Ontwerprapport Terrein, Zeewering en Havendam [ref 9]

L.1. Introduction

In ref 9 preliminary designs are made for the sea defence, reclaimed area and harbour breakwater. The main difference with earlier studies lies within the adjusted requirements, especially concerning the used safety level. The emphasis in this abstract lies on sea defence related subjects.

L.2. Requirements

General requirements

- The design life cycle of the sea dike at the northern side of Maasvlakte 2 depends on the type of structure that is applied. The minimum period must be 50 years [ref 20].
- The designs are based on a safety level of 1/10,000 per year;
- The average overtopping discharge may not exceed 10 l/s/m;

Table 26 Hydraulic design conditions [ref 9]

Exceedance frequencies	10^{0}	10^{-1}	10^{-2}	10^{-3}	10^{-4}
H_{m0} (m)	5.10	6.20	7.05	7.80	8.40
T_m (s)	7.10	7.80	8.30	8.80	9.10
T_p (s)	9.50	10.6	11.4	12.1	12.7
Max. water level (NAP + m)	2.30	2.89	3.52	4.21	4.95
Min. water level (NAP + m)	0.48	1.07	1.70	2.39	3.13

L.3. Cross section

The height of the berm is increased compared to Kortlever to increase its effect. The crest level is calculated at NAP+15.8m. The armour layer consists of quarry stone with a 6-10 ton grading.

M. Overtopping theory

PC Overslag, the software used for the calculations, used the basic formula for wave run-up having the form (Battjes 1974):

$$\frac{R_{ui\%}}{H_s} = (A\zeta_{op} + C)\gamma_r \gamma_b \gamma_h \gamma_\beta$$

where

$R_{ui\%}$ = run-up level exceeded by I percentage of the incident waves

ζ_{op} = surf-similarity parameter $= \dfrac{\tan\alpha}{\sqrt{H_s / L_{op}}}$

A, C = coefficients

γ_r = reduction factor for the surface roughness

γ_b = reduction factor for the influence of a berm

γ_h = reduction factor for the influence of shallow water conditions

γ_β = factor for the influence of angle of incidence β of the waves

α = slope angle

L_{op} = deep water wave length $= \dfrac{gT^2}{2\pi}\tanh\left(\dfrac{2\pi d}{L_{op}}\right)$

H_s = significant wave height

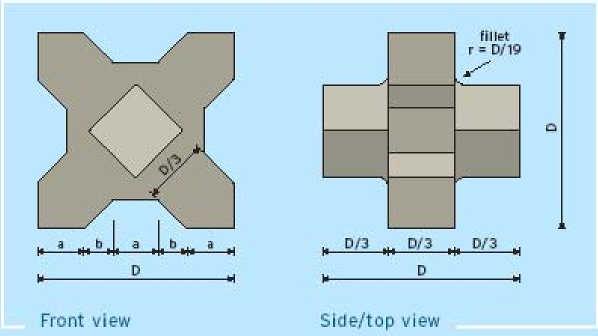
d = depth

The roughness coefficient for the Accropode

N. Basic design rules by Delta Marine Consultants

Main dimensions

The volume of the units varies between 0.75 m³ and 20 m³, depending upon the extreme sea-state.



Front view   Side/top view

Table for concept design

Xbloc® unit height $D \geq \frac{H_s}{1.92\Delta}$; $\Delta = \left[\frac{\rho_{concrete}}{\rho_{seawater}} - 1\right]$

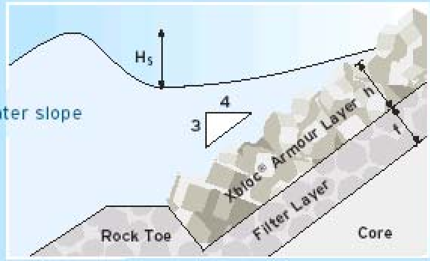
Xbloc® unit weight $W_{Xbloc} = \rho_{concrete} \frac{D^3}{3} = \rho_{concrete} V_{Xbloc}$

Filter layer weight $W_{filter} = \frac{W_{Xbloc}}{15}$ to $\frac{W_{Xbloc}}{7}$

Equations and design table are based on:

Slope: 3V : 4H
$\rho_{concrete}$: 2400 kg/m³
$\rho_{seawater}$: 1030 kg/m³
K_D: 16 for trunk section
13 for head section [armour units are 25% heavier than for trunk section]

