A FLOATING FACTORY FOR THE MAASVLAKTE 2 CAISSON BREAKWATER

-Design and construction method of reusable caissons for the Maasvlakte 2 breakwater-

Final thesis for M.Sc. C.E.
Technical University of Delft,
Faculty of Civil Engineering,
Department: Hydraulic Engineering

Afstudeerrapport aan de
Technische Universiteit Delft
Faculteit der Civiele Techniek
Vakgroep: Waterbouwkunde

student:	**S.Mann**
student number:	**542709**
date:	**March 1999**

Supervisors TU Delft:

prof. drs. ir. J.K. Vrijling
ir. W.H. Tutuarima
ir. T.H.W. Horstmeier

TU Delft
Technische Universiteit Delft

Supervisors Ministry of Transport,
Public Works and Watermanagement:

ir. H.A. Lavooij
ing. R. Camerik

Preface

This report has been prepared for the department of Civil Engineering of the Technical University of Delft and contains the thesis: 'A floating factory for the Maasvlakte 2 caisson breakwater, 'The design and construction method of reusable caissons for the Maasvlakte 2 breakwater', which analyses the feasibility to construct the Maasvlakte 2 breakwater by means of reusable caissons. A floating construction method for these caissons has been analysed as well.

The thesis has been written under the supervision of prof. drs. ir. J.K. Vrijling, ir. W.H. Tutuarima and ir. T.H. Horstmeier from the Technical University of Delft, and from the Ministry of Transport, Public Works and Watermanagement under the supervision of ir. H.A. Lavooij head of the Department of Hydraulic Engineering and ing. R. Camerik of the Dry Infrastructure department.

I would like to express my gratitude to all these people for their time and assistance in the preparation of this thesis. Also I would like to thank the Ministry of Transport, Public Works and Watermanagement for their facilities which were made available to me.

S.Mann, March 1999

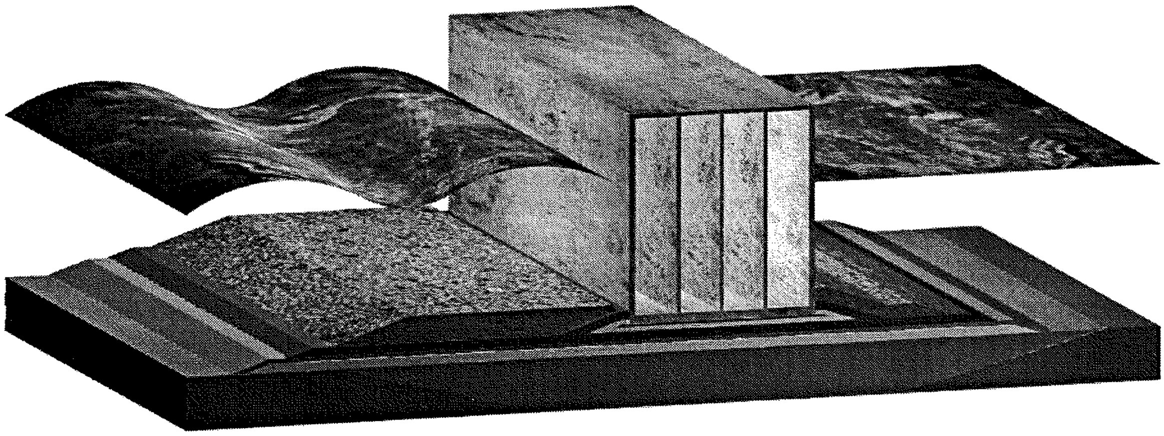


Figure 1. Cross-section of the Maasvlakte 2 caisson breakwater.

Abstract

Due to its excellent geographical location and its modern facilities, the Port of Rotterdam has a leading position in the world trade. In order to maintain this position, the port must compete with the rapid development of other European ports, and investments on the harbour facilities must constantly be made. A problem the harbour will face during the next decades, will be the lack of area to expand.

A typical Dutch solution to create the necessary area for harbour and industrial terrain, is reclamation of land from the sea. This can be achieved by expanding Maasvlakte 1 with Maasvlakte 2. The exact amount of required terrain is subject to many elements which are continuously changing such as political, economical, environmental and social views, and therefore is very difficult to predict accurately. A solution to meet this uncertainty, is to create the Maasvlakte 2 in several phases. By maintaining a flexible planning of the reclamation, the expansion works can be adapted to changed views if necessary.

The construction works of the Maasvlakte 2 are very extensive, and consist of 3 main elements, the terrain itself which is to be created, sea-defence works which must protect the terrain against inundation and erosion, and a breakwater, which must reduce the height of the incoming waves from the North sea to an acceptable level in order to give access for shipping to the harbour under storm conditions and limit downtime of the harbour activities.

For the phased execution of Maasvlakte 2, sections of the newly gained terrain must be protected against the sea by a breakwater during each phase. As breakwaters are very expensive structures, a flexible breakwater, a breakwater which can be reused several times, might be economical. Caissons are an ideal alternative for such a breakwater as these can be brought afloat again and repositioned at a new location.

The caisson dimensions have been determined using the Goda design formulas to calculate the wave forces on vertical walls and the wave transmission over the caisson, and the formulas of Brinch-Hanssen to calculate the bearing capacity of the soil layers. These formulas have been implemented in the computer program '**O**uter **C**aisson **D**imensions', (O.C.D.) written in this study. This program indicates that for the conditions of the future Maasvlakte 2 breakwater site, slip of the subsoil is the decisive failure mode. In order to prevent the occurrence of this failure mode, the caisson must have a width of at least **21.1** m. In order to sufficiently reduce the wave transmission of the crest the construction height of the caisson must be **23,5** m.

The outer walls have a construction thickness of 0.70 m and are prestressed to absorb the outward ground pressure on the shaft. The inner walls have a construction thickness of 0.50 m, and are not prestressed. The caisson consists of 4 cells in width direction and 15 cells in length direction. These cells will have an internal length and width of **4.55 m.**

Construction of the caissons will be on a floating construction yard moored in the Europe Harbour of the Maasvlakte 1 equipped with slip formwork. The main advantage of the floating caisson construction method is that there is no need of an expensive construction dock or specialised lifting equipment.

When the caisson is completed, it is moored at a temporary location where it is trimmed for stability and prepared for transport to the breakwater site. The caissons are towed to the breakwater construction site by tugs, and are lowered onto the foundation by flooding the cells.

Finally the cells are filled with sand, concrete capping plates are placed, and the rocks of the rubble mound bottom protection are placed. The caissons can be considered as building blocks, easily reusable components of the Maasvlakte 2 breakwater.

For caisson reuse, the capping plates must be removed and the sand content of the cells replaced with ballast water. When the caisson is ready to be transported, the water is pumped from the cells and the caissons become buoyant. They can now be transported to their new destination. The costs to reuse the

caisson are relatively low, and therefore the costs of the breakwater for construction phase 2 of Maasvlakte 2 are also relatively low.

Total costs/m[1] of this reusable caisson breakwater are (based on a 4.0 km long section):

activity:	Floating Construction Yard Method		
	costs in Nlg. /m	**costs for a 4km long section [in Nlg.·10^6]**	**percentage [%]**
material costs caisson (section 7.7)	46.455,-	185.9	33
labour costs (working full time) (section 7.7)	34.600,-	138.5	24
construction yard costs (section 8.4)	5.000,-	20.0	4
construction costs foundation (phase 1)	50.600,-	202.4	35
transport and placement costs of caissons (section 10.2)	5.200,-	20.8	4
Total costs construction phase 1:	**141.900,-**	**568**	**100**
costs construction phase 2: (section 11.4)			
reuse of caissons:	3.500,-	14.0	6
new foundation	57.100,-	228.4	94
Total costs construction phase 2:	**60.600,-**	**242.4**	**100**

Table 1 Overview breakwater costs Floating construction yard method[1].

The construction method of caissons on a floating construction yard is technically feasible and financially competitive with other construction methods, such as the Dutch traditional construction method in a dock or the Japanese construction method on a yard located above the ground water level, with use of heavy lifting equipment. By designing the caisson in such a manner that it is capable to float on its own buoyancy, it is an ideal solution to form the components of a reusable breakwater.

By construction of only a section of the Maasvlakte 2 terrain (approximately 50%) the investment in the year 2010 would be 3023 million, approximately Nlg. 1200 million less than required for the construction of the complete MV2. This money can be spent on other (money generating) projects and may lead to a lower threshold for the decision makers of the MV2 to invest on this project.

The investment which can be saved by a breakwater construction of reusable caissons opposed to construction of a new breakwater is Nlg. 160 million (based on the reuse of a 4 km breakwater section).

Another important advantage of a phased execution concerns the flexibility. If there is no need for enlargement of MV2 in 2030, this will not be done. If a terrain which is twice as large as predicted is required, this is also possible. In this manner the chance that money will be invested in a wrong project is limited.

[1] It must be noted that the accuracy of the values in this table serves to find back the essence of the value in the rest of the text. The costs are not as accurate as might be suggested by the values presented in this table.

1. Layout of this report

1.1 Reading guide

Section I (see Flowchart 1.2) of this report is meant as a reading guide of this thesis. It contains an overview of the layout of this study, an introduction to the Maasvlakte 2 project, the problem definition, and the aim of this study.

Section II contains the terms of reference and presents the required design parameters. These consist of the project area boundary conditions, the demands of the design as stated by the Mainport Rotterdam and assumptions.

In section III the caisson breakwater is dimensioned. Based on the forces which are exerted on the construction during different phases of its lifetime (construction, transport, placement, operational and reuse), the outer caisson dimensions -caisson length, width and height-, and the concrete and steel dimensions of the caisson base slab and shaft are determined.

Section IV analyses the construction method and costs of the caisson breakwater. The construction process of the foundation and the floating construction process of the caissons on a floating construction yard, transport to the breakwater site, placement and completion works are analysed. Also the reuse procedure and costs of the breakwater will be discussed.

In the final part of the study, section V, the breakwater design and construction process on a floating construction yard is evaluated on technical feasibility and economical competitiveness with other construction methods; construction in a dock and the 'Japanese construction method'. Finally the economical advantages of a phased execution of Maasvlakte 2 are discussed.

The appendices contain the information on which the boundary conditions, the demands of the design and the assumptions are based. Also the formulas of the calculation methods applied to determine the caisson dimensions are presented in the appendices. Some of the appendices can be read as background information, and are not essential to understand this report.

1.2 Flowchart of this report

The layout of this report is schematically presented in the following flowchart:

Overview: layout of this report

I. Introduction
- layout of this report (Ch. 1.)
- Introduction of the Maasvlakte 2 project (Ch. 2.)
- Problem definition
- Aim of thesis

↓

II. Terms of reference
- Boundary conditions of the project area (Ch. 3.)
- Demands of the design as stated by the Mainport Rotterdam (Ch. 4.)
- Assumptions made for the design (Ch. 5.)

↓

III. Design of the caisson breakwater
- Outer dimensions of the caisson (Ch. 6.)
- Concrete and steel dimensions of the caisson (Ch. 7.)

↓

IV. Construction of the caisson breakwater
- Floating construction method of caissons (Ch. 8.)
- Construction of the rubble mound foundation (Ch. 9.)
- Transport and placement of the caisson (Ch. 10.)
- Reuse of the caisson (Ch. 11.)

↓

V. Evaluation
- Cost comparison different construction methods (Ch. 12.)
  - Floating construction method
  - Japanese construction method
  - Construction in Dock method
- Phased construction of Maasvlakte 2, economical perspective (Ch. 13.)
  - Costs construction in 1 phase
  - Costs construction in 2 phases
- Conclusions and recommendations (Ch. 14.)
  - Elements of risk

Appendices
- (A. to N.)

Figure1.1. Overview layout of this report.

2. Introduction

2.1 General

Due to its excellent geographical location and its modern facilities, the Port of Rotterdam has a leading position in the world trade. In order to maintain this position, the port must compete with the rapid development of other European ports, and investments on the harbour facilities must constantly be made. A problem the harbour faces during the next decades, will be the lack of area to expand.

A possible solution to create the necessary area for the harbour and industrial terrain is a typical Dutch solution, reclamation of land from the sea by expanding Maasvlakte 1 with Maasvlakte 2. Due to uncertainties regarding the required area, an alternative is to execute the project in several phases.

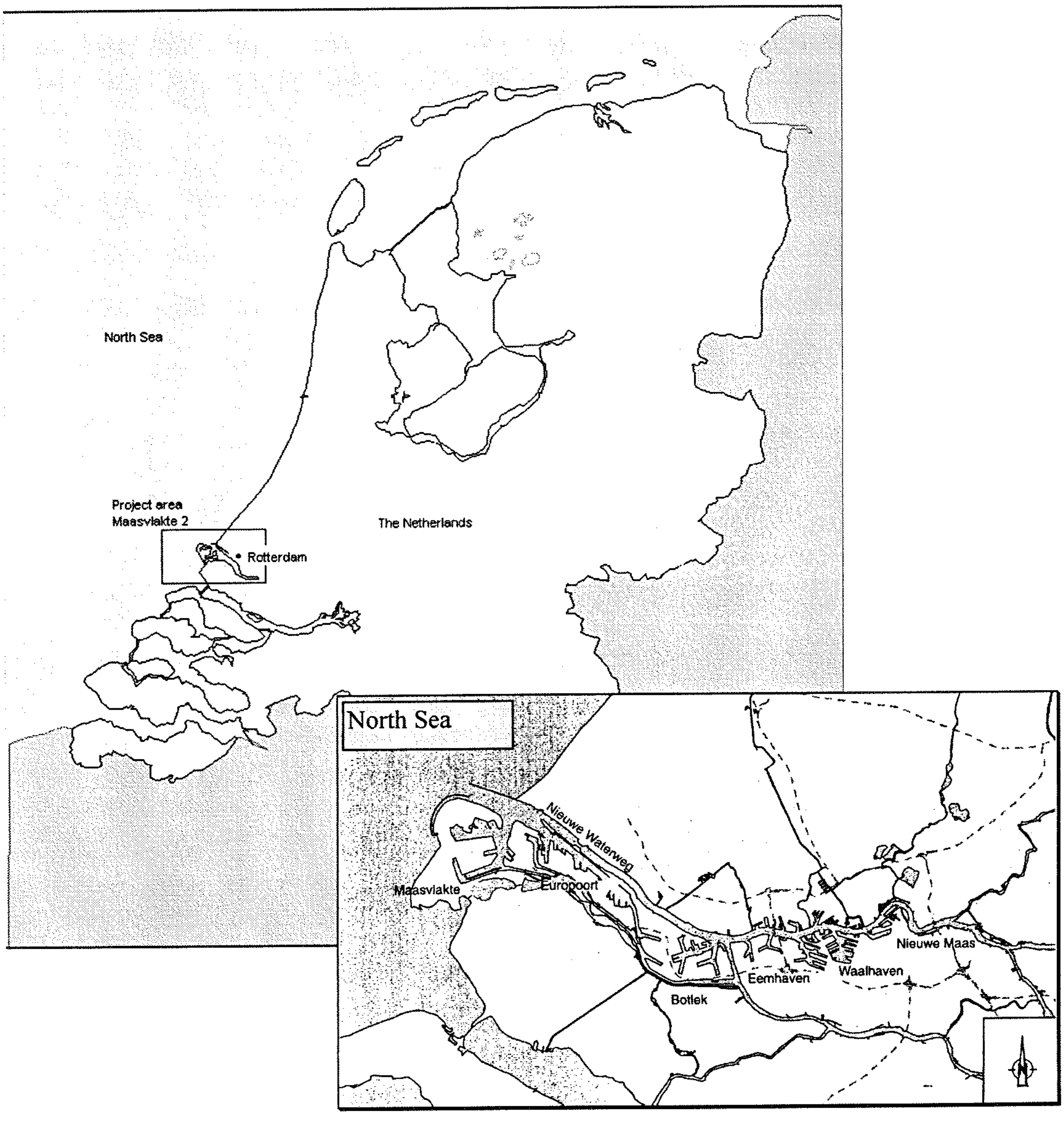


Figure 2.1 Illustration of the project area.

The construction works of the Maasvlakte 2 are very extensive, and consist of 3 main elements. The terrain itself which is to be created, sea-defence works which must protect the terrain against inundation and erosion, and a breakwater, which must reduce the incoming waves from the North sea to an acceptable level in order to give access for shipping to the harbour under storm conditions and limit downtime of the harbour activities.

As the precise contour of the Maasvlakte 2 is not known at this moment of time, the breakwater site can not be exactly specified yet. Therefore a breakwater must be designed which is independent of the exact Maasvlakte 2 contour lines. Location studies of the Maasvlakte 2 so far indicate that by the year 2010 an area of approximately 10 km^2 must be gained, which is to be extended by an additional area of 10 km^2 by the year 2030. Part of this contour will have to be protected by fixed shore protection works, in the form of a breakwater.
During each construction phase of the Maasvlakte 2 the terrain and harbour area must be protected by shore protection works. The phased construction implies that a new shore protection works must be built for each phase. Considering the high costs of a breakwater, it might be advantageous to design a reusable breakwater which can be positioned first on the breakwater site during construction phase 1, and can be easily reused at a new location during a later construction phase.

An alternative for the Maasvlakte 2 breakwater is a caisson breakwater, a type of breakwater which is suited for construction in deep water. By designing the caisson breakwater in such a manner that it can be reused in the future without major difficulties at a new location, the breakwater construction costs can be limited. Appendix F: 'Structural types of breakwaters' and Appendix G: 'Caisson shapes' contain an overview of the different types of breakwaters and the different types of caisson shapes.

The traditional caisson construction method in the Netherlands is by means of a construction dock. A construction dock with sufficient capacity to construct the large number of required caissons is not available in the proximity of the Maasvlakte 2 breakwater site, and the costs to construct a new dock are very high. An alternative which has been proposed for the construction of the caissons is the so called 'Japanese method'. In this construction method the caissons are constructed on a terrain above the ground water level from where they are lifted onto pontoons for transport. A heavy lifting vessel is used to place the caissons on the foundation. This study will focus on yet another construction method, construction of the caissons on a floating construction yard. The main advantage of this construction method is that it doesn't require the use of a large construction dock or an expensive heavy lifting vessel.

In this thesis a reusable caisson breakwater will be designed for the future Maasvlakte 2. The outer caisson dimensions width, height and length and the build up of the rubble mound foundation layers will first be focused upon. Then the concrete and steel dimensions of the caisson shaft and base slab will be dimensioned. Finally the floating construction method of caissons will be analysed. The technical feasibility of the floating construction method will be studied and the economic competitiveness with other construction methods will be analysed. An plan view of the contour is presented in Appendix A: 'Geographic boundary conditions'.

2.2 Problem definition:

In order to create the additional area required for the harbour and industrial activities of the Port of Rotterdam, an alternative is to construct the Maasvlakte 2. This land reclamation project extends into the sea, in north western direction of the Maasvlakte 1. An alternative to protect this land reclamation is by means of a caisson breakwater. As the Maasvlakte 2 is likely to be further expanded in the future, the caisson breakwater will be designed in such a way that it can be reused.
A breakwater section of 4 km is to be constructed within 4 years. To construct the large number of caissons required for this breakwater, an effective caisson construction method is essential. The traditional caisson construction method in the Netherlands is by means of a construction dock. A construction dock with sufficient capacity to construct the required number of caissons is not available in the proximity of the Maasvlakte 2 breakwater site. Therefore an alternative construction method which doesn't require the use of a large construction dock, construction of caissons on a floating construction yard, will be examined in this thesis.

2.3 Aim of thesis:

The goal of this thesis is to design a caisson breakwater which is sufficiently strong to survive the design conditions of the Maasvlakte 2 breakwater site. First the outer caisson dimensions will be determined (length, width and height), and then the required concrete dimensions and the amount of steel reinforcement will be determined. The caisson must be designed in such a manner that it can be built and transported to the breakwater site making use of its own buoyancy, and so that it can be reused after a time period of 20 years without any major problems or costs.

The floating construction method of caissons will be examined on technical feasibility and economic competitiveness with other construction methods. Possible problems of the construction process will be indicated and inventoried.
Finally an economic comparison will be made of the construction costs of the complete Maasvlakte in one phase, and the construction costs of the Maasvlakte 2 in 2 phases.

3. Boundary conditions of project area:

3.1 Introduction

This section focuses on the boundary conditions of the project area which are required for the design of the caisson breakwater. The following boundary conditions will be analysed in this section:

- Geographic;
- Geologic;
- Hydraulic;

The boundary conditions are presented as short and to the point as possible. The appendixes A, B and C contain the theoretical backgrounds and calculations. These will be referred to in the text.

3.2 Geographic boundary conditions

Water depth:
The exact contour of the Maasvlakte 2 is not exactly known at this moment in time (see Appendix N: 'Maasvlakte 2 project'). This study assumes that the breakwater contour will be in the project area indicated in Appendix A: 'Geographical boundary conditions'. The water depth of this breakwater site varies from NAP -17.3 m to NAP -22.0 m. The design of the caisson breakwater of this study will assume a bottom level of NAP -18.0 m.

As demanded by the Mainport Rotterdam, the Maasvlakte must be constructed in 2 phases: construction phase 1 must create an area of 1000 ha. harbour, industrial and recreational terrain, and construction phase 2 must create an additional 1000 ha. of similar terrain. Appendix A contains a plan view of the project area and the layout of the harbour during the construction phases.

The layout of Maasvlakte 2 is dependant of many aspects. The design must meet the following demands:
- nautical, these determine the dimensions of the access channels and harbour basins;
- morphological, these determine acceptable contours of the terrain with respect to erosion and sedimentation of the surrounding coastal area;
- environmental, the design must be fit into the natural ecosystem as well as possible,
- infrastructural, the design must fit with the infrastructure of the Maasvlakte 1;
- economical, the design must be able to be financed;
- hydraulic, wave penetration into the harbour basin must be minimised (seiches);
- technical;

It is beyond the scope of this study to take all these aspects into account. Therefore the design presented in appendix A must be interpreted as merely an indication of the harbour contour and its layout. The aspects which have been taken into account are:

- construction of the Maasvlakte 2 in 2 phases of 10 km^2;
- maintaining the current harbour entrance;
- a contour which fits into the northern contour alternative as defined by the project group Maasvlakte 2 (see Appendix N: 'Maasvlakte 2 project');

The plan views presented in Appendix A serve to indicate the aspect of the phased construction of the Maasvlakte, and illustrate how a reusable caisson breakwater fits into this design. Appendix N: 'Maasvlakte 2 project' presents an insight to some of the other demands mentioned above in more detail, and can be read as background information.

3.3 Geologic boundary conditions

The only available information about the geological boundary conditions of the breakwater site are measurements which have been taken from a sounding taken at the Europe Harbour in Rotterdam. These conditions are assumed to be representative for the total breakwater site. Local trenches due to ancient river beds will not be taken into account in this phase of the design. In Appendix B: 'Geologic boundary conditions' the sounding itself is presented, and based on the average cone resistance and the average friction number, the depths and quality of the different soil layers are determined. For each soil layer the specific weight of the ground γ_{sat} is determined, the angle of internal friction φ', and the cohesion value c'. The results are presented here:

	depth of layer	material	average cone resistance [MPa]	average friction number [%]	γ_{sat} [kN/m^3]	φ' [°]	c' [kPa]
layer 1	from harbour bottom to NAP-24.0m	weak sandy clay	1.5	2	18	22.5	10
layer 2	from NAP-24.0m to NAP-32.0m	sand	24	0.8	21	37.5	0
layer 3	below NAP-32.0m	sand	30	0.8	21	40.0	0

Table 3.1. Overview geologic boundary conditions.

The top ground layer consists of weak sandy clay and the second and third layer consist of clean sand. The top layer of sandy clay possesses so little bearing capacity that it will be excavated and replaced with clean sand of similar consistence as the second soil layer. In this study the angle of internal friction of the second soil layer will be maintained, φ' = 37.5°, cohesion will be assumed c'=0 kPa and γ_{sat} =21 kN/m^2.

3.4 Hydraulic boundary conditions

The hydraulic boundary conditions presented in this section are based on the documents 'Tide levels of The Netherlands' and 'Information bulletin 6' of the project group Maasvlakte 2. In Appendix C: 'Hydraulic boundary conditions' the values are determined for the water levels, wave heights, wave periods, wave lengths and currents. The results are presented in Table 3.2.

The following average water levels, wave heights and currents will be maintained in the calculations:

Aspect	**level**
Water level due to tide	
Grenspeil[1]	NAP + 2.60 m
Average high tide	NAP + 1.11 m
Mean sea level	NAP + 0.07 m
Average low tide	NAP - 0.63 m
Low lower water neap-tide	NAP - 0.84 m
Design condition water levels	
high water level (1/ year)	NAP + 2.45 m
high water level (1/1000 years)	NAP + 4.40 m
low water level (1/100 years)	NAP - 1.90 m
low water level (1/1000 years)	NAP - 2.05 m
sea level rise (every 100 years)[2]	+ 0.50 m
Design condition wave heights	
wave height for **deep water** (1/ year)	H_s = 5.00 m
wave height for **deep water** (1/1000 years)	H_s = 7.89 m
maximum wave steepness	s = 3,5 %
Wave periods serviceability limit state (1/year)	
zero crossing period	T_z = 6.7 s

[1] Grenspeil is the local water level which in average is reached or exceeded once every 2 years.
[2] According to the IPCC (International Panel on Climate Change) the expected sea level rise due to global warming until the year 2100 is 0.50 m.

peak period significant period	$T_p = 9.6$ s $T_s = 9.1$ s
Wave periods ultimate limit state (1/1000 years) zero crossing period peak period significant period	 $T_z = 8.4$ s $T_p = 12.0$ s $T_s = 11.4$ s
Longshore currents north western direction	 1.5 m/s
Density of sea water	$\rho_{seawater} = 1030$ kg/m^3

Table 3.2. Overview hydraulic boundary conditions at Hook of Holland.

3.5 Conclusion

The boundary conditions discussed in the previous section will be maintained in the further calculations of this study. A note must be made of the fact that the values are based on very few measurements, and in a later phase of the design these must be studied more intensively. Where there is insufficient information about the values of the boundary conditions, assumptions will be made. These are discussed in the following section.

4. Design criteria demanded by the Mainport Rotterdam

4.1 Introduction

The demands of the design which have been stated by the Mainport Rotterdam are presented in this section. These demands concern aspects of the functions of the breakwater and the design conditions under which these functions must be fulfilled. Also the constructional demands of the caisson breakwater design are stated.

4.2 Functions of the breakwater

The function which must be fulfilled by the breakwater is to protect vessels moored to the quay located directly behind the breakwater during transhipment of goods (limit downtime of harbour activities). The largest vessels which will be moored to the quay are container vessels with a length of 290 m, a beam of 39.40 m, and a draught of 13.0 m with a minimum keel clearance of 1.0 m. The design water level maintained for sufficient keel clearance is $LWL_{1/100}$ = NAP - 1.90m, which leads to a minimum bottom level of the harbour basin of NAP-15.9m. The acceptable wave conditions in the harbour under which transhipment of containers may continue is H_T = 0.20 m.

Appendix D: 'Ship dimensions and acceptable wave transmission' contains an overview of the types of ships which are to be moored behind the breakwater, their dimensions and the acceptable wave conditions under which transhipment may continue. Figure 4.1 illustrates the function which the caisson breakwater must fulfil.

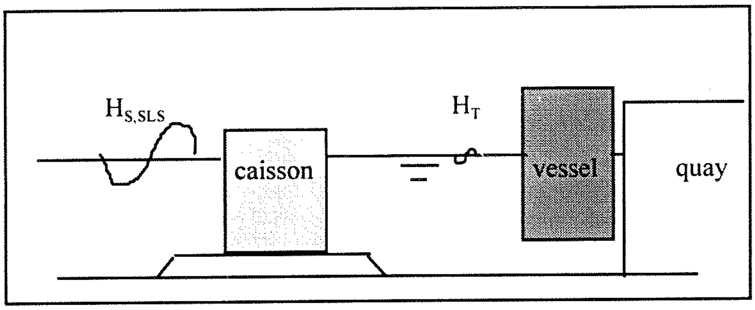


Figure 4.1. Function of the caisson breakwater.

The following assumptions will be further maintained in the design of the breakwater:

- H_T = 0.20 m.
- d_{basin} = NAP - 15.9 m.
- bottom level = NAP -18.0 m

4.3 Design conditions (limit states)

The breakwater design must be based on the following design criteria:

- The design period is 100 years;
- The design storm for Serviceability Limit State (SLS) is the storm with a return period of 1 year;
- The design storm for Ultimate Limit State (ULS) is the storm with a return period of 1000 years;
- The design level for which sufficient keel clearance must be available for all ships in the harbour basin is LWL with a return period of 100 years;
- Based on a bottom level NAP - 18.0 m for the caisson breakwater (see section 3.2 'Geographic boundary conditions'), the local significant wave height[1] $H_{s,SLS}$ = 4.57m, and the local significant wave height $H_{s,ULS}$ = 7.43m.

[1] $H_s=H_{s,0}\cdot K_S\cdot K_R$ with K_s is shoaling factor and K_R is the refraction factor. H_s represents the local wave height at the breakwater site (local bottom level is NAP-18.0m).

4.4 Length profile of the breakwater and caisson length

The length of the reusable breakwater section which must be constructed for the Maasvlakte 2 is 4.000 m. Appendix N: 'Maasvlakte 2 project', discusses the origin of this value. Caissons built until now usually have a length which is approximately 2 to 3 times the caisson width. Assuming a caisson width of approximately 25 m, the caisson length will be 50 to 75m. In this stage of the design a caisson length of 75 m will be maintained, in a later stage of the design the caisson length can be optimised. Under the assumption of a caisson length of 75 m, 54 caissons are required for the first construction phase of the Maasvlakte 2.

- $L_{breakwater}$ = 4.000 m;
- $L_{caisson}$ = 75 m;
- 54 caissons required;

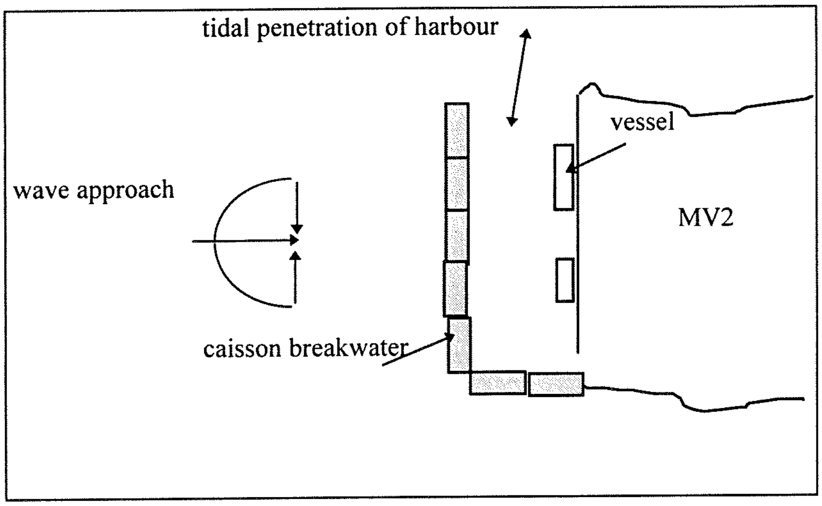


Figure 4.2. Plan view of the project area.

4.5 Bottom level of project area during construction phase 1

The contour of the caisson breakwater for the Maasvlakte 2 is at a depth of NAP-18.0m for phase 1 and NAP -20.0 m for phase 2.

- bottom level phase 1 = NAP -18.0m.
- bottom level phase 2 = NAP -20.0m.

4.6 Available construction time for the breakwater

The maximum construction time which is available for the construction of the breakwater is 4 years. Appendix N: 'Maasvlakte 2 project' discusses the origin of this value.

- $T_{construction}$ = 4 years;

4.7 Reusable caissons

With respect to the phased construction of Maasvlakte 2 the breakwater caissons must be reusable without many problems or costs. This means that once the caissons have been operational for a time period of several years, it must be possible to empty them of their ballast material, bring them afloat under controlled conditions, transport them to their new location floating on their own buoyancy, and reposition them at the new breakwater site.

- Caisson must be reusable;

4.8 Conclusion

The Mainport Rotterdam states the following demands concerning the caisson breakwater for Maasvlakte 2:

• $H_T = 0.20$ m (SLS)	Acceptable maximum wave height in harbour basin, return period 1/year;
• d_{basin} = NAP - 15.9 m	Minimum required bottom level of harbour basin;
• $H_{s,SLS} = 4.57$m	Local significant wave height with return period of one year;
• $H_{s,ULS} = 7.43$m	Local significant wave height with return period once every 1000 years;
• Design period = 100 years	Time period for which the breakwater must fulfil its function;
• P(SLS) = 1/year	The design storm for serviceability limit state is the storm with a return period of one year;
• P(ULS) = 0.001/year	The design storm for ultimate limit state is the storm with a return period of 1000 years;
• P(insufficient keel clearance) = 0.01	Design level for which sufficient keel clearance must be maintained for all ships in the harbour basin is the lower-low-water level with a return period of 100 years;
• $L_{breakwater}$ = 4.000 m	Total length of the caisson breakwater;
• b.l.$_{phase\ 1}$ = NAP -18.0m. • b.l.$_{phase\ 2}$ = NAP -20.0m.	Design bottom level during construction phase 1 of the Maasvlakte 2; Design bottom level during construction phase 2 of the Maasvlakte 2;
• $T_{construction}$ = 4 years	Available construction time for breakwater;
• Reusable caisson	The caissons must be able to be reused after a time period of several years, without major problems or costs;

Table 4.1. Overview design criteria stated by the Mainport Rotterdam.

The demands stated by the Mainport Rotterdam and the boundary conditions of the project area are the main input parameters of the design. Aspects which are not covered by these demands or boundary conditions, and which are essential for the breakwater design are taken up in assumptions. These are discussed in the next section.

5. Further assumptions made for the design

5.1 Introduction

Aspects required for the design of the Maasvlakte 2 caisson breakwater which have not been covered by the boundary conditions and the demands of the Mainport Rotterdam are presented in this section. A cross section of the Maasvlakte 2 caisson breakwater is presented Appendix M: 'Technical drawings'.

5.2 Assumptions of the subsoil

The geographic boundary conditions indicate the water depth of the project area, NAP-18.0m, is larger than the demand stated by the mainport, d_{bassin}= NAP-16.9 m. Therefore a bottom level of NAP-18.0m will be maintained in this study. As stated in the geologic boundary conditions the top 6 meters of soil, from NAP -18.0m to NAP -24.0m consist of sandy clay with little bearing capacity. This design assumes that the layer of sandy clay will be excavated and replaced by sand with a similar consistence to the sand layer below NAP -24.0m. If necessary, measures as compacting of the soil will be taken to achieve the required ground consistency. In this phase of the design the following values are assumed for the subsoil (see Appendix B: 'Geologic boundary conditions', Table B.2.):

- angle of internal friction: $\varphi_{sub} = 37.5\ °$
- cohesion of sand: $c_{sub} = 0$ kPa
- density of submerged sand: $\rho_{sub,submerged} = 11$ kN/m^3
- grain size of sand: 300μ (0.3 mm)

In section 9. the process of the sand improvement and compacting of the subsoil is discussed.

5.3 Assumptions of the rubble mound foundation

In average the costs of 1.0m construction height of rubble mound are higher than the costs of 1.0m construction height of caisson. Therefore this study assumes that it is most economic to construct the rubble mound as low as possible with a high caisson.

The design of the rubble mound foundation is based on the rules for geometric filters. A basic rule of the thumb is that the proportion D_{50} of one layer to the next is not larger than 1:5. Another rule is that the thickness of a layer must be at least $2D_n$. The values of the standard stone gradings are presented in Appendix E: 'Standard stone gradings'.

The foundation will be built up of the following layers:

function:	stone dimensions:	layer thickness[1]:
subsoil	0.3 mm	6m
filter layer a.	0-60 mm	1.0m
filter layer b.	80-200 mm	0.5 m
core	60-300 kg	1.0 m
levelling layer a.	10-60 kg	0.5 m
levelling layer b.[2]	30-80 mm	0.25 m
reverse filter layer a.	10-60 kg	0.5 m
reverse filter layer b.	60-300 kg	1.0 m
bottom protection sea-side	6-10 T	3.0 m
bottom protection harbour side	1-3 T	1.8 m

Table 5.1. Foundation layers.

[1] Note that the layer thickness presented here is a theoretical value and from a practical point of view may be designed thicker.
[2] For the largest part this levelling layer will fall into the openings of levelling layer a.

The function of the filter layers is to prevent the soil improvement to wash out through the core material and to prevent liquefaction of the subsoil due to large wave pressures. The levelling layers are meant to provide a sufficient smooth surface on which the caisson can be placed without danger of rock penetration through the caisson base slab. The function of the reverse filter is to prevent the levelling layers to be washed out through the bottom protection from underneath the caisson, which can lead to piping and induce instability. The bottom protection must extend at least ¼ of the wave length, 25 m, seaward of the vertical wall in order to prevent erosion of the sea bottom due to large wave velocities caused by standing waves.

The total thickness of the foundation construction is 3.25 m, and the angle of internal friction of rubble mound is assumed to be $\varphi_{rm} = 40°$. The density of submerged stone is $\gamma_{submerged} = 16.5$ kN/m^3. Goda [lit. 8] advises a value $\mu = 0.6$ for the friction coefficient between the concrete base slab and the rubble mound foundation.

Figure 5.1 contains a sketch of the cross section of the caisson breakwater. Appendix M: 'Technical drawings' contains a technical drawing of the rubble mound foundation and the caisson.

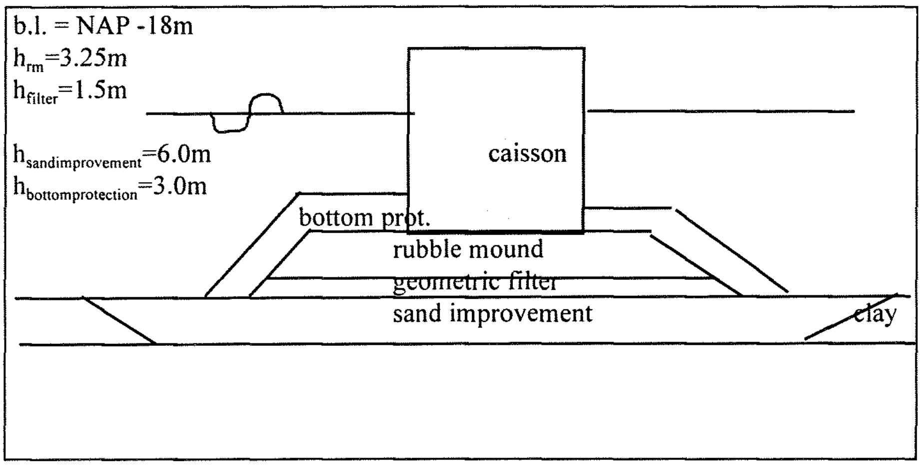


Figure 5.1. Cross section of the caisson breakwater design.

There are several alternatives for the construction of the rubble mound foundation. Alternatives of a geometric filter are so called 'zinkstukken', or prefabricated filter mats, which are lowered onto the sea bottom from vessels as was done for the Eastern Scheldt works. These alternatives will not be further analysed in this study.

The construction process and a price estimation of the foundation construction are discussed in section 9. 'Construction of the rubble mound foundation'.

5.4 General assumptions

- In this study a simple rectangular cross section of the caisson is maintained. A note is made that there are many alternative caisson shapes. Instead of a closed vertical wall also caissons with sloping walls or with slits in the wall to absorb the waves can be used. An overview of different caisson shapes is presented in Appendix G: 'Caisson shapes'.
- Sufficient penetration of the tide can occur through the harbour entrance so that head level difference between the sea side and the harbour side of the caisson does not need to be taken into account in the design criteria;
- The incidence angle of the waves for which the caissons must be designed is perpendicular wave approach;
- The weather conditions under which construction of the breakwater will continue (transport and placement on the foundation) will be such that the maximum wave length $L << L_{caisson}$ and $H_s \leq$ 1.0m;
- Sufficient building materials are available (gravel, quarry run, concrete, steel, etc.);

- Sufficient sand is available of good quality, however it must be taken into account that there are also other sand-consuming projects: regular beach nourishment along the Dutch coast and projects as an airport located on a (reclaimed) island in sea are under discussion;
- The effects on ground water levels and -currents resulting from the construction of Maasvlakte 2 on Maasvlakte 1 are not taken into account;
- Morphological effects as erosion and sedimentation which result from the breakwater construction will not be taken into account in this phase of the design;
- Nautical aspects will be taken in account in less extent, an important aspect which is taken into account concerns the minimum required length of the breakwater to provide calm water for connection of tugs to incoming vessels (3500m);
- Hydraulic aspects as wave penetration, tide penetration and currents in the harbour basin due to the layout of the breakwater and basin are beyond the scope of this study and are not taken into account;
- In this phase of the design it is assumed that the density of a caisson is $\rho_{caisson}$=21.0 kN/m^3, this is based on a caisson which consists of 25% concrete and 75% ballast sand;

5.7 Conclusion

Together with the boundary conditions and demands of the Mainport Rotterdam, the assumptions form the program of requirements of the Maasvlakte 2 caisson breakwater.

Based on these input parameters the following dimensions of the Maasvlakte 2 caisson breakwater will be determined:

- required height and width of the caisson;
- required concrete and steel dimensions of the base slab and shaft;
- construction method of the breakwater (foundation and caisson);

6. Outer dimensions of the caisson, functional requirements

6.1 Introduction

The outer dimensions of the caisson determined in this section are:

- Caisson height;
- Caisson width;
- Caisson length;

These dimensions depend on many input parameters, which have been presented in the Terms of Reference (TOR, sections 3-5). The Goda design formulas for vertical breakwaters have been applied to calculate the wave forces on the caisson, the formulas of Brinch-Hanssen have been applied to calculate the bearing capacity of the rubble mound and subsoil. In order to calculate the outer caisson dimensions, the computer program OCD (**O**uter **C**aisson **D**imensions) has been written in this study. The calculation method is based on the calculation method discussed in this section. The program is presented in Appendix J: 'EXCEL computer model OCD'. The results of this model are presented in this section.

6.2 Caisson height

The height of the caisson is determined by the acceptable wave transmission over the crest and through the rubble mound foundation. The Mainport Rotterdam has set H_T= 0.20m as maximum acceptable wave transmission for storm conditions with a return period of 1 year (SLS). The local wave height under these design conditions is $H_{s,SLS}$=4.57m.

The required crest elevation above the water level h_c=5.66m is calculated with the transmission model of Goda, (see Appendix I: 'Transmission model of Goda'). The water level for these design conditions is NAP+2.45m. The sea level rise of the next 100 years (0.50m) must also be taken into account.

This gives:

h_{crest} = sea level rise [m] + HWL_{SLS} [NAP + m] + h_c [m]
= 0.50m + NAP+2.45m + 5.66 = NAP + 8.61m

The top of the rubble mound foundation is at NAP-14.75m. As the required crest height is NAP+8.61m the total construction height of the caisson is 23.36m.

h_{cais} = NAP -h' [m] + NAP + h_{crest} [m]
= 14.75m + 8.61m = 23.36 m

In Figure 6.1 the cross section of the breakwater has been schematised.

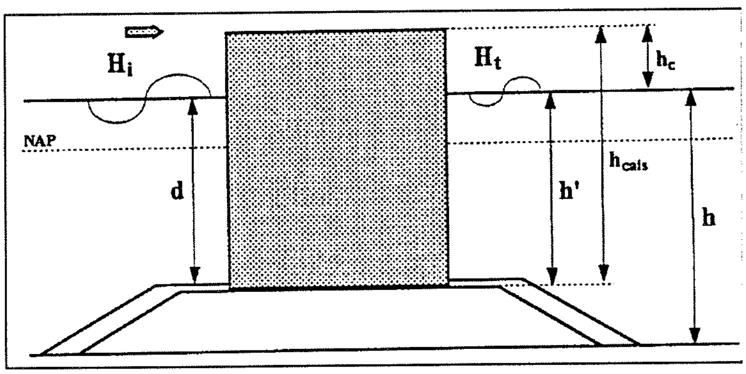


Figure 6.1 Schematised transmission according to Goda.

6.3 Caisson width

The width of the caisson must be in such a way that the following failure modes will not occur:

- Sliding;
- Overturning;
- Circular slip of the rubble mound foundation;
- Circular slip of the subsoil;

Figure 6.2 illustrates the principle failure modes of a caisson on a rubble mound foundation. The breakwater must be capable to absorb the forces exerted on it under ultimate limit state conditions (ULS):

- HWL_{ULS}= NAP+4.40m,
- $H_D=1.8{\cdot}H_s=13.37$m (Goda design wave[1])

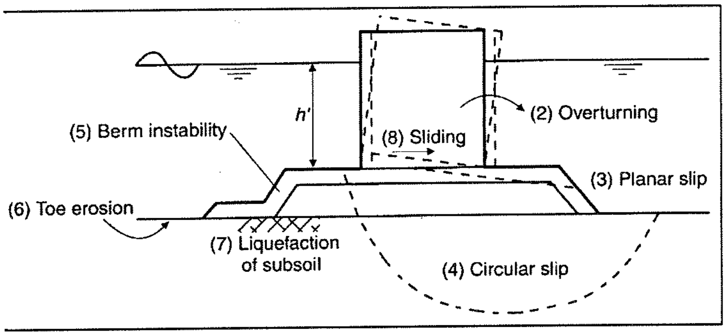


Figure 6.2 Failure mechanisms of a caisson breakwater[lit.3].

In this phase of the design it is assumed that the design of the geometric filter and bottom protection of the rubble mound foundation are in such a way that the remaining failure mechanisms -berm instability, toe erosion and liquefaction of the subsoil- will not occur.

In the next section the calculation method maintained for each of these failure modes is presented. As each failure mode results in a different value for the required caisson width, which is dependent of many factors, the actual calculations are done in the computer model OCD (see Appendix J: 'EXCEL computer model OCD'). The results are presented in this section.

[1] Many caissons are constructed with the Goda design formulas. These formulas make use of several coefficients to calculate, amongst others, wave forces due to impulsive wave pressures. Currently there are no theoretical models available to calculate the forces of breaking waves on a vertical wall. Goda assumes there is little danger of breaking waves if the bottom slope i_b is lower than 1:50 (MV2 situation $i_b \approx$ 1: 300. The heighest wave in the design sea state is to be employed. Its height is taken as $H_D = 1.8\text{x}\ H_{s,ULS}$.

6.3.1 Forces on the construction

Figure 6.3 presents a definition sketch of the total wave pressure, uplift force and their moments under a wave crest as defined by the Goda design formulas for vertical breakwaters.

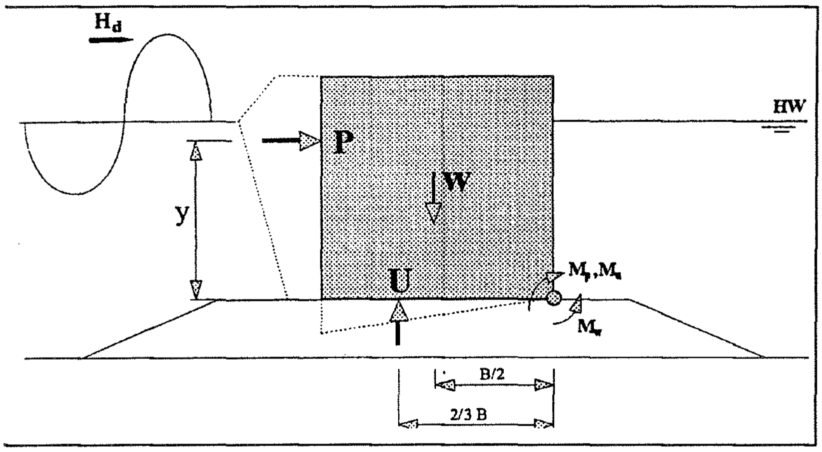


Figure 6.3 Definition sketch of total wave pressure, uplift force

and their moments under a wave crest.

P	[kN]	: horizontal wave force on the caisson per unit extension
V = W - U	[kN]	: vertical wave force on the caisson per unit extension

with:
U is the total uplift pressure caused by waves
W is the underwater weight of the caisson per unit extension in still water.

In this phase of the calculations specific weight of the upright section is taken to be $\rho_{subm} = 11$ kN/m^3 (concrete caisson filled with sand) for the submerged portion, and $\rho = 21$ kN/m^3 for the section of the caisson which is above the still water level. The total horizontal and vertical wave forces P and U on the caisson are determined with the Goda formulas, presented in Appendix H: 'Goda design formulas for vertical breakwaters'.

The horizontal wave pressure is independent of the caisson width, and for the defined design conditions P = 2500 kN per unit length extension and $M_p = 36.6$ MNm per unit length extension will be maintained in the calculations.

The vertical wave pressure U and the own weight of the caisson W are dependant of the caisson width. Therefore these values will not be presented for each failure mode individually, but for the caisson width which attains a sufficiently high safety factor for all failure modes. To give an indication of the required caisson width (B) for each failure mode, the value B will be presented for each failure mode.

6.3.2 Sliding

Sliding of the caisson occurs when the horizontal force on the caisson is larger than the maximum friction force between the caisson bottom and the rubble mound foundation.

Sliding stability criterion: $\mu\,(W-U) \geq P$

According to Goda the angle of friction μ between the concrete caisson and rubble mound foundation can be taken as 0.6.

The required caisson width to achieve sufficient stability against sliding is B = 16.99m for the defined design conditions.

6.3.3 Overturning

Overturning of the caisson will occur when there is no moment equilibrium. The rotation of the caisson due to instability of the underground is an important failure mode. Due to eccentric loading of the foundation, the underground can slip causing the construction to overturn.

The pivot point of the tipping caisson is the heel of the caisson.

Tipping moments caused by the wave pressure:

$$M_P = P \cdot y$$
$$M_U = U \cdot \tfrac{2}{3} \cdot B$$

The corrective tipping moment caused by the own weight of the structure:

$$M_W = W \cdot \tfrac{1}{2} \cdot B$$

Overturning Stability criterion[2]: $M_P + M_U \leq M_W$

The required caisson width to achieve sufficient stability against overturning is B = 17.63m for the defined design conditions.

6.3.4 Bearing capacity of the rubble mound foundation

The bearing capacity of the rubble mound foundation is calculated with the Brinch-Hansen model. The bearing capacity of the rubble mound is determined by the crush of the stones under the heel of the caisson. It is assumed that a trapezoidal or triangular distribution of the bearing pressure exists beneath the caisson. In order to prevent crush of the stones, the pressure $p_{caisson}$ resulting from the vertical force V must be lower than $p_{bearingcap,rm}$, the bearing capacity of the effective section B_e of the rubble mound.

Figure 6.4 shows the forces which are exerted on the rubble mound foundation by the caisson.

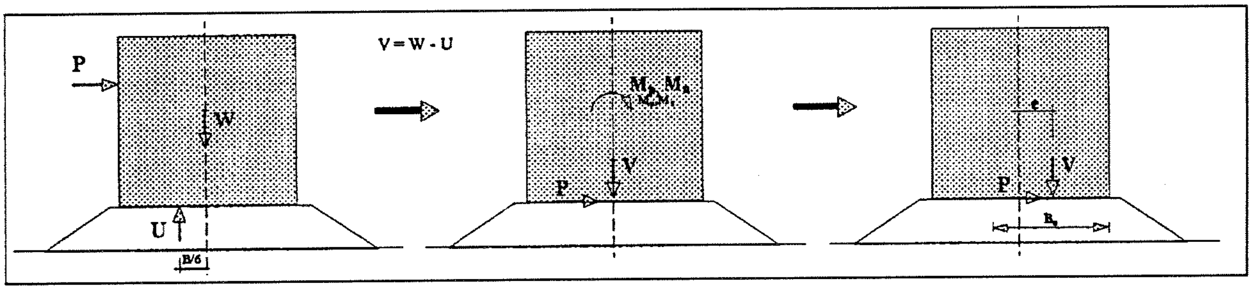


Figure 6.4 Forces on the rubble mound foundation.

According to Brinch-Hansen the bearing capacity of the rubble mound is:

$$p_{bearingcap,rm} = s_c i_c N_c c + s_q i_q N_q q + s_\gamma i_\gamma N_\gamma 0.5 \rho B_e$$

where:

c	= cohesion of the ground
q	= the loading force
ρ	= own weight of the soil
i_c, i_q, i_γ	= coefficients of the loading force direction

[2] Goda only weighs the horizontal wave force P and its' tipping moment M_P with the safety factor γ

s_c, s_q, s_γ = shape coefficients
B_e = the effective width of the foundation
N_c, N_q, N_γ = dimensionless coefficients

The average ground pressure exerted by the caisson on the rubble mound foundation is:
$p_{caisson} = V/B_e$

where:
$V = (W - U)$
$B_e = B - 2e$
$e = (M_p + M_u)/ V$
$M_u = 2/3 \cdot B \cdot U$

Sufficient bearing force of the rubble mound foundation means:
$\gamma_{bearingcap,rm} = p_{bearingcap,rm} / p_{caisson} \geq 1.2$

The required caisson width to achieve a sufficiently large value for $\gamma_{bearingcap,rm}$ is B = 20.46m for the defined design conditions.

6.3.5 Bearing capacity of the subsoil

The forces are transferred from the rubble mound foundation to the subsoil. For the total vertical force V_{sub} exerted on the subsoil, the weight of the rubble mound foundation is added to the vertical force V. The effective width of the subsoil, $B_{e,sub}$, must be capable to bear V_{sub}.

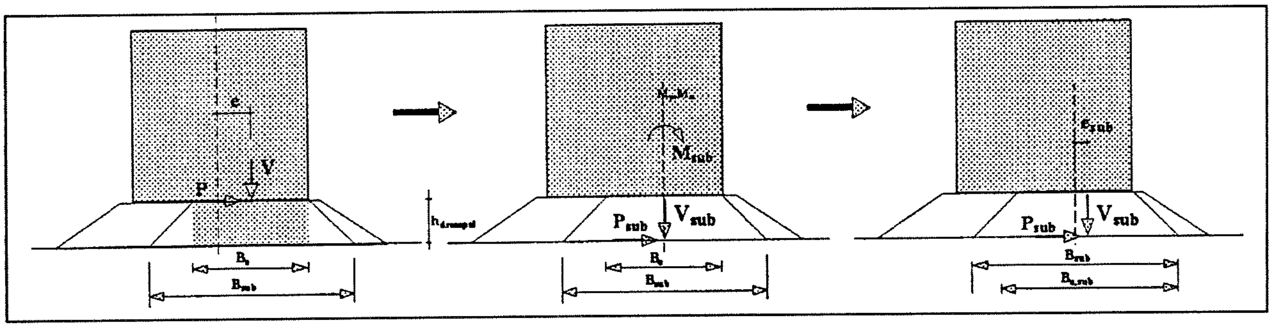


Figure 6.5 Transmission of the forces from rubble mound to the subsoil.

The coefficients from Figure 6.5 are defined as:

$V_{sub} = V + \rho_{mound} \cdot h_{mound} \cdot B_e$
$P_{sub} = P$
$M_{sub} = P \cdot h_{mound}$
$e_{sub} = M_{sub} / V_{sub}$

where:
h_{rm} = the height of the foundation mound
ρ_{mound} = the underwater weight of the rubble mound

The effective width $B_{e,sub}$ of the subsoil follows from B_{sub} and e_{sub}:

$B_{sub} = B_e + 2 \cdot h_{mound} \cdot \tan 45°$
$B_{e,sub} = B_{sub} - 2 \cdot e_{sub}$

The average ground pressure from the rubble mound foundation on the subsoil is:
$p_{rm} = V_{sub} / B_{e,sub}$

Sufficient bearing force of the rubble mound foundation means:

$\gamma_{bearingcap,sub} = p_{bearcap,sub} / p_{rm} \geq 1.2$

where:
$p_{bearcap,sub} = s_c i_c N_c c + s_q i_q N_q q + s_\gamma i_\gamma N_\gamma 0.5 \rho B_e$

The required caisson width to achieve a sufficiently large value for $\gamma_{bearingcap,sub}$ is B = 21.09m for the defined design conditions.

6.3.6 Determining failure mode

The boundary conditions and assumptions of the MV2 lead to a minimum caisson width B = 21.09 m, which is determined by the failure mode: bearing capacity of the subsoil. The safety factors for each failure mode for this caisson width are:

safety factor	formula	minimum value	calculated value
$\gamma_{sliding}$	$= \mu\ V / P$	1.20	1.49
$\gamma_{overturning}$	$= M_w / (M_p + M_u)$	1.20	1.72
$\gamma_{bearingcap,rm}$	$= p_{rm} / p_c$	1.20	1.54
$\gamma_{bearingcap,sub}$	$= p_{bear,sub} / p_{rm,sub}$	1.20	1.21

Table 6.1 Safety factors for a caisson width B = 21.09m.

6.4 Caisson length

The optimum caisson length depends on the following factors:

- Number of caissons which are to be placed;
- Control during placement procedure;
- Required caisson width (results form shape coefficients in Brinch- Hanssen model);
- Constructional possibilities of the caisson fabrication facility;

The caisson length will be determined in a later phase of the design.

6.5 Effect of different parameters on the required caisson width

The computer program OCD makes it possible to quickly analyse the influence of different parameters on the caisson dimensions. The main cost aspect of the caisson breakwater is determined by the construction width (*B*) and -height of the caisson, required to prevent failure. Therefore it is interesting to analyse the effect of different input parameters on B.

The following figure presents the input screen and the obtained results of the program OCD. The complete program is presented in Appendix J: 'EXCEL computer model OCD'.

The following relationships will be illustrated:

- Effect of the caisson width on the different safety factors;
- Effect of the angle of internal friction of the subsoil on the caisson width;
- Effect of the incidence angle of waves on the caisson width;
- Effect of the caisson length on the caisson width;
- Effect of the wave steepness on the caisson width;
- Effect of $H_{s,ULS}$ on the caisson width;
- Effect of the acceptable wave transmission H_t on the caisson height h_{caison};

Computer model Outer Caisson Dimensions, (O.C.D.)
by: S. Mann
date: 22-02-99

MODIFIED GODA CALCULATION

INPUT

	Parameter	Value	Dimension	Definition
Wave conditions	$H_{s,sls}$	**4,57**	[m]	*local wave height with return period of 1year*
	HWL_{sls}	**2,45**	[m tov NAP]	*water level with return period of 1year*
	$H_{s,uls}$	**7,43**	[m]	*local wave height with return period of 1000 years*
	HWL_{uls}	**4,40**	[m tov NAP]	*water level with return period of 1000 years*
	$H_{t,sls}$	**0,20**	[m]	*acceptable wave transmission 1/year*
	LWL_{ULS}	-2,05	[m tov NAP]	*design low water level, 1/100 years*
	s	3,50	[%]	*wave steepness*
Breakwater dimensions	b.l.	**-18,00**	[m tov NAP]	*bottom level*
	h_{rm}	**3,25**	[m]	*total rubble mound height (filter & leveling layer included)*
	B_m	5,00	[m]	*berm width*
	θ	0,00	[rad]	*incedence angle of waves*
	α	2,20	[-]	*Goda coefficient*
	L_c	75,00	[m]	*caisson length*
subsoil coef.	φ_{sub}	37,5	[deg.]	*internal angle of friction of the subsoil*
	c_{sub}	0	[kN/m^2]	*cohesion of the subsoil*
	$\rho_{sub,subm}$	11	[kN/m^3]	*underwater weight of the subsoil*
Rubble mound coef.	μ	0,60	[-]	*friction coefficient between concrete and rubble mound*
	φ_{rm}	40,0	[deg.]	*internal angle of friction of the rubble mound*
	c_{rm}	0	[kN/m^2]	*cohesion of the rubble mound*
	$\rho_{rm,subm}$	16,50	[kN/m^3]	*underwater weight of the rubble mound*
Densities	ρ_{water}	10,30	[kN/m^3]	*density of the sea water*
	$\rho_{cs,emerged}$	21,00	[kN/m^3]	*average density of the emerged caisson*
	g	9,81	[m/s^2]	*gravity*
Sea level rise	slr	0,50	[m/century]	*sea level rise*
Safety factor	γ	1,20	[-]	*minimal safety factor*

RESULTS

Outer caisson dimensions:				
Caisson height (sls)	$h_{caisson}$	**23,36**	**[m + NAP]**	*total construction height of caisson*
Caisson width (uls)	**B**	**21,09**	**[m]**	*total construction width of caisson*
Safety factors:				
sliding	$\gamma_{sliding}$	**1,49**	**[-]**	*safety factor*
overturning	$\gamma_{overturning}$	**1,72**	**[-]**	*safety factor*
bearing capacity rm	$\gamma_{bearcap,rm}$	**1,54**	**[-]**	*safety factor*
bearing capacity subsoil	$\gamma_{bearcap,sub}$	**1,21**	**[-]**	*safety factor*

Figure: Input and results of O.C.D.

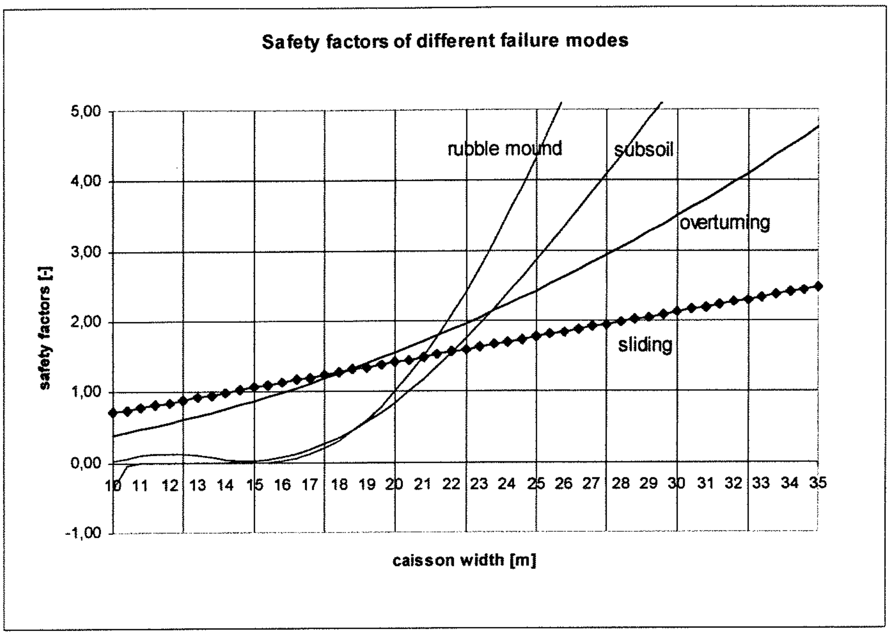


Figure 6.6 Overview safety factors.

For the defined design conditions Figure 6.6 indicates which failure mode is decisive, and which minimum value must be maintained for B to ensure the breakwater does not fail. Maintaining a safety factor γ=1.2, the following values for B are determined for each failure mode:

Failure mode		required caisson width [m]
sliding:	$\gamma_{sliding}$=1.2	16.99
overturning:	$\gamma_{overturning}$=1.2	17.63
slip of the rubble mound:	$\gamma_{r.m.}$=1.2	20.46
slip of the subsoil:	$\gamma_{subsoil}$=1.2	21.09

Table 6.2 Required caisson width for certain failure mode.

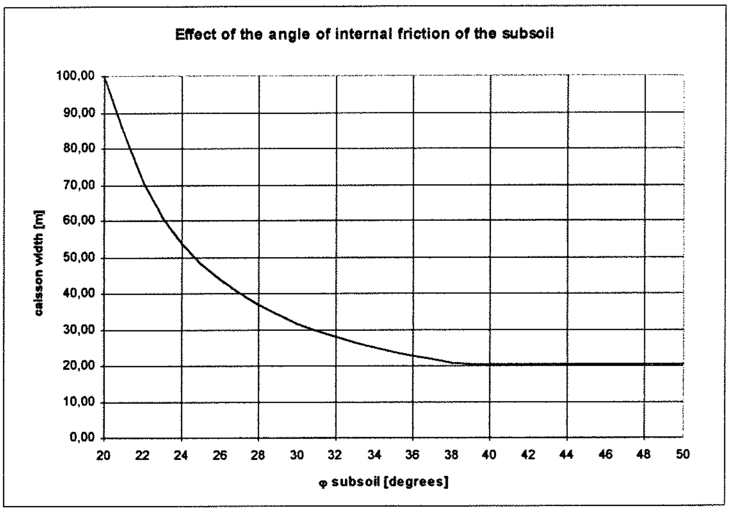


Figure 6.7 Effect of the angle of internal friction of the subsoil.

In the design conditions $\varphi_{subsoil}$ = 37.5° was determined. The effect of $\varphi_{subsoil}$ on the caisson width can be seen in Figure 6.7. From this figure it can be concluded that for $\varphi_{subsoil} \leq 25°$ the bearing capacity of the subsoil is insufficient, leading to an extreme wide caisson design. If a soil improvement hadn't been applied for the top 6.0 m of soil, the angle of internal friction would be 22.5°, which would require a caisson width of approximately 67 m. Therefore it is sufficient to compact the sand improvement until this angle of internal friction is achieved.

For values $\varphi_{subsoil}$ >38° the caisson width remains constant, and the angle of internal friction of the rubble mound φ_{rm} can be analysed in similar manner. Usually $\varphi_{r.m.}$ = 40° is maintained for the angle of internal friction of quarry run.

An important conclusion is that B (and the total breakwater costs) rapidly increases when φ_{sub} or φ_{rm} decreases. It is therefore extremely important to gain more accurate information concerning the exact properties of the subsoil. If this is much lower than the 37.5° which was determined in this study, the width of the caisson will increase significantly.

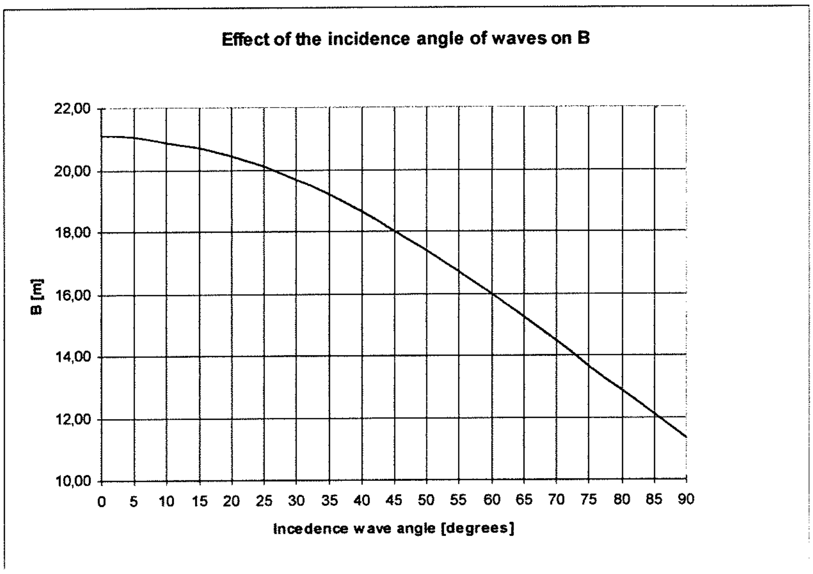


Figure 6.8. Effect of the incidence angle of waves on B.

Figure 6.8 illustrates the effect of the incidence angle of the waves. As can be expected, B is largest for perpendicular incoming waves (θ=0°).

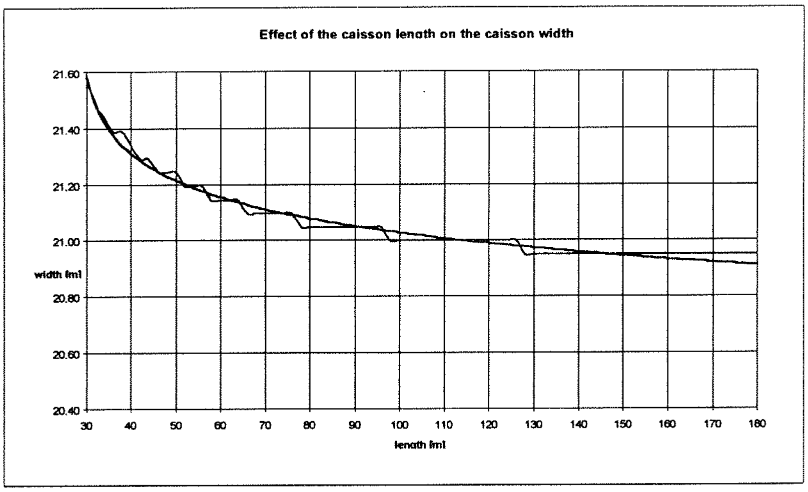


Figure 6.9 Effect of the caisson length on B.

An increase of the caisson length allows for a smaller caisson width. This is due to shape coefficients implemented in the formulas of Brinch-Hanssen. This effect is strongest for relatively short caissons and becomes of less effect for caissons longer than 120 m. Longer caissons will lead to less transport and placement procedures, and will therefore be economical. However the bending moments and tension moments will also become larger with an increase in caisson length. This will require extra investments on the construction strength.

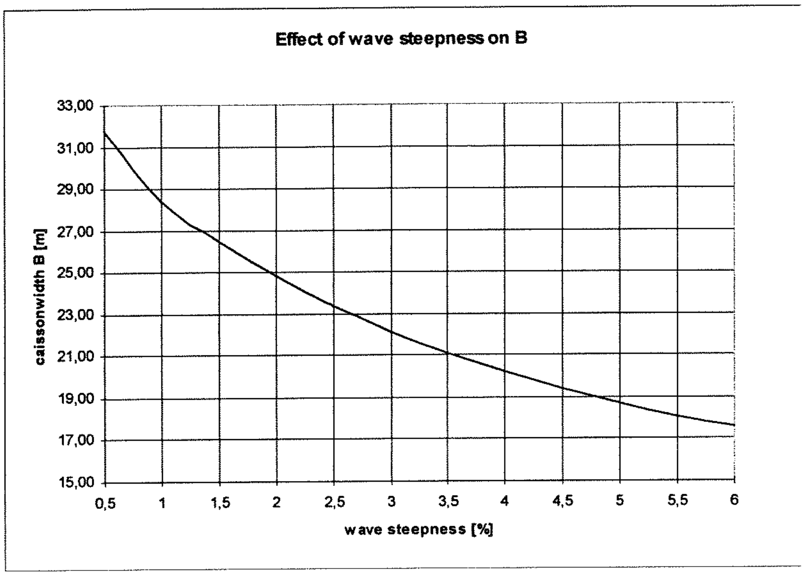


Figure 6.10 Effect of the wave steepness on B.

An increase of the wave steepness leads to lower wave forces on the caisson, and therefore less caisson width is required. The average wave steepness on the North Sea is 3.5%.

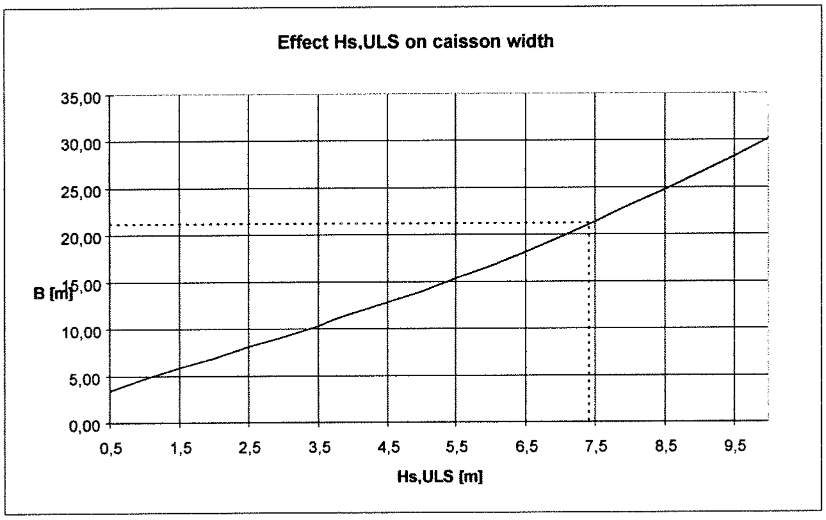


Figure 6.11 Effect of $H_{s,ULS}$ on B.

The local significant wave height for ultimate limit state is taken as $H_{s,ULS}$ = 7.43 m. This is the local wave height which has a return period of 1000 years, and with which the failure modes have been calculated. From Figure 6.11 it can be seen that if a more strict criterion is maintained for $H_{s,ULS}$, the required caisson width rapidly increases.

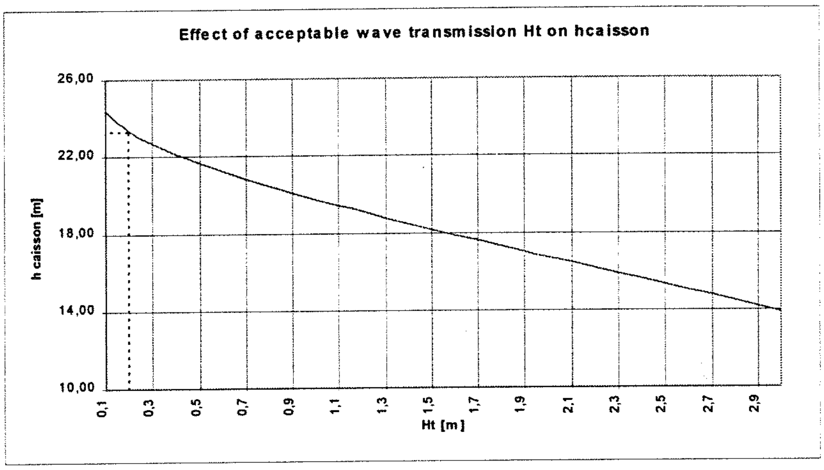


Figure 6.12 Effect of the acceptable wave transmission H_t on $h_{caisson}$

The acceptable level of wave transmission is strongly dependant upon the function which the basin behind the breakwater must fulfil. In this study it was assumed that transhipment of container vessels must be able to take place, for which the acceptable height of transmitted waves is H_t= 0.20 m. As can be seen in Figure 6.12 this criterion is very strict, and means the crest height, and therefore total construction height of the caisson, is large. By maintaining a less strict H_t, $h_{caisson}$ can be reduced significantly.

Note that wave penetration through the harbour entrance and local generated wave fields are not included in H_t.

6.6 Conclusion

The outer caisson dimensions which will be maintained for further calculations in this study are:

total construction height caisson:	$h_{caisson}$	= 23.40 m
construction width caisson:	B	= 21.10 m
caisson length:	L	≈ 75.00 m

For this design the safety factors of the analysed failure modes are:

safety factor	[-]
$\gamma_{overturning}$	1.72
$\gamma_{sliding}$	1.49
$\gamma_{r.m.}$	1.54
$\gamma_{subsoil}$	1.21

Table 6.3 Safety factors of analyzed failure modes, caisson width B=21.09 m.

The required minimum width of the caisson strongly depends on the input values of the boundary conditions and assumptions. In this study it is assumed that the subsoil is improved in order to create a higher bearing capacity. This makes it possible to construct the caisson less wide, which is economical. This reduction of the caisson costs must be compared with the extra expenses of the improvement of the subsoil. This will not be analysed further in this study.

The maintained value for the acceptable transmitted wave height over the breakwater is quite strict, and leads to a very large construction height of the caissons. By maintaining a less strict criterion for H_t the breakwater costs can be reduced significantly.

7. Concrete and steel dimensions

7.1 Introduction

The concrete caissons for the Maasvlakte 2 breakwater consist of a base slab and a shaft which consists of outer walls and inner walls in the form of a cell structure. During the caissons lifetime it is subjected to different forces resulting from hydraulic pressure, waves, currents and ground pressure. These forces are inventoried in this section. Based on these forces the concrete and steel dimensions of the caisson cell structure will be designed which are required to resist these forces.

7.2 Design philosophy

To schematise the cell structure of the caisson, the cells can be interpreted as plates. The following three possibilities were considered to realise the steel reinforcement of these plates.

- Completely prestressed concrete implies that on the tension side of the plate only a minimal tension is acceptable. How high this tension may be depends on the function of the element and the amount of outer steel reinforcement.

- Partially prestressed concrete implies that bending moments are absorbed by a the prestressing steel reinforcement in combination with the outer steel reinforcement. The prestressing steel absorbs the largest part of the bending moment, and the outer steel reinforcement absorbs restmoments. An import aspect of partially prestressed concrete is limitation of the acceptable cracks.

- By increasing the thickness of the plate, complete absorption of the bending moments may be possible, which eliminates the need for prestressing steel.

The optimum proportion between prestressing steel and steel reinforcement from a structural or economical perspective is difficult to indicate. An economical optimisation is dependant of the going market rates. In this study it is assumed that the caissons will be fabricated of partially prestressed concrete. In a later phase of the design further study can be done to determine the optimal form of steel reinforcement from an economical perspective.

7.2.1 Placement of the prestressing steel reinforcement

By effectively placing the prestressing steel reinforcement according to the extant bending moments and tension resulting from the forces on the cell wall and floors (Figure 7.1), the amount of steel can be minimised. There are several manners in which this can be achieved:

By placing the prestressing steel reinforcement in rounded position it is possible to economise on the amount of steel required, however the installation is more complex (Figure 7.2). Placing the prestressing steel reinforcement in a straight line but slightly asymmetrical to the concrete centre line is less effective from a structural perspective, but the installation is less complicated (Figure 7.3). In this phase of the study it is assumed that the prestressing steel is placed in the concrete centre line (Figure 7.4). Further optimisation of the design can be done in a later phase of the study.

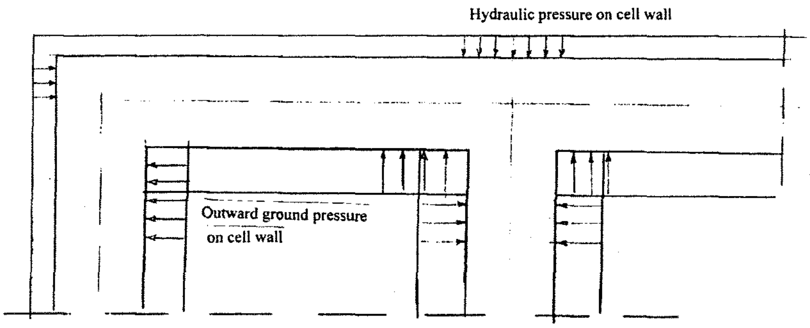


Figure 7.1. Forces on the cell wall.

Effective

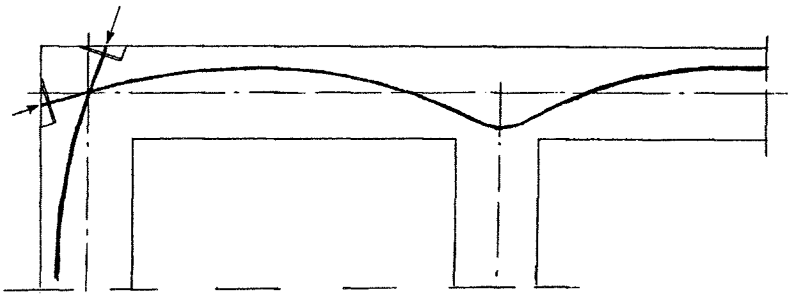


Figure 7.2. Rounded placement of the prestressing steel reinforcement

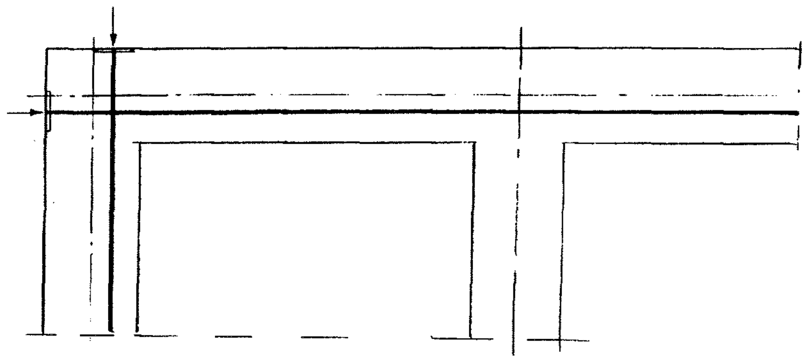
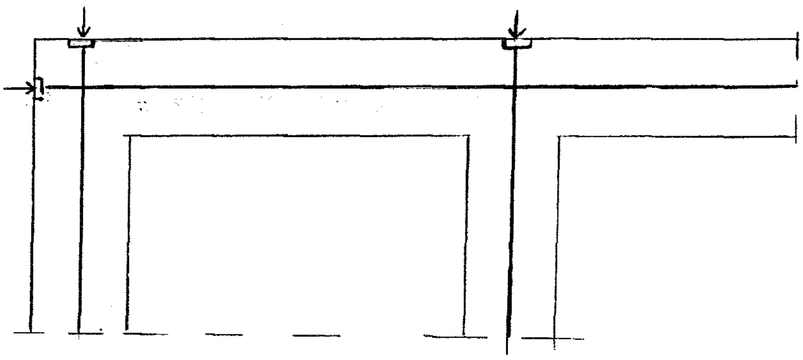


Figure 7.3. Eccentric placement of the prestressing steel reinforcement.



Less effective

Figure 7.4. Symmetrical placement of the prestressing steel reinforcement.