Cross-section of breakwater slope

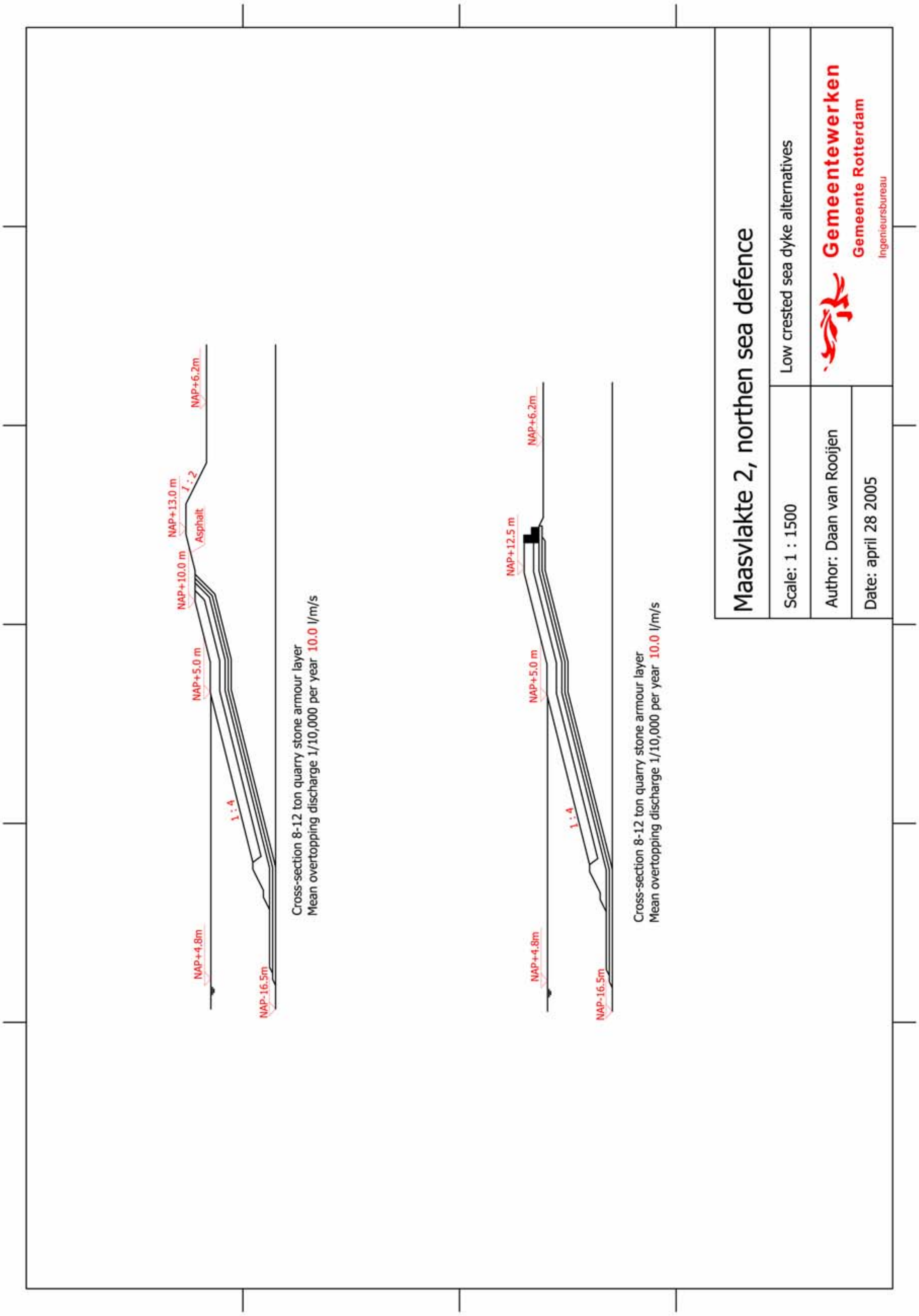


Unit volume V_{Xbloc} [m³]	Design wave height H_s [m]	Unit height D [m]	Unit weight W_{Xbloc} [ton]	Thickness Xbloc® armour layer h [m]	Density [number of units per 100 m²]	Advised placement grid dx.D, dy.D: dx	dy	Concrete volume per m² [m³/m²]	Porosity Xbloc® armour layer [%]	Standard rock grading for filter layer [ton-ton]	Thickness filter layer f [m]
0.75	3.35	1.31	1.8	1.3	70.00	1.300	0.640	0.53	59	0.06-0.3	0.8
1.0	3.69	1.44	2.4	1.4	57.78	1.300	0.640	0.58	59	0.06-0.3	0.8
1.5	4.22	1.65	3.6	1.6	44.10	1.300	0.640	0.66	59	0.3-1.0	1.3
2.0	4.65	1.82	4.8	1.8	36.40	1.300	0.640	0.73	59	0.3-1.0	1.3
2.5	5.01	1.96	6.0	1.9	31.37	1.300	0.640	0.78	59	0.3-1.0	1.3
3.0	5.32	2.08	7.2	2.0	27.78	1.300	0.640	0.83	59	0.3-1.0	1.3
4.0	5.86	2.29	9.6	2.2	22.93	1.300	0.640	0.92	59	0.3-1.0	1.3
5.0	6.31	2.47	12.0	2.4	19.76	1.300	0.640	0.99	59	1.0-3.0	1.8
6.0	6.70	2.62	14.4	2.5	16.71	1.330	0.655	1.00	61	1.0-3.0	1.8
7.0	7.06	2.76	16.8	2.7	15.08	1.330	0.655	1.06	61	1.0-3.0	1.8
8.0	7.38	2.88	19.2	2.8	13.80	1.330	0.655	1.10	61	1.0-3.0	1.8
9.0	7.67	3.00	21.6	2.9	12.75	1.330	0.655	1.15	61	1.0-3.0	1.8
10.0	7.95	3.11	24.0	3.0	11.89	1.330	0.655	1.19	61	1.0-3.0	1.8
12.0	8.44	3.30	28.8	3.2	10.53	1.330	0.655	1.26	61	1.0-3.0	1.8
14.0	8.89	3.48	33.6	3.4	9.08	1.360	0.670	1.27	63	3.0-6.0	2.4
16.0	9.29	3.63	38.4	3.5	8.31	1.360	0.670	1.33	63	3.0-6.0	2.4
18.0	9.67	3.78	43.2	3.7	7.68	1.360	0.670	1.38	63	3.0-6.0	2.4
20.0	10.01	3.91	48.0	3.8	7.16	1.360	0.670	1.43	63	3.0-6.0	2.4