7.3 Calculation of the forces on the caisson

The concrete and steel dimensions of the shaft and base slab are determined by the forces exerted on the caisson during all the phases it undergoes during its lifetime. The following phases with each their specific forces must be taken into account:

- construction phase, hydrostatic pressure on outer wall;
- transport phase, hydrostatic pressure on outer wall;
- placement phase, hydrostatic pressure on walls;
- completion phase, filling of the cells with sand, ground pressure on walls;
- operational phase, combinations of hydrostatic pressure, wave pressure, and ground pressure on shaft;

In section 6. the required height and width of the caisson has been calculated. In this phase of the design it is assumed that the base slab has a construction height of 1.00m, and the capping is 0.50m thick. This leads to the caisson dimensions as indicated in the figure below:

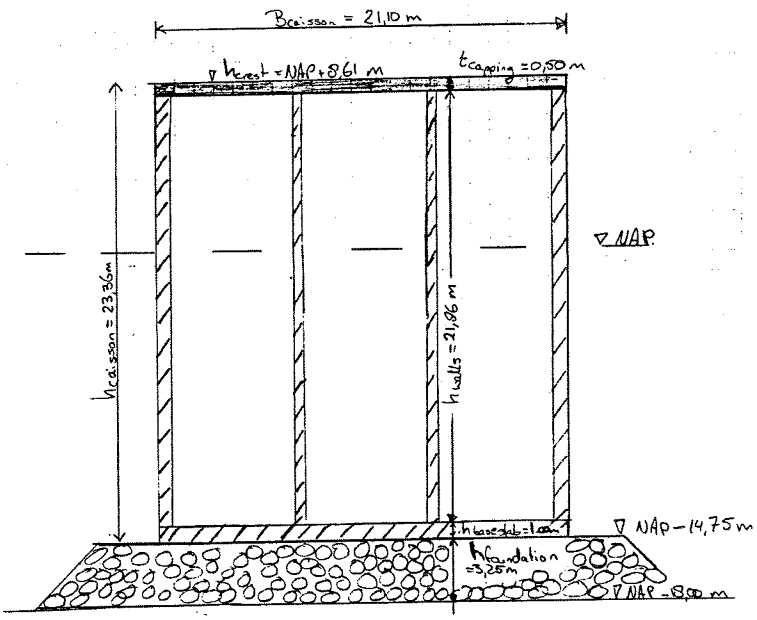


Figure 7.5. Cross section of caisson.

7.3.1 Schematised design forces on the caisson during construction phase

The shaft will be constructed using slip formwork. During the construction phase of the shaft, the caisson and FCY will sink, due to increase in dead weight, as the construction height of the shaft increases. The submerged section of the caisson is subjected to hydrostatic pressure. As the concrete which is subjected to this pressure is still new, this hydrostatic pressure may not be too large.

The hydrostatic pressure exerted on the caisson shaft can be controlled by the lifting speed of the formwork. The acceptable pressure on the newly cast concrete is dependant of the type of concrete used, climate conditions and additives.

The hydrostatic pressure during this phase is considerably less than the pressures exerted on the shaft during the operational phase, and therefore does not determine the required concrete thickness.

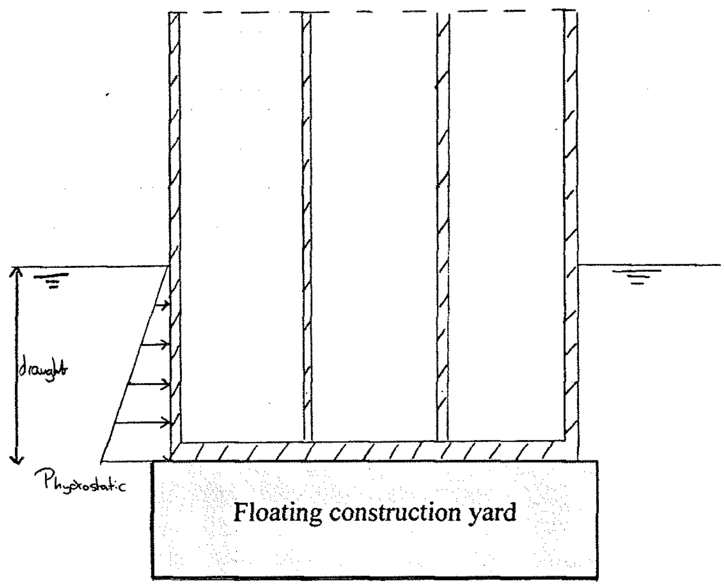


Figure 7.6. Hydrostatic forces on outer walls during construction phase.

7.3.2 Schematised design forces on the caisson during transport phase

After completion of the caisson it is trimmed to guarantee stability during transport (see Appendix L: 'Caisson stability'). The height of the shaft is h_{shaft}= 21.86 m, height of the base slab is 1.0 m and the draught of the caisson after trimming is d=13.50 m. The maximum acceptable wave height which is taken into account during the transport phase is $H_{s,transport}$= 1.0 m. The outward force of the ballast sand in the cells reduces the inward force caused by the hydrostatic pressure, as this effect is minimal it is not taken into account.

The hydrostatic pressure on the shaft during this phase is:

$p_{transport}$= 12.50 m x 10.30 kN/m^3 = 130 kN/m^2

The caissons will only be transported during calm weather conditions with maximum wave heights of 1.0 m.

Conclusion: no extreme loads during construction phase.

7.3.3 Schematised design forces on the caisson during placement phase

During the placement phase the caisson cells will be flooded and the caisson will sink onto the foundation. As the inner cell walls are not dimensioned to absorb large bending moments, care must be taken that the cells are filled to a more or less even level. During sand filling of the cells the maximum head level difference between neighbouring cells is 5.0 m. It is assumed that the top level of the foundation mound has been levelled so that there are no extreme peak loads resulting from protruding rocks on the base slab. The levelling procedure is discussed in 'section 9.3 Rubble mound foundation'.

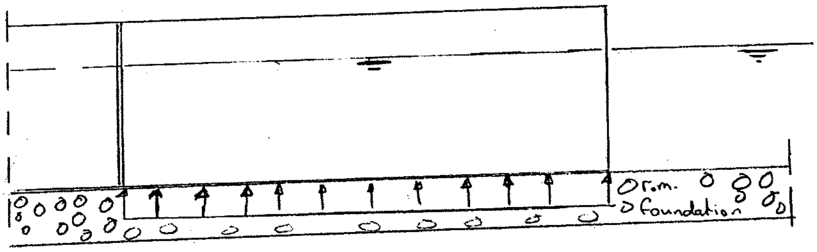


Figure 7.7 Schematised forces during placement.

Conclusion: No extreme loads on the caisson during this phase.

Risk: Obstacle on rubble mound foundation, prevent by dragging beam or scrading, see section 9.3 Rubble mound foundation. It is assumed that the rubble mound foundation is sufficiently flexible to level out any remaining unevenness.

7.3.4 Schematised design forces on the caisson during operational phase

During the operational phase there are 3 situations which must be analysed:

A. Maximum wave load on the outer wall;
B. Maximum ground pressure on the outer wall;
C. Peak loads on base slab due to uneven support from rubble mound foundation;

A. Maximum wave load on the outer wall

The maximum wave load to which the outer wall is subjected is the Goda design wave H_D=1.8x $H_{s,ULS}$=13.37m combined with the ultimate limit state high water level, HWL_{ULS}=NAP+4.40m. The values $P_{1,GODA}$, $P_{2,GODA}$ and $P_{3,GODA}$ for these conditions have been calculated with the EXCEL computer program OCD written in this thesis. These calculations are presented in Appendix: J. The theoretical background of these calculations is presented in Appendix H: 'Goda design formulas for vertical breakwaters'.

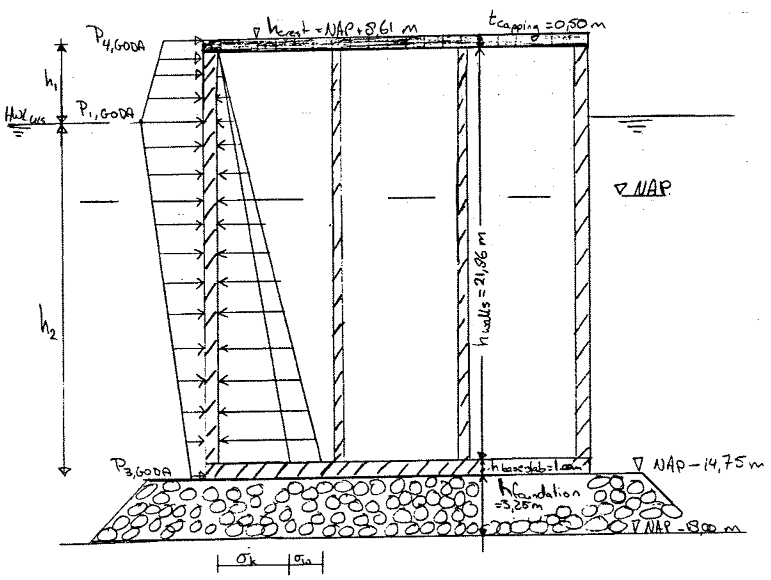
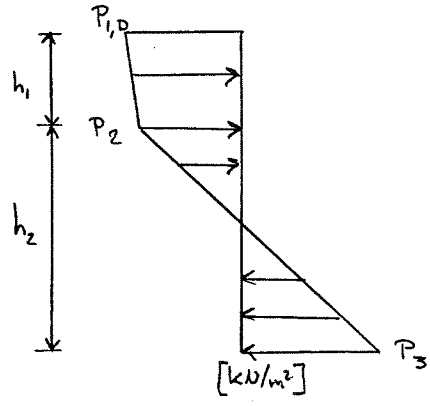


Figure 7.8. Maximum wave load on the outer wall.

Input:
HWL_{ULS} = NAP + 4.40 m
$H_{s,ULS}$ = 7.43 m
H_D = $1.8 \cdot H_{ULS}$ = 13.37m
$P_{1,\,Goda}$ = 108 kN/m^2
$P_{3,\,Goda}$ = 71 kN/m^2
$P_{4,\,Goda}$ = 52 kN/m^2
λ_n = 0,5 [-]
h_1 = $h_{crest} - t_{capping} - HWL_{ULS}$ = 3.71m
h_2 = slab level - t_{slab} + HWL_{ULS} = 18.15m



Result:

$p_{1,D}$ $= P_{4,\,Goda}$
$= 52$ kN/m^2

p_2 $= P_{1,\,Goda} - P_{2,soil}$
$= P_{1,\,Goda} - h_1 \cdot [\lambda_n \cdot (\rho_{sat} - \rho_{wat}) + \rho_{wat}]$
$= 108 - 3.71 \cdot [0.5 \cdot (21.0-10.30) + 10.30]$
$= 108 - 58.1$
$= 50$ kN/m^2

p_3 $= P_{3,\,Goda} - p_{2,soil} - h_2[\lambda_n \cdot (\rho_{sat} - \rho_{wat})]$
$= 71-58.1-18.15 \cdot [0.5 \cdot (21.0-10.30)]$
$= -85$ kN/m^2

Conclusion: $p_{1,D} = 52$ kN/m^2 is the decisive pressure for the calculation of the upper-outer wall section.

B. Maximum ground pressure on the outer wall

The maximum ground pressure to which the outer wall is exerted is for a caisson filled with saturated ground, at the moment of a wave trough of H_D, with the water level LLWS.

Design conditions:
- LLWS = NAP -0.84 m
- H_D = 13.37m (Goda)
- Caisson saturated with water

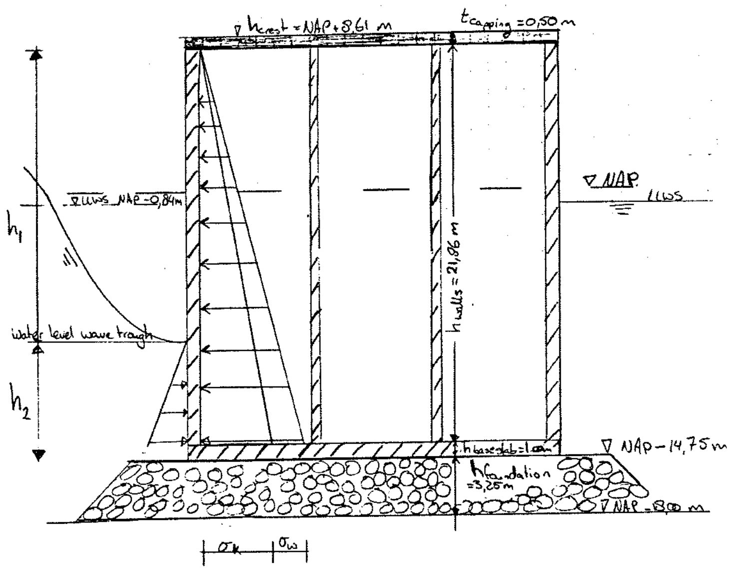
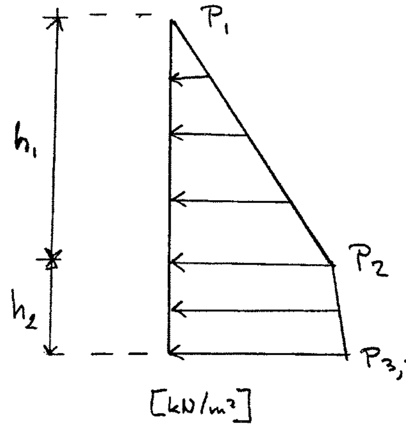


Figure 7.9. Maximum ground pressure on the outer wall.

Input:
water level = LLWS - $0.5 \cdot H_D$ = NAP - 7.53 m

h_1 = 7.53+8.61-0.50=15.64m
h_2 = 14.75-1-7.53=6.22m



Result:

p_1 = 0 kN/m²

$$p_2 = h_1 \cdot [\lambda_n \cdot (\rho_{sat} - \rho_{wat}) + \rho_{wat}]$$
$$= 15.64 \cdot [0.5 \cdot (21-10.30)+10.30]$$
$$= 245 \text{ kN/m}^2$$

$$p_{3,D} = p_2 + h_2[\lambda_n \cdot (\rho_{sat} - \rho_{wat})]$$
$$=245+7.53 \cdot [0.5 \cdot (21-10.30)]$$
$$=285 \text{ kN/m}^2$$

Conclusion: $p_{3,D}$=285 kN/m² is the decisive pressure for the calculation of the lower section of the outer shaft.

C. Peak loads on base slab due to uneven support from rubble mound foundation

The maximum pressure on the base slab is due to uneven support from the rubble mound foundation. If foundation material is washed from underneath the caisson, the base slab must absorb the weight of the ground in the cell above. This pressure is largest when the water level is low, and the ground is saturated with water.

Design conditions:
- LWL_{ULS} = NAP - 2.05 m
- H_s = 0.0 m
- Caisson saturated with water
- One cell not supported

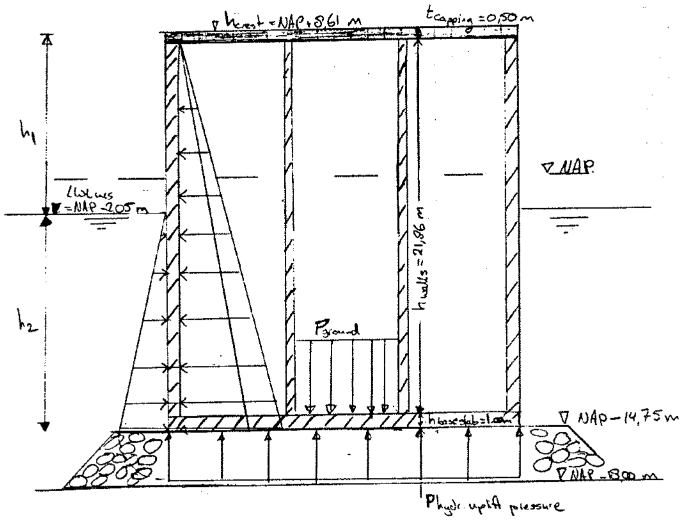
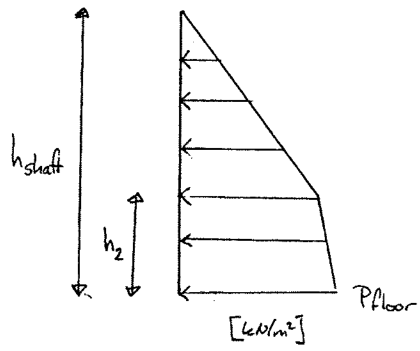


Figure 7.10. Peak loads on base slab due to uneven support from rubble mound foundation.

h_2 $= LWL_{ULS}$ - slab level
$= 14.75-2.05$
$= 12.70\text{m}$

p_{floor} $= (h_{shaft} + h_{baseslab})\, \rho_{sat} - (h_2 \cdot \rho_{water})$
$= 22.86 \cdot 21 - 12.70 \cdot 10.30$
$= 350\ \text{kN/m}^2$



7.3.5 Schematised design forces on the caisson during reuse phase

During this phase the sand filling must be removed from the cells. As the inner cell walls are not dimensioned to absorb large bending moments, care must be taken that the cells are emptied to a more or less even level. The maximum acceptable head level difference between neighbouring cells is 5.0 m.

Conclusion: no extreme forces on the caisson during reuse phase;

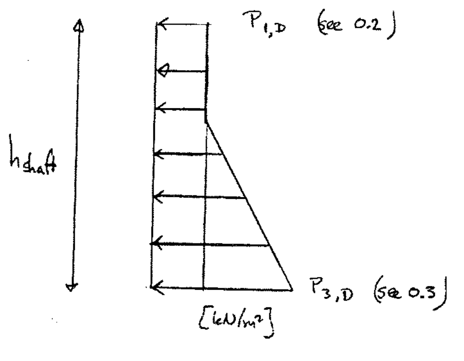
Risk: caisson might stick to rubble mound foundation when it is floated up;
maximum head level difference between neighbouring cells is exceeded;

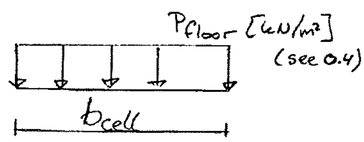
7.3.6 Conclusion design forces

The design forces are determined by the outward ground pressure combined with the inward wave pressure. The combination of these forces will be maintained for further calculations of the concrete and steel dimensioning.

Design force on shaft:
$p_{1,D} = 52\ \text{kN/m}^2$
$p_{3,D} = 285\ \text{kN/m}^2$

Design force on floor:
$p_{floor} = 350\ \text{kN/m}^2$





7.4 Calculation of the bending moments in the plates

After all the forces on the cell walls have been inventoried and the construction has been schematised, the bending moments and side loading forces in the plates are calculated. This can be done with the 'GTB tables' or 'Platten', [lit.10]. These calculations are presented in Appendix K: 'Concrete and steel dimensions'. The results are presented in the following sections.

The caisson has been schematised into the following elements:

- Outer walls : plate clamped at three sides;
- Inner walls : plate clamped at three sides;
- Base slab : plate clamped at four sides;

7.5 Calculation of the concrete and steel dimensions

The following approach is maintained to calculate the concrete and steel dimensions:

1. Choose a height of the plate element;
2. Choose a concrete quality;
3. Choose a prestressing steel quality and a reinforcement steel quality;
4. Determine outer reinforcement on both sides of the plate;
   assume: $A_t = A_d$, heart to heart- distance rods 150 to 200 mm.
5. Determine the bending moments and the tension in the plate resulting from the design forces;
6. Chose type and size of the prestressing steel reinforcement;
7. Determine the position of the prestressing steel reinforcement;
8. Determine the resulting (reduced) bending moments due to effective placement of prestressing steel reinforcement;
9. Using the resulting bending moments, tension forces and position of the prestressing steel reinforcement determine the outer steel reinforcement required to meet the limited crack criteria;
10. Control the VBC-criteria [lit.16];
11. Control capacity to absorb side loading forces[1];

7.5.1 Computer program RCA

The calculation of the required concrete and steel dimensions as discussed in the previous section can be done with the EXCEL-program RCA, **R**isk **CA**lculation. As mentioned earlier, the structurally and economically most efficient combination between concrete thickness, placement of prestressing steel reinforcement and outer steel reinforcement is dependant on the current market prices. Using the computer program RCA several combinations for the dimensions have been examined. The calculations and concrete design method are presented in Appendix K: 'Concrete and steel reinforcement dimensions'. The following values were determined for the caisson:

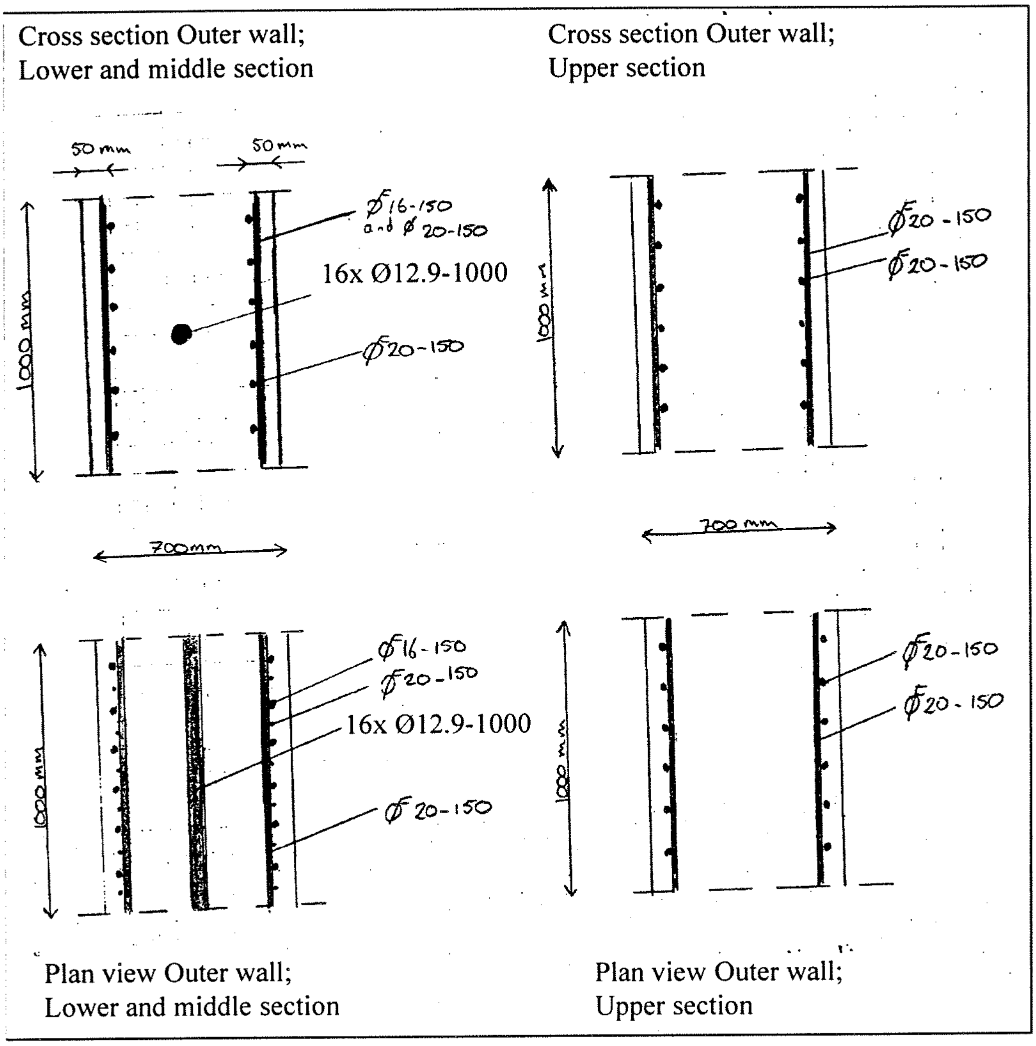
element	construction height	concrete thickness (B45)	outer steel reinforcement (FeB 500)	prestressing steel reinforcement (FeP 1860)
Outer cell wall, x-direction				
• lower section	5.47m	700mm	Ø20-150	16x Ø12.9-1000
• middle section	10.93m	700mm	Ø20-150	16x Ø12.9-1000
• upper section	5.47m	700mm	Ø20-150	-
Outer cell wall, y-direction				
• lower section	5.47m	700mm	Ø20-150 + Ø15-150	-
• middle section	10.93m	700mm	Ø20-150 + Ø15-150	-
• upper section	5.47m	700mm	Ø20-150	-
Inner cell wall, x direction				
• all sections	21.86m	500mm	Ø20-150	-
Inner cell wall				
• all sections	21.86m	500mm	Ø20-150	-
Floor, x-direction = y-direction	1.00m	1000mm	Ø25-150	-

Table 7.1. Overview concrete and steel dimensions.

[1] The steel reinforcement in combination with the concrete thickness is sufficiently strong to absorb the side loading forces from the ground pressure on the cell walls, and therefore no specialised reinforcement needs to be installed for this purpose.

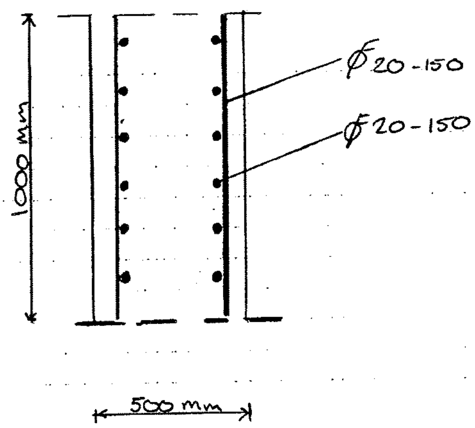
7.6 Technical drawings

7.6.1 Outer wall

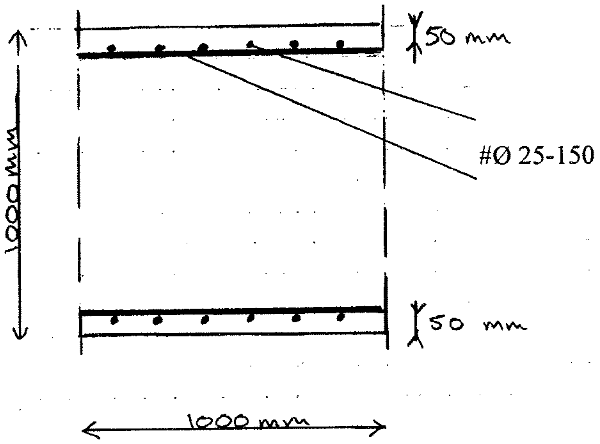


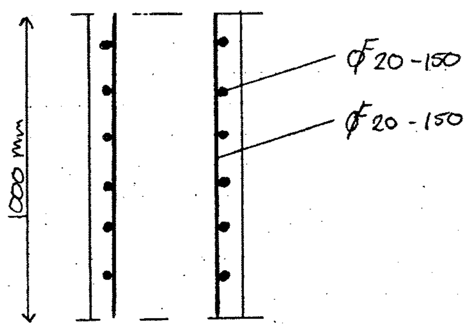
7.6.2 Inner wall and base slab

Cross section Inner wall;
all sections

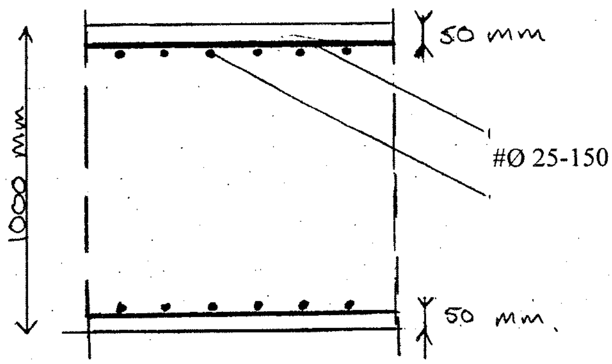


Cross section base slab;
length direction





Plan view Inner wall;
all sections



Cross section base slab;
width direction

Element	section height [m]	concrete: concrete thickness	concrete: concrete volume[2] $[m^3]$	outer steel reinforcement: dimensions	outer steel reinforcement: kg steel $/m^3$	outer steel reinforcement: ton steel /section	prestressing steel reinforcement: dimensions	prestressing steel reinforcement: kg steel $/m^3$	prestressing steel reinforcement: ton steel /section	Formwork $[m^2]$
Outer cell wall, x-direction										
• lower section	5.47	700mm	**775**	Ø20-150	47	**36.4**	16xØ12.9-1000	17.9	**13.9**	**1288**
• middle section	10.93	700mm	**1475**	Ø20-150	47	**69.4**	16xØ12.9-1000	17.9	**26.4**	-
• upper section	5.47	700mm	**775**	Ø20-150	47	**36.4**	-	-	-	-
Outer cell wall, y-direction										
• lower section	5.47	700mm	775	Ø20-150 + Ø16-150	77	**59.7**	-	-	-	-
• middle section	10.93	700mm	1475	Ø20-150 + Ø16-150	77	**113.6**	-	-	-	-
• upper section	5.47	700mm	775	Ø20-150	47	**36.4**	-	-	-	-
Inner cell wall, x direction										
• all sections	21.86	500mm	**5268**	Ø20-150	47	**247.6**	-	-	-	**1288**
Inner cell wall										
• all sections	21.86	500mm	5268	Ø20-150	47	**247.6**	-	-	-	-
Floor, x-direction	1.00	1000mm	**1618**	Ø25-150	73	**118.1**	-	-	-	**196**
Floor, y-direction	1.00	1000mm	1618	Ø25-150	73	**118.1**	-	-	-	-
Subtotal	-	-	**9911**	-	-	**1083**	-	-	**40.3**	**1484**
Overlap factor	-	-	-	-	-	1.3[3]	-	-	-	-
Total [4]	-	-	**9911**	-	-	**1407**	-	-	**40.3**	**1484**

Table: Overview used materials per caisson, caisson dimensions: Lx Bx h= 76.65m x 21.10m x 23.36m.
Remark: a plan view of the caisson is presented in Appendix P: 'Technical drawings'.
Materials: Concrete: B45; Outer steel reinforcement: FeB 500; Prestressing steel: FeP 1860;

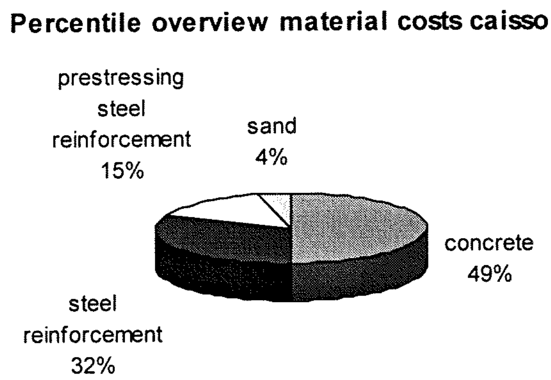
[2] $A_{outerwalls}=135m^2$, $A_{innerwalls}=241m^2$ and $A_{baseslab}=1618m^2$.
[3] 30% overlap of reinforcement is assumed.
[4] Total is of the values printed in bold print.

7.7 Conclusions

The following concrete, steel and sand quantities will be maintained to determine the construction process (logistic) and price estimation[5]. The average material costs of the caisson[6] are Nlg. 100,-/m³.

aspect	units required	unit costs	material costs / caisson [Nlgx10⁶]	material costs / m [Nlg.]	material costs 4.0km long section [Nlgx10⁶]	[%]
concrete[7]	9911m³	Nlg. 180,-/m³	1.78	23.274,-	93.1	49
steel reinforcement	1407 Tonnes	Nlg. 0.80/kg	1.13	14.685,-	58.8	32
prestressing steel reinforcement[8]						
length direction	34	9.950,-/unit	0.34	4.415,-	17.7	10
width direction	34	5.200,-/unit	0.18	2.310,-	9.2	5
sand[9]	27.150	5,-	0.14	1.771,-	7.1	4
total:	-	-	**3.57**	**46.455**	**185.9**	**100**

Table 7.2 Overview material costs caisson (based on caisson length of 76.65m).



[5] The costs have been determined with the DIBK (Droge Infrastructuur Bedrijfszaken Kostprijszaken), the cost calculation department of the Ministry of Transport, Public Works and Watermanagement.
[6] Material costs/m³ = Nlg. $3.57 \cdot 10^6$ / 35.400 m³ = Nlg 100,-/m³.
[7] Concrete:
Concrete type: B45, environment class 4, delivery Nlg 180,-/m³;
[8] Prestressing steel reinforcement:
34 units length direction, A_{steel} =16x Ø12,9-1000; L=76.65m Nlg 9.950,-/unit
34 units width direction, A_{steel} = 16x Ø12,9-1000, L=21.10m Nlg 5.200,-/unit
[9] Sand: delivery, Nlg 5,-/m³ ,the pumping costs are taken up in the placement procedure, section 10.2.

The Maasvlakte 2 breakwater caisson will have the following dimensions:

element	dimension [m]
total caisson height	23.36
caisson length	76.65
caisson width	21.10
15 cells in length direction	4.55
4 cells in width direction	4.55
outer walls height thickness	 21.86 0.70
inner walls height thickness	 21.86 0.50
base slab height	 1.00
height capping plates	0.50
surface area cells	$1242m^2$

Table 7.3. Overview caisson dimensions.

Caisson weight:

concrete and steel:	23.786 Tonnes	($9.911m^3 \cdot 2.400kg/m^3$)
sand filling:	48.870 Tonnes	($27.150m^3 \cdot 1.800kg/m^3$)
total:	72.656 Tonnes.	

Labour costs:	**material**	**price**	**amount**	**costs for 4 km section [Nlg. x10^6]**	**costs/m [Nlg.]**
working only daytime	concrete	60,-/m^3	9911 m^3	32.7	8.200,-
	steel[10]	0.80 /kg	1500 T	66	16.500,-
total	-	-	-	98.7	**24.700,-**
working full time	concrete	80,-/m^3	9911 m^3	43.6	10.800,-
	steel	1.15/kg	1500 T	94.9	23.800,-
total	-	-	-	138.5	**34.600,-**

Table 7.4 Overview labour costs.

Labour costs will be more expensive when works continue full time. Labour costs = Nlg 60,-/ hour in the daytime, and are approximatelty 33% more expensive when works structurally continue full time, Nlg 80,- /hour (Assumption pouring: 1 m^3/ manhour, reinforcement steel placement Nlg 0.80 /kg).

[10] Assume total amount of steel = 1407 T. + 2x 36 T. ≈ 1500 T.

8. Floating construction method of caissons

8.1 Introduction

Since the 1920's many caissons have been constructed for different purposes. They have formed the core of several dams constructed for the Deltaworks (Veerse Dam, Brouwers Dam) in The Netherlands, have been extensively applied as quay wall constructions in the Port of Rotterdam, and have been used as breakwater constructions, as for example in Japan and Italy. During the progress of time and at different locations on earth various construction methods for these caissons have been applied. The four main construction methods can be divided into the following categories:

1. Construction of caissons in a dock;
2. Construction of caissons on a yard;
3. Floating construction of caissons;
4. Other alternatives;

This study will focus upon the feasibility of the floating construction method of caissons for the Maasvlakte 2 caisson breakwater.

8.2 Construction method of caissons in a floating construction yard

The floating construction method has been much applied in Spain and Italy. This caisson construction method makes use of a floating dock in which the caissons are constructed. The platform must have sufficient uplifting capacity to bear the dead weight of the caisson during the first construction phase in which the caisson itself does not have any buoyancy. As the shaft height of the caisson increases, so does its buoyancy. In this manner the pressure of the caisson on the construction deck remains low.

There are many types of floating construction methods which mainly differ in the aspect whether they are mobile or not mobile. The advantage of a mobile platform is that it can be located at any convenient construction site nearby the breakwater site. In this study the construction process of caissons on a floating construction yard (F.C.Y.) will be focused on.

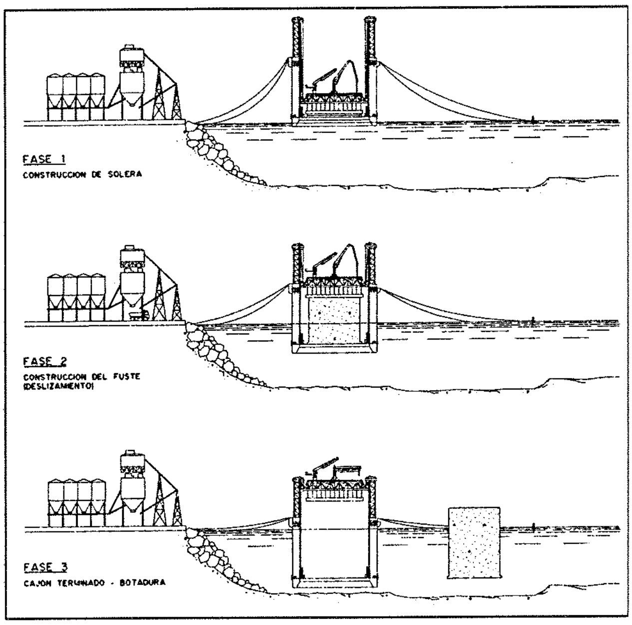


Figure 8.1 Floating Construction Yard[1].

The following floating construction methods can be distinguished:

a) Floating shipdock;
b) Rebuild bulkcarrier to floating dock;
c) Construction on pontoons;
d) Floating formwork;
e) Floating floor construction;
f) 'HBG method';
g) Floating construction yard (mobile);
h) Floating construction yard (not mobile);

[1] Illustrations FCY from Dragados Y Construccciones and Grandi Lavori Fincosit, [lit. 5 & 9].

8.2.1 Sequence of works

Four shifts of 6 hours will work full time to carry out the construction works.

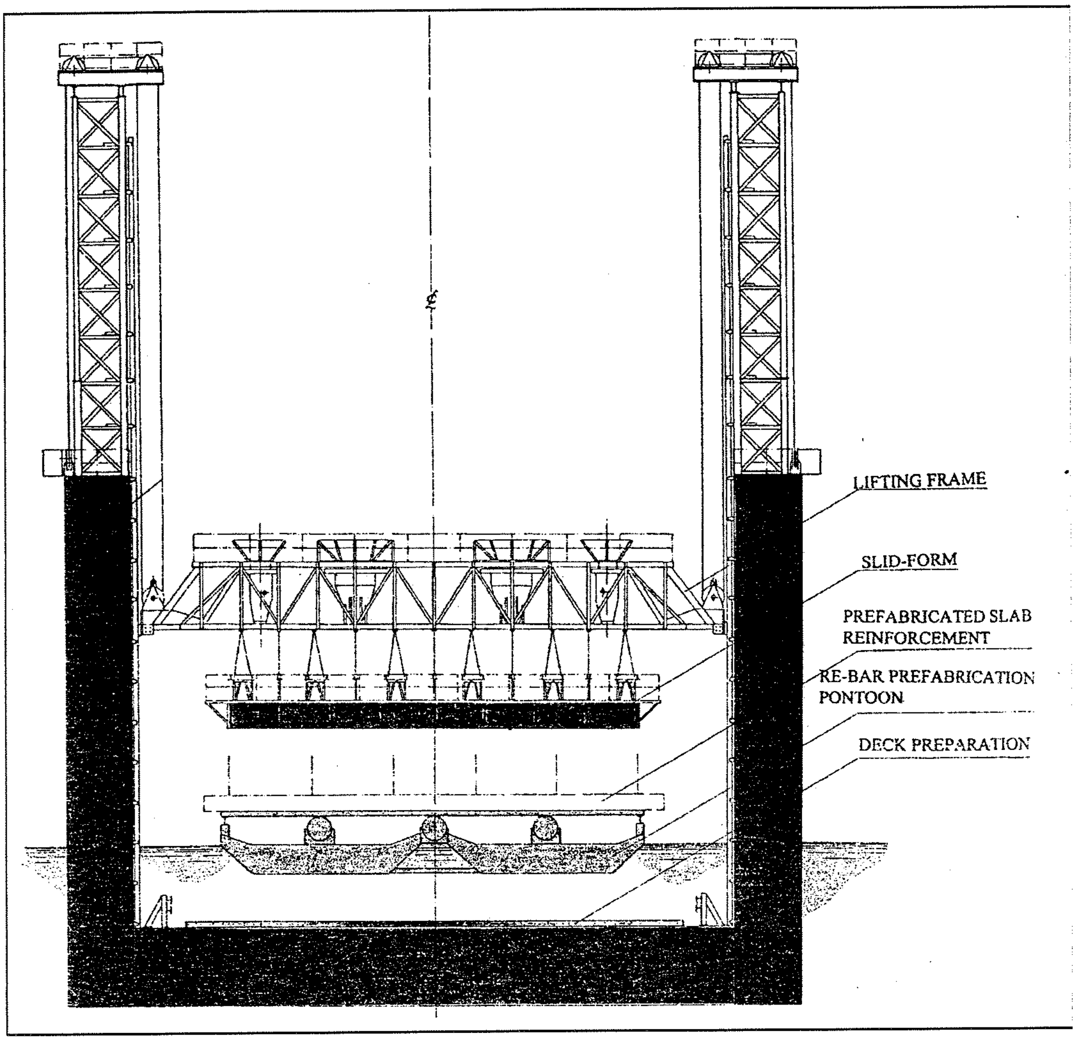


Figure 8.2 Construction phase I.

Phase I

The floating construction yard is filled with water until the deck is submerged deep enough so that a pontoon with prefabricated base slab reinforcement can be towed inside. The prefabricated base slab reinforcement is attached to the lifting frame of the FCY and lifted off the rebar prefabrication pontoon. After the pontoons are towed from the dock, ballast water is pumped out of the dock until the deck is emerged. The installation of the base slab reinforcement takes half a day.

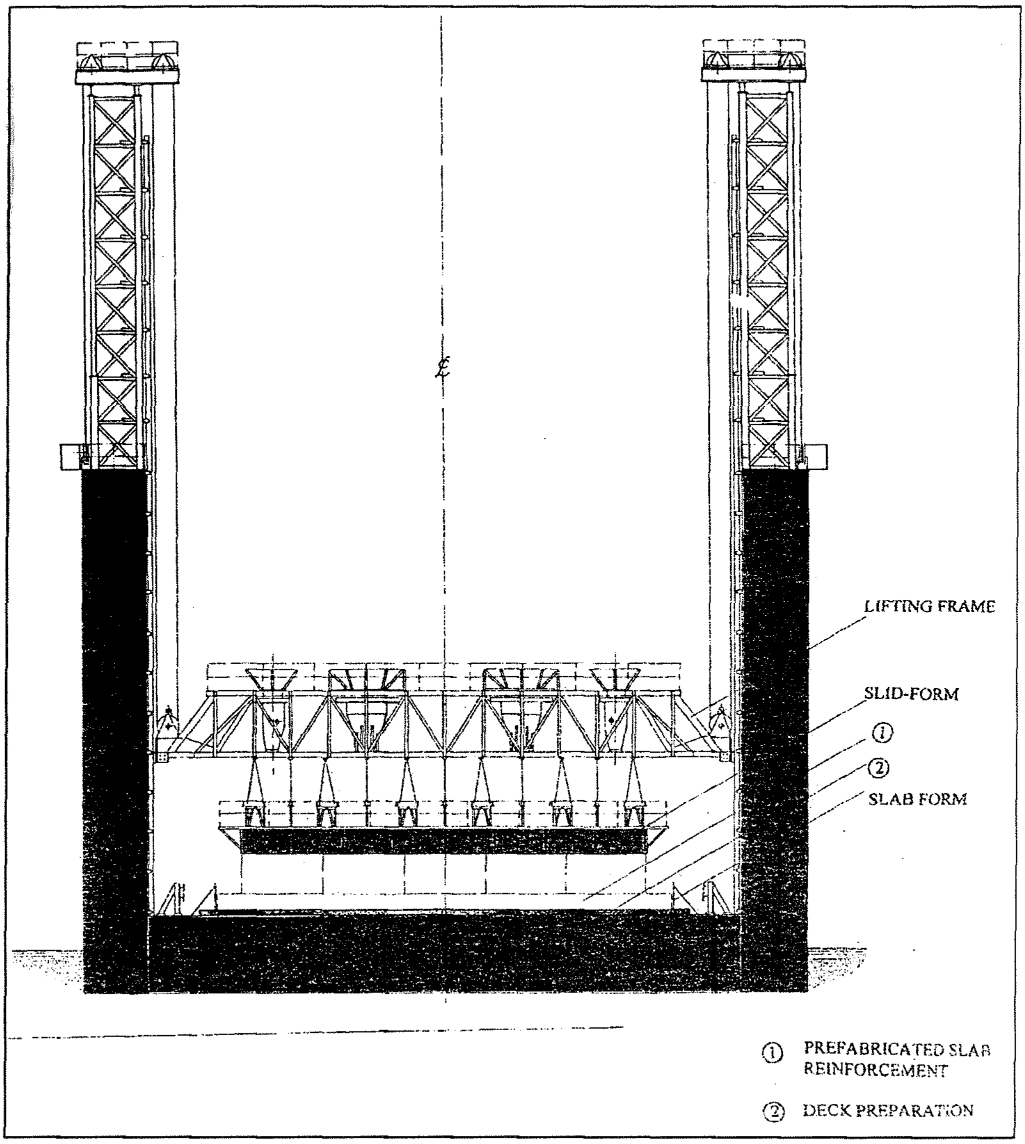


Figure 8.3 Construction phase II.

Phase II
The base slab reinforcement is lowered onto the deck preparation (wooden boards form the bottom formwork of the base slab) and the side formwork of the slab is installed. The base slab is now ready for concreting. Installation of the base slab side formwork takes 1 day.

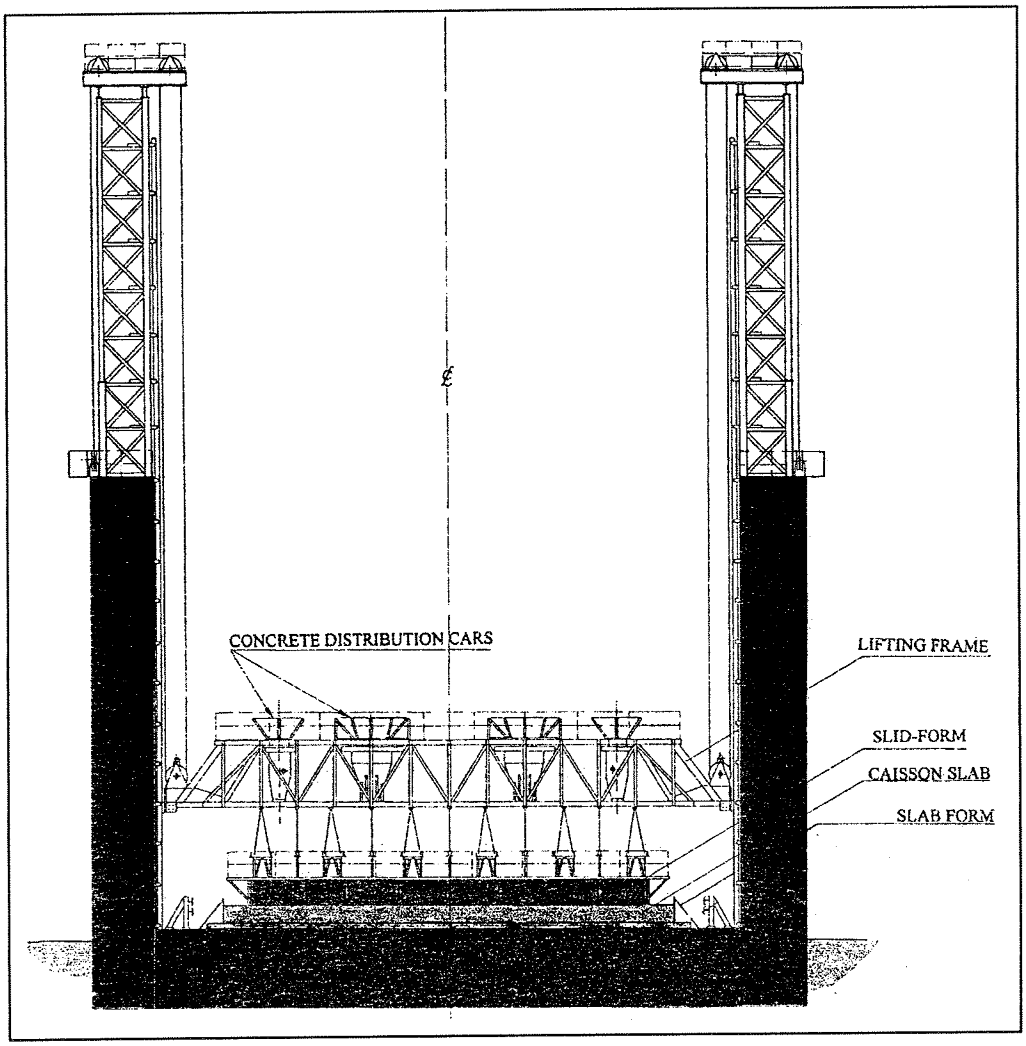


Figure 8.4 Construction phase III.

Phase III

In this phase the concrete of the base slab is poured. The concrete is supplied from a land based factory nearby and applied to the construction through the concrete distribution cars. The concrete pouring is done in one day. Sufficient hardening of the base slab in order to remove the side formwork and install the first lift of shaft reinforcement takes 2 days. The required concrete pouring capacity is 1625 m^3/24 hrs = 70 m^3/hour, which will be achieved by 2 pumps with a capacity of 50m^3/hour.

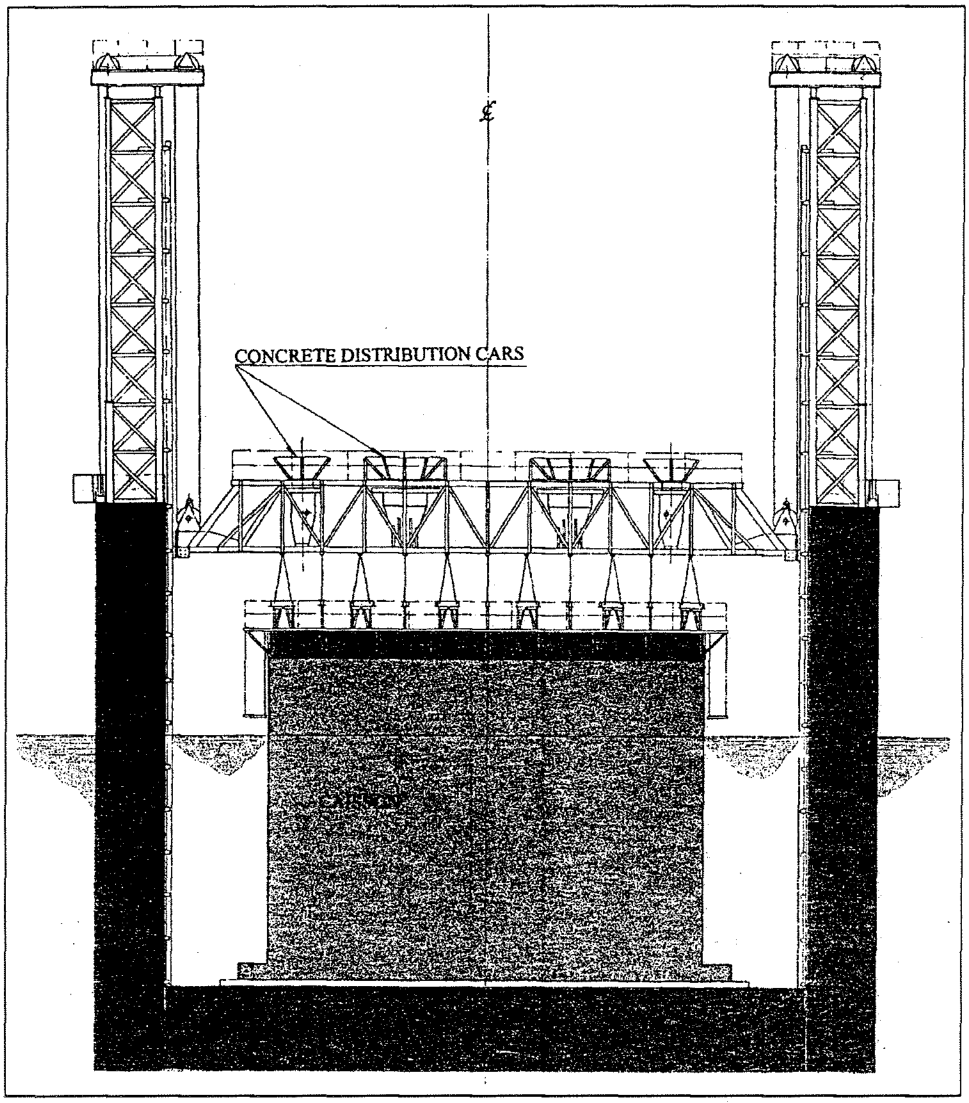


Figure 8.5 Construction phase IV.

Phase IV

In this construction phase the shaft will be concreted. The shaft will be concreted by means of slip formwork.
After sufficient hardening of the base slab, the first lift of reinforcement is installed. The steel reinforcement will be installed in 5 lifts of 5.10m each (including 0.70m overlap). Installation of each lift of reinforcement takes one day. Next the slip-formwork (1.00m high), which is suspended to the lifting frame is lowered into position above the base slab, the time required is half a day. The first lift is now ready for concreting. The lifting speed of the slip formwork is 0.18m/hour, or 4.40 m/day.

One lift of the shaft will be realised in two days, one day installation of the reinforcement and one day concreting, allowing completion of the shaft in 10 days.
The total surface area of the shaft (horizontal cross section) is 376m^2, with a lifting speed of 0.18m/hour the required concrete pouring capacity is 70m^3/ hour.

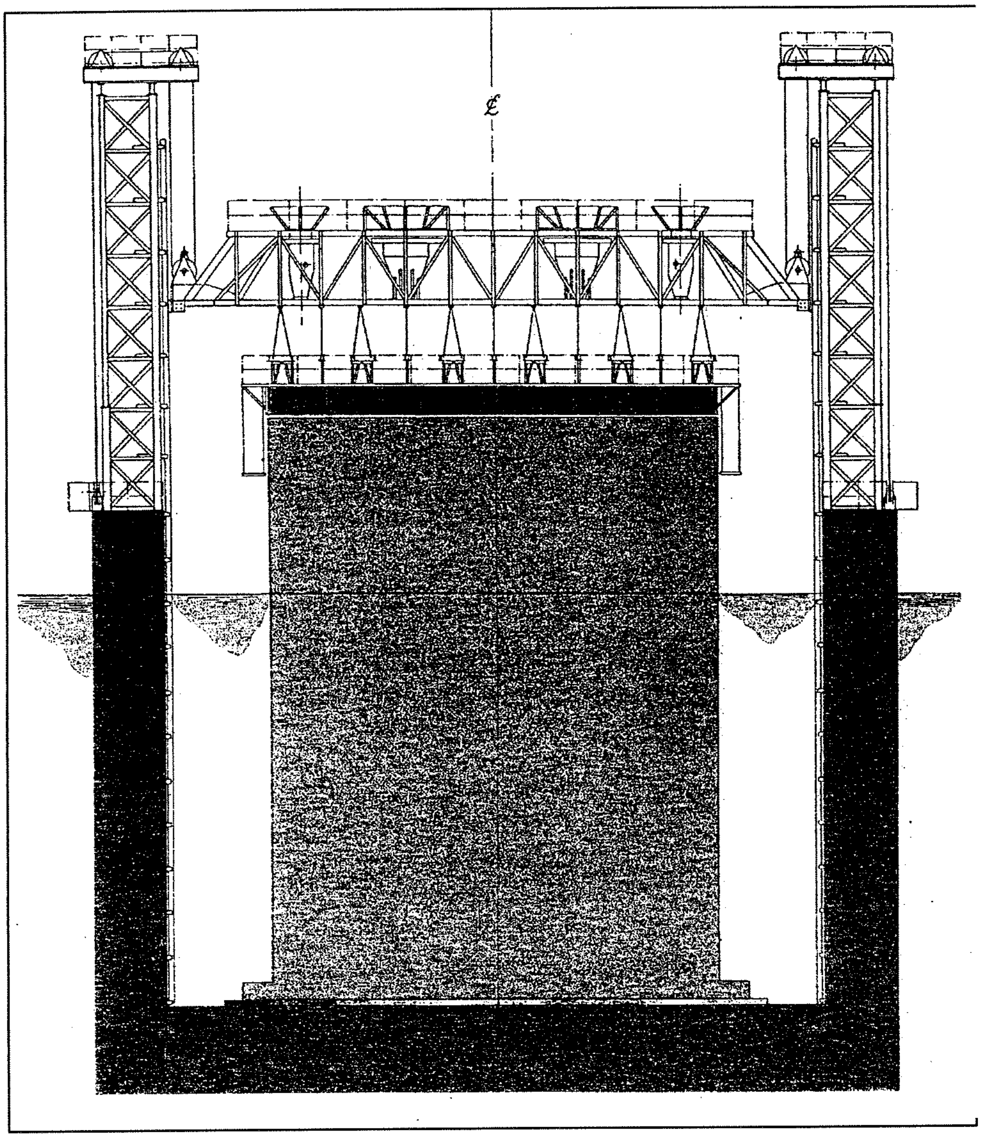


Figure 8.6 Construction phase V.

Phase V

In this phase the caisson shaft reaches the required height (21.86m) and the concreting process is completed. The immersion of the concreted part of the caisson is dependant of the concrete strength and the structural arrangement of the caisson. The contact between the submerged caisson base and deck will be maintained by means of inert ballast material if the uplift forces are higher than the dead weight of the structure.

After completion of the concreting, the formwork is lifted clear from the caisson and the caisson is ballasted in order to achieve sufficient stability for transportation. The caisson is now ready for launching. This process takes half a day.

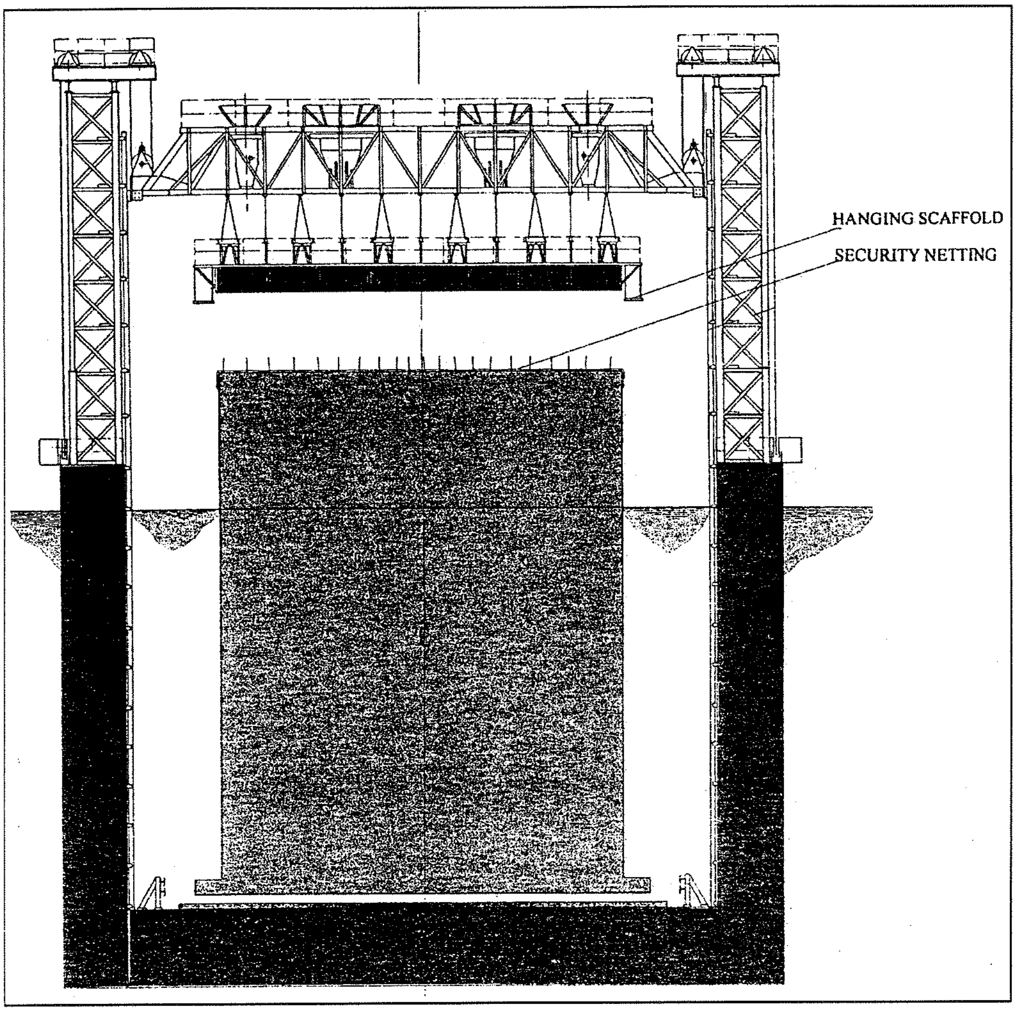


Figure 8.7 Construction phase VI.

Phase VI
To launch the caisson, the dock is ballasted with water in order to increase its immersion until the caisson floats up under the effect of its own hydrostatic uplift. The caisson can now be towed out to a storage location. This process takes half a day.

The fabrication of one caisson takes 18 working days. A scheme of the working schedule is presented in section 8.2.5.

8.2.2 Formwork

The constant dimensions of the caisson shaft form ideal conditions for sliding formwork. The sliding formwork (Figure 8.8). of the shaft consists of a couple of steel panels (1) of approximately 1.0m high suspended by clamps (2) and installed according to the wall geometry. To avoid concrete dragging during striking the panels are mounted slightly flared at the bottom and contact surfaces will be lubricated. The sliding formwork is raised by hydraulic screw jacks (3) fixed to bars (4) which are inserted in the caisson walls and rest on the caisson slab. Concrete and reinforcement steel will be applied from the working floor (5 and 6). Control and completion of the freshly emerged concrete from the formwork is done from scaffolding (7 and 8) hanging from the clamps. Control of equal lifting is done by means of plummets (12) or laser equipment.

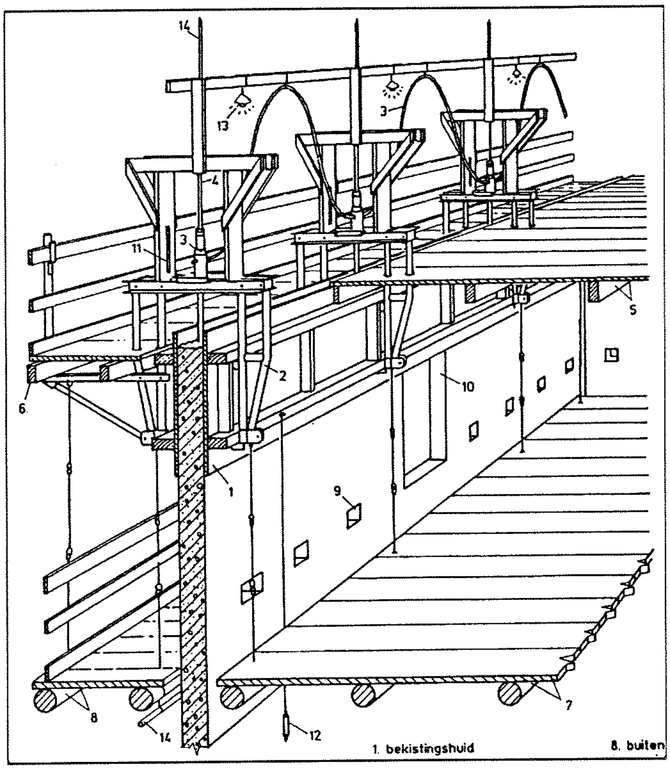


Figure 8.8 Sliding formwork [lit.30].

8.2.3 Concreting

On the quay besides the FCY there will be a concrete factory. The concrete will be supplied to the placement location by belt conveyers, land fed or by a bucket crane, or alternatively by concrete pumps. Before each pour the concrete joints are carefully cleaned by means of compressed air. Spacers will keep rebar cover constant.

Base slab
The base slab sections will be concreted in one continuous pour of one day. For the volume of the base slab ($1625m^3$) a concrete pouring capacity of $1625m^3/24$ hours = $70\ m^3$/hour is required.

Shaft
The surface area (horizontal cross section) of the shaft $A_{shaft}= 376m^2$ and the lifting speed of the slip formwork is v_{lift}=0.18 m/ hour, this means a concrete pouring capacity of $376m^2$ x 0.18m/hour = $70\ m^3$/ hour is required.

8.2.4 Stability during construction

As the base slab is poured, the deck of the FCY gradually sinks into the water. In this phase the overall buoyancy of the FCY must be sufficient to provide a dry construction area. In the next phase of construction the shaft is poured. Figure 8.9 illustrates that in this construction phase the shaft rises much more quickly than the pontoon sinks. In other words in this phase the caissons positive buoyancy develops much more quickly than its gravity weight.

Eventually, when the walls have risen to a certain height (see point A), the element has so much buoyancy that it is capable to float without support from the FCY. In order to guarantee sufficient contact pressure between the caisson and the FCY deck during the final construction phase, the caisson is filled with inert ballast material to provide negative buoyancy.

Compartments within the FCY will also be filled in order to guarantee stability[2] of the caisson-FCY-system. A pump system connected to a series of tanks is also required to compensate for accidental asymmetric concrete pours.

It is apparent that during construction of the caisson the bottom slab and the walls must resist the hydrostatic pressure. During the construction phase these forces will be considerably less than the final outward ground pressure during the operational phase of the element, and therefore can be absorbed by the construction without problems. The buoyancy of the FCY, B(FCY), must be sufficient to guarantee a dry working space during construction of the base slab and the first lift of the shaft.

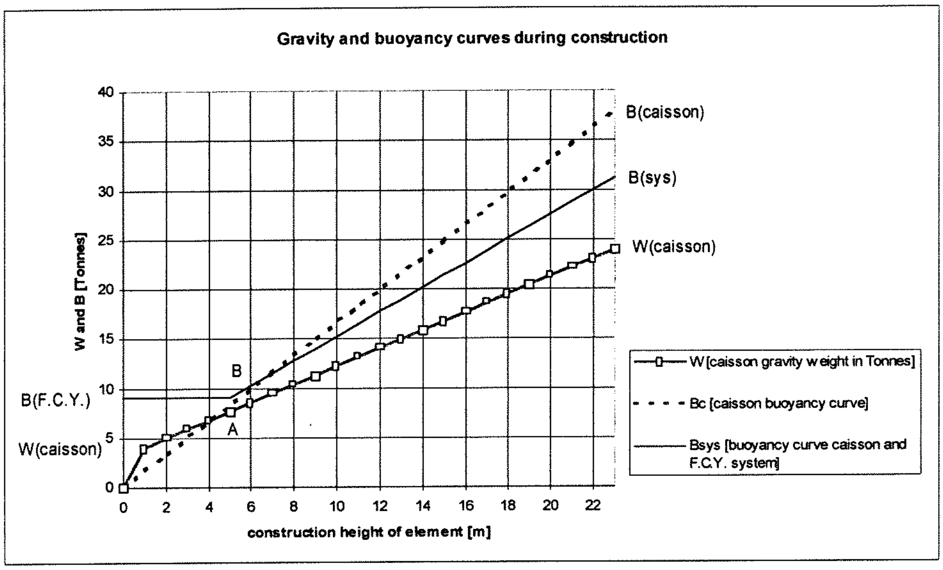


Figure 8.9 Gravity and buoyancy during construction.

8.2.5 Logistic process

The construction works of the total caisson breakwater for the Maasvlakte 2 involve the following activities:

1. Preparation of the rubble mound foundation (section 9);
2. Construction of caissons (section 8);
3. Transport of caissons to breakwater site (section 10);
4. Placement of caissons on the rubble mound foundation (section 10);
5. Filling and capping of the caisson (section 10);
6. Installation of the top layer bottom protection (section 10);

Total available construction time[3]:	3.5 years (=3.5x48 wk=168 wk)
Total required length of caisson breakwater:	4.000m
Caisson length:	76.65m
Number of caissons to be placed:	53+2=55 (2 caissons reserve)

The critical path of the construction cycle is determined by the placement of the caissons on the foundation. All the other works can be tuned to synchronise this procedure. The available construction time for one caisson is determined by the following aspects:

[2] See Appendix L: 'Caisson stability'.

[3] Total available construction time has been discussed in Appendix N: 'Maasvlakte 2 project'.

Total available construction time for breakwater:	168 wk
Installation construction site:	6 wk
Unworkable time (e.g. frost, 3 wks/year)	9 wk
completion works after placement last caisson:	6 wk
available construction time for caissons:	147 wk

Caissons to be constructed per week: 55/147 = 0.4 caissons/ wk, so each 2.7 weeks, or 19 working days, one caisson must be completed. This value gives an indication of the available time for the construction cycle of one caisson.

The following options can be maintained for the capacity of the construction yard:
- One single construction facility which fabricates one caisson every 19 days;
- Two construction facilities which each fabricate one caisson every 38 days;
- Three construction facilities which each fabricate one caisson every 57 days;
- Four construction facilities which each fabricate one caisson every 76 days;

In this phase of the design it is assumed that one FCY will be installed to fabricate all 55 caissons.

The logistic process of the caisson construction is based on the following assumption:

The construction activities will be full continuous. 4 shifts of 6 hours will continue works 48 weeks per year 7 days a week. Based on this labour capacity the following time work schedule is set up for the construction of one caisson:

Phase:		activity:		1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.	14.	15.	16.	17.	18.			
			Days	m	t	wd	th	fri	sat	s	m	t	wd	th	fri	sat	s	m	t	wd	th	fri	sat	s
0.	1.	construction works																						
	2.	prepare floating dock for construction cycle (maintenance)	1																					
	3.	formwork maintenance	1																					
I.	4.	install base slab reinforcement	1																					
II.	5.	install base slab side formwork	1																					
III.	6.	slab concreting	1																					
	7.	slab hardening	2																					
	8.	remove base slab formwork	0.5																					
IV.	9.	install shaft formwork	0.5																					
	10.	install shaft reinforcement (5 lifts)	5																					
	11.	concreting of shaft[1] (5 lifts) (5x4,5m)	5																					
	12.	shaft hardening	1																					
V.	13.	caisson finalisation (stability trimming)	0.5																					
VI.	14.	caisson launching	0.5																					
	15.	storage	0																					
0.'	16.	construction works (next caisson)	18																					
Gangs:			-	2	3	3	3	2	2	3	3	3	3	3	3	3	3	3	3	3	2	2	3	3

Table 1. Planning of construction activities.

Remarks:
- construction time 1 caisson: 18 work days, (18 calendar days);
- 4 shifts of 6 hours, 24/24, also weekends;
- Gang consists of 4 men and 1 leader;

[1] Slip speed = 0.18m/hour.

Tender program of works Maasvlakte 2 caisson breakwater; 4.0km section for construction phase 1						
General	**year:**	**2006**	**2007**	**2008**	**2009**	**2010**
design	**month** / **activity**	**1 2 3 4 5 6 7 8 9 10 11 12**	**1 2 3 4 5 6 7 8 9 10 11 12**	**1 2 3 4 5 6 7 8 9 10 11 12**	**1 2 3 4 5 6 7 8 9 10 11 12**	**1 2 3**
	Total construction time					
	• evaluation of tender and award of contract					
	• soil investigations (24 weeks)					
	• detailed design (36 weeks)					
construction	**Milestones**					
	• installation of construction site (6 weeks)					
	• fabrication of caissons (55 units) (142 wks construction +9 unworkable wks)					
	• construction of foundation (124 weeks)					
	• dumping of levelling layer (6x2.5 weeks)					
	• transport and placement of caissons (6x2.5 weeks)					
	• filling caissons and capping (6x2.5 weeks)					
	• completion of bottom protection (6x2.5 weeks)					
	• finishing (4 weeks)					

Table 1. Tender program of works Maasvlakte 2 caisson breakwater (4.0 km section for construction phase 1).

Remarks:

1. Placement of the caissons on the foundation will be executed during the summer months (from April to September). In this period the average maximum wave height is less than 1.00m for 53% of the time;
2. In total 55 caissons will be constructed (2 more than required as reserve). Construction of one caisson will take 18 days;
3. 9 weeks have been reserved for unworkable days (e.g. frost);

The following time schedule has been set for the construction works:

date:	activity:
± 2001	Governmental discussion and juridical procedures of Maasvlakte 2 construction phase 1.
2003	Decision of Maasvlakte 2 phase 1 layout and construction.
2006	Begin construction of phase 1 (award of contract).
2010	Completion of phase 1 (10 km^2).

8.2.6 Demands of the construction site

A possible location for the floating construction site of the caissons is the Europe Harbour. This harbour is situated on Maasvlakte 1, close to the Maasvlakte 2 caisson breakwater site, and possesses over free area with sufficient water depth. The distance from this caisson fabrication site to the construction site of the breakwater is approximately 7 nautical miles. Appendix A: 'Geographic boundary conditions' shows this location.

In order to execute the construction of the large number of caissons, the construction site must meet the following requirements:

As the principle of the construction works is 'conveyor belt' it is very important to prevent stagnation of all activities of the construction process. One aspect which could lead to this, is delayed delivery of raw construction materials. It will be assumed that construction works must be able to continue for 2 months without delivery of construction materials as sand, gravel, cement or steel reinforcement. In a time period of 2 months 3 caissons are constructed. With one caisson under construction this means that there must be a storage capacity for the construction materials of 4 caissons. For the steel reinforcement the required space is 15.000m^2 (6.000 Tonnes). For the concrete the required space is 40.000m^3 of sand, gravel and cement. When the storage of these materials is assumed as cone-shaped, the required surface area for the concrete raw materials is 11.000m^2.

Remarks:

- The terrain must be accessible by roads and waterways for the provision of equipment, materials and people;
- Activities must not cause excessive inconvenience to the environment;
- There are two options concerning the prefabrication of the steel reinforcement, one option is to install a reinforcement bending centre on the construction yard, another possibility is to have the reinforcement prefabricated elsewhere in bending centres/ factories and shipped to the construction site. In this study it will be assumed that the steel reinforcement will be prefabricated in factories located elsewhere and supplied to the construction yard in elements. These elements can be stored on the quay, and installed on the floating construction yard by a land based crane. In order to prevent delay of the total construction sequence due to late deliveries, three complete sets of steel reinforcement are in storage on the construction yard.
- installation costs of the construction terrain are taken up in overhead costs;

A plan view of the floating construction site is presented in Figure 8.10.

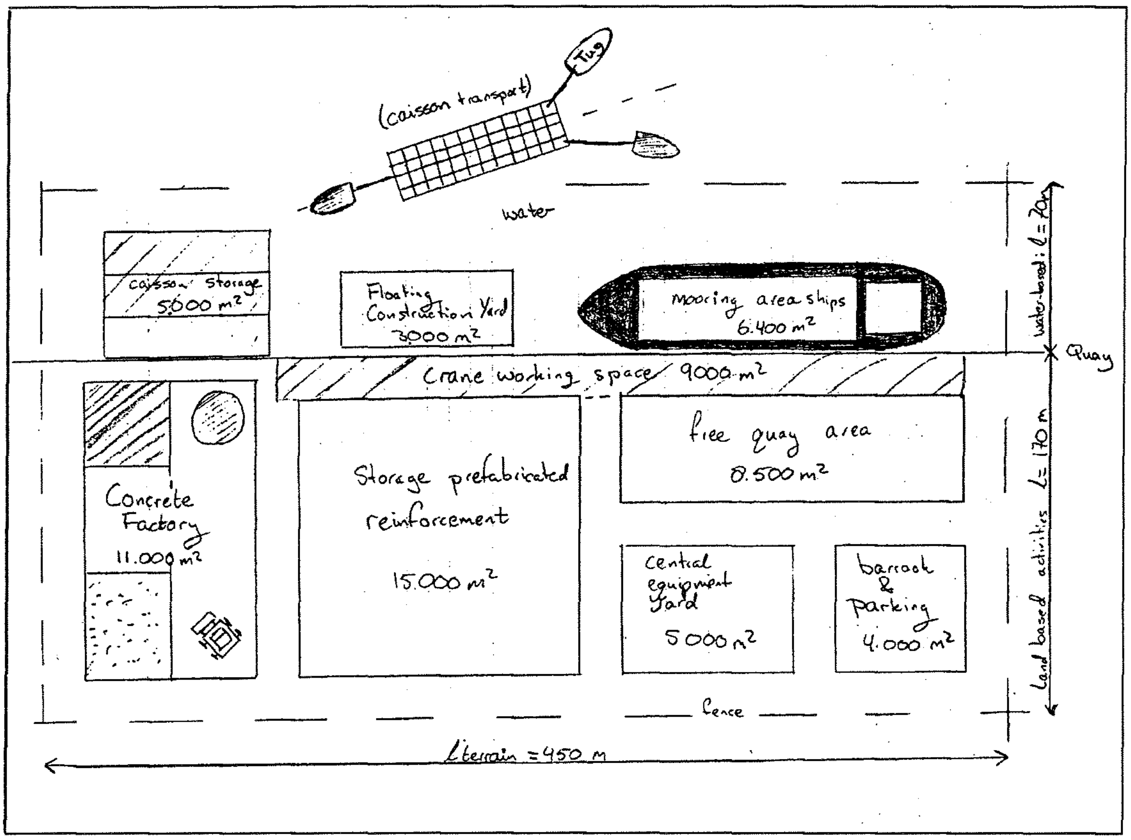


Figure 8.10 Plan view floating construction site.

8.2.7 Equipment & costs

The costs of the floating construction yard are based on unit prices, how these have been set is presented in the footnotes.

Costs F.C.Y.	**costs (new) [Nlg.]**
Platform[4] (pontoon 80x25x6):	8.500.000,-
Steel towers (vertical floats)[5]	1.700.000,-
4 lifting towers[5];	1.500.000,-
Lifting frame from which the concrete distribution cars are suspended[5];	2.200.000,-
pontoons for base slab reinforcement[4];	850.000,-
Other: set of pumps to distribute the ballast water; electrical installations;	1.000.000,-
Total floating construction yard costs (formwork not included)	**15.750.000,-**

Table 8.1 Overview equipment costs.

[4] Document: 'Grondslagen voor de kostennormen, Pontoons'.

[5] Costs of the steel towers and framework (Nlg 6,50/kg) are based on the amount of steel required for these constructions. Market price of steel (Nlg. 1,30/kg), fabrication/installation (Nlg 3,50/kg), conservation (Nlg 1,-/kg) and 10% other costs.

The new costs of the floating construction yard are approximately Nlg. 15.750.000,-. Assume 50% of these costs will be written off on the construction works of the Maasvlakte 2 caisson breakwater. This means the investment costs for the construction yard are Nlg. 8.0 million. Based on the construction of 55 caissons construction costs per caisson are Nlg. 145.500,- , or based on construction of a 4.0km long breakwater section Nlg. 2.000,-/m. The costs of the formwork are presented in Table 8.2.

Costs formwork	**costs [Nlg.]**
land based crane; 3T lift capacity at 40m	1.000.000,-
Formwork[6]: base slab formwork (m^2); slip-formwork for the caisson shaft (m^2);	 3.000.000,- 8.000.000,-
Total formwork costs	**12.000.000,-**

Table 8.2 Overview equipment costs.

It is assumed that the costs of the formwork, Nlg $12.0 \cdot 10^6$ will be completely written off on the construction works of the Maasvlakte 2 caissons. Based on the construction of 55 caissons formwork costs per caisson are Nlg. 218.000,- , or based on construction of a 4.0km long breakwater section Nlg. 3.000,- /m.

8.3 Conclusions

The construction costs of the caissons for the Maasvlakte 2 breakwater are Nlg. 20.0 million, or Nlg. 5.000,-/m. This is based on the assumption that 50% of the construction costs of the Floating construction yard are written off to this project, and that the formwork costs will be completely written off on this project.
Overview construction yard costs:

aspect	**costs per m breakwater [Nlg]**	**costs for 4 km long section [Nlg.x10^6]**
Floating construction yard:	2.000,-	8.0
Formwork:	3.000,-	12.0
Total construction yard costs:	**5.000,-**	**20**

Table 8.3 Overview construction yard costs.

Aspect	**costs per m breakwater [Nlg]**	**costs for 4 km long section [Nlg.x10^6]**
Labour: (full time construction)	34.600,-	138.5

Table 8.4 Labour costs.

---

[6]Slip-formwork: slip formwork will be completely renewed once. Total required slip formwork: 2x $1.288m^2$= $2.576m^2$, costs slip-formwork: Nlg. 1.000,-/m^2, and 15% engineering costs, total costs slip-formwork: ($2.576m^2$xNlg. 1.000,-/m^2)x1.15 = Nlg. $2.96 \cdot 10^6$.
Handling: 10 men, 55 caissons, 22.00 m high, slip speed 0.18m/hour. Total labour costs: 10x 55x 22/0.18 = 67.200 man hours, at Nlg. 75,-/hour, total labour costs: Nlg $5.0 \cdot 10^6$.
Total costs slip formwork: Nlg. $2.96 \cdot 10^6$ + Nlg.$5.0 \cdot 10^6$ $\approx$ Nlg. $8.0 \cdot 10^6$.

Costs base slab formwork: A_{floor}= $1618m^2$ x Nlg. 20,-/m^2 x 55 caissons = Nlg. $1.78 \cdot 10^6$ and A_{side}= $196m^2$xNlg 100,-/m^2x55 caissons = Nlg. $1.08 \cdot 10^6$
Total costs base slab formwork: Nlg. $1.78 \cdot 10^6$ + $1.08 \cdot 10^6$ $\approx 3.0 \cdot 10^6$.

9. Rubble mound foundation

9.1 Introduction

The design of the rubble mound foundation has been discussed in section 5.3 'Assumptions of the rubble mound foundation'. In this section the construction method of the foundation will be briefly discussed. An important aspect of the rubble mound foundation concerns the flatness of the top layer which is required in order to place the caissons without danger of damage to the base slab due to protruding rocks. Also the costs of the foundation are determined in this section.

A note must be made of the fact that the design of the rubble mound foundation in this phase is to give an indicative view of the construction and the costs. During a later phase of the design the rubble mound foundation must be dimensioned more accurately. Appendix M: 'Technical drawings' contains a cross section of the caisson breakwater foundation.

9.2 Subsoil

The top soil layer of the breakwater site consists of sandy clay which has insufficient bearing capacity to form the foundation of the caisson. Therefore the top 6 meters of soil will be excavated by cutter suction dredgers (Nlg 10,- /m^3) and replaced with clean sand (Nlg 5,-/m^3) which is compacted with specialised equipment (Nlg 10,-/m^3)[1] in a similar manner as was done for the Eastern Scheldt Barrier (Figure 9.1). In the cross section of the breakwater it can be seen that the volume of this sand improvement is 384 m^3 per meter breakwater length. The total costs of the soil improvement are Nlg. 25,-/m^3x384m^2 =Nlg. 9.600,- /m^1 breakwater.