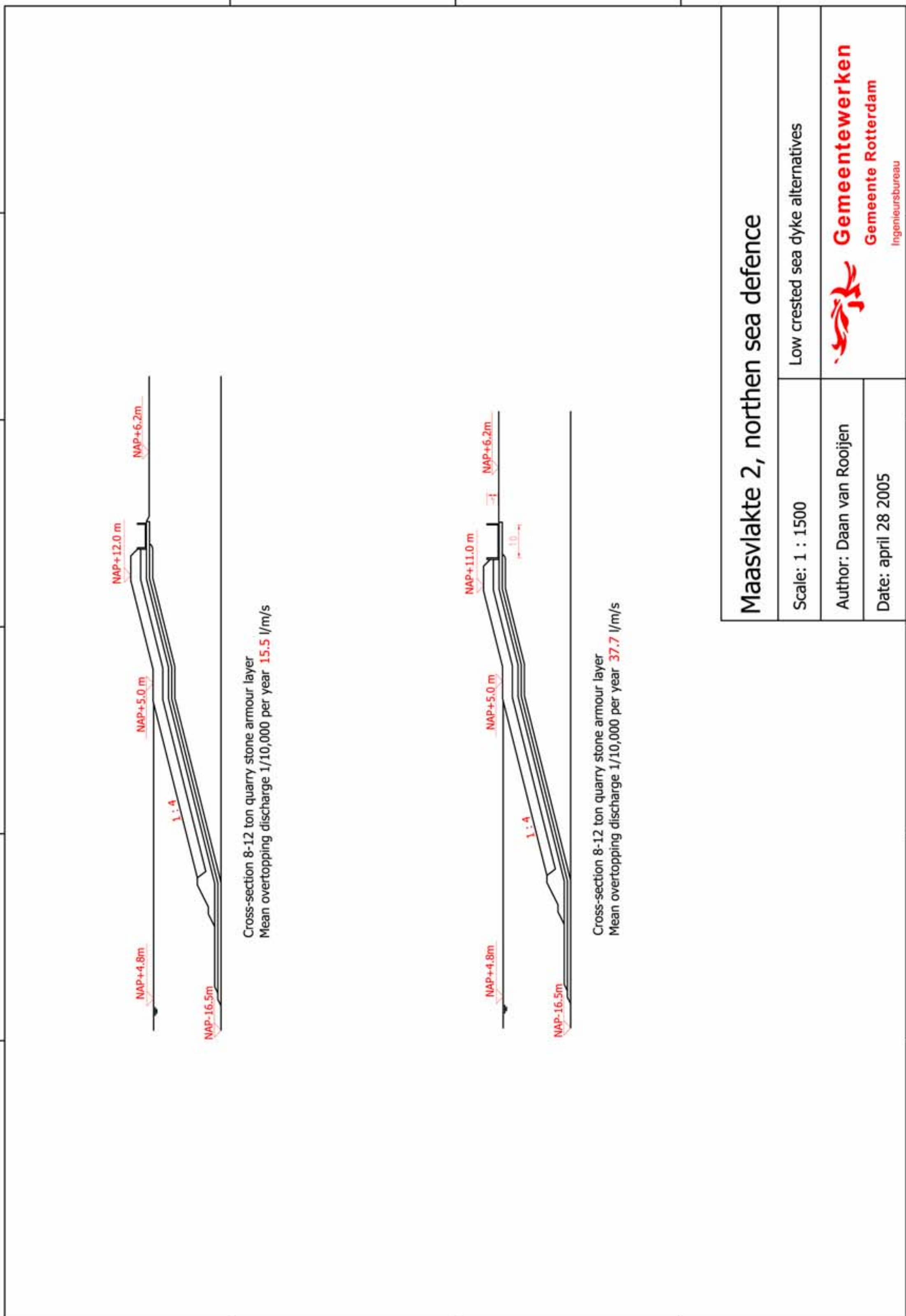
O. Discharge formula parameters

Crest height	Mean overtopping discharge 10^{-4} cond.	Q canal End	Canal width	Canal height	Hydraulic	Concrete Qty	Concrete Costs
[NAP+m]	[l/m/s]	[m3/s]	[m]	[m]	radius [m]	[m3/m1]	[€/m1]
	XBloc armour layer						
34.8	0.1	0	-	-	-	-	-
28.9	1	0	-	-	-	-	-
27.1	2	0	-	-	-	-	-
24.8	5	0	-	-	-	-	-
23	10	0	-	-	-	-	-
20	15.7	35	7.4	2.5	1.5	5	550
19	24.1	54	8.8	2.9	1.7	5	502
18	37	83	10.5	3.5	2.1	6	682
17	57	128	12.4	4.1	2.5	7	770
16	88	198	14.8	4.9	2.9	8	832
15	137	308	17.7	5.9	3.5	10.0	1100
14	210	473	21.0	7.0	4.2	12	1320
13	325	731	25.0	8.3	5.0	14	1540
12	500	1125	36.5	8.2	5.7	18	1980
11	770	1733	62.9	7.2	5.9	25	2750
10	1190	2678	115.5	6.2	5.6	40	4400
	Quarry stone armour layer						
18.6	0.1	0	-	-	-	-	-
15.8	1	2	-	-	-	-	-
13	10	23	-	-	-	-	-
12	15.5	35	7.4	2.5	1.5	5	550
11	37.7	85	10.5	3.5	2.1	6	682
10	92	207	15.1	5.0	3.0	9	979
9	224	504	33.5	4.9	3.8	14	1573

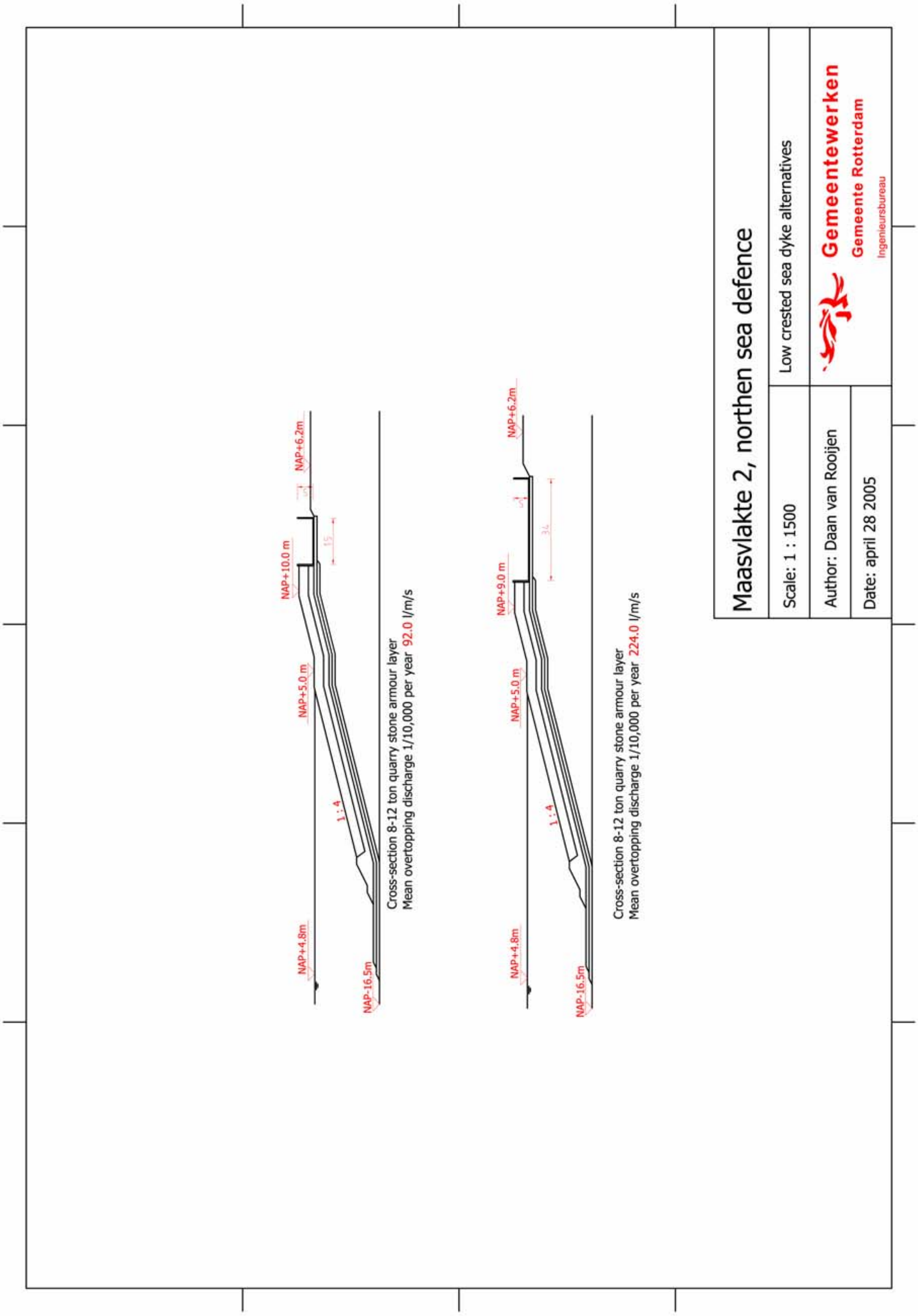
P. Cross-sections low crested sea-dikes, quarry stone



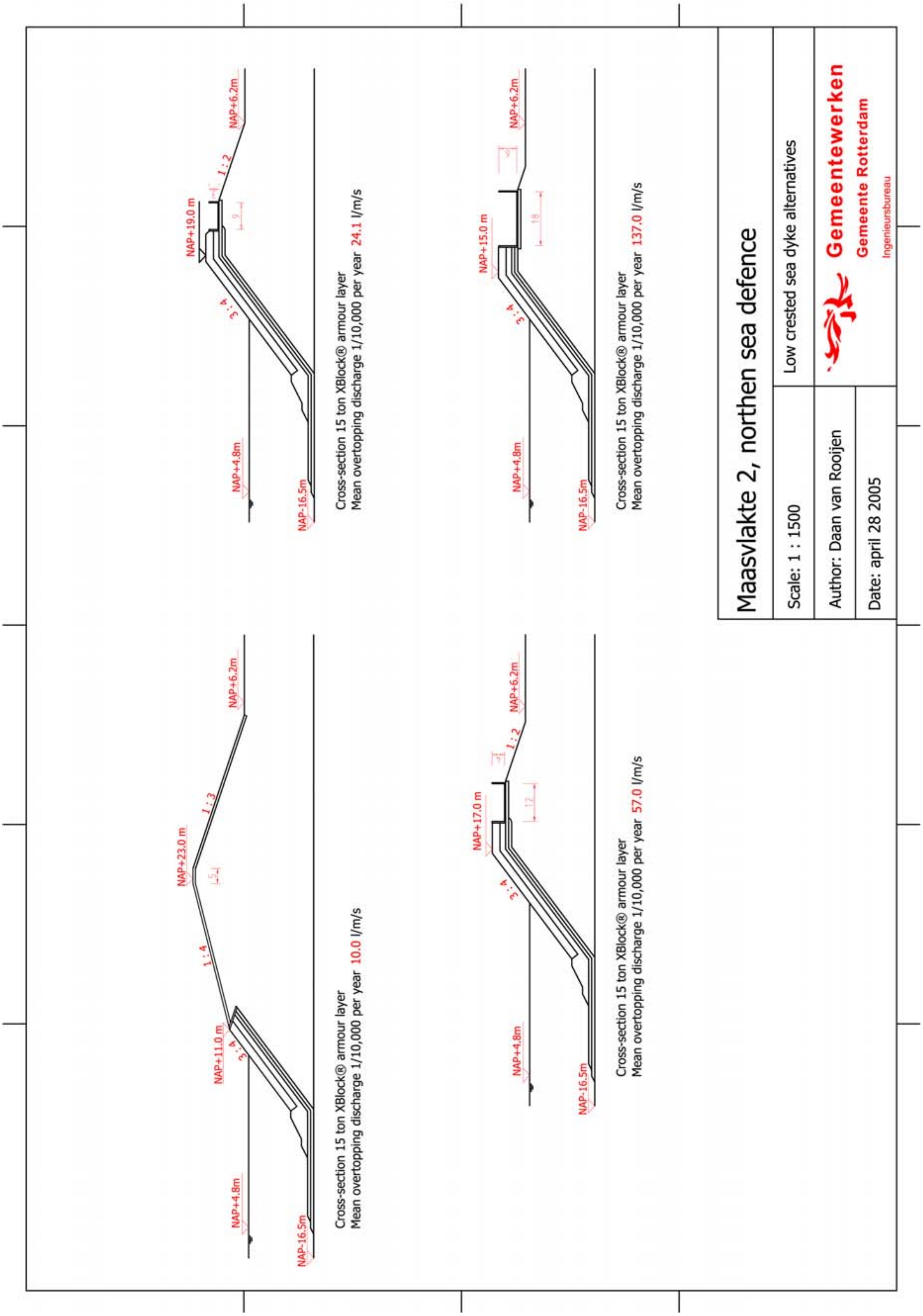
Q. Cross-sections low crested sea-dikes, quarry stone