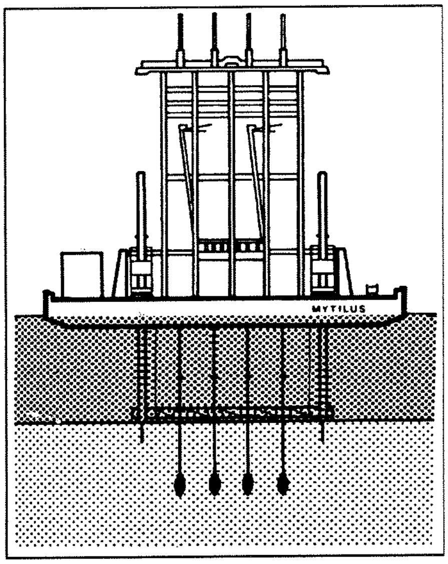


Figure 9.1 Ship to compact newly dumped sand.

9.3 Rubble mound foundation

The Rubble mound foundation consists of several layers of which the design has been discussed in section 5.3. The bottom two filter layers (0-60 mm and 80-200 mm) will be dumped from trailing dredgers (Figure 9.2 top) and the 60-300 kg stones will be dumped from barges with side unloading (Figure 9.2 bottom). The levelling layer which consists of stones 10-60 kg is used to smoothen out the

[1] The new costs of the compacting ship 'Mytilus' are Nlg 30 million. It will be assumed that 50% of these costs will be written off on compacting the 1.5 million m^3 of newly dumped sand for the MV2 caisson breakwater works. This gives: (Nlg 30.10^6x 0.50) / $1.5.10^6$ m^3 ≈ Nlg 10,-/m^3. Note that the compacting process for the MV2 breakwater is less complex as those for the Eastern Scheldt works.

top layer in order to provide a sufficient flat surface on which the caissons can be placed without danger of rock penetration into the base slab. This levelling layer will also be dumped from barges with side unloading. Just before placement of the caisson on the foundation, a tug dragging a heavy steel beam (bulldozer) over the levelling layer will even out the top layer.

After placement of the caisson on the foundation 10-60 kg and 60-300 kg stones are placed on the levelling layer. These layers prevent the material of the levelling layer to be washed out through the bottom protection from underneath the caisson, eventually leading to piping. Instead of a geometric filter 'gravel bags' can be installed to immobilise the levelling layer. These are geotextile bags which are filled with gravel and placed by the caisson toe.

Finally the bottom protection rocks of 6-10 tons on the sea-side and 1-3 tons on the harbour side are placed. In order to prevent damage to the caisson the rocks will be placed by a crane vessel nearby the caisson. Further away the rocks can be dumped from barges with side unloading.

In the cross section of the breakwater it can be seen that the volume of the rubble mound is approximately 315 m^3 per meter breakwater length. An average price of rubble mound (material costs and placement) can be set at Nlg 50,-/ton or Nlg 130,-/m^3 (ρ_{rubble}=2650 kg/m^3). The total costs of the rubble mound are Nlg. 130/m^3x315m^2= Nlg 41.000,- /m^1 breakwater.

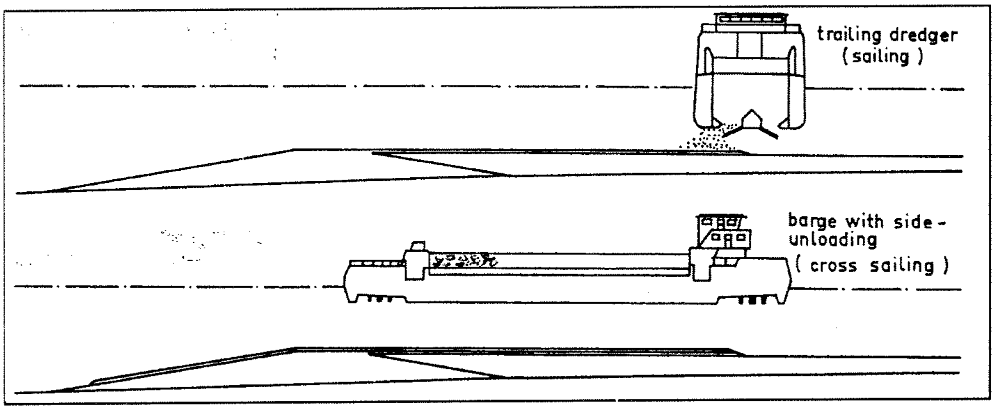


Figure 9.2 Barges for dumping rubble mound layers.

9.3.1 Scrading

There is much discussion concerning the required smoothness of the levelling layer which can be achieved by dragging a steal beam over the top layer. An alternative is the so called Multi-Purpose Scrader (Figure 9.3). This specialised equipment deposits and levels the toplayer using a telescopic fall pipe which is mounted on a surface support vessel. The bottom of the pipe is kept at the desired level. The scrade material is fed into the fall pipe by a conveyor. By moving the pipe horizontally (the bottom making contact with the scrade layer), material is deposited and levelled at the same time (Figure 9.4). By applying a negative overlay, trenches are created. These trenches can be especially useful in the drainage of water during the placement of the caisson on the foundation. Just before the caisson bottom contacts the levelling- or scrade layer, a large amount of water must be pressed away between the two surfaces, which can cause hovering of the caisson. This hovering may cause problems with accurate placement of the caisson. The scrading technique was developed and applied by Boskalis for the caisson foundation at Pasir Panjang, Singapore in 1996. Since then it has been used for several other projects, amongst which the gravel bed foundation of the Øresund tunnel between Denmark and Sweden. If required the accuracy of this system can be approximately 25 mm. Figure 9.4 Illustrates the configuration of the scrading system.

A note must be made that before this equipment can execute works under the North Sea wave conditions, (expensive) adaptations must be made to the vessel.

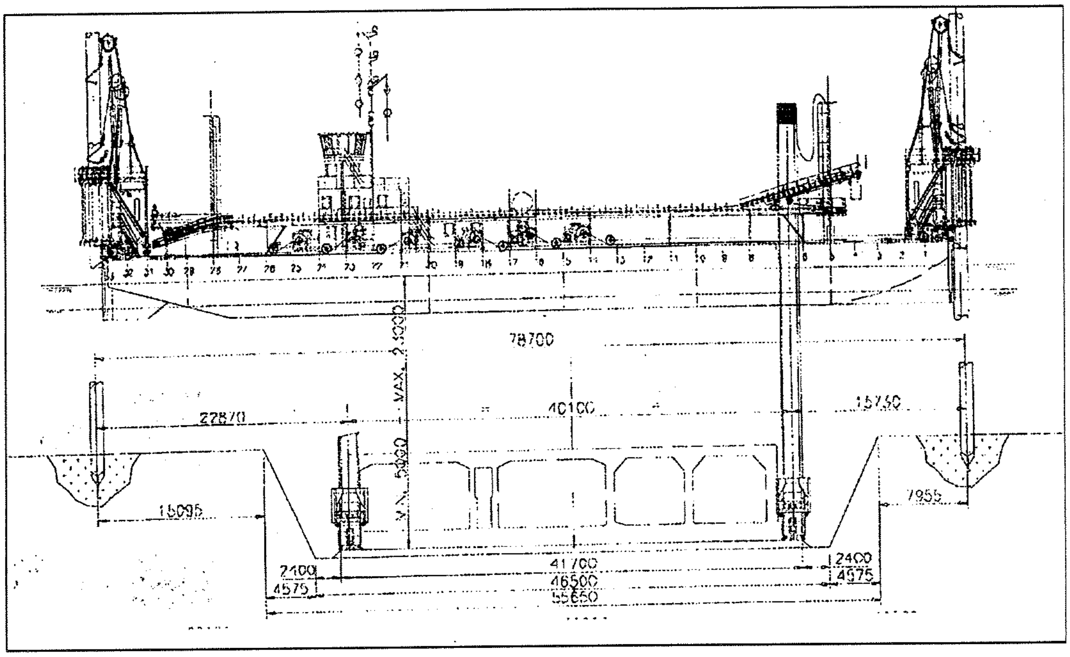


Figure 9.3 General plan of the Multi -Purpose Pontoon[2].

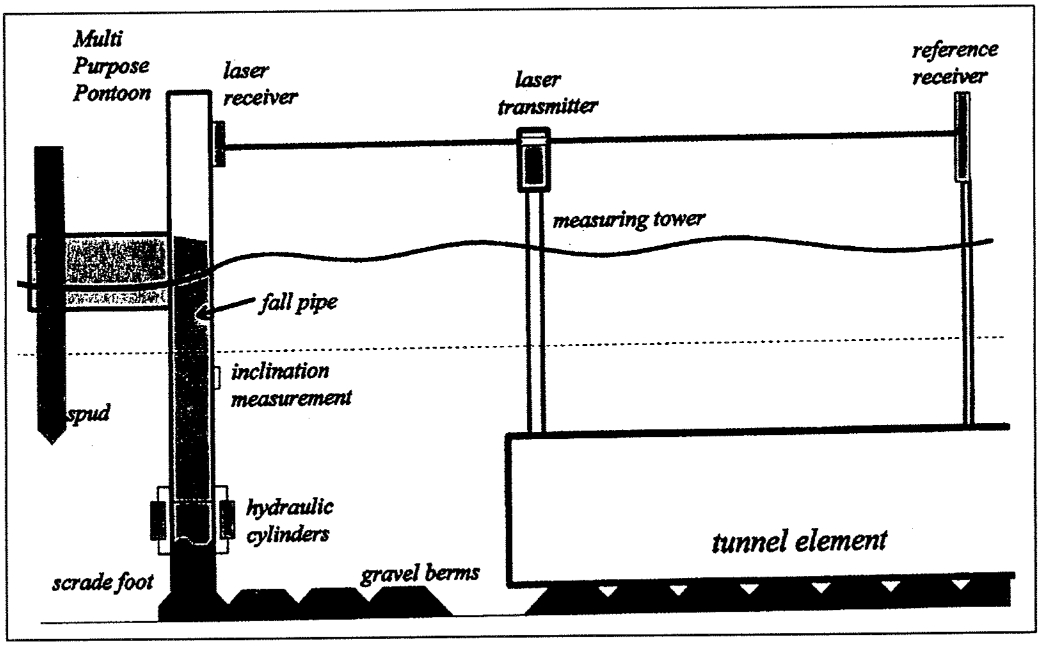


Figure 9.4 Configuration of the scrading system[2].

[2] Documentation Boskalis: Multi-Purpose Scrader Concept: New Technology for Seabed Treatment, R.F.J. Neelisen a.o.[lit.15]

9.4 Conclusion

The costs of the rubble mound foundation will be approximately Nlg 50.600,-/m. These costs include the materials and equipment required for the improvement of the subsoil and construction of the rubble mound and will be used in the cost calculation of the total breakwater. It must be noted that the foundation discussed in this section is a first design of the construction, and must be analysed in more detail in a later phase. Table 9.1 gives an overview of the foundation costs:

aspect	**foundation costs per m breakwater [Nlg.]**	**foundation costs for a 4 km long section [Nlg. x10^6]**
excavation of subsoil and sand improvement	9.600,-	38.4
rubble mound	41.000,-	164
Total foundation costs:	**50.600,-**	**202.4**

Table 9.1 Overview breakwater site preparation costs.

10. Transport and placement of the caissons

10.1 Introduction

The transport and placement of the caissons can be done with simple and inexpensive equipment. Several aspects influence the total transport and placement time, for example the total number of elements which have to be placed and the amount of unworkable days due to high waves.

10.2 Procedure

Before transport of the caissons they must be trimmed in order to guarantee stability during transport, and transport installations as boulders, winches and positioning equipment must be installed.

As the transport route from the Europe Harbour to the Maasvlakte 2 caisson breakwater site is within the harbour of Rotterdam and the extremely busy shipping lane Maas Geul, it is essential that this procedure is completely under control. The hinder to the shipping during transportation of the very large amount of caisson elements must be limited to an absolute minimum Therefore 3 tugs will be used for transportation of the 55 caissons. The distance is approximately 7 nautical miles, which with an average speed of 2 knots will be sailed in 3.5 hours.

The caissons will be placed on the foundation in the summer months, from April to September, when the average wave heights are lower than 1.0m for 53% of the time, see Figure 10.5.

The 55 caissons will be placed in 5 shifts of 9 caissons and one shift of 10 caissons. The required transport and placement time for one element is 1 day. Based on 47% unworkable time, the placement time of 9 caissons is 13 days. Total tug costs are Nlg. $3.1 \cdot 10^6$,-[1] or Nlg. 775,-/m.

The planning must be in such a way that the caisson arrives at its destination shortly before high tide, so that the caisson can be floated above the foundation and connected to the winches of the previously installed caisson. At the moment of high tide the currents are least for a time period of approximately 2 hours. During this time the caisson must be accurately positioned and filled with water so that it sinks onto the foundation. When the base slab is touching the foundation bed, ballasting is interrupted in order to definitively set the caisson making use of one or more topographic stations. The caisson cells are grouped into independent compartments, cells of the same compartment are interconnected through holes built into the inner walls during construction.

Total costs tugs, pumps, positioning equipment = Nlg. 1.000,-/m

Next the caisson cells must be filled with sand. Large head level differences between neighbouring cells will lead to large forces on the inner cell walls, and therefore the cells must be filled more or less evenly. The maximum acceptable head level difference is 5.0m between neighbouring cells (see section 11.2). This will require specialised pumping equipment. The total volume of sand to be pumped into the cells is 27.150m^3. The costs of the sand itself has been taken up in section 9.7 Materials per caisson, and the pumping costs of the specialised equipment are assumed to be Nlg. 4,-/m^3. Total sand filling costs for 55 caissons: Nlg. 5.973.000,- ,or Nlg. 1.500,-/m.

Once the cells are filled, capping in the form of 0.5 m high prefabricated concrete panels can be placed, forming the caisson roof. This can either be done by crane vessels or by construction over the crest. The costs of the concrete capping plates (Nlg. 250,-/m^3) is approximately Nlg. 203.000,-/caisson (Nlg. 2.700/m).

After placement of the caisson, the bottom protection can be placed. This is discussed in section 9.3.

---

[1] Tug costs: 6 shifts x 3 tugs x 13 days x 24 hours x Nlg. 550,-/hour = Nlg. 3.088.800,- = Nlg. 775,-/m.
Costs pumping water: Nlg. 0,10 /m^3 x 27.150 m^3 x55 caissons ≈ Nlg. 149.325,-/4000 = Nlg. 36,-/m.

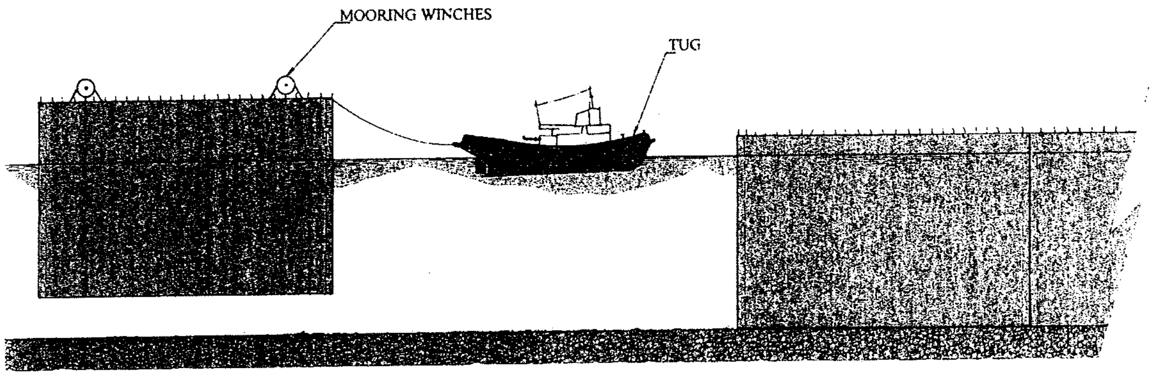


Figure 10.1. Caisson towing to site.

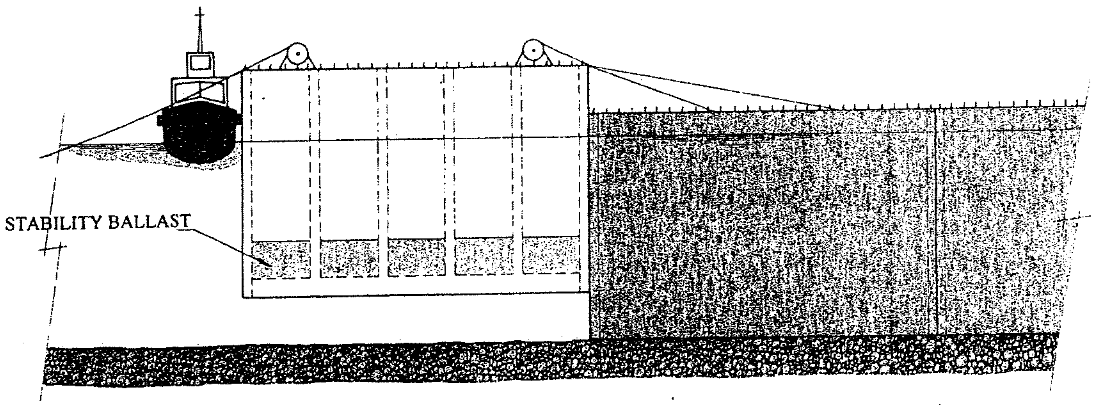


Figure 10.2. Temporary mooring.

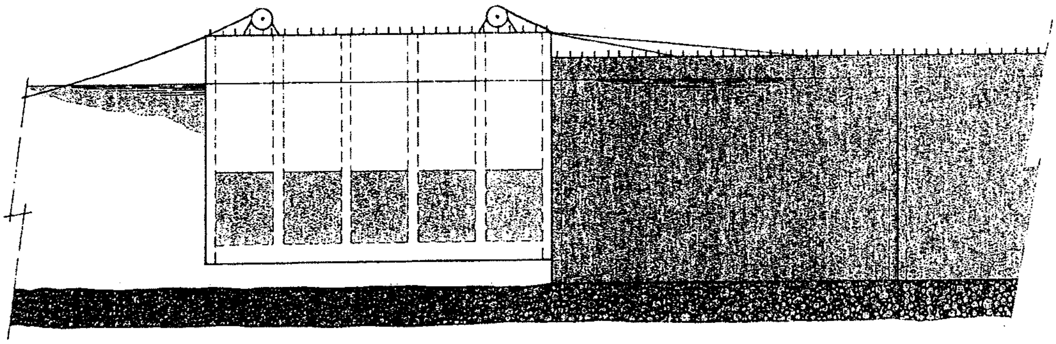


Figure 10.3. Water ballasting.

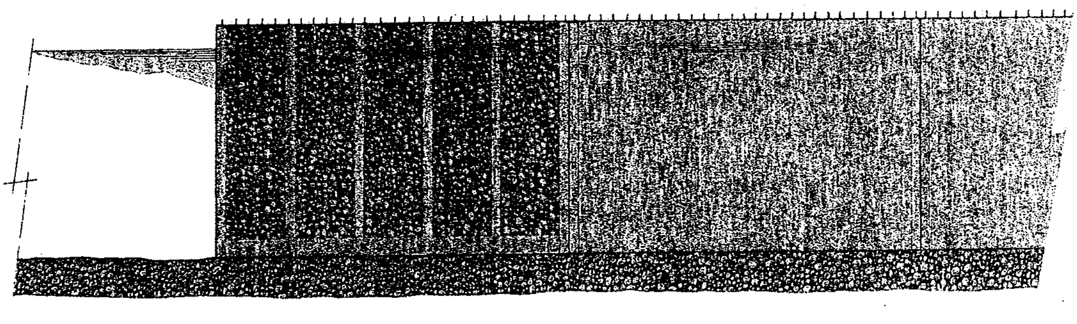


Figure 10.4. Caisson placement and sand filling.

The average wave heights are lower than 1.0m for 53% of the time from April to September.

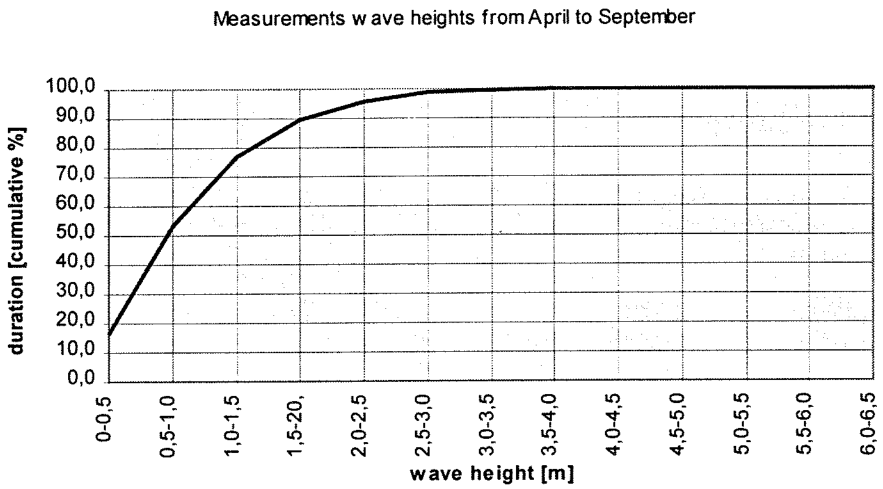


Figure 10.5. Measurements wave heights April to September.[2]

10.3 Conclusion

Based on a caisson length of 76.65 m the total transport and placement costs are Nlg. 20 million, or Nlg 5.000,-/m. These costs include all the equipment and material costs required to transport the caisson from the Europe Harbour to the Maasvlakte 2 caisson breakwater site, position and place the caisson on its foundation, fill the cells with sand and place the capping. These costs will be used to calculate the total construction costs of the breakwater.

As the transport and placement costs of the caissons are relatively low, further study should be done to investigate whether it is economical to construct the breakwater of more caissons of shorter length. The same construction time can be achieved by installation of more caisson placement teams.

Table 10.1 contains an overview of the total transport and placement costs of the caisson.

aspect	costs per m breakwater [Nlg.]	costs for 4 km long section [Nlg.]
tugs and positioning equipment, winches, boulders, hydraulic pumps	1.000,-	$4.0 \cdot 10^6$
sand filling process[3]	1.500,-	$6.0 \cdot 10^6$
capping-plates	2.700,-	$10.8 \cdot 10^6$
total transport and placement:	5.200,-	$20.8 \cdot 10^6$

Table 10.1 Overview caisson transport and placement costs.

[2] 3 -hour interval measurements taken at the Euro0-platform from 1985 to 1991.
[3] Only sand pumping costs are considered here, delivery costs of sand are taken up in section 9.7 Materials per caisson.

11. Reuse of the caissons

11.1 Introduction

The Mainport Rotterdam has stated that the caisson breakwater must be reusable after a time period of several years without any major problems or expenses. The repositioning of these caissons is a relatively simple and cheap procedure, which doesn't require much specialised equipment. An important aspect of the reuse of the caissons concerns the protection of the harbour during the repositioning phase.

11.2 Procedure

In order to mobilise the caisson, the capping plates must be removed by cranes (over the crest) and the sand in the caisson liquefied by means of jets. A series of Toyo pumps can remove the dense liquid and replace it with sea water (Figure 11.1). In order to prevent high ground pressure on the inner walls of the shaft, care must be taken to prevent large head-level differences between neighbouring cells. This can be achieved by removing the sand in a stepwise manner, see Figure 11.2. The maximum acceptable head level difference is approximately 5.0m.

After the cells are emptied of their sand content, controlled uplifting can be achieved by pumping the water from the caisson cells. The caissons can now be tugged to their new location and placed on the foundation as was done in the first phase.

An important aspect concerning the reuse phase concerns the protection of the harbour during this phase. The hinder to shipping and downtime of the harbour must be kept to an absolute minimum.

A possible way to execute the works is by first extending the Northern Dam and southern coastline (beach), and then repositioning the caisson breakwater (see Figure 11.3). During the repositioning phase of the caissons, part of the harbour will temporarily be shut down (downtime). In order to limit this time period, several teams of tug boats will simultaneously work full time to transport and place the caissons at their new location. Costs of such *'caisson-repositioning-teams'* are relatively low. Under the assumption that one team can reposition one caisson a day, and formation of 3 such teams, the 55 caissons of the breakwater can be repositioned in 55/3 = 18 days. These works will be executed in the summer months, when the chance on high waves is least. Under the assumption of 47% unworkable days (see section10.2), the total placement time will be 27 days. During this time period a section of the harbour must be shut down. After repositioning of the caissons, transhipment can recommence, and the harbour terrain (1000 ha.) can be constructed.

A cost evaluation must determine whether it is economic to reuse the foundation material. In this study it is assumed that new material will be used for the new foundation.

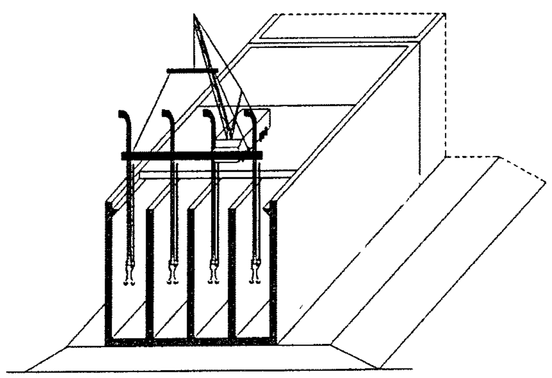


Figure 11.1 Series of pumps to remove sand filling from cells.

Removal of sand during reuse phase:

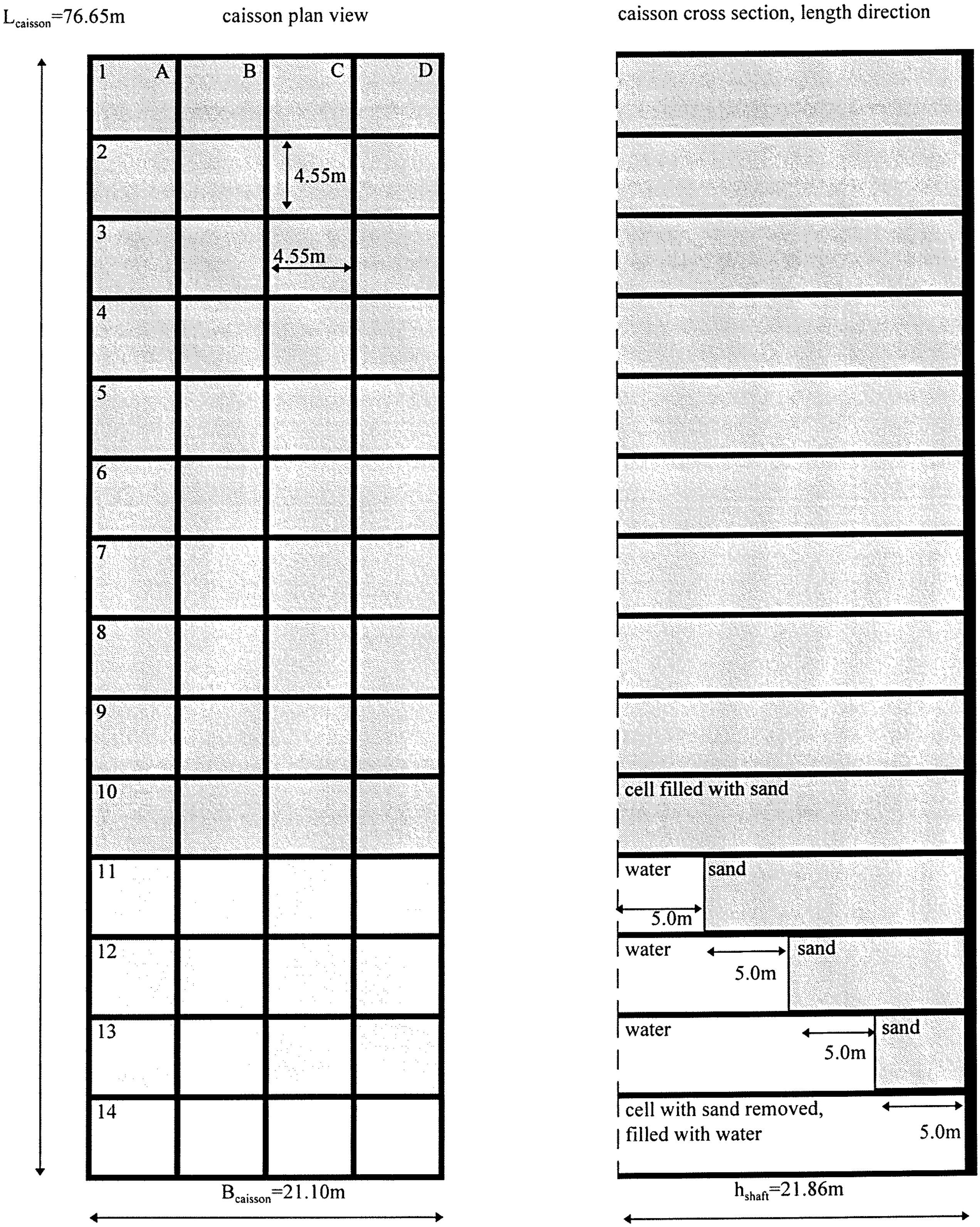
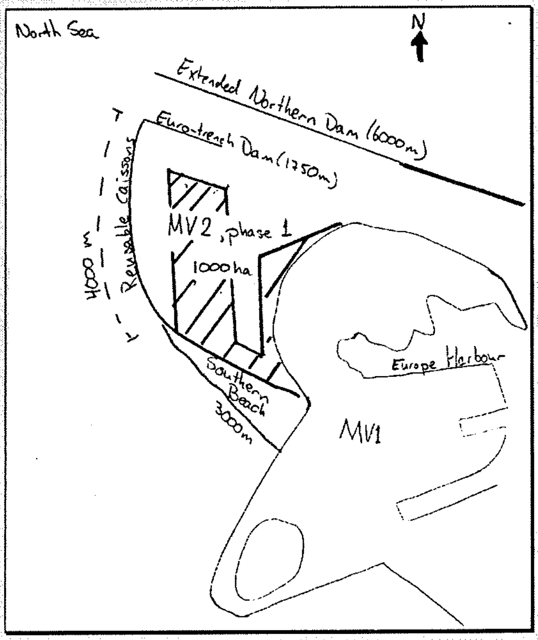


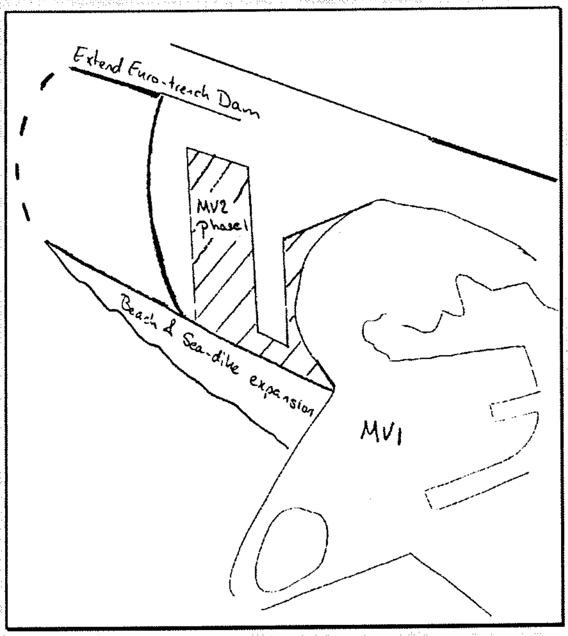
Figure 11.2 Stepwise removal of sand from cells.

Remarks:
- internal cell dimensions: 4.55x4.55m
- max. head level difference of sand between neighbouring cells =5.00m (based on strength of inner walls);

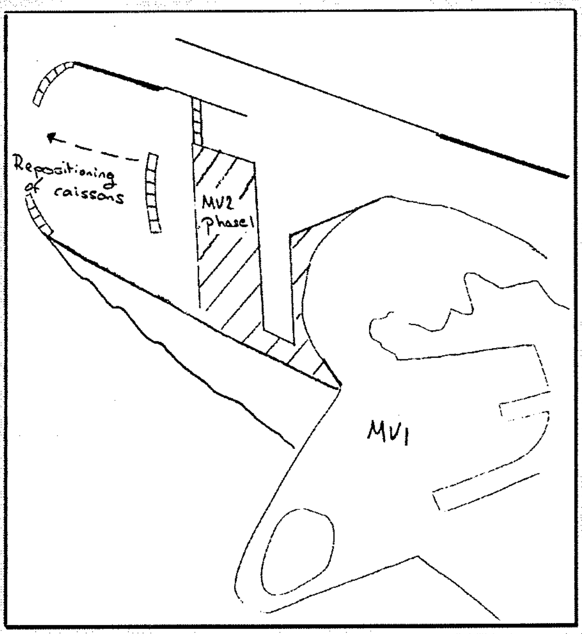
Overview phased execution MV2 construction phases



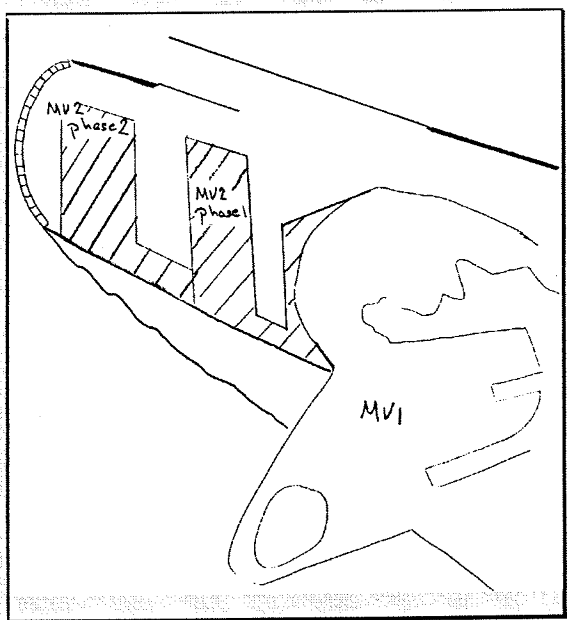
Construction phase 1 MV2
completion year 2010;



Intermediate phase, extension of Euro-trench Dam and Southern beach;
year: 2028;



Intermediate phase, repositioning of caissons,
duration: approximately 1 month
year 2029;



Construction phase 2 MV2.
year 2029;

Figure 11.3 Construction phases of the MV2.

11.4 Conclusion

Due to the relative simple and cheap equipment required to reposition the caissons, the costs of this procedure are relatively low (6% of reuse costs). The main costs of this construction phase are determined by the new rubble mound foundation which is to be constructed (94 % of reuse costs).

During the repositioning of the caissons, a section of the harbour must temporarily be shut down. In order to limit this downtime, 3 'caisson repositioning teams' will work full-time and simultaneously. The total time required to reposition the 55 caissons will be 27 days, 47% unworkable days included.

Aspect	**costs per m breakwater [Nlg.]**	**costs for 4 km long section [Nlg. x10^6]**
tugs and other positioning equipment[1]	1.000,-	4.0
sand filling[2]	1.500,-	6.0
reuse of capping plates	1.000,-	4.0
Subtotal reuse of caissons	3.500,-	14.0
new foundation[3]	57.100,-	228.4
Total transport and placement:	**60.600,-**	**242.4**

Table 11.1 Overview caisson reuse costs (based on 4 km section).

[1] Costs of tugs, pumps and other positioning equipment as calculated in section 10.2.
[2] Sand filling see section 10.2.
[3] Due to the larger water depth the foundation will be 2.0 m higher over a 25.0m wide section than in the first construction phase, and therefore more expensive: 2m x25m xNlg. 130,-/m^3=Nlg. 6.500,-/m.

12 Caisson construction methods

12.1 Introduction

Goal of this section is to compare three different construction methods suited for the Maasvlakte 2 caisson breakwater. The following construction methods will be compared on technical and economical feasibility:

1. Floating construction method, based on the results of this study;
2. Japanese construction method, based on the results of the thesis of C. Spanjers [lit.27];
3. Traditional construction in docks, based on statistics of the Ministry of Transport, Public Works and Watermanagement [lit. 18, 20, 21, 22];

These construction methods are based on the following principles:

a. make use of caisson buoyancy (1 and 3);
b. partial use of caisson buoyancy;
c. no use of caisson buoyancy (2);

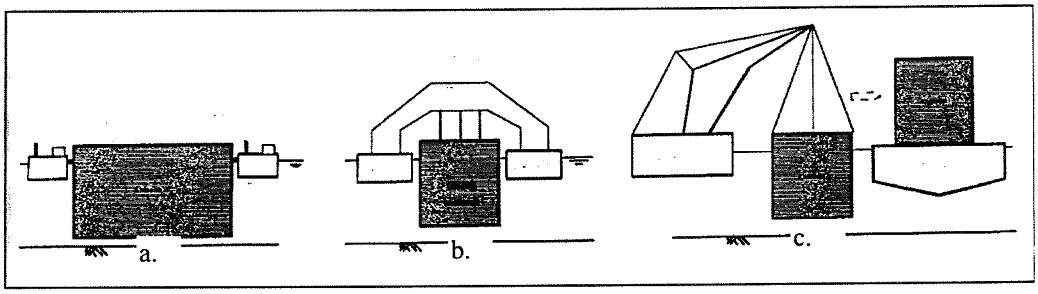


Figure 12.1 Use of caisson buoyancy.

Assumptions made for the costs calculations:

- The construction costs calculated in this section do not contain the following aspects: 17.5% Tax, 5% profit and risk, 5% costs construction site and unpredicted events, 2.5% overhead costs;
- All analysed yards are located on the Maasvlakte 1;
- The costs calculation is based on the construction of a 4 km caisson breakwater section in 4 years;

Remark:
The cost calculations in this section are indicative. As approximately 90% of total construction costs consist of materials and labour and only 10% of the costs are for the construction yard, it must be noted that a relatively low change in e.g. concrete or rubble mound costs will have a large effect on the total construction costs.

12.2 Floating construction yard method

The floating construction method of caissons has been discussed in sections 7 to 11 of this study.

12.2.1 Positive and negative aspects of the floating construction yard method

Positive aspects:
a) no large construction terrain required;
b) no dock and drainage system is required;
c) use is made of the buoyant capacity of the caissons, therefore during the transport and placement phase only tugs and positioning equipment is required;
d) caisson dimensions are not limited by the size of the dock;
e) caissons can be launched one by one;
f) caissons can be stored in floating condition;

Negative aspects:
a) relatively little experience with this construction method in The Netherlands;
b) floating construction yard must be built or bought;
c) deep water is required for the floating construction yard, and a sufficiently deep trench from the construction site to the breakwater site (Europe Harbour meets these requirements);
d) delay of one element will cause delay of all elements;

12.2.2 Overview breakwater costs Floating construction yard method:

An overview of the costs of the Maasvlakte 2 caisson breakwater is presented in Table 12.1.
It must be noted that the accuracy of the values in this table serves to find back the essence of the value in the rest of the text. The costs are not as accurate as might be suggested by the values presented in this table.

activity:	**Floating Construction Yard Method**		
	costs in Nlg /m	**costs for a 4km long section [in Nlg·10^6]**	**percentage [%]**
material costs caisson (section 7.7)	46.455,-	185.9	32.8
labour costs (working full time) (section 7.7)	34.600,-	138.5	24.4
construction yard costs (section 8.4)	5.000,-	20.0	3.5
construction costs foundation (phase 1)	50.600,-	202.4	35.6
transport and placement costs of caissons (section 10.2)	5.200,-	20.8	3.7
Total costs construction phase 1:	**141.900,-**	**568**	**100**
costs construction phase 2: (section 11.4) reuse of caissons: new foundation	 3.500,- 57.100,-	 14.0 228.4	 5.8 94.2
Total costs construction phase 2:	**60.600,-**	**242.4**	**100**

Table 12.1 Overview breakwater costs Floating construction yard method.

12.3 Japanese construction method

12.3.1 Procedure

An alternative method to construct the caissons required for the Maasvlakte 2 is the so called Japanese construction method. Caissons are constructed on a terrain which is above the water level of the connecting river or waterway, and above the ground water level. This way there is no need of an extensive drainage system.

Several caissons are constructed simultaneously on a yard, and are horizontally shifted to the quay by means of specialised equipment (e.g. winches/ hovering/ cranes). From the quay they are placed on pontoons and towed to the breakwater site. The caissons will be lifted off the pontoons by a specialised lifting vessel, and placed on the rubble mound foundation.

In a study done by C.M. Spanjers [lit. 27] this construction method has been analysed and in co-operation with the firm Heeremac -offshore lifting-, a vessel was designed which could place the caissons on the foundation.

In his study Spanjers came to the following conclusions:

- The thickness of the caisson wall is determined by the outward ground pressure of the soil in the cells, this is similar to the caisson designed in this study;
- One aspect is principally different for methods which don't make use of the caisson buoyancy. During the placement of the caisson from the quay onto the pontoon, the caisson is only supported in 2 places (Figure 12.2), this induces large bending moments in the structure, which must be absorbed by large amounts of prestress steel in the base slab. These measures are not necessary for caissons which make use of their own buoyancy;
- Special provisions will have to be installed to connect the lifting frame of the lifting vessel to the caisson. These measures are not necessary for caissons which make use of their own buoyancy (Figure 12.4).
- In order to allow the sea water to flow into the caisson unhindered during placement on the foundation, openings are required in the bottom of the caisson shaft. Provisions will be required to close the openings when the cells are filled with sand. These measures are not necessary for caissons which make use of their own buoyancy;

Caissons of a breakwater constructed in the Japanese method will be constructed slightly different than caissons which make use of their buoyant capacity. The costs of these provisions is considered to be [1] 14% in perspective to the total material costs of the caisson. Total material costs caisson: Nlg. $213 \cdot 10^6$, $\approx$ 53.520/m.