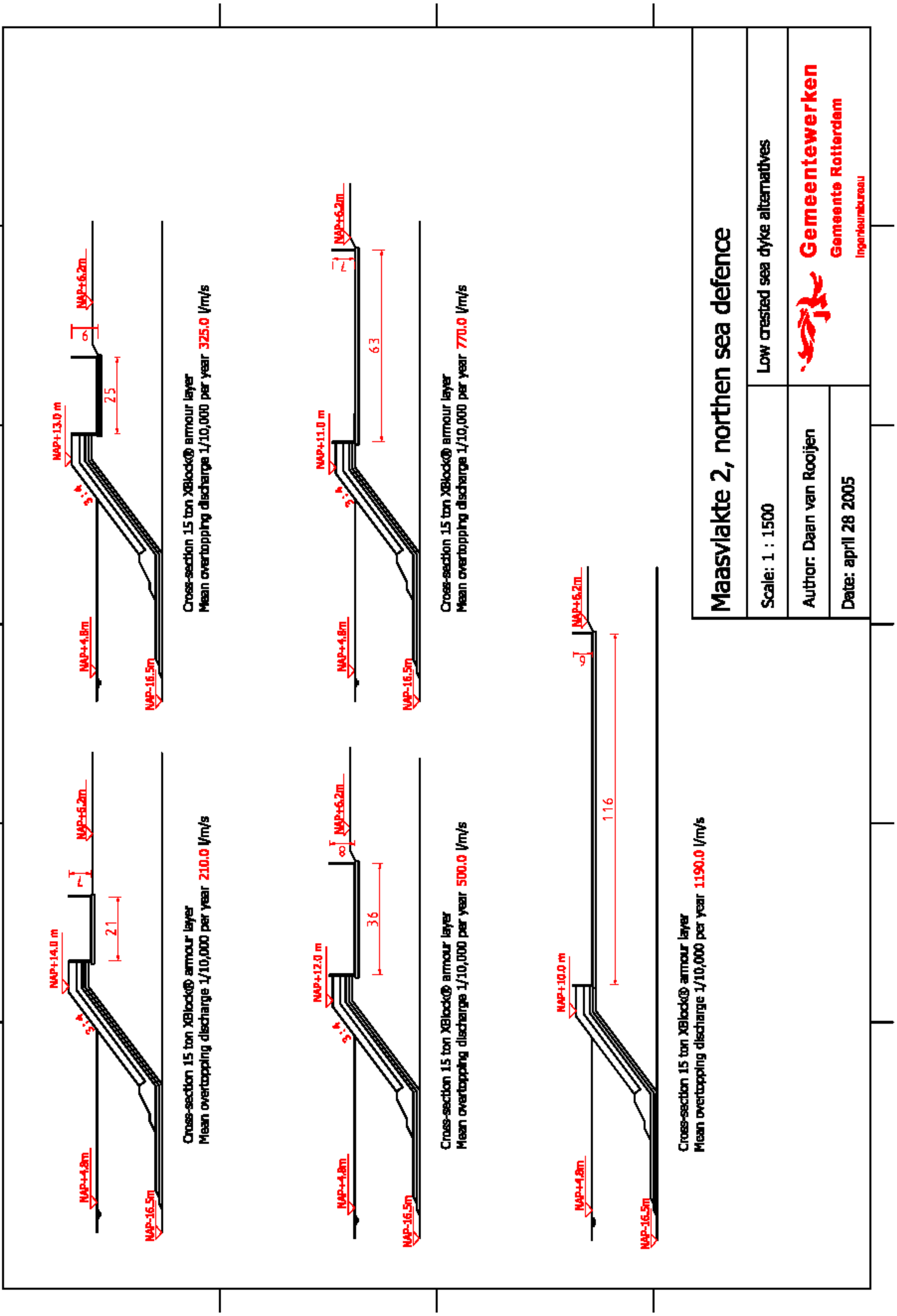
R. Cross-sections low crested sea-dikes, quarry stone



S. Cross-sections low crested sea-dikes, XBloc units



T. Cross-sections low crested sea-dikes, XBloc units



U. Cost calculation low crested sea dike alternatives

U.1. Unit costs: building materials + construction costs

Table 27 Calculation of quarry stone unit prices

	Delivery cost building materials	Added percentage for construction	Total cost for building and construction
Grading	[€/ton]	[%]	[€/ton]
40-100 mm	11,25	5,00	11,8
5-40 kg	13,00	6,00	13,8
10-60 kg	13,00	6,00	13,8
60-300 kg	15,40	7,00	16,5
300-1000 kg	18,50	8,50	20,1
1-3 ton	18,50	10,00	20,4
(average 3 tons)	18,90	30,00	24,6
(average 4 tons)	19,30	32,86	25,6
3-6 ton	19,50	34,29	26,2
(average 5 tons)	19,71	35,71	26,8
(average 6 tons)	20,14	38,57	27,9
(average 7 tons)	20,57	41,43	29,1
6-10 ton	21,00	44,29	30,3
(average 9 tons)	21,50	47,14	31,6
8-12ton	22,00	50,00	33,0

Building material/unit	Unit prices (production and construction costs)	unit
XBloc	127,50	€/m^3
Sand	4,-	€/m^3
Canal concrete	110,-	€/m^3

U.2. Costs low crested sea-dikes[14]

	Quarry stone alternative with asphalt top part				
	Crest height	13	10 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	2444	m3/m1	2444	m2	9776
quarry stone 5-40 kg	139	m3/m1	221	ton	3050
quarry stone 60-300 kg	133	m3/m1	211	ton	3489
quarry stone 1-3 ton	220	m3/m1	350	ton	7136
blocks 30 ton	0	m3/m1	0	m2	0
quarry stone 8-12 ton	277	m3/m1	440	ton	14534
asphalt	13	m2/m1	13	m2*	195
					38180
					172.000.000