There are many types of construction yards which mainly differ in the specialised equipment used for the horizontal and vertical transport of the element on the yard. The following yard-methods can be distinguished:

a) Segment or sliding method;
b) Hovercraft method;
c) Synchrolift;
d) Japanese-method, (vessel with large lifting capacity);

Further details concerning the different yard construction methods are discussed in the document 'Handbook Specific Tunnel Design - SATO', of the Bouwdienst R.W.S., Department of Dry Infrastructure, Tunnel engineering.

[1] Assume extra prestressing steel is used in the base slab to absorb the forces. The total amount of prestressing steel is 2x that of a caisson which makes use of its buoyant capacity, which will increase the material costs of the caisson by approximately 14%, (see section 7.7 Table 2.). Total material costs caisson: Nlg. $213 \cdot 10^6$, $\approx$ 53.520/m.

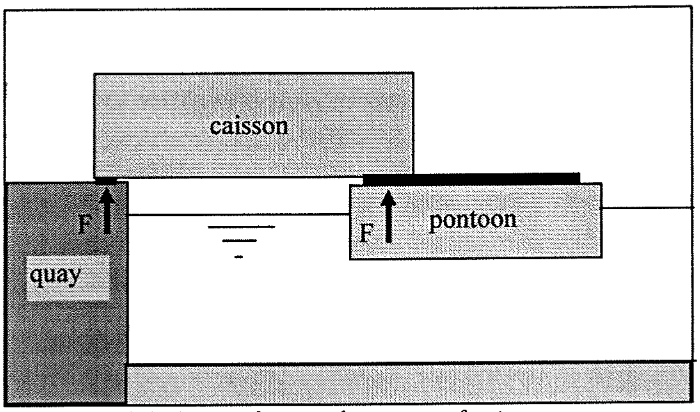


Figure 12.2 Forces during placement of caisson on pontoon.

Costs of construction yard:
Based on the study of C.M. Spanjers it is assumed that the following specialised yard equipment must be installed:

- As the caissons will not use their buoyant capacity, special equipment is installed on the yard for the horizontal shifting of the caissons to the quay;
- 2 sets of formwork and crane installations are required to achieve sufficient construction capacity, costs: Nlg. $12{\cdot}10^6$ (see section 8.2.7);
- Expenses can be saved on the soil excavation costs, the costs of the surrounding dikes, and on the drainage system;

In this study it is assumed that the building costs of a (new) high construction yard on the shore of Maasvlakte 1 with specialised equipment for horizontal shifting of the caissons are of the same order as the building costs of a (new) deep construction dock with an extensive drainage system. The costs of a construction dock are determined in section 12.4.1, Table 12.3, and are set at Nlg. 37.10^6. This dock is suited to construct a 4 km section of caissons in a time period of 4 years.

Total costs construction facilities: Nlg. $37.0{\cdot}10^6$ + Nlg. $11.8{\cdot}10^6$ = Nlg. $48.8{\cdot}10^6$.

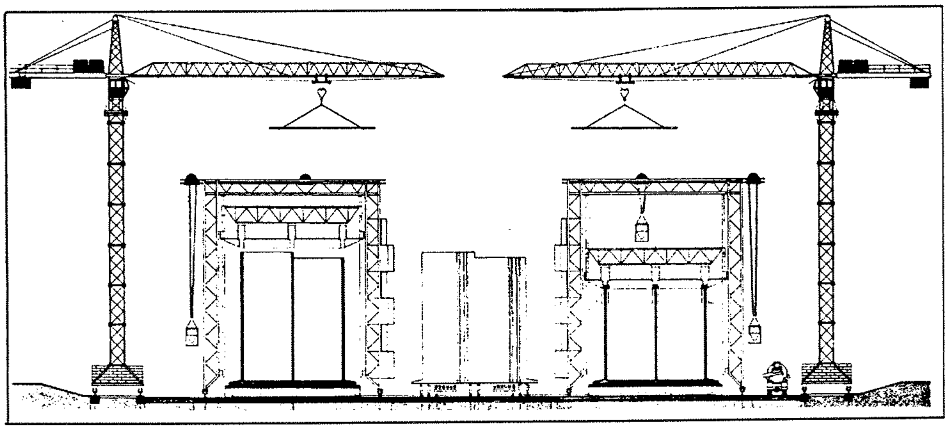


Figure 12.3 Construction of caissons on a construction yard.

Near Shore Construction Vessel, NSCV
The lifting vessel which was designed has a maximum lifting capacity of 12.000 Tonnes. Based on required caisson height and width of the caisson for the Maasvlakte 2 breakwater, the maximum caisson length is 50.00 m.
According to Heeremac the placement capacity of this vessel is 2 caissons/day. As the operational costs of this vessel are very high, work will continue 24 hours a day, 7 days a week.

The required time to place 80 caissons of 50.00m length for the 4.0 km breakwater section is 40 days. 2 weeks extra are reserved for possible delay[2] (e.g. high waves or damage). The total operational time of the Near Shore Construction Vessel is 54 days, approximately 8 weeks.

Costs of this vessel:
New costs of a vessel which can operate in shallow water and has sufficient lifting capacity are approximately Nlg. 75.10^6. However if use is made of modified Heeremac equipment, costs for such a vessel are Nlg. 40.10^6, and operational costs: Nlg. 750.000,-/week, which includes the use of tugs and pontoons. It is reasonable to assume that 50% of these costs are written off on the Maasvlakte 2 works.

Total costs transport and placement:
(50% x 40.10^6 + 8 x 750.000,-) =Nlg. 26.10^6. The transport and placement costs per m breakwater are: 26.10^6 /4.000=Nlg 6.500,-/m. According to the study of Spanjers and Heeremac, transport and placement costs for the second construction phase will be twice the costs of the first construction phase, Nlg. 13.000,-/m.

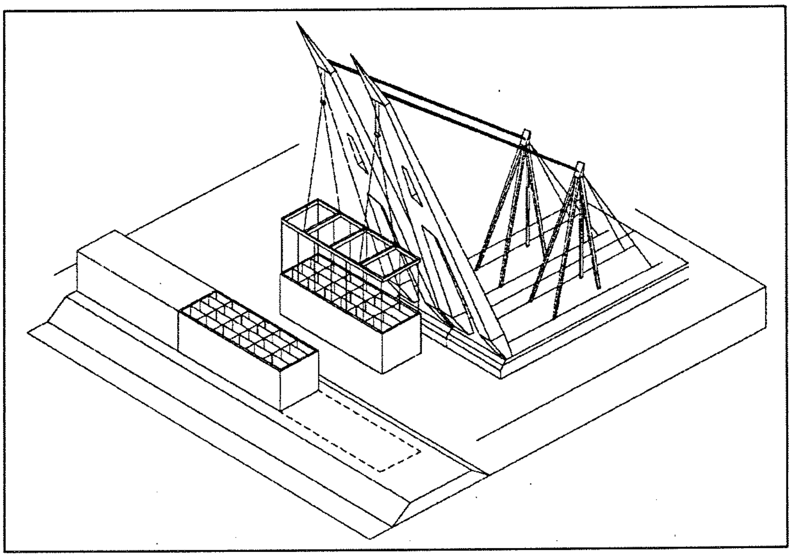


Figure 12.4 Near Shore Construction Vessel.

12.3.2 Positive and negative aspects Japanese method

Positive aspects:
a) caissons can be built a construction terrain above the water level of the connecting river or waterway, and above the ground water level. No deep dock and extensive drainage system or deep access channel is required;
b) transport and placement is possible under higher wave conditions;

[2] Unworkable days calculated under the assumption that transport and placement works can continue at slightly higher wave conditions when executed by the NSCV(H_s<1.25m for 65% of time) than with only tugs(H_s≤1.0m for 53% of the time), see section 10.3 Figure 5.
Total placement time: 40 days x 1.35 = 54 days.

Negative aspects:
a) large construction yard is required;
b) expensive horizontal transport system on yard (winches);
c) extra prestressing steel is required in the base slab to absorb the large forces which are exerted on the caisson during placement of the caissons from the yard on the pontoon (see Figure 12.2);
d) provisions must be installed to lift the caisson with the NSCV;
e) expensive Near Shore Construction Vessel is required;
f) relative high costs reuse due to not making use of caisson buoyancy;
g) special attention must be paid to water inlet slits;
h) caisson length limited by lifting capacity of NSCV;
i) caissons must be stored on pontoons or on yard;

12.3.3 Overview breakwater costs Japanese construction method:

An overview of the costs of the Maasvlakte 2 caisson breakwater is presented in Table 12.1. It must be noted that the accuracy of the values in this table serves to find back the essence of the value in the rest of the text. The costs are not as accurate as might be suggested by the values presented in this table.

	Japanese Construction Method		
activity:	**costs in Nlg /m**	**costs for a 4km long section [in Nlg·10⁶]**	**percentage [%]**
material costs caisson	53.520,-	213	36.2
labour costs (working only daytime) (section 7.7)	24.700	98.7	16.8
construction yard costs	12.200	48.8	8.3
construction costs foundation	50.600,-	202.4	34.4
transport and placement costs of caissons (phase 1)	6.500,-	26.0	4.3
total costs construction phase 1:	**147.500,-**	**589**	**100**
costs construction phase 2: transport and placement caissons new foundation	 13.000,- 57.100,-	 52.0 228.4	 22.8 77.2
total costs construction phase 2:	**70.100,-**	**280.4**	**100**

Table 12.2 Overview breakwater costs Japanese-construction method.

12.4 Dock-construction method

12.4.1 Procedure

This is the most common way in which large concrete elements with buoyant capacity as tunnel elements and bridge piers have been fabricated in The Netherlands. To construct the dock a large basin surrounded by a dike is excavated with a bottom level which is below the water level of the connecting river or waterway. After the basin has been drained, several elements can be fabricated simultaneously on a gravel bed placed on the bottom of the dock. The construction of the caissons can be done in similar manner as on yard located above the groundwater level. A dry working space is guaranteed by a drainage system. Once all the elements are completed the dock (compartment) is flooded, the surrounding dike is partially excavated and the elements are launched from the dock and transported to their destination by tugs. Sometimes a sluice is built in the surrounding dike so that partial excavation is not necessary. There are many types of docks which mainly differ in the height of the bottom level and the construction of the vertical wall. The main principle of a construction dock is illustrated in Figure 12.5.

The following types of docks can be distinguished:

a) Cement or sheetpile vertical walls which reach to a horizontal watertight layer can be used as an alternative instead of a dike;
b) Cement or sheetpile vertical walls with additional drainage;
c) Sheetpile walls with underwater concrete floor with foundation (dry dock);
d) Building dock with foil construction;
e) Half deep dock with deep trench and foil construction;
f) High dock with deep trench and foil construction;
g) Slipway;

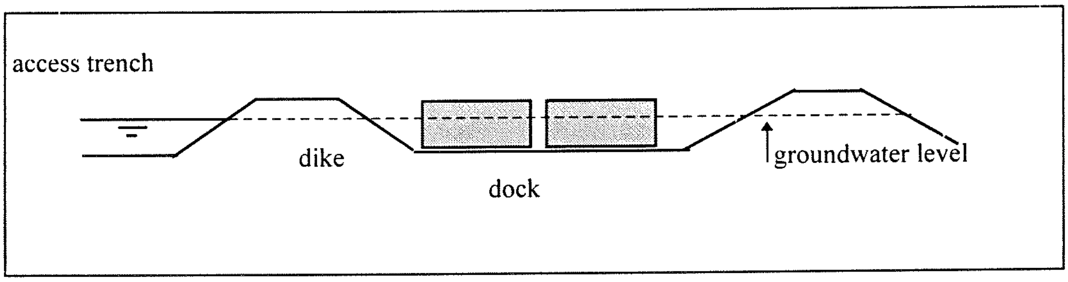


Figure 12.5 Cross section construction dock.

Costs of construction dock:
The project group Maasvlakte 2 has studied the construction costs of a building dock which is suited to fabricate caissons for a 4.0 km breakwater section in 4 years. This is based on the use of 2 sets of formwork. An overview of the construction costs of a building dock is presented in Table 12.3. The values in this table are taken from the document [BD/GWR 1997, notitie 3c]. These costs include the excavation of ground, construction of a surrounding dike, preparation costs of the working space, and drainage costs.

The construction dock which will be maintained in this study is a construction dock with a bottom level of NAP-15 m, and for a 4 km caisson breakwater section. K_T refers to the acceptable wave transmission over the caissons. K_T=10% will be maintained here, costs Nlg. 37.10^6, or Nlg. 9.250,-/m.

	construction costs of 1 km caisson breakwater section [in Nlg. x10^6]		construction costs of 4 km caisson breakwater section [in Nlg. x10^6]	
bottom level	K_T=10%	K_T=50%	K_T=10%	K_T=50%
NAP-10m	17.0	15.0	34.0	30.0
NAP-15m	18.5	16.5	**37.0**	33.0
NAP-20m	20.0	18.0	40.0	36.0

Table 12.3 Overview costs of construction dock.

Costs of formwork:
The construction process of caissons in a dock is similar to that in a yard, and the costs of the formwork and other construction equipment will be set as the Japanese construction method; Nlg. $11.8 \cdot 10^6$.

Total costs construction facilities:
Dock + formwork = Nlg. $37.0 \cdot 10^6$ + Nlg. $11.8 \cdot 10^6 = 48.8 \cdot 10^6$.

Costs transport and placement:
As this construction method also makes use of the buoyant capacity of the caisson, these costs are assumed to be similar to the floating construction yard method, Nlg. 5.200,- (see section 10.3 Table 10.1).

12.4.2 Positive and negative aspects dock method

Positive aspects:
a) much experience with this construction method in The Netherlands;
b) low transport and placement costs of the caissons;
c) no specialised equipment is required;

Negative aspects:
a) Large amount of ground must be excavated and dumped, a surrounding dike must be constructed;
b) all the caissons must be completed in dock (section) in order to flood dock;
c) During construction of the elements a dry working space must be guaranteed, and a drainage system must be installed. Drainage of the dock will have a negative effect on the ground water level of the surrounding terrain;
d) Delay of one element will cause delay of all the elements;
e) A sufficiently deep access trench is required from the dock to the breakwater site;
f) Dependant on the proximity of the dock to the breakwater site, the elements must be transported over a large distance with all risks involved;

12.4.3 Overview breakwater costs Dock-construction method:

An overview of the costs of the Maasvlakte 2 caisson breakwater is presented in Table 12.4.

activity:	Dock-Construction Method		
	costs in Nlg /m	**costs for a 4km long section [in Nlg.10^6]**	**percentage [%]**
material costs caisson	46.455	185.9	33.4
labour costs (working only daytime) (section 7.7)	24.700	98.7	17.7
construction dock costs	12.200,-	48.8	8.7
construction costs foundation	50.600,-	202.4	36.4
transport and placement costs of caissons	5.200,-	20.8	3.8
Total costs construction phase 1:	**139.200,-**	**557**	**100**
costs construction phase 2: reuse of caissons new foundation	 3.500,- 57.100,-	 14.0 228.4	 5.8 94.2
Total costs construction phase 2:	**60.600,-**	**242.4**	**100**

Table 12.4 Overview breakwater costs dock-construction method.

12.5 Other alternatives

Besides the construction methods discussed in this section there are many other innovative construction methods for caissons. It is beyond the scope of this thesis to examine these closely. Examples:

a) slipway (for ship repair);
b) On site construction;
c) Steel structure;

12.6 Conclusions

The caissons of the different construction methods must be designed to absorb the same design forces during the operational phase, the outward force of the ground pressure on the outer walls. During the transport phase caissons which don't make use of their buoyant capacity must possess over extra strength to absorb bending moments during shifting over the construction yard and placement on the pontoon. The required extra strength can be achieved by placement of extra prestressing steel in the base slab. This increases the material costs of the caisson by approximately 14%.

The construction costs of a dock and a yard are approximately twice as high as those of a FCY, and also these construction methods required a large terrain. Transport and placement costs of the methods are competitive with one another for the first construction phase, but the Japanese method is significantly more expensive for the second construction phase.

Because works continue full time for the FCY-construction method, labour costs are higher than for construction methods when works are only carried out in the day time. When works are structurally carried out full-time, labour costs are approximately 33% more. On the other hand the yard costs of the FCY-construction method are lower than the other construction methods because it is used much more efficiently.

Concluding it can be said that the three construction methods are all technically feasible and price competitive with one another. The traditional dock-construction method is the cheapest alternative, but further study should be done with more specific price information to determine the costs more precisely.

Cost comparison caisson construction methods:

Costs caisson breakwater Construction phase 1						
	Floating Construction Yard Method		**Japanese Construction Method**		**Construction in Dock Method**	
activity	costs [Nlg·10^6]	%	costs [Nlg·10^6]	%	costs [Nlg·10^6]	%
materials caisson	185.9	32.8	213	36.2	185.9	33.4
labour costs	138.5	24.4	98.7	16.8	98.7	17.7
construction facility	20.0	3.5	48.8	8.3	48.8	8.7
material and construction foundation	202.4	35.6	202.4	34.4	202.4	36.4
transport and placement caissons	20.8	3.7	26	4.3	20.8	3.8
Total:	**568**	100	**589**	100	**557**	100

Table 12.5 Overview cost comparison construction methods, construction phase 2.

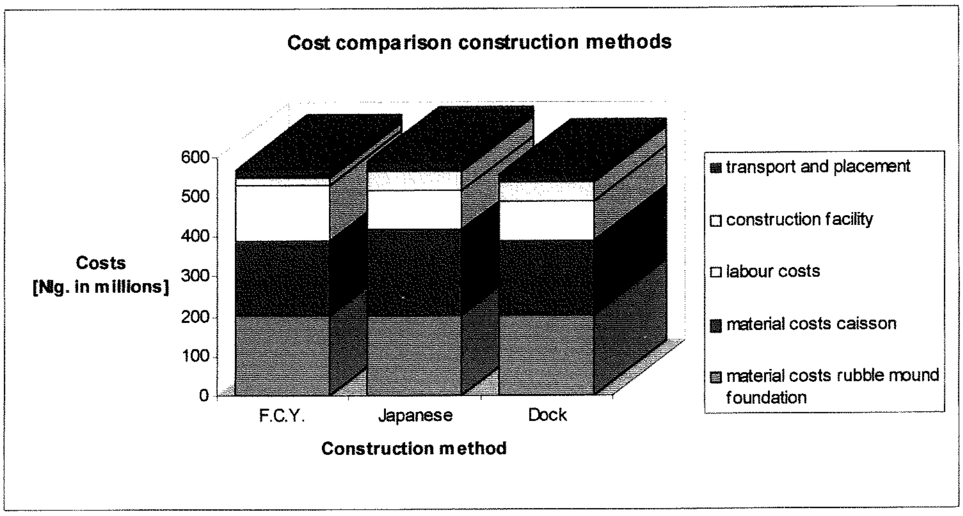


Figure 12.6 Overview construction costs caisson breakwater construction phase 1.

Costs caisson breakwater Construction phase 2						
	Floating Construction Yard Method		**Japanese Construction Method**		**Construction in Dock Method**	
activity	costs [Nlg·10^6]	%	costs [Nlg·10^6]	%	costs [Nlg·10^6]	%
caisson reuse: foundation:	14.0 228.4	5.8 94.2	52.0 228.4	22.8 77.2	14.0 228.4	5.8 94.2
Total:	**242.4**	100	**280.4**	100	**242.4**	100

Table 12.6 Overview cost comparison construction methods.

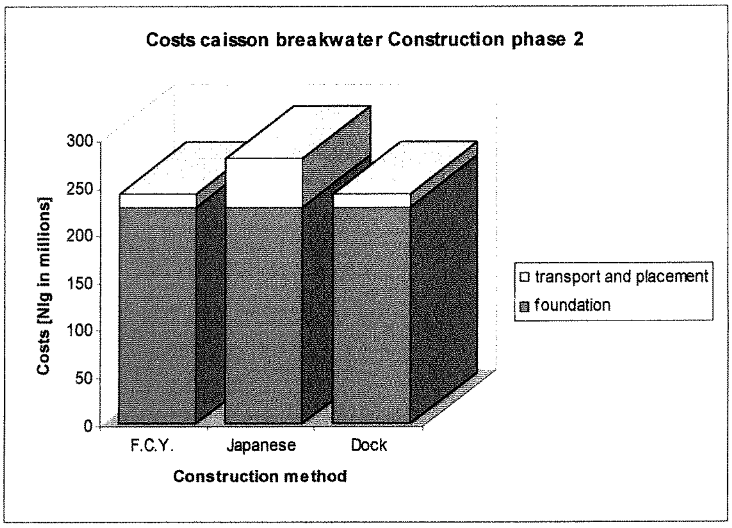


Figure 12.7 Overview construction costs caisson breakwater, reuse phase.

13 Phased construction of Maasvlakte 2, economical perspective

13.1 Introduction

Goal of this section is to indicate the economical investment which can be saved by construction of the Maasvlakte 2 in 2 phases. Also the effect of the reusable caissons will be calculated.

13.2 Advantages and disadvantages of phased construction

2 phases	1 phase
+ A phased execution of MV2 requires a lower investment in the year 2010. This money can be spent on other projects and may lead to a lower threshold of the decision makers of MV2 to start construction;	- Invest all money in one moment, great economical loss of terrain which is unused;
+ Flexibility, contour is adaptable to changed economical, political, technical changes;	- Not flexible;
+ Learning aspect from earlier phase;	- No learning aspect;
- To restart the construction process for the second phase, equipment must be mobilised for a second time, which will bring extra expenses, also there will be hinder to the environment for a second time;	+ 1x mobilisation and hinder;
+ works can be spread more regularly;	- concentration of construction works, great capacity is required;

Table 13.1 Overview advantages and disadvantages of phased execution.

Assumptions made for the cost calculations:

- To calculate the total construction costs of MV2, the breakwater costs calculated in the previous sections for the FCY construction method, are raised by 30%. This consists of the following aspects: 17.5% Tax, 5% profit and risk, 5% costs construction site and unpredicted events, 2.5% overhead costs;
- The time between construction phase 1 and 2 is 20 years;
- The rent including inflation is estimated to be 5% per year;
- the average bottom level of the MV2 terrain is NAP -15m;
- the contours of the MV2 breakwater are at NAP - 18m;
- caissons are the cheapest solution for breakwaters at NAP-18m[1];

The surface area of the MV2 phase 1 and 2, the length and costs/m of the Northern Dam and Southern Beach are based on the values discussed in Appendix N. The costs of the caisson breakwater are based on the calculations determined in this study.

[1] Price calculations of the project group Maasvlakte 2 indicate the a rubble mound breakwater (with similar transmission as the caissons) at NAP- 18m are approximately Nlg. 185.000,-/m.

Four alternatives are considered:

A) Total construction of MV2 (2000 ha.) in one phase in the year 2010;

B) Construction of MV2 in 2 phases, 1000 ha in 2010 and 1000 ha. in 2030, with a **complete new breakwater** constructed for the second construction phase;

C) Construction of MV2 in 2 phases, 1000 ha in 2010 and 1000 ha. in 2030, with a **reusable breakwater** for the second construction phase;

D) Only construction of the first phase of MV2 (e.g. if terrain is not required);

E) Construction of the MV2* in 2 phases, 1000 ha in 2010 and 2000 ha in 2030, with a **reusable breakwater** for the second construction phase;

13.3 Cost calculations of the alternatives

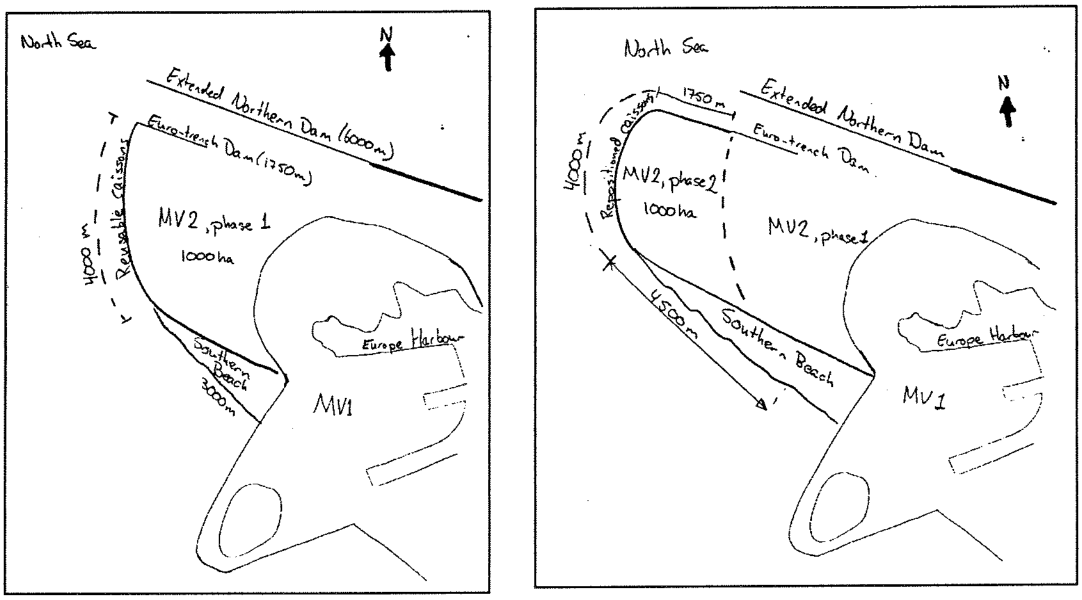


Figure 13.1 Construction phase 1 and 2 of Maasvlakte 2.

Alternative A): Total construction of MV2 (2000 ha.) in one phase in the year 2010;

Construction costs total MV2 in one phase = **Nlg. 4200·10⁶:**

element	section	costs [Nlg]	costs section [Nlg. x10⁶]	costs section [Nlg. x10⁶] (30% extra)
Northern Dam	6000 m	141.900,- /m	851.4	1106.8
Euro-trench Dam	3500 m	141.900,- /m	496.7	645.7
Southern Beach	6000 m	32.500,- /m	195	253.5
reusable caissons	4000 m	141.900,- /m	567.6	737.9
terrain[2]	14 km^2	80.10^6 /km^2	1120	1456
Total:	-	-	**3231**	**4200**

Table 13.2 Costs construction MV2 in one phase in 2010.

Construction costs for the complete MV2 in 2010: Nlg. $4200 \cdot 10^6$.

[2] Assume 70% of required 10 km^2 consists of terrain, 30% water surface;

Alternative B): Construction of MV2 in 2 phases, 1000 ha in 2010 and 1000 ha. in 2030, with a **complete new breakwater** constructed for the second construction phase;

Construction costs phase 1 MV2 = **Nlg. 3023·10⁶:**

element	**section**	**construction costs [Nlg]**	**construction costs section [Nlg. x10⁶]**	**costs section [Nlg. x10⁶]** (30% extra)
Northern Dam	6000 m	141.900,-/m	851.4	1106.8
Euro-trench Dam	1750 m	141.900,-/m	248.3	322.8
Southern Beach	3000 m	32.500,- /m	97.5	126.8
caissons (not reused)	4000 m	141.900,-/m	567.6	737.9
terrain[1]	7 km^2	80.10^6 /km^2	560	728
Total:	-	-	**2325**	**3023**

Table 13.3 Costs construction phase 1 of MV2 in 2010.

Construction costs phase 2 MV2 = **Nlg. 1979·10⁶.**

Element	**section**	**costs [Nlg]**	**costs section [Nlg. x10⁶]**	**costs section [Nlg. x10⁶]** (30% extra)
Euro-trench Dam	1750 m	141.900,- /m	248.3	322.8
Southern Beach	4500 m	32.500,- /m	146.3	190.1
caissons (not reused)	4000 m	141.900,- /m	567.6	737.9
terrain[1]	7 km^2	80.10^6 /km^2	560	728
Total:	-	-	**1522**	**1979**

Table 13.4 Costs construction phase 2 of MV2 in 2030.

For the situation that the caisson breakwater is not reused, but a new one is installed the total costs of construction phase 2 MV2 would then be: Nlg. $1979 \cdot 10^6$. The present worth value of these costs in 2010 is ($P_n = 1979 \cdot 10^6$):

$P_0 = 1979 \cdot 10^6 / (1+0.05)^{20}$ = **Nlg. $746 \cdot 10^6$.**

Construction costs for the complete MV2 in 2010: Nlg. $3023 \cdot 10^6$ + Nlg. $746 \cdot 10^6$ = Nlg. $3769 \cdot 10^6$.
Saved expenses compared with Alternative A) = Nlg. $431 \cdot 10^6$.

Alternative C): Construction of MV2 in 2 phases, 1000 ha in 2010 and 1000 ha. in 2030, with a **reusable breakwater** for the second construction phase;

Construction costs phase 1 MV2 = **Nlg. 3023·10⁶.** (identical to Alternative B)
Construction costs phase 2 MV2 = **Nlg. 1556·10⁶.**

Element	**section**	**costs [Nlg]**	**costs section [Nlg. x10⁶]**	**costs section [Nlg. x10⁶]** (30% extra)
Euro-trench Dam	1750 m	141.900,- /m	248.3	322.8
Southern Beach	4500 m	32.500,- /m	146.3	190.1
reusable caissons	4000 m	60.600,- /m	242.4	315.1
terrain[1]	7 km^2	80.10^6 /km^2	560	728
Total:	-	-	**1197**	**1556**

Table 13.5 Costs construction phase 2 of MV2 in 2030.

The present worth value[3] of the future investment for MV2 construction phase 2 ($P_n = 1556 \cdot 10^6$) is:
$P_0 = 1556 \cdot 10^6 / (1+0.05)^{20}$ = **Nlg. $586 \cdot 10^6$**.

Construction costs for the complete MV2 in 2010: Nlg. $3023 \cdot 10^6$ + Nlg. $586 \cdot 10^6$= Nlg. $3609 \cdot 10^6$.
Saved expenses compared with Alternative A) = Nlg. $591 \cdot 10^6$.

Alternative D): Only construction of the first phase of MV2 (e.g. if terrain is not required);
In this situation the second phase of MV2 will not be executed due to changed views.
Construction costs for the complete MV2 in 2010: Nlg. $3023 \cdot 10^6$.
Saved expenses compared with Alternative A) = Nlg. $1177 \cdot 10^6$.

Alternative E): In this situation the MV2* will be built with a total surface of 3000 ha. (1000 ha extra due to changed views). Assume construction costs of 2000 ha (Nlg $3112 \cdot 10^6$) in 2030 are twice as high as for 1000 ha, (Nlg $1556 \cdot 10^6$).

The present worth value of the future investment for MV2* construction phase 2 ($P_n = 3112 \cdot 10^6$) is:
$P_0 = 3112 \cdot 10^6 / (1+0.05)^{20}$ = **Nlg. $1173 \cdot 10^6$**.

Construction costs for the complete MV2 in 2010: Nlg. $3023 \cdot 10^6$ + Nlg. $1173 \cdot 10^6$ = Nlg. $4196 \cdot 10^6$.
Saved expenses compared with Alternative A) = Nlg. $4 \cdot 10^6$.

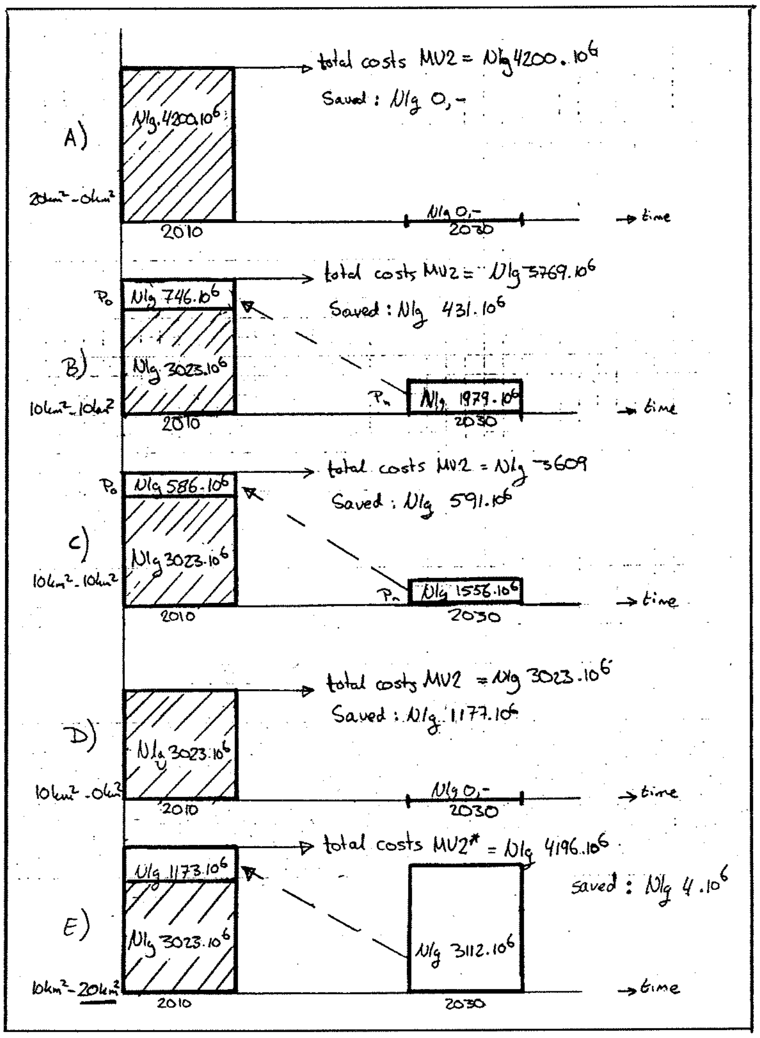


Figure 13.2 Required investments and present worth value for different plannings.

[3] $P_0 = P_n / (1+r)^{\Delta t}$.

13.4 Conclusions

In order to limit the investment made for the construction of MV2, the project can be executed in 2 phases. If the total MV2 were to be constructed in one phase in 2010, the investment would be Nlg. 4200 million in 2010.

By construction of only a section of this terrain (approximately 50%) the investment in 2010 would be 3023 million, approximately Nlg. 1200 million less. This money can be spent on other (money generating) projects (Figure 13.2) which may lead to a lower threshold for the decision makers of phase 1.

If necessary, in the year 2030 an additional section of terrain can be constructed. When the present worth value of the future investment for this second construction phase is added to the investment to be made in the year 2010 for phase 1 of MV2, total costs will be significantly lower then if the complete MV2 is constructed in the year 2010. If a new breakwater is constructed, the saved investment is Nlg. $431 \cdot 10^6$ and if a reusable caisson breakwater is used, the saved investment is Nlg. $591 \cdot 10^6$.

If it is decided that the second construction phase is not necessary, MV2 phase 2 will not be constructed and the saved investment is Nlg. 1200 million. Even if the second construction phase of MV2* is twice as large (total area MV* = 3000 ha.), the total costs are still lower than complete construction of MV2 in 2010. It can be concluded that it is economical to construct the MV2 phased.

The investment which can be saved by a breakwater construction of reusable caissons as opposed to construction of a new breakwater is Nlg. 160 million (based on a 4 km breakwater section).

Another important advantage of a phased execution concerns the flexibility. If there is no need for enlargement of MV2 in 2030, this will not be done. If a terrain which is twice as large as predicted is required, this is also still possible. By maintaining a phased construction method for the MV2, the chance that money is invested in a wrong project is limited.

14. Conclusions and recommendations

14.1 Introduction

In this study a design has been made for the Maasvlakte 2 breakwater. Critical aspects of the design, and possible consequences of these aspects are reviewed in this section. Also possible measures which can be taken to reduce the chance that these critical aspects will occur are mentioned.

The critical elements for the following phases will be analysed:

- construction of caissons;
- construction of foundation;
- transport of caissons to breakwater site;
- placement of caissons on the foundation;
- operational phase;
- reuse phase;

Also the conclusions which can be drawn from this study are discussed and recommendations are made concerning aspects which demand further study.

14.2 Elements of risk

No.	Critical point	Consequences	Measures
Construction of caissons			
1.	late delivery of materials: • steel reinforcement • concrete	construction activities can not continue;	create sufficient **stock pile**;
2.	breakdown of equipment: • formwork • concrete factory • crane • pontoon	construction activities can not continue;	maintenance & repair shop spare parts; **Use of 2 Floating construction yards,** (see conclusions and recommendations);
3.	higher market prices: • materials • equipment • labour	total construction costs will be higher;	calculate total construction costs with high price alternative;
4.	weather conditions: • frost	construction activities can not continue;	calculate extra unworkable days;
5.	errors made in construction process: • asymmetric concrete pour • uneven lift of formwork	asymmetric loading forces on FCY; formwork is stuck;	ballast water compartments in FCY; control system even lifting;
6.	• changes of the design	FCY is not suited for caisson fabrication;	make adjustable installations on FCY;
7.	• other unexpected events:	unknown;	calculate extra unworkable days;
Construction of rubble mound foundation			
1.	weather conditions: • rougher waves than predicted • fog	rubble can't be dumped; collision danger dumping barges with other ships;	calculate extra unworkable days; radar equipment;
2.	• extreme waves	foundation material is	survey and repair;

		disturbed;	
3.	• damaged equipment:	construction works can't proceed;	regular maintenance and spare parts;
4.	late deliveries of materials • sand • rubble	construction works can't proceed;	create sufficient stock pile
5.	• other unexpected events:	unknown;	calculate extra unworkable days;
Transport of caissons to breakwater site			
1.	weather conditions: • rougher waves than predicted • fog	caissons can't be transported; collision danger with ships;	calculate extra unworkable days; radar equipment; transport at special time periods;
2.	caisson is lost: • collision with ship • broken tow-cable • gets stuck in shallow water	caisson is obstacle in shipping lane;	special equipment to remove caisson (e.g. inflatable bags which can be inserted into cells); stand-by tug;
3.	leakage: • leak in caisson shaft	instability; cell fills with water, damage to reinforcement steel;	pump water from cell; repair cell; regular inspection of damaged section;
4.	damage: • navigation errors of tugs (e.g. caisson bangs against quay)	caisson is damaged;	install fenders on caisson; use more tugs for more control;
5.	• other unexpected events:	unknown;	calculate extra unworkable days;
Placement of caissons on the foundation			
1.	weather conditions • rougher waves than predicted • fog	caissons can't be placed;	calculate extra unworkable days;
2.	failure of equipment: • positioning equipment malfunctions • winches break • pumps malfunction	problems with accurate positioning; caisson cells can not be filled;	use different positioning system (e.g. DGPS and laser positioning); spare parts;
3.	foundation: • foundation has been washed away • foundation is not sufficiently level	uneven loading forces on caisson;	extra survey procedure; design caisson so it can absorb some unevenness;
4.	• caisson is placed incorrectly	uneven breakwater, extra forces are induced;	tolerate some misplacement; reposition if required;
5.	• other unexpected events:	unknown;	calculate extra unworkable days;
Operational phase			
1.	extreme loading conditions: • design wave ULS is exceeded • ice loads	caisson will slip, **failure of construction !** extra loads on caisson;	Replace caisson; (difficult and expensive!) design to absorb ice forces;

2.	wave forces: • extreme impulsive wave forces • filter is damaged	damage to concrete; instability; can lead to failure of breakwater;	extra concrete thickness; survey and maintenance;
3.	• ship collision:	damage to breakwater;	keep navigation channel away from breakwater; repair of breakwater;
4.	• other unexpected events:	unknown;	reserve caisson;
Reuse phase			
1.	• caissons don't need to be reused	no problems;	no measures;
2.	• more caissons are required then available:	shortage of caissons;	construction of extra caissons;
3.	• repositioning takes longer than predicted (e.g. due to waves):	more downtime of harbour section;	calculate extra unworkable days;
4.	• difficulties with removal of sand from cells:	sand can't be removed from cells ;	extra jets and pumps;
5.	caissons damaged: • too large head level differences • to many extreme loads exerted during operational phase • caisson floats up uncontrolled	walls damaged;	prevent large head level difference by control system; repair if possible; extra thickness of walls;
6.	• other unexpected events:	unknown;	calculate extra unworkable days;

Table 14.1. Overview Elements of risk.

14.3 Conclusions

The floating construction method of caissons is technically well feasible and economically competitive with other construction methods (Japanese method or in a construction dock).

In principle one yard is sufficient to construct the 55 caissons of 76,65m length in 3.5 years. A large risk of the construction method is that the yard works according to the conveyor belt principle. If one aspect of the construction process is seriously disturbed, this will have a (negative) effect on the total construction process of the caisson, and also on the construction process of the complete breakwater. This risk can be reduced significantly by installing 2 construction yards, which each construct caissons which have a shorter length. As the costs of transport and placement of the caissons are relatively low, more placement procedures will not influence total breakwater costs extremely.