	Quarry stone alternative				
	Crest height	12,5	10 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	1849	m3/m1	1849	m2	7396
quarry stone 5-40 kg	149	m3/m1	237	ton	3269
quarry stone 60-300 kg	150	m3/m1	239	ton	3935
quarry stone 1-3 ton	246	m3/m1	391	ton	7979
blocks 30 ton	19	m3/m1	19	m2	2090
quarry stone 8-12 ton	334	m3/m1	531	ton	17525
					42195
					190.000.000

	Quarry stone alternative				
	Crest height	12	15,5 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	1843	m3/m1	1843	m2	7372
quarry stone 5-40 kg	147	m3/m1	234	ton	3225
quarry stone 60-300 kg	151	m3/m1	240	ton	3961
quarry stone 1-3 ton	242	m3/m1	385	ton	7850
blocks 30 ton	0	m3/m1	0	m2	0
quarry stone 8-12 ton	325	m3/m1	517	ton	17053
					39461
					178.000.000

	Quarry stone alternative				
	Crest height	11	37,7 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	1785	m3/m1	1785	m2	7140
quarry stone 5-40 kg	143	m3/m1	227	ton	3138
quarry stone 60-300 kg	150	m3/m1	239	ton	3935
quarry stone 1-3 ton	234	m3/m1	372	ton	7590
blocks 30 ton	0	m3/m1	0	m2	0
quarry stone 8-12 ton	312	m3/m1	496	ton	16371
					38174
					172.000.000

	Quarry stone alternative				
	Crest height	10	92 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	1755	m3/m1	1755	m2	7020
quarry stone 5-40 kg	139	m3/m1	221	ton	3050
quarry stone 60-300 kg	150	m3/m1	239	ton	3935
quarry stone 1-3 ton	227	m3/m1	361	ton	7363
blocks 30 ton	0	m3/m1	0	m2	0
quarry stone 8-12 ton	302	m3/m1	480	ton	15846
					37214
					167.000.000

	Quarry stone alternative				
	Crest height	9	224 l/m/s		1,00E+04
	Surface		Quantity		€/m
sand	1889	m3/m1	1889	m2	7556
quarry stone 5-40 kg	135	m3/m1	215	ton	2962
quarry stone 60-300 kg	165	m3/m1	262	ton	4329
quarry stone 1-3 ton	220	m3/m1	350	ton	7136
blocks 30 ton	0	m3/m1	0	m2	0
quarry stone 8-12 ton	289	m3/m1	460	ton	15164
					37147
					167.000.000

[14] Costs of alternatives are base on unit costs in appendix U.1.

U.3. Costs low crested sea-dikes[15]

	XBloc®				
	Crest height	23	10 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	3597	m3/m1	3597	m2	14388
quarry stone 5-40 kg	81	m3/m1	129	ton	1777
quarry stone 60-300 kg	79	m3/m1	126	ton	2073
quarry stone 1-3 ton	118	m3/m1	188	ton	3827
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	85	m3/m1	33	m2	4227
					26292
					118,000,000

	XBloc®				
	Crest height	19	24.1 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1790	m3/m1	1790	m2	7160
quarry stone 5-40 kg	97	m3/m1	154	ton	2128
quarry stone 60-300 kg	103	m3/m1	164	ton	2702
quarry stone 1-3 ton	152	m3/m1	242	ton	4930
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	139	m3/m1	54	m2	6912
					23833
					107,000,000

	XBloc®				
	Crest height	17	57 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1600	m3/m1	1600	m2	6400
quarry stone 5-40 kg	94	m3/m1	149	ton	2063
quarry stone 60-300 kg	103	m3/m1	164	ton	2702
quarry stone 1-3 ton	146	m3/m1	232	ton	4736
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	131	m3/m1	51	m2	6514
					22414
					101,000,000

	XBloc®				
	Crest height	15	137 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1261	m3/m1	1261	m2	5044
quarry stone 5-40 kg	110	m3/m1	175	ton	2414
quarry stone 60-300 kg	86	m3/m1	137	ton	2256
quarry stone 1-3 ton	140	m3/m1	223	ton	4541
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	123	m3/m1	48	m2	6116
					20371
					92,000,000

	XBloc®				
	Crest height	14	210 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1076	m3/m1	1076	m2	4304
quarry stone 5-40 kg	111	m3/m1	176	ton	2436
quarry stone 60-300 kg	84	m3/m1	134	ton	2204
quarry stone 1-3 ton	137	m3/m1	218	ton	4444
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	119	m3/m1	46	m2	5917
					19304
					87,000,000

	XBloc®				
	Crest height	13	325 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1142	m3/m1	1142	m2	4568
quarry stone 5-40 kg	114	m3/m1	181	ton	2501
quarry stone 60-300 kg	83	m3/m1	132	ton	2178
quarry stone 1-3 ton	134	m3/m1	213	ton	4346
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	115	m3/m1	45	m2	5718
					19312
					87,000,000

	XBloc®				
	Crest height	12	500 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1334	m3/m1	1334	m2	5336
quarry stone 5-40 kg	123	m3/m1	196	ton	2699
quarry stone 60-300 kg	81	m3/m1	129	ton	2125
quarry stone 1-3 ton	131	m3/m1	208	ton	4249
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	110	m3/m1	43	m2	5470
					19879
					89,000,000

	XBloc®				
	Crest height	11	265 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	1825	m3/m1	1825	m2	7300
quarry stone 5-40 kg	148	m3/m1	235	ton	3247
quarry stone 60-300 kg	79	m3/m1	126	ton	2073
quarry stone 1-3 ton	128	m3/m1	204	ton	4152
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	106	m3/m1	41	m2	5271
					22043
					99,000,000

	XBloc®				
	Crest height	11	666 l/m/s		1.00E+04
	Surface		Quantity		€/m
sand	2842	m3/m1	2842	m2	11368
quarry stone 5-40 kg	198	m3/m1	315	ton	4345
quarry stone 60-300 kg	78	m3/m1	124	ton	2046
quarry stone 1-3 ton	125	m3/m1	199	ton	4055
blocks 30 ton	0	m3/m1	0	m2	0
XBloc®	143	m3/m1	56	m2	7111
					28924
					130,000,000

Material	Euro	unit
sand	4	m3
quarry stone 5-40 kg	13.8	ton
quarry stone 60-300 kg	16.5	ton
quarry stone 1-3 ton	20.4	ton
blocks 30 ton	110	m3
XBloc®	127.5	m3

specific density concrete	2.5	ton/m3
specific density stone	2.65	ton/m3
porosity blocks - double	0.47	-
porosity blocks - single	0.15	-
Porosity quarry stone	0.4	-
porosity Accropodes	0.61	