Figure 14.2 illustrates that approximately 33% of the breakwater construction costs consist of material costs for the caisson, and 35% of material costs for the foundation. The costs of the construction facility (4%) and of the transport and placement procedure of the caissons (4%) are relatively low. Labour costs form 24% of the construction costs. Therefore it is important to examine how the design can be further optimised, and minimise the quantities of used materials. Market price changes of e.g. concrete or steel will significantly influence the total construction costs.

In this study it was assumed that directly behind the caisson breakwater harbour activities, transhipment of container vessels, are to take place. In order to achieve this, the maximum acceptable wave transmission is H_t = 0.20m, for waves which have a return period of 1 year. This is quite a strict demand, and leads to a high crest level, which implies high investments. A combination of a different layout of the harbour and a less strict demand of the acceptable wave transmission can lead to a lower crest level. Another aspect which influences the wave climate within the harbour basin is wave penetration through the harbour entrance and reflection within the basin. This has not been taken into account in this study.

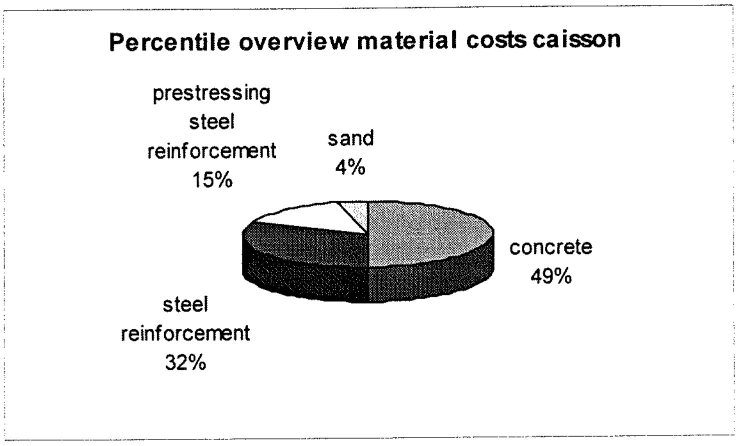


Figure 14.1 Percentile overview material costs caisson.

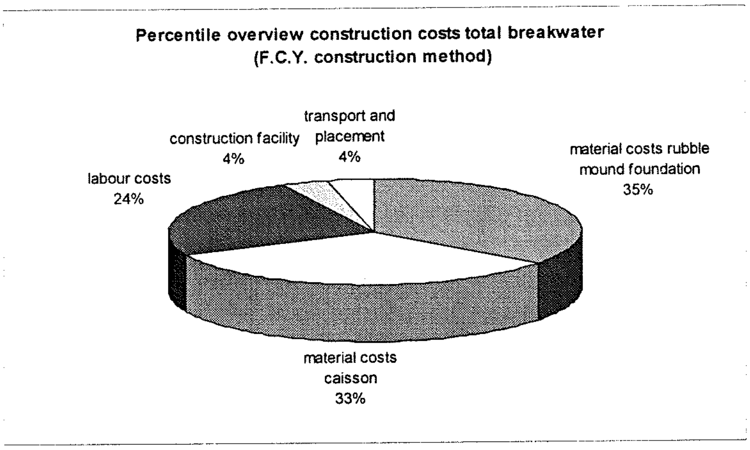


Figure 14.2 Percentile overview construction costs total breakwater.

In order to limit the investment made for the construction of MV2, the project can be executed in 2 phases. If the total MV2 were to be constructed in one phase in the year 2010, the investment would be Nlg. 4200 million.

By construction of only a section of this terrain (approximately 50%) the investment in the year 2010 would be 3023 million, approximately Nlg. 1200 million less than required for the construction of the complete MV2. This money can be spent on other (money generating) projects and may lead to a lower threshold to invest for the decision makers of the MV2.

If necessary, in the year 2030 an additional section of terrain can be constructed. When the present worth value of the future investment of the second construction phase is added to the investment to be made in the year 2010 for phase 1 of MV2, total costs will be significantly lower. Nlg. 431 million if a new breakwater is constructed and Nlg. 591 million if a reusable caisson breakwater is used. Even if the second construction phase of MV2* is twice as large (total area MV* = 3000 ha.), the costs are still lower than complete construction of MV2 in 2010. It can be concluded that it is economical to construct the MV2 phased.

The investment which can be saved by a breakwater construction of reusable caissons opposed to construction of a new breakwater is Nlg. 160 million (based on the reuse of a 4 km breakwater section).

Another important advantage of a phased execution concerns the flexibility. If there is no need for enlargement of MV2 in 2030, this will not be done. If a terrain which is twice as large as predicted is required, this is also possible. The chance that money will be invested in a wrong project is limited.

If the water depth is more than NAP -15 m, caissons are a cheaper solution than rubble mound or sea-dike breakwaters. Also caisson breakwaters have the advantage that they are reusable (flexible).

In this study a reusable breakwater section of 4 km for the MV2 was analysed, but caissons could also serve as a breakwater or as sea defence works for other large projects which are located in deep water (more than NAP-15 m). Within The Netherlands for example they could be used as a combination of a breakwater and sea defence works for an island in sea to locate an airport (replace Schiphol), or as combination of a breakwater and quay for the harbour expansions of Vlissingen.

14.4 Recommendations for further study

The following aspects should be further analysed:

- Scale model tests should be performed to analyse the actual wave penetration in the basin;
- An alternative is to construct a lower caisson in combination with a superstructure to limit wave transmission, or other caisson shapes, e.g. sloping walls (see Appendix G: 'Caisson shapes');
- It should be analysed if there is a cheaper way to construct the rubble mound foundation, as this is an expensive part of the construction costs, 35% of the total costs during the first construction phase and 94% of the total costs during the second construction phase;
- An economic analysis of the investment costs of the breakwater (e.g. a less strict criterion for H_t will lead to a lower investment of the breakwater but to more economic loss during the operational phase due to downtime of the harbour (container terminal);
- Analyse the relationship between extra installation costs of several F.C.Y.'s and the reduced risk of stagnation of the construction process;
- Analyse if further optimisation is possible of the concrete and steel dimensions of the caisson;

References

1. A.A. Brudno and A.R. Lancellotti, Senior Professional Associates, Parsons Brinckerhof, Floating Factory, immersed tube fabrication, Tunnels & Tunnelling, January 1995

2. Agema, J.F. Overzicht golfbrekers/ zeeweringen ten behoeve van het project Maasvlakte 2, June 1996

3. CUR/RWS publication 169, Manual on the use of Rock in Hydraulic Engineering, June 1995, CUR, Gouda, The Netherlands

4. D.I.B.K., Droge Infrastructuur Bedrijfszaken Kostprijszaken, Bouwdienst R.W.S., 1998

5. Dragados Y Construcciones, S.A. -Division Peyma. Flota, Company Folder, March 1998

6. EAU 1990, Recommendations of the Committee for Waterfront Structures Harbours and Waterways, Committee for Waterfront Structures, 1992

7. Gemeentewerken Rotterdam, Hydraulische en meteorologische gegevens voor het Rotterdamse Havengebied, Informatiebulletin nr 6, 1988

8. Goda, Y. Random Seas and Design of Maritime Structures, University of Tokyo Press, 1985

9. Grandi Lavori Fincosit, Caisson construction method, March 1998

10. GTB Deel 2, Grafieken en Tabellen voor Beton, Betonvereniging, Gouda, 1992

11. HP 16: Stroomatlas Benedenrivieren en aanlopen: (Hoek van Holland, Scheveningen, IJmuiden, Texel, Den Helder), Chef der Hydrografie, 's-Gravenhage, January 1992

12. Ing. Mantelli & C, Impressa Generale di Costruzioni S.p.A. Mantelli Marine Works, 1988

13. Massie, W.W. Coastal Engineering Volume III Breakwater Design, Technical University Delft, January 1986

14. Ministry of Transport, Public works and Water management, Design Plan Oosterschelde Storm-Surge Barrier, Overall Design and Design Philosophy, 1994

15. Neelissen, R.F.J. e.a., Multi-Purpose Scrader Concept: New technology for Seabed Treatment, Boskalis, 1998

16. NEN 6720, Voorschriften Beton, TGB 1990, Constructieve eisen en rekenmethoden (VBC 1995), Nederlands Normalisatie-instituut, 1995

17. Proceedings of the international workshop on 'Wave barriers in deepwaters', Port and Harbour Research Institute, January 1994, Yokosuka, Japan

18. Project Organisatie Maasvlakte 2, Bouwstenen "Terrein, Zeewering en Golfbreker" voor Maasvlakte 2, Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, Bouwdienst Rijkswaterstaat and Gemeentewerken Rotterdam, Ingenieursbureau Havenwerken, July 1997

19. Project Organisatie Maasvlakte 2, *De nautische beoordeling van de ontwerpvarianten voor MV2*, Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, Bouwdienst Rijkswaterstaat and Gemeentewerken Rotterdam, Ingenieursbureau Havenwerken, July 1997

20. Project Organisatie Maasvlakte 2, *Principeoplossingen Zeeweringsconstructies*, Bouwdienst Rijkswaterstaat and Gemeentewerken Rotterdam, June 1995

21. Project Organisatie Maasvlakte 2, Rapportage Voorstudie Maasvlakte II, fase IA, Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, Bouwdienst Rijkswaterstaat and Gemeentewerken Rotterdam, Ingenieursbureau Havenwerken, July 1997

22. Project Organisatie Maasvlakte 2, Tussenrapportage, Fase 1B, Ministerie van Verkeer en Waterstaat, Directoraat-Generaal Rijkswaterstaat, Bouwdienst Rijkswaterstaat and Gemeentewerken Rotterdam, Ingenieursbureau Havenwerken, January 1997

23. Rijksinstituut voor Kust en Zee (RIKZ), Getiijtafels Voor Nederland, 1997 Sdu Uitgevers, Den Haag

24. Samenwerkingsverband Maasvlakte 2 Varianten, Golfdoordringingsonderzoek Maasvlakte 2, November 1998

25. Samenwerkingsverband Maasvlakte 2 Varianten, Memo: Prognose zeescheepvaart Maasvlakte 2, July 1998

26. Samenwerkingsverband Maasvlakte 2 Varianten, Memo: Trend in de ontwikkeling van maatgevende schepen, July 1998

27. Spanjers, C.M. Thesis: 'Ontwerp van een verplaaatsbare caissongolfbreker voor Maasvlakte 2'; Technical University of Delft, August 1997

28. Takahashi, S. Reference Document No. 34 Design of Vertical Breakwaters, Port and Harbour Research Institute, Ministry of Transport, August 1996

29. Verruijt, A. Grondmechanica, Delftse Uitgevers Maatschappij b.v.; Delft, 1990

30. Vos, C. & de Jager, Uitvoeringstechnologie van betonconstructies, Technical University Delft, 1995

31. Vrijling, J.K. and Glerum, A. Waterbouwkundige Kunstwerken, Technical University Delft, January 1995

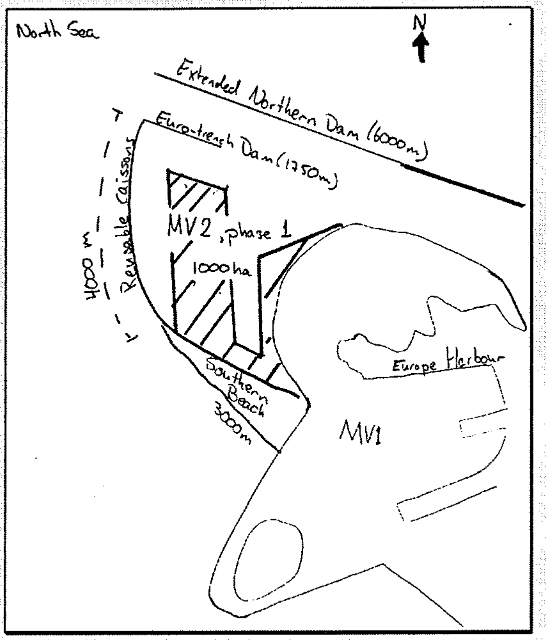
32. V. The history of caisson construction with the Hollandsche Beton Groep nv from 1920 to 1977, hbg 1977, Rijswijk

33. Z. Bouwdienst RWS, handboek Specifieke Aspecten Tunnel Ontwerp, SATO, Utrecht, 1993

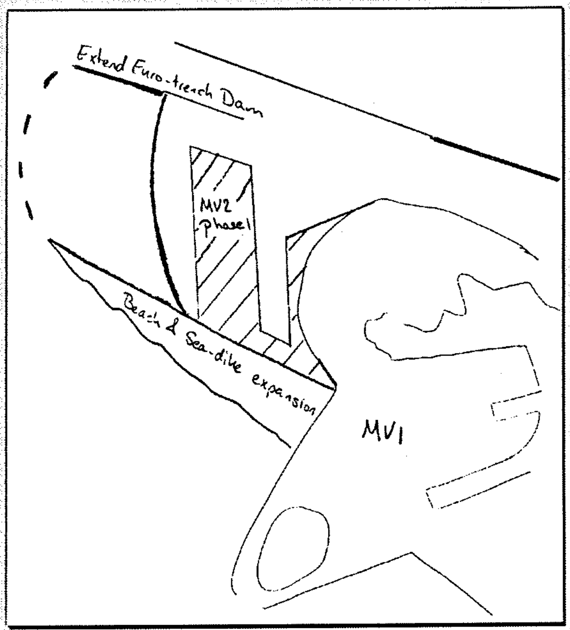
Appendix A: Geographic Boundary Conditions

- **Project area Maasvlakte 2 & location Europe Harbour**
- **Construction phases Maasvlakte 2**
- **Birds-eye view of the Maasvlakte 2**

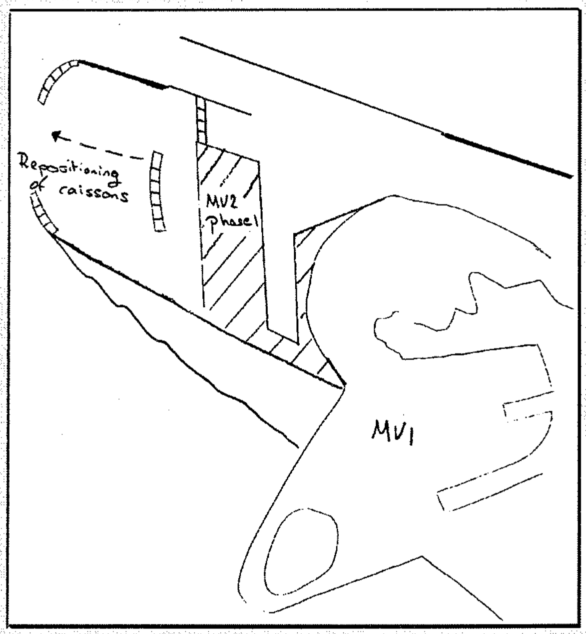
Overview phased execution MV2 construction phases



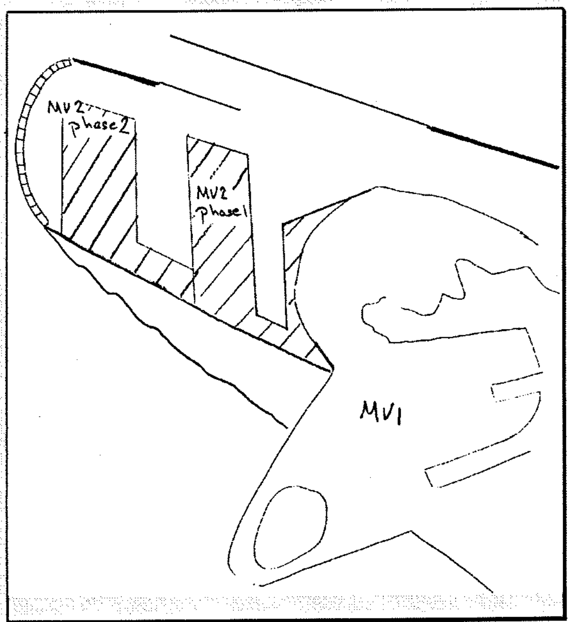
Construction phase 1 MV2 completion year 2010;



Intermediate phase, extension of Euro-trench Dam and Southern beach; year: 2028;



Intermediate phase, repositioning of caissons, duration: approximately 1 month year 2029;



Construction phase 2 MV2. year 2029;

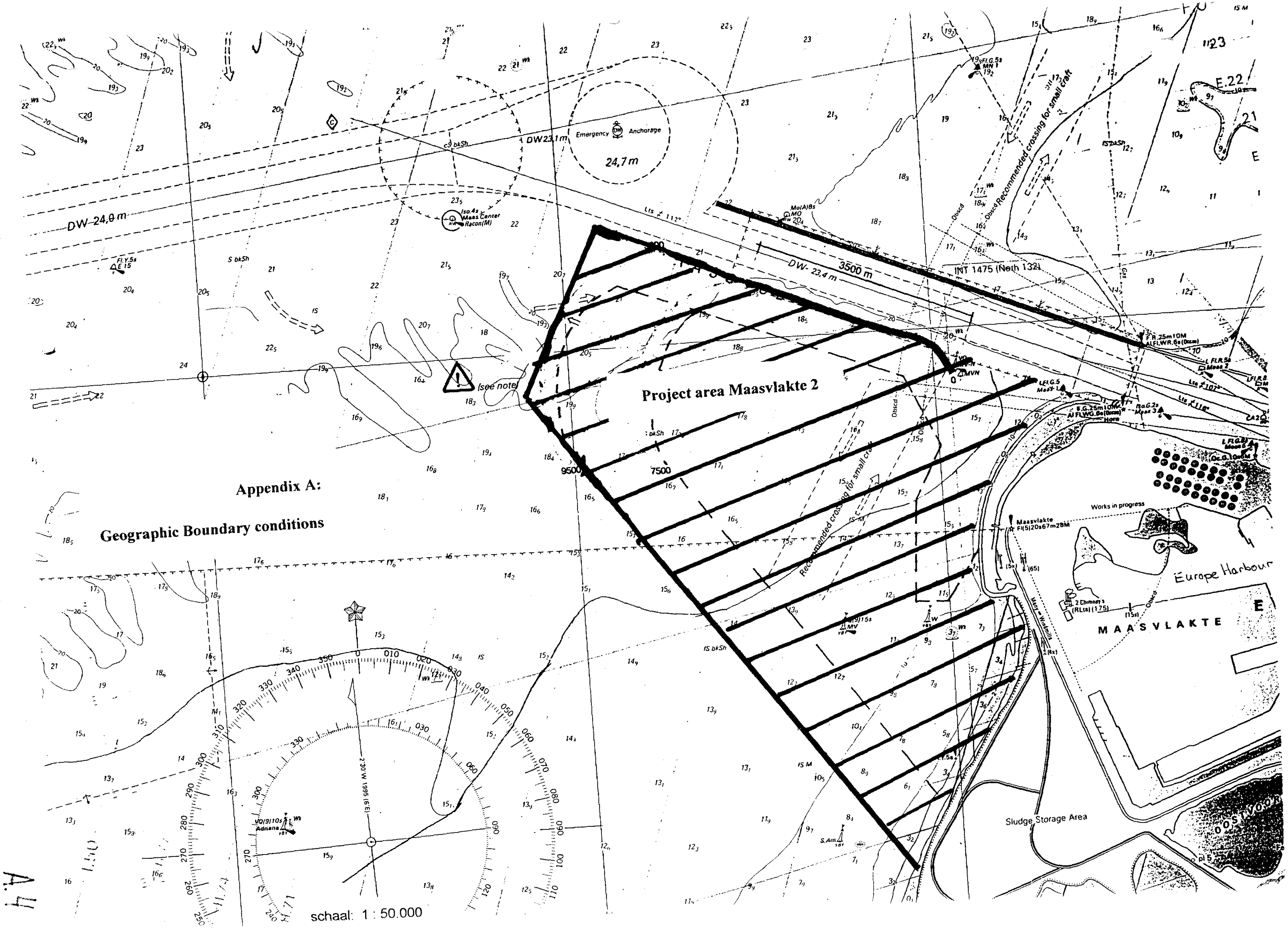
Birds-eye view of the Maasvlakte 2,

- after completion of the second construction phase in 2030;
- total surface area MV2 2000 ha.;

Figure: Birds-eye view of the Maasvlakte 2 in the year 2030.

Appendix A:

Geographic Boundary conditions



Appendix B:

Geologic Boundary conditions

- **Sounding taken at the Europe harbour**
- **Determination of specific weight of soil γ_{sub}**
- **Determination of angle of internal friction φ'**
- **Determination of cohesion value c'**

The values measured in the Europaharbor are also assumed to be representative for the location where the Maasvlakte 2 breakwater is to be constructed.

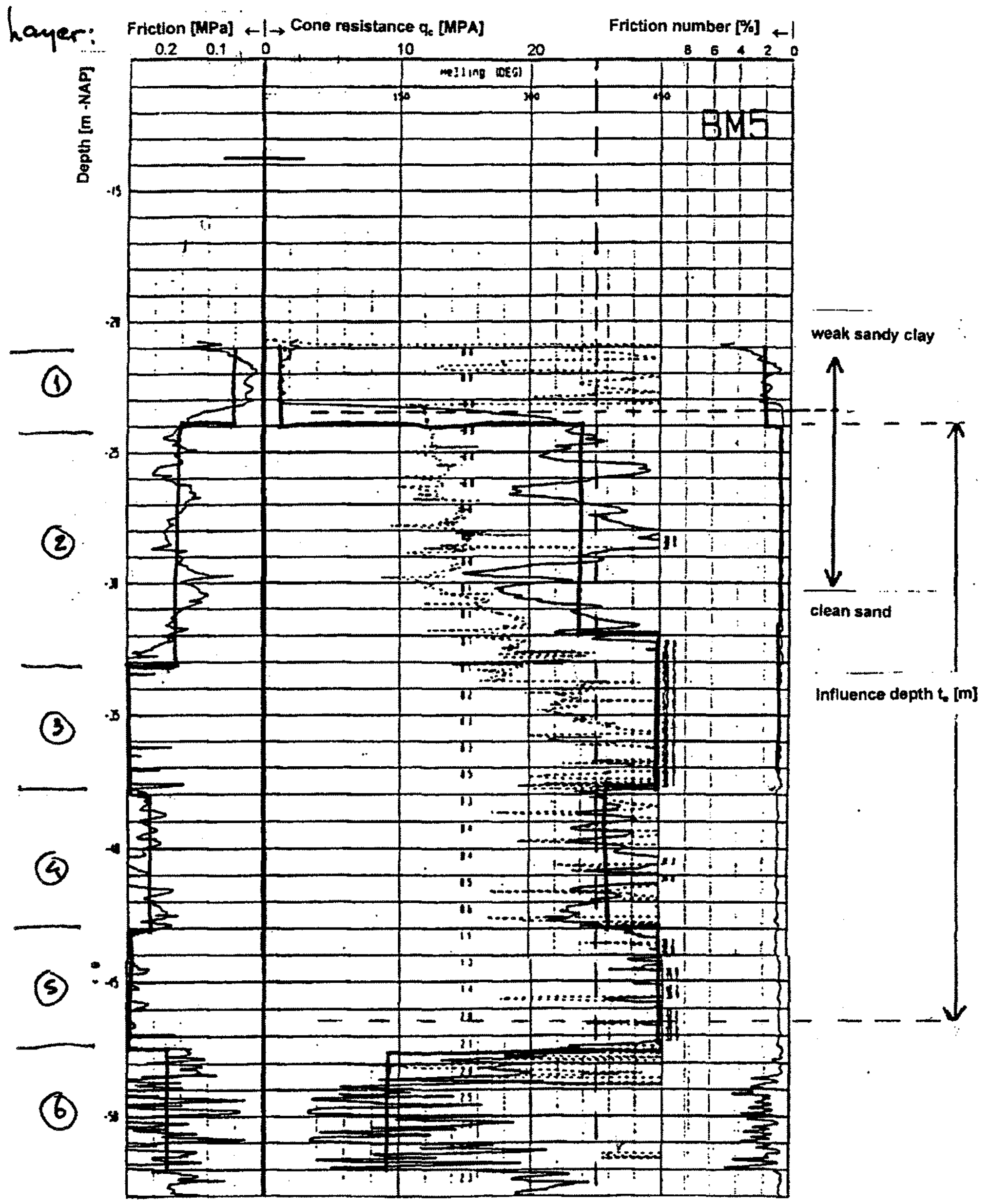


Figure 1. Sounding taken at the Europe Harbour.

The following conclusions can be drawn from figure 1:

	depth	average cone resistance q_c [MPa]	friction number [%]	friction [MPa]
layer 1	from harbour bottom to NAP -24.0m	1.5	2	0.05
layer 2	from NAP-24.m to NAP-32.0m	24	0.8	0.15
layer 3	below NAP-32.0m	30	0.8	0.25

Determining γ_{sub} , φ' and c' of the underground

Figure 2 shows a diagram of the relationship between the friction number, the cone resistance and the angle of internal friction of the soil belonging to these values. From this diagram the type of soil can be determined, which in combination with Table 1 leads to the required soil parameters.

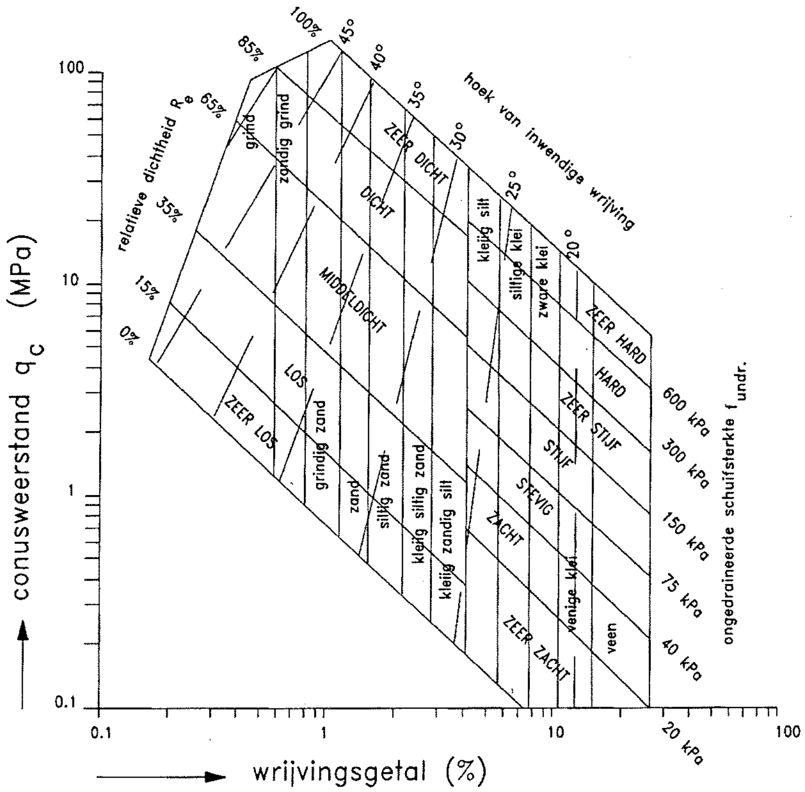


Figure 2. Relationship between friction number, cone resistance and angle of internal friction of the soil, CUR-publication about sheet piling [litt].

grondsoort		representatieve gemiddelde waarde van de grondeigenschappen												
hoofd-naam	bijmengsel	consistentie [1]	γ [2] kN/m³	γ_{sat} kN/m³	q_c [3] [6] MPa	C'_p	C'_s	C_c	C_α [5]	C_{sw}	E [6] MPa	ϕ' °	c' kPa	f_{undr} kPa
grind	zwak siltig	los	17	19	15	500	-	0,008	0	0,003	75	32,5	n.v.t.	n.v.t.
		matig	18	20	25	1000	-	0,004	0	0,002	125	35		
		vast	19 of 20	21 of 22	30	1200 of 1400	-	0,003 of 0,002	0	0,001 of 0	150 of 200	37,5 of 40		
	sterk siltig	los	18	20	10	400	-	0,009	0	0,003	50	30	n.v.t.	n.v.t.
		matig	19	21	15	600	-	0,006	0	0,002	75	32,5		
		vast	20 of 21	22 of 22,5	25	1000 of 1500	-	0,003 of 0,002	0	0,001 of 0	125 of 150	35 of 40		
zand	schoon	los	17	19	5	200	-	0,021	0	0,007	25	30	n.v.t.	n.v.t.
		matig	18	20	15	600	-	0,006	0	0,003	75	32,5		
		vast	19 of 20	21 of 22	25	1000 of 1500	-	0,003 of 0,002	0	0,001 of 0	125 of 150	35 of 40		
	zwak siltig kleiig		18 of 19	20 of 21	12	450 of 650	-	0,008 of 0,005	0	0,003 of 0,001	25 of 35	27 of 32,.5	n.v.t.	n.v.t.
	sterk siltig kleiig		18 of 19	20 of 21	8	200 of 400	-	0,019 of 0,009	0	0,006 of 0,001	20 of 30	25 of 30		
leem [4]	zwak zandig	slap	19	19	1	25	650	0,168	0,004	0,056	2	27,5 of 30	0	50
		matig	20	20	2	45	1300	0,084	0,002	0,028	5	27,5 of 32,5	2	100
		vast	21 of 22	21 of 22	3	70 of 100	1900 of 2500	0,049 of 0,030	0,001	0,017 of 0,005	10 of 20	27,5 of 35	5 of 7,5	200 of 300
	sterk zandig		19 of 20	19 of 20	2	45 of 70	1300 of 2000	0,092 of 0,055	0,002	0,031 of 0,005	5 of 10	27,5 of 35	0 of 2	50 of 100
klei	schoon	slap	14	14	0,5	7	80	1,357	0,013	0,452	1	17,5	0	25
		matig	17	17	1,0	15	160	0,362	0,006	0,121	2	17,5	10	50
		vast	19 of 20	19 of 20	2,0	25 of 30	320 of 500	0,168 of 0,126	0,004	0,056 of 0,042	4 of 10	17,5 of 25	25 of 30	100 of 200
	zwak zandig	slap	15	15	0,7	10	110	0,759	0,009	0,253	1,5	22,5	0	40
		matig	18	18	1,5	20	240	0,237	0,005	0,079	3	22,5	10	80
		vast	20 of 21	20 of 21	2,5	30 of 50	400 of 600	0,126 of 0,069	0,003	0,042 of 0,014	5 of 10	22,5 of 27,5	25 of 30	120 of 170
	sterk zandig	-	18 of 20	18 of 20	1,0	25 of 140	320 of 1680	0,190 of 0,027	0,004	0,063 of 0,025	2 of 5	27,5 of 32,5	0 of 2	0 of 10
	organisch	slap	13	13	0,2	7,5	30	1,690	0,015	0,550	0,5	15	0 of 2	10
		matig	15 of 16	15 of 16	0,5	10 of 15	40 of 60	0,760 of 0,420	0,012	0,250 of 0,140	1,0 of 2,0	15	0 of 2	25 of 30
veen	niet voorbelast	slap	10 of 12	10 of 12	0,1	5 of 7,5	20 of 30	7,590 of 1,810	0,023	2,530 of 0,600	0,2 of 0,5	15	2 of 5	10 of 20
	matig voorbelast	matig	12 of 13	12 of 13	0,2	7,5 of 10	30 of 40	1,810 of 0,900	0,016	0,600 of 0,300	0,5 of 1,0	15	5 of 10	20 of 30
variatiecoëfficiënt			0,05		-			0,25				0,10		0,20

De tabel geeft de lage representatieve waarde van gemiddelden van de betreffende grondsoort. Binnen een gebied, gedefinieerd door de rij van het bijmengsel en de kolom van de parameter (een "vakje"), geldt:

- voor γ, γ_{sat}, C_p, C_s, E, ϕ', c' en f_{undr}: als een verhoging van de waarde leidt tot een ongunstige situatie (grotere afmetingen fundering), dan moet de rechterwaarde op dezelfde regel zijn gebruikt, of, indien er geen rechterwaarde is vermeld, de waarde op de regel eronder;
- voor C_c, C_α en C_{sw}: als een verlaging van de waarde leidt tot een ongunstige situatie, dan moet de rechterwaarde op dezelfde regel zijn gebruikt, of, indien er geen rechterwaarde is vermeld, de waarde op de regel daaronder.

1) los : $0 < R_n < 0{,}33$
   matig : $0{,}33 \le R_n \le 0{,}67$
   vast : $0{,}67 < R_n < 1{,}00$
2) bij natuurlijk vochtgehalte
3) hier gegeven q_c-waarden (conusweerstand) dienen als ingang in de tabel, en mogen niet worden gebruikt in berekeningen
4) gerekend wordt verzadigde leem
5) C_α-waarden zijn geldig voor een spanningsverhogingstraject van ten hoogste 100 %.
6) q_c en E zijn genormeerd op een effectieve verticale spanning van 100 kPa. Om een juiste ingang in de tabel via q_c te verkrijgen zijn de gemeten waarden van q_c te converteren naar een effectief verticaal spanningsniveau van 100 kPa. De conversiefactor C_N moet worden bepaald met de grafiek in figuur 2A.

Table 1 Representative values for soil characteristics (TGB 1990: NEN 6744).

According to the document TGB 1990: NEN 7644 (Table 1) a value φ' = 35° to 40° must be maintained for clean tightly packed sand with a cone penetration resistance of 25 MPa. For the calculations a value φ' = 37.5° will be maintained here.

Overview geological boundary conditions

The top ground layer consists of weak sandy clay and the second and third layer consist of clean sand. In this study the angle of internal friction of the second soil layer will be maintained, φ' = 37.5°. The top layer will be excavated and replaced with sand with bearing capacity similar to that of layer 2.

	depth of layer	material	average cone resistance [MPa]	average friction number [%]	γ_{sat} [kN/m^3]	φ' [°]	c' [kPa]
layer 1	from harbour bottom to NAP -24.0 m	weak sandy clay	1.5	2	18	22.5	10
layer 2	from NAP-24.0 m to NAP-32.0m	sand	24	0.8	21	37.5	0
layer 3	below NAP-32.0m	sand	30	0.8	21	40.0	0

Table 2. Overview geological boundary conditions.

Appendix C:

Hydraulic Boundary conditions

- **Water levels**
- **Wave heights**
- **Wave periods**
- **Wave length**
- **Currents**

Water levels

To predict the water levels at Hook of Holland, the following sources were examined:
1. Tide levels of The Netherlands [lit.11]
2. Informationbulletin 6 of Project group Maasvlakte 2 [lit.19]

Tide levels of the Netherlands
The document Tide levels of The Netherlands shows the water levels at Hook of Holland resulting from the astronomical tide and river discharges.
As the influence of the river discharges on the water level is relatively small (order of several centimeters) compared to the influence of the astronomical tide (order of several meters), these will not be taken into account in the tide level calculations.

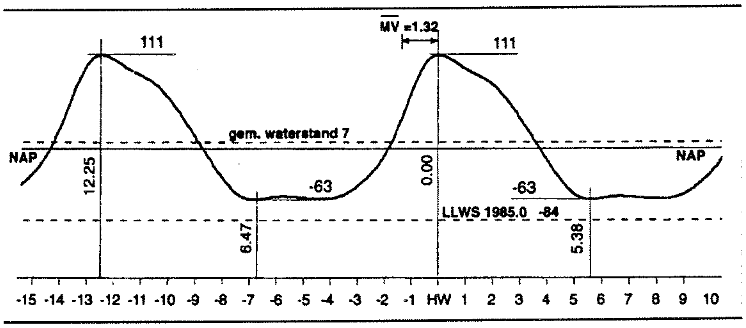


Figure 1. Average tide levels at Hook of Holland.

The exceedence values of the high water level can be read from Table 1.

Tabel IX. SECTOR WEST-HOLLAND
Overschrijdingswaarden in cm t.o.v. NAP

	overschrijdings-frequentie HW/jaar	Hoek van Holland	Scheve-ningen	IJmuiden	Petten
hoge vloeden	5	210	210	190	180
	2	230	230	210	200
	1	245	245	235	215
lage stormvloeden	0,5	260	270	250	235
	0,2	280	290	270	260
middelbare stormvloeden	0,1	300	305	290	280
	5.10^{-2}	315	325	310	300
	2.10^{-2}	340	350	340	325
hoge stormvloeden	10^{-2}	360	370	360	345
	5.10^{-3}	380	390	380	365
	2.10^{-3}	410	420	410	390
buitengewoon hoge stormvloeden	10^{-3}	430	440	435	410
	5.10^{-4}	450	460	460	430
	2.10^{-4}	480	490	490	450
extreme stormvloeden	10^{-4}	505	515	515	470
gemiddeld HW (slotgemiddelde 1991.0)		111	107	97	81
aantal minuten later dan Hoek van Holland		0	15	60	90
grenspeil	0,5	260	270	250	235
1 februari 1953		385	397	385	320
ontwerppeil	10^{-4}	505	515	515	470
basispeil	10^{-4}	505	515	515	470

- De stormvloedkering te Krimpen aan de IJssel wordt bij een verwachte HW-stand te Hoek van Holland boven ca. 220 cm + NAP gesloten.

- De stormvloedkering in de Nieuwe Waterweg wordt bij een verwachte HW-stand te Rotterdam (Boerengat) boven 300 cm + NAP gesloten. **Tot 1 oktober 1998 is deze grens echter nog 320 cm + NAP.**

Table 1. High tide level excedence values in cm relative to NAP.

Another source of tide levels and wave heights is the Informationbulletin 6 [litt], which contains measurements taken at the Light Island Goeree in the period 1979 to 1991. This measuring station is located nearby the Maasvlakte 2 site. Figure 2 directly shows wave heights and their corresponding chance of excedence.

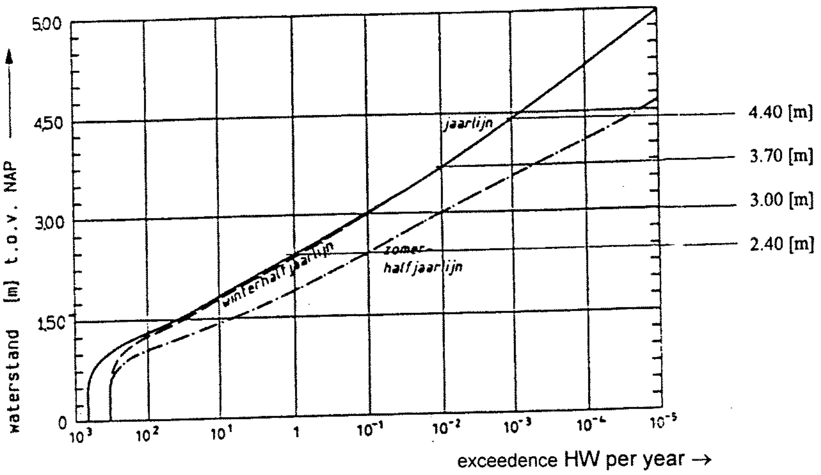


Figure 2. Exceedence of the high water level at Hook of Holland.

Table 2 contains an overview of the water levels from the different sources. The values which will be maintained during further calculations are the maximum values of both sources.

Storm duration of 6 hours	source A: Tide levels of The Netherlands 1998	source B: Information-bulletin 6 of Project group Maasvlakte 2	maximum values of source A and B	Low water levels taken from Information bulletin of the Gemeentewerken Rotterdam [litt]
Exceedence frequency	HWL	HWL	HWL_{max}	LWL
[per year]	[m+NAP]	[m+NAP]	[m+NAP]	[m -NAP]
2	2.30	2.20	2.30	1.35
1	2.45	2.40	2.45	1.50
0.1	3.00	3.00	3.00	1.70
0.01	3.60	3.70	3.70	1.90
0.001	4.30	4.40	4.40	2.05

Table 2 Overview tide levels from different sources.

Sea level rise
According to the IPCC[1] the expected sea level rise due to global warming until the year 2050 is 0.50 m. This must be taken into account when dimensioning constructions in the project area.

Wave heights

The exceedence values of the significant wave height H_s will be determined from the measurements taken the Light Island Goeree (Figure 3).

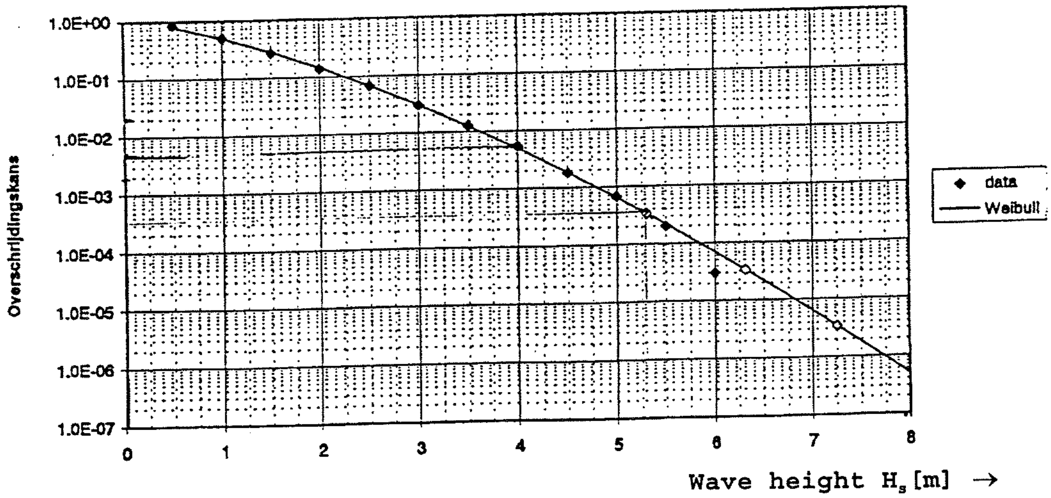


Figure 3 Excedence of the significant wave height h_s.