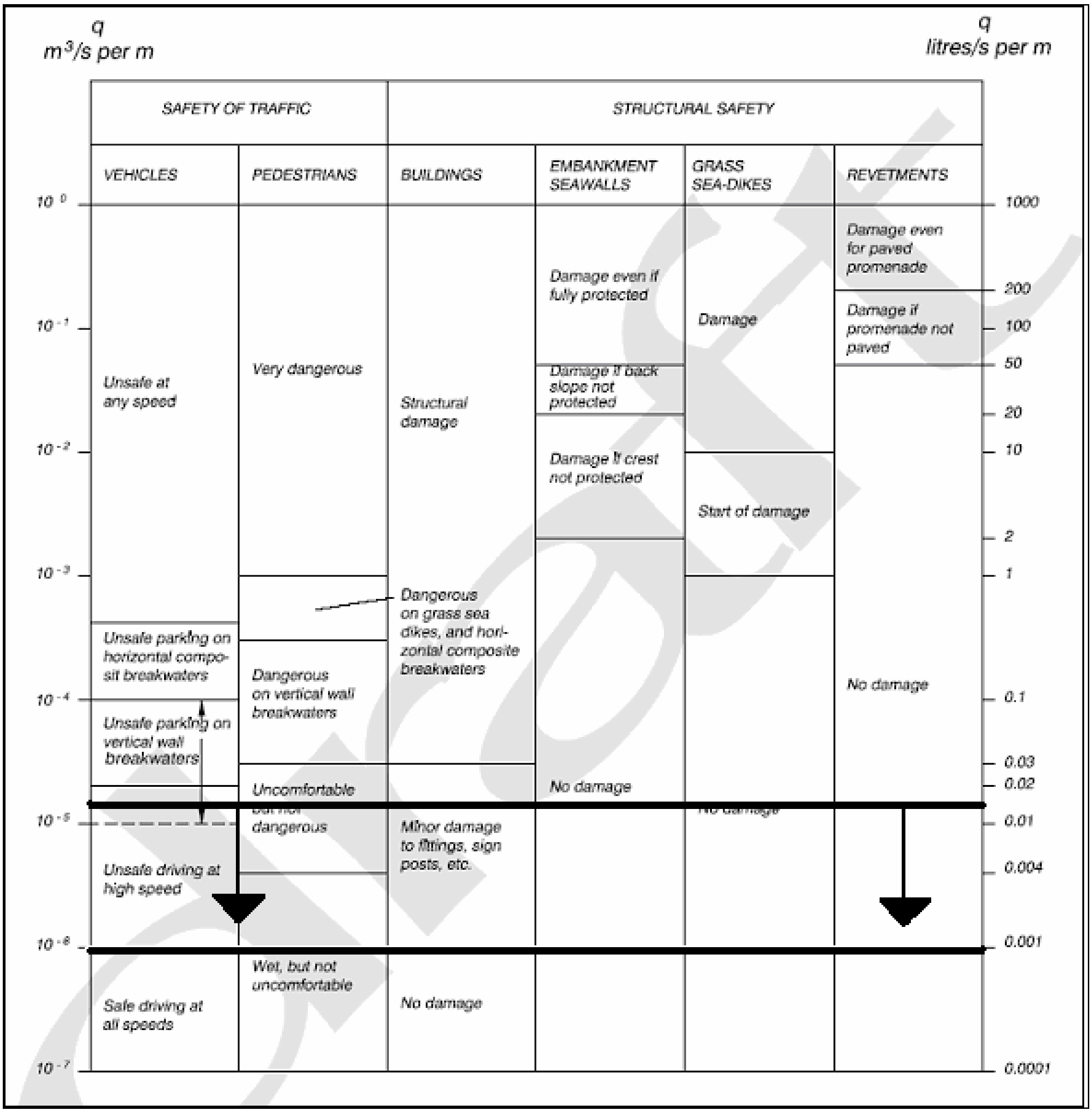
[15] Costs of alternatives are base on unit costs in appendix U.1.

U.4. Costs low crested dike alternatives[16]

Crest height	Overtopping 1/10,000 per year	Canal cost	End construction	sub total extra costs	Constr. costs dike (canal excl.)	Total constr. costs	Total cost
[m]	[l/m/s]	[€/m1]	[€/m1]	[€/m1]	[€/m1]	[€/m1]	[€]
XBloc armour layer							
23	10	0	0	0	26,292	26,292	118,000,000
19	24.1	502	222	724	23,833	24,556	111,000,000
17	57	770	222	992	22,414	23,406	105,000,000
15	137	1,100	222	1,322	20,371	21,693	98,000,000
14	210	1,320	222	1,542	19,304	20,846	94,000,000
13	325	1,540	222	1,762	19,312	21,074	95,000,000
12	500	1,980	222	2,202	19,879	22,081	99,000,000
11	770	2,750	222	2,972	22,043	25,015	113,000,000
10	1190	4,400	222	4,622	28,924	33,546	151,000,000
Quarry stone armour layer							
13	10	0	222	222	38,180	38,402	173,000,000
12	16	550	222	772	39,461	40,233	181,000,000
11	38	682	222	904	38,174	39,078	176,000,000
10	92	979	222	1,201	37,214	38,415	173,000,000
9	224	1,573	222	1,795	37,147	38,942	175,000,000

[16] Costs of alternatives are base on unit costs in appendix U.1.

V. Overtopping criteria



W. Return levels potential wind speed at HvH

Return level (m/s) Location: 330 Hoek van Holland, Season: Year

T (year)	010-030	040-060	070-090	100-120	130-150	160-180	190-210	220-240	250-270	280-300	310-330	340-360	omni
0.5	13.1	11.4	10.0	9.9	11.5	14.1	16.4	17.4	17.1	16.1	15.1	14.2	18.7
1	14.2	12.5	11.0	10.9	12.5	15.1	17.5	18.6	18.4	17.4	16.4	15.4	19.8
2	15.3	13.5	11.9	11.7	13.4	16.1	18.5	19.7	19.5	18.6	17.7	16.6	20.7
5	16.6	14.6	13.0	12.8	14.4	17.2	19.6	20.8	20.7	20.0	19.2	18.1	21.8
10	17.6	15.4	13.7	13.5	15.1	17.9	20.4	21.6	21.6	20.9	20.2	19.2	22.5
20	18.5	16.3	14.4	14.1	15.7	18.6	21.0	22.3	22.3	21.8	21.0	20.1	23.2
50	19.6	17.3	15.2	14.8	16.5	19.4	21.8	23.1	23.2	22.7	22.1	21.2	24.0
100	20.4	18.0	15.9	15.4	17.1	20.0	22.4	23.7	23.9	23.4	22.7	21.9	24.5
200	21.1	18.8	16.5	15.9	17.7	20.5	22.9	24.3	24.4	23.9	23.3	22.6	25.0
500	22.0	19.7	17.3	16.6	18.3	21.1	23.5	25.0	25.2	24.6	24.0	23.4	25.7
1000	22.6	20.3	17.8	17.0	18.9	21.5	24.0	25.5	25.6	25.1	24.5	23.9	26.1
2000	23.2	20.9	18.4	17.5	19.3	21.9	24.4	25.9	26.1	25.6	25.0	24.3	26.5
5000	23.9	21.7	19.0	18.0	20.0	22.4	25.0	26.4	26.6	26.2	25.6	25.0	27.0
10000	24.3	22.3	19.5	18.4	20.4	22.7	25.4	26.8	27.0	26.5	26.0	25.4	27.3

source: http://www.knmi.nl/samenw/hydra

X. Construction costs for *light constructions*

X.1. Construction costs 1-3ton and 8-12ton alternative

	Quarry stone alternative with asphalt top part					Quarry stone alternative with asphalt top part				
	Crest height 13		top layer: 8-12 ton			Crest height 13		top layer: 1-3 ton		
	Surface		Quantity		€/m	Surface		Quantity		€/m
sand	2444	m3/m1	2444	m2	9776	2769	m3/m1	2769	m2	11076
quarry stone 5-40 kg	139	m3/m1	221	ton	3050	134	m3/m1	213	ton	2940
quarry stone 60-300 kg	133	m3/m1	211	ton	3489	130	m3/m1	207	ton	3411
quarry stone 1-3 ton	220	m3/m1	350	ton	7136	161	m3/m1	256	ton	5222
blocks 30 ton	0	m3/m1	0	m2	0	0	m3/m1	0	m2	0
quarry stone 8-12 ton	277	m3/m1	440	ton	14534	0	m3/m1	0	ton	0
asphalt	13	m2/m1	13	m2*	195	13	m2/m1	13	m2*	195
					38180					22844
					172,000,000					103,000,000

X.2. Damage criteria and costs[17]

				Damage level			
				S>12	9<S≤12	6<S≤9	S≤6
				(2/3)*costs new constr.	2*costs for new rock	1.5*costs for new rock	1*costs for new rock
Grading	D_{n50} [m]	W_{50} [ton]	Constr. costs [€/m1]	Repair costs [€/m1]	Repair costs [€/stone]	Repair costs [€/stone]	Repair costs [€/stone]
1-3 ton	0.88	2	22844	15237	81.4	61.1	40.7
(average 3 tons)	1.04	3	24761	16516	147.4	110.6	73.7
(average 4 tons)	1.10	4	26678	17794	205.1	153.8	102.6
3-6 ton	1.18	4.5	27637	18434	235.7	176.8	117.8
(average 5 tons)	1.24	5	28595	19073	267.6	200.7	133.8
(average 6 tons)	1.31	6	30512	20352	334.9	251.2	167.5
(average 7 tons)	1.38	7	32429	21630	407.3	305.5	203.7
6-10 ton	1.43	8	34346	22909	484.8	363.6	242.4
(average 9 tons)	1.50	9	36263	24188	569.4	427.1	284.7
8-12ton	1.56	10	38180	25466	660.0	495.0	330.0

X.3. Damage number, S

			Return period		
Grading	**10000**	**1000**	**100**	**10**	**1**
1-3T	165,51	105,36	54,98	19,97	3,61
2-4T	71,03	45,22	23,59	8,57	1,55
1-6T	54,94	34,97	18,25	6,63	1,20
3-6T	39,00	24,83	12,95	4,71	0,85
3-7T	30,32	19,30	10,07	3,66	0,66
4-8T	22,37	14,24	7,43	2,70	0,49
5-9T	17,30	11,02	5,75	2,09	0,38
6-10T	14,61	9,30	4,85	1,76	0,32
7-11T	11,38	7,25	3,78	1,37	0,25
8-12T	9,55	6,08	3,17	1,15	0,21

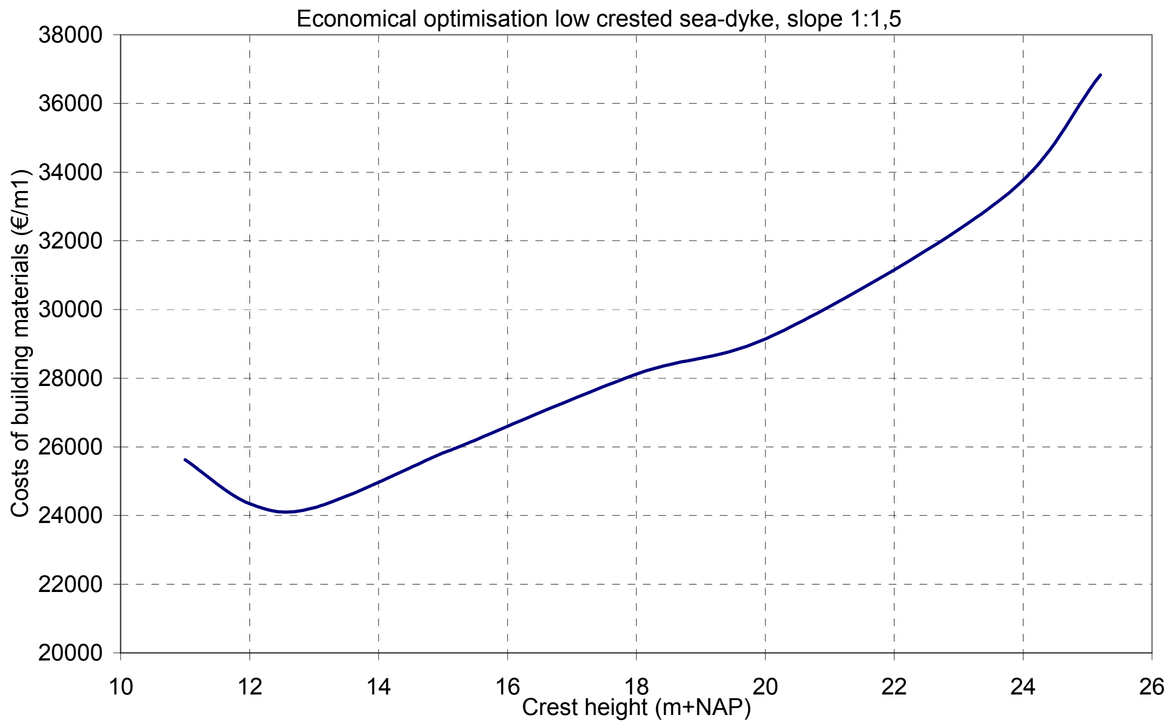
[17] Costs of alternatives are base on unit costs in appendix U.1.

X.4. Total costs[18] light construction

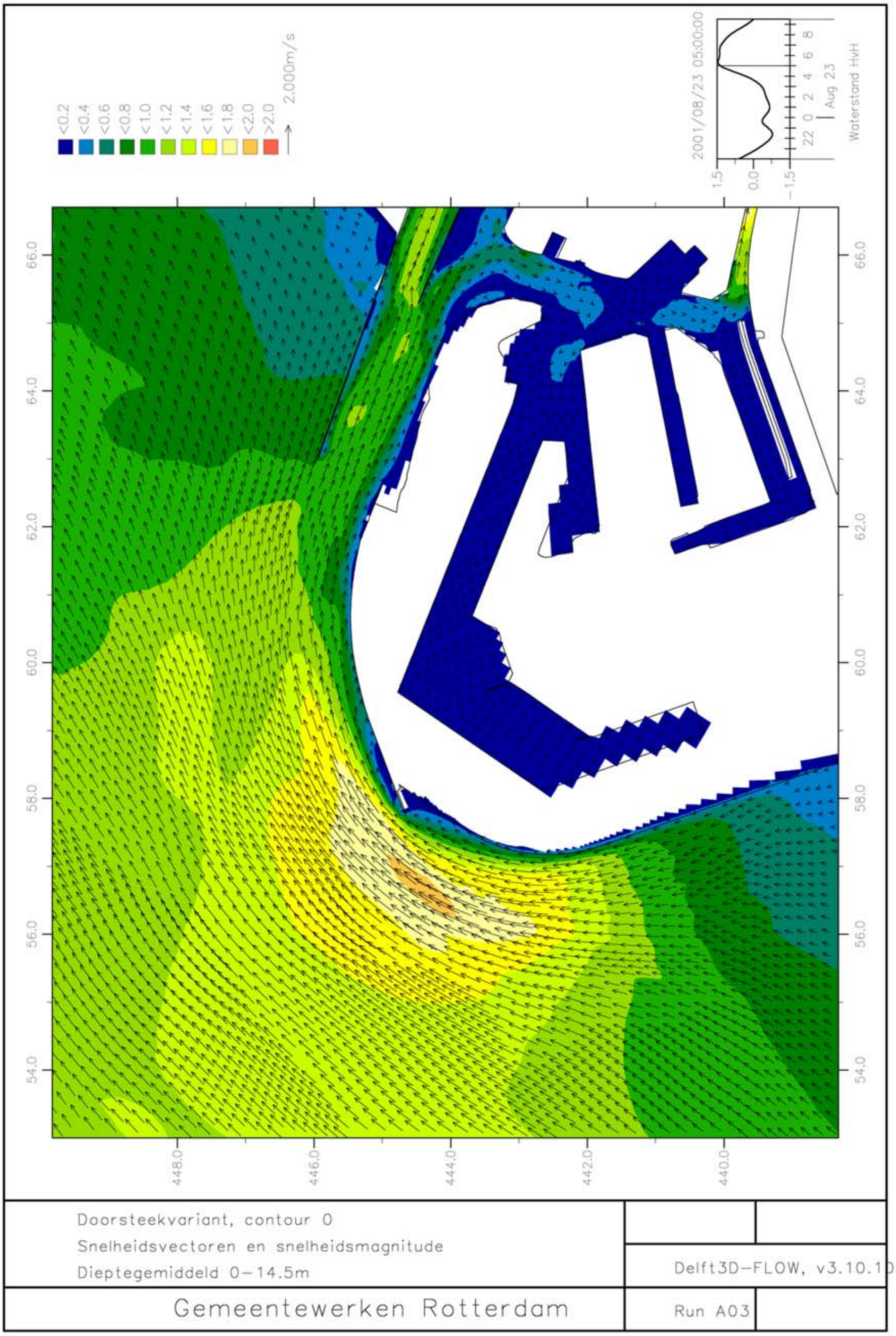
Grading	Repair [€/m1]	Initial construction [€/m1]	Total [€/m]
1-3 ton	51796	22844	74640
(average 3 tons)	9252	24761	34013
(average 4 tons)	7498	26678	34176
3-6 ton	7179	27637	34816
(average 5 tons)	3687	28595	32283
(average 6 tons)	3146	30512	33658
(average 7 tons)	2451	32429	34881
6-10 ton	2383	34346	36729
(average 9 tons)	2035	36263	38298
8-12ton	1910	38180	40090

[18] Costs of alternatives are base on unit costs in appendix U.1.

Y. Economical optimisation XBloc slope 1:1.5

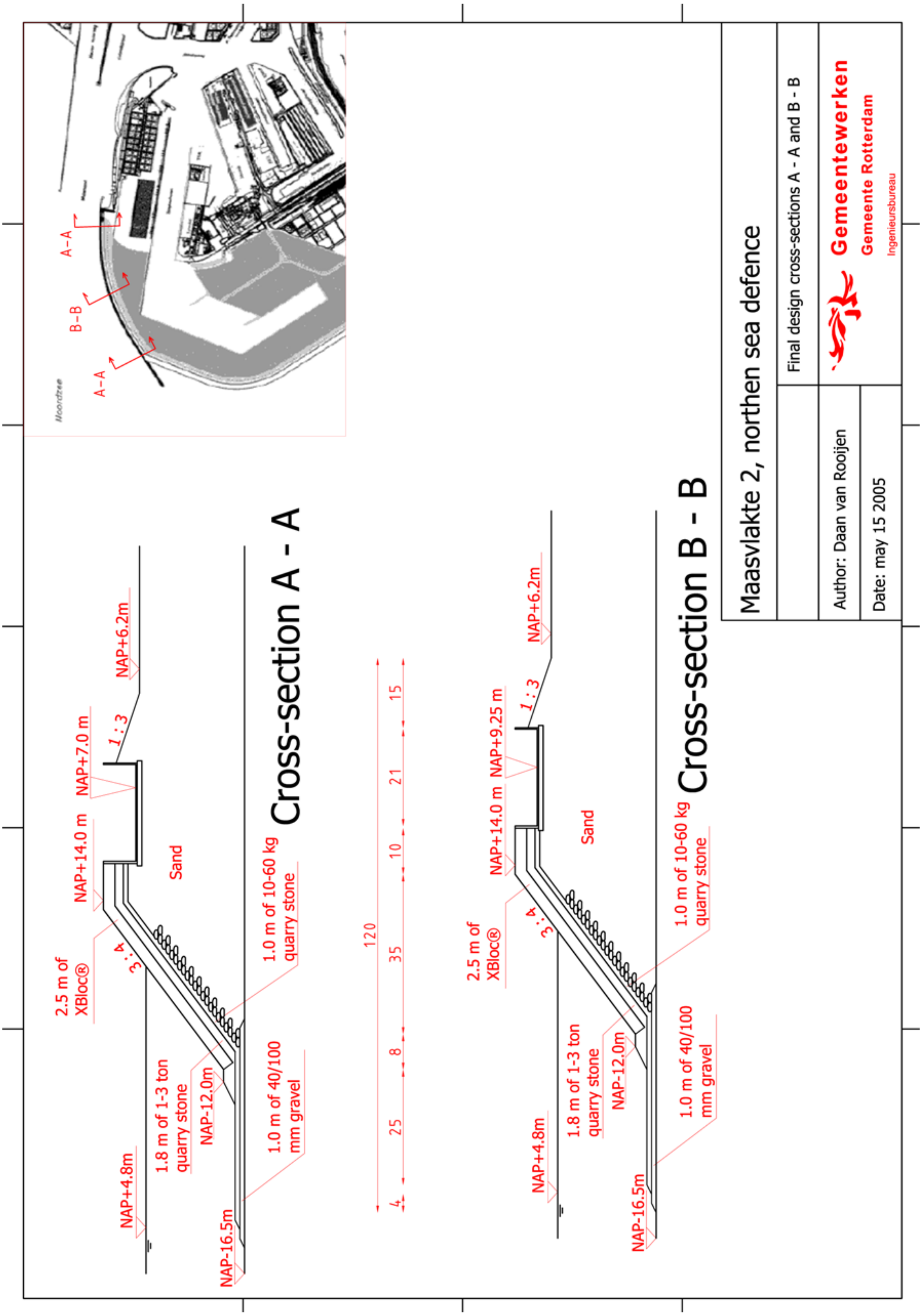


Z. Maximum depth averaged tidal flow



AA. Final Design

AA.1. Cross-sections final design



AA.2. Costs final design

	XBloc®	cross-section A-A					XBloc®	Cross-section B-B				
	Crest height	21	10	l/m/s	1.00E+04		Crest height	19	24.1	l/m/s	1.00E+04	
	Surface		Quantity		€/m		Surface		Quantity		€/m	
sand	1489	m3/m1	1489	m3	5956		1538	m3/m1	1538	m2	6152	
gravel 40/100 mm	71	m3/m1	113	ton	1332		71	m3/m1	113	ton	1332	
quarry stone 10-60 kg	86	m3/m1	137	ton	1887		86	m3/m1	137	ton	1887	
quarry stone 1-3 ton	106	m3/m1	169	ton	3438		106	m3/m1	169	ton	3438	
concrete slabs	12	m3/m1	12	m3	1320		10	m3/m1	10	m2	1100	
15 ton XBloc® units	131	m3/m1	51	m3	6514		131	m3/m1	51	m2	6514	
geotube	33	-	33	-	224		33	m2/m1	33	m2	224	
					20672						20648	

Material	Euro	unit
sand	4	m3
gravel 40/100 mm	11.8	ton
quarry stone 10-60 kg	13.8	ton
quarry stone 1-3 ton	20.4	ton
concrete slabs	110	m3
15 ton XBloc® units	127.5	m3
geotube	6.8	m1

specific density concrete	2.5	ton/m3
specific density stone	2.65	ton/m3
porosity blocks - double	0.47	-
porosity blocks - single	0.15	-
Porosity quarry stone	0.4	-
porosity XBloc® units	0.61	-