The excedence levels of the wave heights are used to determine the value of the significant wave height H_s and the return period T_r of this wave height.

The chance of excedence per year of a storm with a return period of T_r can be defined as:

$$P\left(H_s > H_{s,T_r}\right) = \frac{1}{v \cdot T_r}$$

with:

v $= 24 \cdot 365/\Delta t$ = number of storm periods per year

$H_{s,Tr}$ = significant wave height with a return period once every T_r years.

[1] International Panel on Climate Change.

With a storm duration Δt of 6 hours, a year devides into 1460 periods. A year contains 1460 independent observations for H_s. A storm with a significant wave height H_s which is exceeded in average once a year, has an average chance of excedence 1/1460, or 0.0685 %.

The values of Figure 3 result in the following wave heights:

storm duration 6 hours	H_s
Excedence frequency [per year]	[m+NAP]
2	4.67
1	5.00
0.1	6.04
0.01	7.00
0.001	7.89

Table 3 Wave heights measured by the Light Island Goeree.

Wave periods

To determine T_p the relationship between wave steepness, s_p and wave period, T_p [litt 6 vrijling] will be used:

$$s_p = \frac{H_s}{L_p} = \frac{H_s}{\frac{g \cdot T_p}{2\pi}} \quad \text{or: } T_p = \sqrt{\frac{2\pi H_s}{s_p g}}$$

with:

s_p = the wave steepness; for storms on the North Sea a design value of approximately 3.5% is usually maintained
H_s = significant wave height [m]
g = gravity force [m/s^2]
L_P = wave length of peak period [m]

For the wave force on a construction the peak period T_p is important. The zero-crossing period T_z basically only indicates the number of waves during a certain time. T_s is used to determine the wavelength.

The relationship between T_z, T_p and T_s is dependent of the wave spectrum, when there is lack of information about these values, relationships advised by Goda are assumed. According to Goda the peak period T_p is empirically related to the zero crossing period T_z in the following manner: $T_z = 0.7\ T_p$. The significant period T_s can be determined from T_p by the relationship $T_s = 0.95\ T_p$. Further investigation of the wave spectrum may lead to alterations of these relationships.

Table 4 contains an overview of the wave periods.

Storm-duration = 6 hours	significant wave height	wave periods based on wave-steepness, Vrijling		
Exceedence frequency	H_s	T_z	T_p	T_s
[per year]	[m]	[s]	[s]	[s]
2	4.67	6.4	9.1	8.6
1	5.00	6.7	9.6	9.1
0.1	6.04	7.4	10.5	10.0
0.01	7.00	7.9	11.3	10.8
0.001	7.89	8.4	12.0	11.4

Table 4 Overview wave periods.

Wave length

For the calculations of the wave length the following relationship is maintained:

$$L = L_0 \tanh(kh) \qquad \text{en} \qquad L_0 = 1.56T_s^2$$

with:

L_0	= wavelength in deep water	[m]
T_s	= significant wave period	[s]
k	= $2\pi/L_0$	[rad/m]
h	= local water depth	[m]

Currents

The average tidal currents at the breakwater site can be found in the Atlas of currents, Hook of Holland [litt #]. The current is strongest during spring tide when it has a north-western direction with a maximum velocity of 3.7 m/s. During neap tide this velocity is 3.0 m/s (Figure 4).

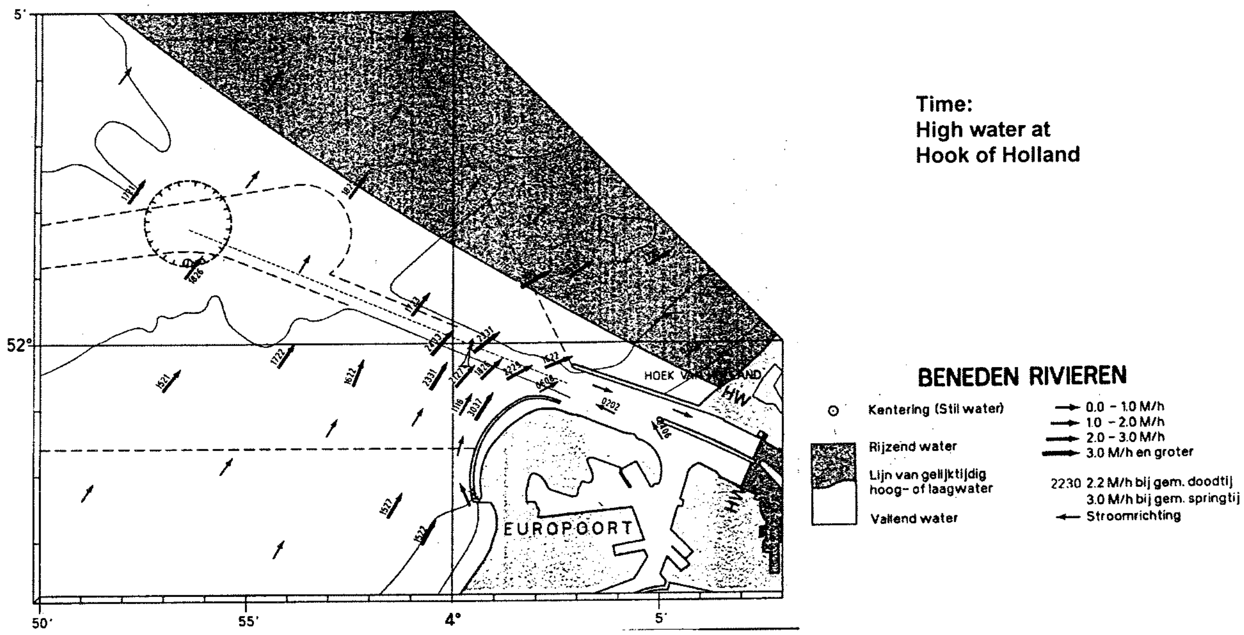


Figure 4 Maximum currents at Hook of Holland.

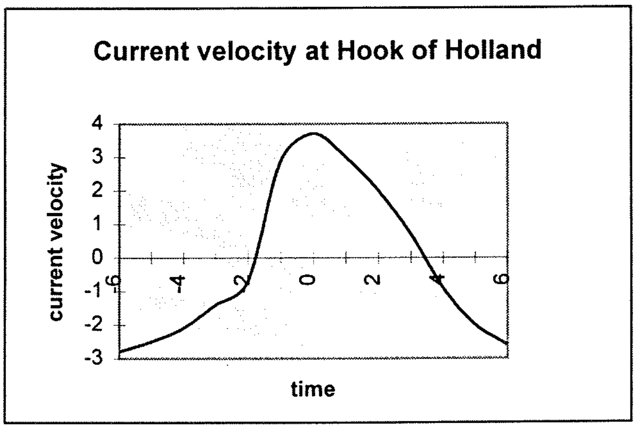


Figure 5 Current velocity at Hook of Holland.

Overview hydraulic boundary conditions

exceedence frequency/ year	LWL [m -NAP]	HWL *maximum values*[2] [m +NAP]	Significant wave heights and wave periods			
			$H_{s,max}$	T_z	T_p	T_s
2	1.35	2.30	4.67	6.4	9.1	8.6
1	1.50	2.45	5.00	6.7	9.6	9.1
0.1	1.70	3.00	6.04	7.4	10.5	10.0
0.01	1.90	3.70	7.00	7.9	11.3	10.8
0.001	2.05	4.40	7.89	8.4	12.0	11.4

Table 5 Overview hydraulic boundary conditions.

[2] In this table the tide and the storm surge have been taken into account.

Appendix D:

Ship dimensions and acceptable wave transmission

- **Vessels to be moored behind breakwater**
- **Ship dimensions**
- **Acceptable wave transmission, $H_{T,lim}$**

Vessels to be moored behind breakwater

Vessels visiting the Maasvlakte 1				
vessel	**harbour basin**	**1995**	**1996**	**1997**
container	Europahaven	3980	3940	3540
	Amazonehaven	0	100	460
dry bulk	Amazonehaven	290	330	410
	Mississippihaven	290	270	310
wet bulk	8e Petr. Haven	170	140	140
total Maasvlakte 1		4730	4780	4860

Table 1. Vessels visiting the Maasvlakte 1.[litt]

Shipping at the Maasvlakte 1 is dominated by container vessels, 80-85% of the total shipment (Table 1.) and in the future an even further explosive growth of the container sector is predicted [litt]. Based on these predictions the harbour basin of the Maasvlakte 2 is designed to accommodate this type of ship. In order to accommodate all the types of container vessels, the harbour basin will be designed for container ships with the largest draft. The future generation of container ships is predicted to increase in beam, and not in draft, and therefore d_{ship}= 13.0 m is maintained for the design of the basin (see Table 2). The acceptable height of the waves transmitted into the harbour basin are illustrated in Table 3. For the transhipment of large container vessels (L_{ship} > 100 m) without hatch (luikloos) $H_{T,lim}$ = 0.2 m is demanded. Local wind waves generated within the basin are assumed not to cause any problems for these ships [litt].

Ship dimensions

Carrying capacity	Displace-ment G	Overall Length	Length between Perps	Beam	Draft	Number of Con-tainers	Gener-ation
DWT	kN	m	m	m	m	circa	
55000	770000	275	260	39.4	12.5	3900	4th
50000	735000	290	275	32.4	13.0	2800	3rd
42000	610000	285	270	32.3	12.0	2380	3rd
36000	510000	270	255	31.8	11.7	2000	3rd
30000	415000	228	214	31.0	11.3	1670	2nd
25000	340000	212	198	30.0	10.7	1380	2nd
20000	270000	198	184	28.7	10.0	1100	2nd
15000	200000	180	166	26.5	9.0	810	1st
10000	135000	159	144	23.5	8.0	530	1st
7000	96000	143	128	19.0	6.5	316	1st

Table 2. Container ship dimensions.

Acceptable wave transmission, $H_{T,lim}$

operatie	Hs,lim	locatie	MV1: T0 bestaande situatie %	MV1: meetpunt	Noord / Open Zee: B0 basis variant %	B0a verlengde h.dammen %	B0b dissipatie constructie %	B0c geen transmissie %	B0d verlengde binnendam %	Noord / Open Zee: meetpunt	Zuid / Estuarium: C1 basis variant %	Zuid / Estuarium: meetpunt	Zuid / Open Zee: D0 basis variant %	D0a geen indringing %	Zuid / Open Zee: meetpunt	Noord - Open Zee via MV1: E0 basis variant %	Noord - Open Zee via MV1: meetpunt	
vastmaken sleepboten	1.8m	havenmond	(3.1)	P1'	(4.6)	(5.3)	(4.7)	(4.5)	(5.3)	P1'	(4.0)	P1'	(2.9)	–	P1'	(2.5)	P1	
		toegangsgebied	0.0	P2	0.5	0.2	0.6	0.5	0.7	P2	0.4	P2	0.0	–	P2	0.5	P2	
			0.0	P3	0.1	0.1	0.1	0.1	0.1	P3	0.0	P3	0.0	–	P3	0.1	P3	
sleepboothulp	1.2m	toegangsgebied	0.3	P2	2.4	1.6	2.5	2.3	3.0	P2	2.6	P2	0.5	–	P2	3.0	P2	
			0.5	P3	0.9	1.0	0.9	0.9	1.1	P3	0.7	P3	0.2	–	P3	1.3	P3	
		binnengebied	0.0	P5	0.0	0.1	0.0	0.0	0.0	P5	0.0	P7t/m8	0.0	0.0	P5t/m7	0.0	P5	
			0.0	P6t/mP11	0.0	0.0	0.0	0.0	0.0	P6t/m11						0.0	P7t/m11	
		exposed terminal	–		–	–	–	–	–		0.4	P4	0.1	0.0	P4	–		
afmeren	1.0m	olie terminal	0.0	P11	0.0	0.0	0.0	0.0	0.0	P11	1.2	P4	0.2	0.2	P4	0.0	P11	
		container / ro-ro terminal	0.0	P8	0.0	0.0	0.0	0.0	0.0	P8	0.0	P7&8	0.0	0.0	P6	0.0	P8	
overslag zeeschepen containers	0.3m	groot (L>100m)	0.0	P8&P11	0.0	0.0	0.0	0.0	0.0	P8	0.3	P7&8	0.0	0.0	P6	0.0	P8	¤
	0.2m	groot luikloos (L>100m)	0.0	P8&P11	0.0	0.1	0.0	0.0	0.0	P8	5.8	P7&8	0.4	0.4	P6	0.0	P8	¤
	0.3m	feeders (L<100m)	0.1	P8	0.2	0.5	0.2	0.3	0.2	P8	8.1	P7&8	1.4	1.2	P6	0.0	P8	
			3.1	P11														
overslag zeeschepen	1.0m	olietankers / LNG-carriers	0.0	P11 (olie)	0.0	0.0	0.0	0.0	0.0	P11	0.9	P4	0.2	0.1	P4	0.0	P11	¤
diverse overige	0.3m	ro-ro	0.0	P8	0.0	0.0	0.0	0.0	0.0	P8	0.3	P8	0.0	0.0	P6	0.0	P8	¤
binnenvaart container / bunker / koppelverband	0.4m	toegang via 'noordroute'	8.0	P4	19.5	19.8	20.2	19.0	18.8	P4	–		–	–		3.7	P7***	
		toegang via 'zuidroute'	1.1	P9	0.4	0.6	0.4	0.4	0.4	P9	4.9	P5	1.8	1.7	P5	–		
		olie/gas-terminal	0.4	P11	0.2	0.3	0.2	0.2	0.2	P11	19.9	P4 via P3	25.0	5.7	P4 via P3	0.0	P11	
		overige terminals	0.0	P8	0.0	0.0	0.0	0.0	0.0	P8	1.7	P7&8	0.1	0.1	P6	0.0	P8	
binnenvaart overig	0.3m	toegang via 'noordroute'	18.1	P4**	34.4	35.7	35.8	32.9	33.7	P4**	–		–	–		15.0	P7**	
		toegang via 'zuidroute'	8.0	P9**	4.4	4.7	4.4	4.3	4.3	P9**	11.4	P5**	5.6	5.0	P5**	–		
		olie/gas-terminal	3.1	P11	2.8	3.5	2.9	2.6	2.6	P11	32.0	P4 via P3	37.4	15.7	P4 via P3	1.0	P11	
		overige terminals	0.1	P8	0.2	0.5	0.2	0.3	0.2	P8	8.1	P7&8	1.4	1.2	P6	0.0	P8	

Table 3. Acceptable wave transmission $H_{T,lim}$ in the harbour basin.

Appendix E: Standard stone gradings

Standaard sorteringen breuksteen

STANDAARDSORTERINGEN (gebaseerd op ρst.=2650 kg/m³)

rho_steen	2.650,00	kg/m3
rho_water	1.025,00	kg/m3
delta	1,59	

Sortering	min/ max	Mgem [kg]	faktor [-]	M50 [kg]	D50 [m]	Dn_{50} [m]	D15 [m]	D85 [m]	D85/D15 [-]	1.5 Dn [m]	2Dn [m]	delta.Dn [-]
	min	0,12		0,16	0,05	0,04	0,02	0,06		0,06	0,08	0,062
30/60 mm	gem	0,24	1,30	0,31	0,06	0,05	0,04	0,07	2,069	0,20	0,20	0,077
	max	0,35		0,46	0,07	0,06	0,05	0,09		0,08	0,11	0,088
	min	0,31		0,40	0,07	0,05	0,04	0,09		0,08	0,11	0,085
40/100 mm	gem	0,62	1,30	0,80	0,08	0,07	0,05	0,10	1,943	0,20	0,20	0,106
	max	0,92		1,20	0,10	0,08	0,07	0,12		0,12	0,15	0,122
	min	0,92		1,20	0,10	0,08	0,05	0,13		0,12	0,15	0,122
50/150 mm	gem	1,65	1,30	2,15	0,12	0,09	0,08	0,15	1,987	0,20	0,20	0,148
	max	2,38		3,09	0,13	0,11	0,10	0,17		0,16	0,21	0,167
	min	2,38		3,09	0,13	0,11	0,08	0,17		0,18	0,21	0,167
80/200 mm	gem	4,77	1,30	6,19	0,16	0,13	0,11	0,20	1,872	0,20	0,27	0,210
	max	7,15		9,30	0,19	0,15	0,14	0,23		0,23	0,30	0,241
	min	10,00		13,00	0,21	0,17	0,16	0,28		0,25	0,34	0,269
5-40 kg	gem	15,00	1,30	19,50	0,24	0,19	0,18	0,31	1,694	0,29	0,39	0,308
	max	20,00		26,00	0,27	0,21	0,20	0,33		0,32	0,43	0,339
	min	20,00		26,00	0,27	0,21	0,20	0,32		0,32	0,43	0,339
10-60 kg	gem	27,50	1,30	35,75	0,30	0,24	0,23	0,34	1,511	0,36	0,48	0,377
	max	35,00		45,50	0,32	0,26	0,25	0,36		0,39	0,52	0,409
	min	80,00		92,00	0,40	0,33	0,31	0,48		0,49	0,65	0,517
40-200 kg	gem	100,00	1,15	115,00	0,44	0,35	0,34	0,51	1,500	0,53	0,70	0,557
	max	120,00		138,00	0,46	0,37	0,37	0,54		0,56	0,75	0,592
	min	130,00		149,50	0,48	0,38	0,36	0,55		0,58	0,77	0,608
60-300 kg	gem	160,00	1,15	184,00	0,51	0,41	0,40	0,58	1,468	0,62	0,82	0,652
	max	190,00		218,50	0,54	0,44	0,43	0,61		0,65	0,87	0,690
	min	540,00		594,00	0,75	0,61	0,60	0,83		0,91	1,21	0,963
300-1000 kg	gem	615,00	1,10	676,50	0,79	0,63	0,63	0,88	1,389	0,95	1,27	1,006
	max	690,00		759,00	0,82	0,66	0,66	0,92		0,99	1,32	1,045
	min	1700,00		1785,00	1,09	0,88	0,88	1,20		1,31	1,75	1,390
1-3 ton	gem	1900,00	1,05	1995,00	1,13	0,91	0,93	1,26	1,357	1,36	1,82	1,442
	max	2100,00		2205,00	1,17	0,94	0,97	1,31		1,41	1,88	1,491
	min	4200,00		4200,00	1,45	1,17	1,26	1,51		1,75	2,33	1,848
3-6 ton	gem	4500,00	1,00	4500,00	1,48	1,19	1,29	1,58	1,221	1,79	2,39	1,891
	max	4800,00		4800,00	1,51	1,22	1,32	1,64		1,83	2,44	1,933
	min	7500,00		7500,00	1,75	1,41	1,57	1,80		2,12	2,83	2,243
6-10 ton	gem	8000,00	1,00	8000,00	1,79	1,45	1,60	1,88	1,172	2,17	2,89	2,291
	max	8500,00		8500,00	1,83	1,47	1,63	1,95		2,21	2,95	2,338

Appendix F:

Structural types of breakwaters[1]

F.1 Introduction to breakwaters

Breakwaters are constructed to provide a calm water ships and to protect harbor facilities. They are also used to protect ports from currents and the intrusion of littoral drift.
There are two main types of breakwaters, rubble mound and composite breakwaters. Rubble mound breakwaters have a rubble mound core with an armor layer that usually consists of shape designed concrete blocks. Due to the development of these blocks, modern day rubble mound breakwaters can strongly resist the destructive power of waves, even in deep waters. Composite breakwaters consist of a rubble foundation and a vertical wall, and are therefore classified as vertical breakwaters. By using caissons as the vertical wall, composite breakwaters provide an extremely stable structure, even in rough and deep seas.
A third type of breakwater which is not so common is the so called non gravity breakwater.

Breakwaters can be classified into three structural types:

1. Rubble mound breakwaters
2. Horizontal composite or vertical composite breakwaters
3. Non gravity breakwaters

F.2 Rubble mound breakwaters

This type of breakwater mainly consists of a rubble mound as shown in Figure 1

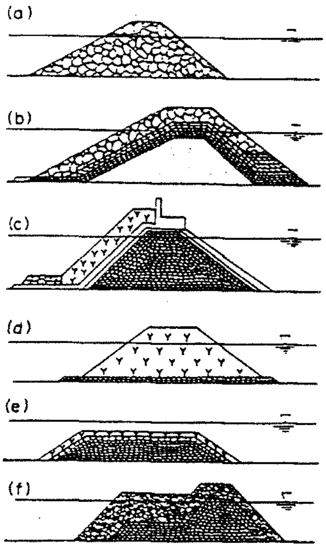


Figure 1 Rubble mound breakwaters.

The most fundamental rubble mound breakwater is one with randomly placed stones (a). To increase stability and decrease wave transmission, as well as to decrease material costs, the multi-layered rubble mound breakwater was developed having a core of quarry run (b). The stability of the armour layer can be strengthened using shape designed concrete blocks, while wave transmission can be reduced using a superstructure, which can also function as an access road to the breakwater (c). Breakwaters made of only concrete blocks (d) are also constructed, especially for use as a detached breakwater providing coastal protection. Although wave transmission is not reduced so much for this breakwater type, its simple construction procedure and the relatively high permeability of the breakwater body are

[1]Shigeo Takahashi, 1996; Reference Document No. 34 'Design of vertical breakwaters'

advantageous. Also reef breakwaters or submerged breakwaters (e) have been constructed for coastal protection, while not interrupting the 'seascape'. Reshaping breakwaters (f) utilize the basic concept of establishing an equilibrium between the slope of the rubble stone and wave action. The rubble mound forms an S-shape slope to stabilize itself against wave actions. This breakwater has a large berm in front, which after some time is reshaped by the wave actions.

F.3 Composite breakwaters

F.3.1 Vertical composite breakwaters

The original concept of the vertical breakwater was to reflect waves, while that for the rubble mound was to break them. Figure 2 shows several vertical type breakwaters having different mound heights. The basic vertical wall breakwater (a) has no rubble mound foundation and is strictly spoken not a vertical composite breakwater. The other breakwaters with a rubble mound foundation are vertical composite breakwaters. The high mound composite breakwater (d) has a mound that is higher than the low water level, and the low mound composite breakwater (c) has a mound that is lower than the low water level. The high mound composite breakwater causes waves to break on the mound, which causes unstability due to wave generated impulsive pressure and scouring. The low mound composite breakwater doesn't have this problem as waves don't break on it's mound.

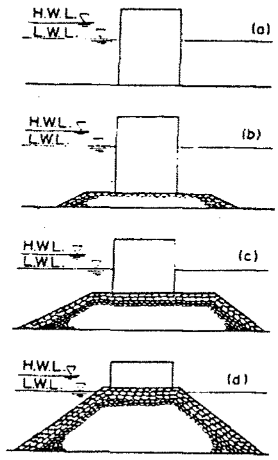


Figure 2 Vertical type breakwaters.

F.3.2 Horizontal composite breakwaters

To reduce wave reflection and the breaking wave force on the vertical wall, concrete blocks, for example tetrapods, can be placed in front of it. These breakwaters are very similar to rubble mound breakwaters with an armor layer. They are basically different however since the concrete blocks of the rubble mound breakwater acts as the armor for the rubble foundation, while the concrete blocks of the horizontal composite breakwater functions to reduce the wave force and size of the reflected waves, see Figure 3.

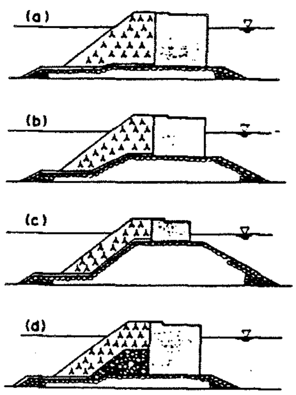


Figure 3 Horizontal composite breakwaters.

F.3.3 Structural features of vertical walls

Figure 4 shows several types of vertical walls. An upright wall with block masonry (b) was initially most popular. Also cellular blocks (c.) have been used. The invention of the caisson, see appendix I, (d) made breakwaters more reliable. Caisson breakwaters have been improved using sloping top caisson (e) or perforated walls (f). Appendix E presents an overview of different types of caisson shapes.

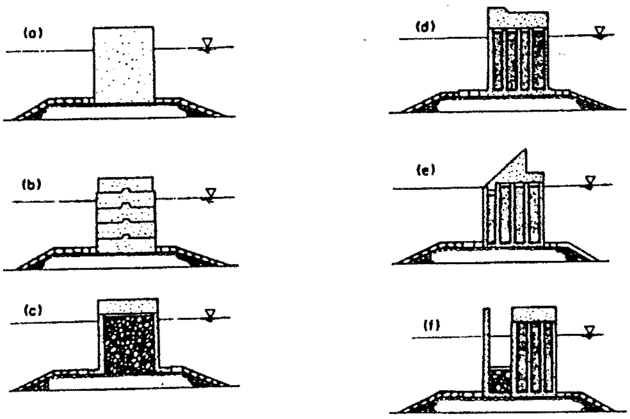


Figure 4 Types of vertical walls.

F.4 Non gravity breakwaters

This type of breakwater employs special features, and is not commonly used (Figure 5). The curtain wall breakwater (a) is used as a secondary breakwater to protect small craft harbors. The vertical wall breakwater has sheetpiling or wooden piles (b) to break relatively small waves. A horizontal plate breakwater (c.) can reflect and break waves. Floating breakwaters (d) are very useful as a breakwater in deep water, but its effect is limited to relatively short waves. The pneumatic breakwater (e) breaks the waves due to a water current induced by air bubble flow and is considered effective for improving nearby water quality, though only being effective for relatively short waves.

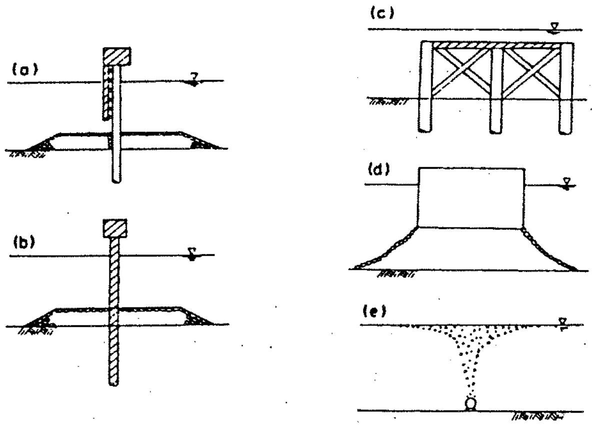


Figure 5 Non-gravity breakwaters.

F.5 Items to be considered in the selection of a breakwater

When selecting the most suited breakwater for a certain situation one must take account of the following factors:

- Layout of the breakwaters;
- Environmental conditions considering aspects as wave reflection;
- Utilization conditions an functions, top loadings, tide and foundation conditions;
- Executive conditions, available equipment and materials;
- Costs of the construction;
- Construction terms;
- Importance of the breakwaters and acceptable risk;
- Available construction materials;
- Maintenance;

F.6 Comparison of rubble mound and composite breakwaters

F.6.1 Advantages of vertical composite breakwaters:

- Smaller seabed occupancy with less impact on flora and fauna;
- Less material quantities;
- Reuse of the dredging material for filling caisson cells;
- Less need of maintenance;
- Safer close navigation, breakwater is clearly visible and reduced underwater obstacles;
- Potential to be removed;
- Reduced environmental impact at construction, less trucks and air pollution, less noise and less water turbidity;
- Rapid construction possible with a reduction of failure during construction;
- Sometimes it is the only option if there is a lack of rubble stones;

F.6.2 Advantages of rubble mound breakwaters:

- Use of natural materials;
- Use of smaller construction equipment;
- Less environmental impact due to smaller reflected waves and more water exchange;
- Creation of a natural reef;
- Potentially lower crest elevation;

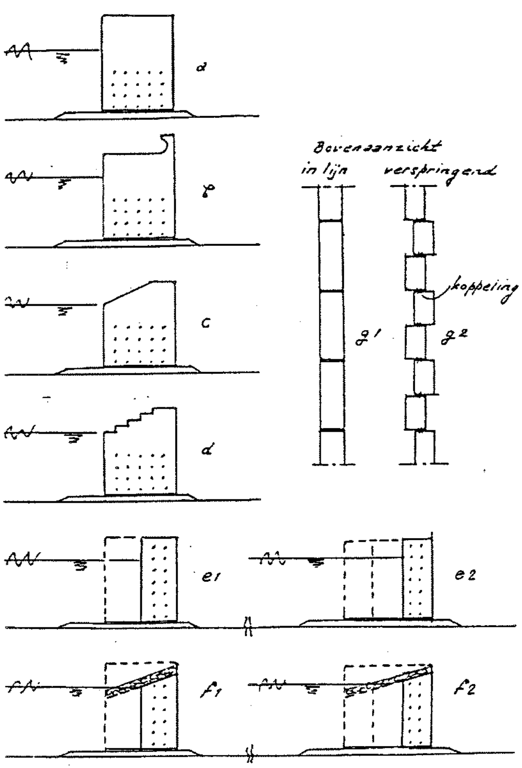
F.7 Conclusion

The most suited type of breakwater for the hard and reusable shore protection of the Maasvlakte 2 is a caisson breakwater. Due to the large water depth at the breakwater site and the small chance of waves breaking against the breakwater, it can be founded on a low rubble mound foundation without concrete blocks placed before it to reduce wave forces. The design rules of Goda for a vertical caisson breakwater are discussed in the next section, and also the dimensions of the Maasvlakte 2 breakwater are calculated.

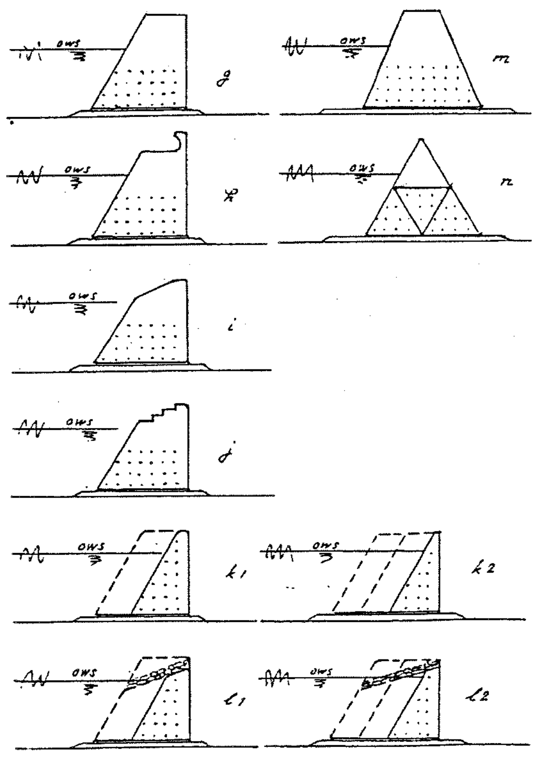
Appendix G:

Caisson shapes

Caissons are not always constructed as square boxes. In order to reflect incoming waves, absorb wave forces, reduce wave transmission and economize on materials, much research has been done in order to obtain more efficient caisson shapes. Here the diversity of caisson shapes is shown.

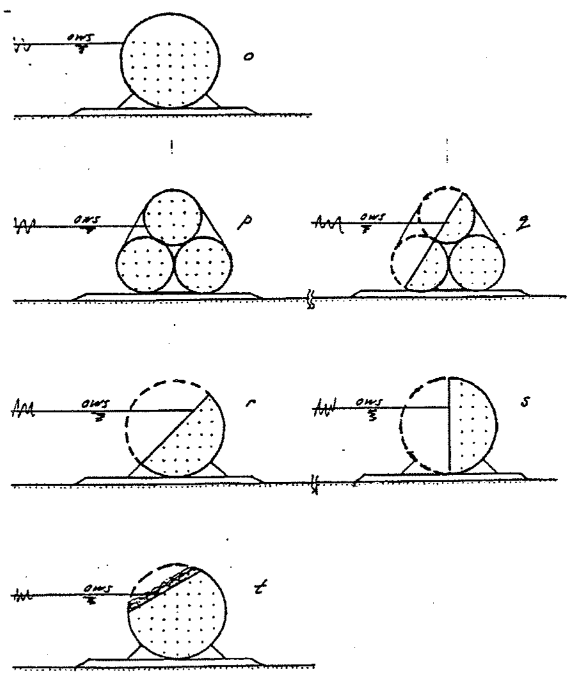


Rectangular shaped caissons[1]

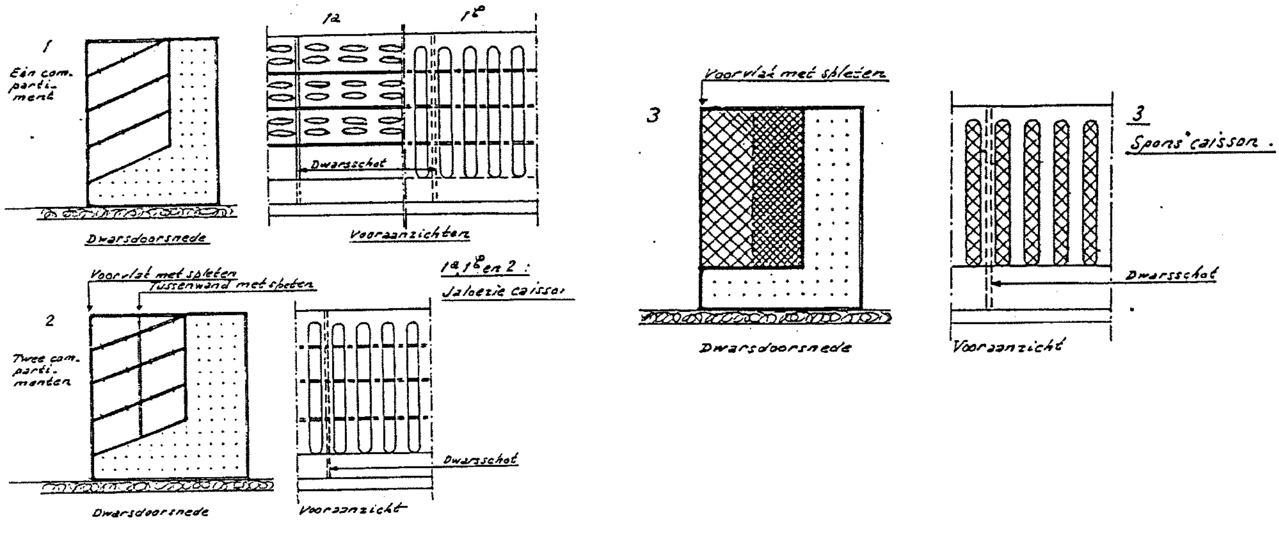


Trapezium shaped caissons

[1] Dotted lines represent a porous caisson front, behind which is a chamber where wave energy is absorbed.



Circular caissons



Caissons with slits[2]

[2] These caissons are sometimes referred to as spundge caissons. The seaward side of the compartments are more porous than the inner side. The spundge can be made of rocks or for example logs.

Appendix H:

Goda design formulas for vertical breakwaters

H.1 Design of vertical breakwaters, Goda

Formulas of wave pressure under wave crests:

The wave pressure formulas proposed by Goda for the design of vertical breakwaters assume the existence of a trapezoidal pressure distribution along a vertical wall, as shown in Figure 1, regardless of whether the waves are breaking or nonbreaking. In this figure, h denotes the water depth in front of the breakwater, d the depth above the armor layer of the rubble foundation, h' the distance from the design water level to the bottom of the upright section, and h_c the crest elevation of the breakwater above the design breakwater level. The wave height for the pressure calculation and the other formulas are specified below.

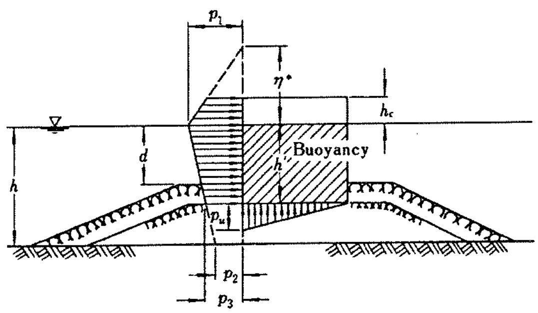


Figure 1 Distribution of wave pressure on an upright section of a vertical breakwater.

Design wave:
The heighest wave in the design sea state is to be employed. Its height is taken as $H_{max} = 1.8\ H_{1/3}$ seaward of the surf zone, whereas within the surf zone the height is taken as the heighest of the random breaking waves H_{max} at the location at a distance 5 $H_{1/3}$ seaward of the breakwater. The wave height $H_{1/3}$ is to be estimated at the depth of the location of the breakwater.
The period of the heighest wave is taken as that of the significant wave, $T_{max} = T_{1/3}$.

Elevation to which the wave pressure is exerted:

$$\eta^* = 0.75\ (1 + \cos\beta)H_{max} \qquad (1.1)$$

in which β denotes the angle between the direction of wave approach and a line normal to the breakwater. The wave direction should be rotated by an amount of up to 15° toward the line normal to the breakwater from the principal wave direction. This directional correction is made in view of the uncertainty in the estimation of the design wave direction.

Horizontal wave pressure on the front of a vertical wall:

$$p_1 = \frac{1}{2}(1+\cos\beta)(\alpha_1 + \alpha_2 \cos^2\beta)\rho g H_{\max} \qquad (1.2)$$

$$p_2 = \frac{p_1}{\cosh(2\pi h / L)} \qquad (1.3)$$

$$p_3 = \alpha_3 p_1 \qquad (1.4)$$

in which

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h / L}{\sinh(4\pi h / L)}\right]^2 \tag{1.5}$$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{\max}}{d}\right)^2, \frac{2d}{H_{\max}}\right\} \tag{1.6}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh(2\pi h / L)}\right] \tag{1.7}$$

min{a,b} : smaller of a and b,
h_b : water depth at a distance 5 $H_{1/3}$ seaward of the breakwater. The above pressure intensities are assumed not to change even if wave overtopping takes place. The value of the coefficient α_1 can be read off of Figure 2 and the value $1/\cosh(2\pi h/L)$ for α_3 is obtained from Figure 3. The symbol L_0 in both figures denotes the wavelength corresponding to the significant wave period in deep water.

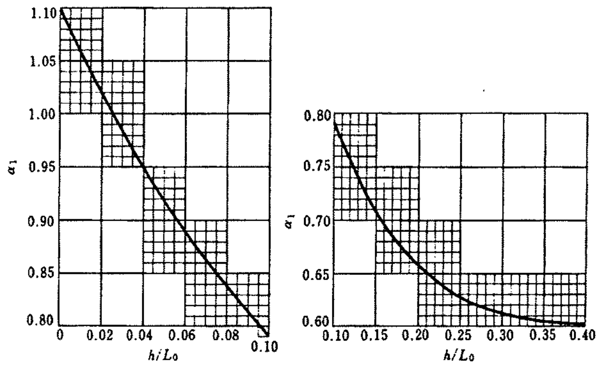


Figure 2 Calculation diagrams for the parameter α_1[litt].

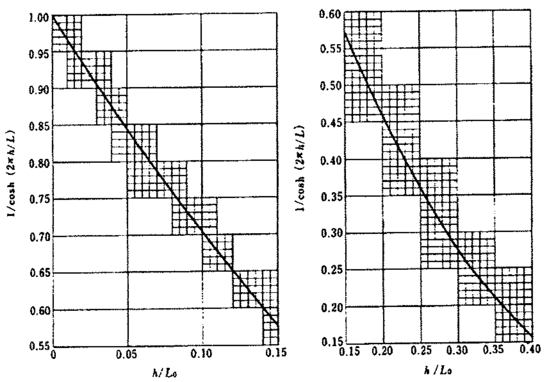


Figure 3 Calculation diagrams for the factor of $1/\cosh/(2\pi h/L)$ [litt].

Impulsive wave pressure:
An impulsive pressure is exerted on a vertical wall when an incident wave begins to break in front of the wall and collides with it whilst the front face of the wave is almost vertical, as shown in Figure 4 . The impinging wave loses its forward momentum in the short time during which the collision takes place. The forward momentum is converted into an impulse which is exerted on the vertical wall. By

denoting the forward momentum of the breaking wave per unit width as M_v, the total impulsive pressure on the wall as P_I, and its duration as τ, the momentum equation for the present situation becomes as follows:

$$\int_0^{\tau} P_I dt = M_V \tag{1.20}$$

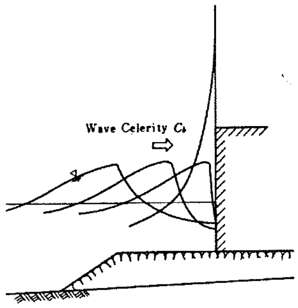


Figure 4 Profiles of a breaking wave colliding with a vertical wall.

To estimate the magnitude of this momentum the case of a water mass in the form of a semi-circular cylinder with a diameter of H_b advancing with the speed of the wave C_b can be considered. This leads to:

$$M_V = \frac{\pi w_0}{8g} C_b H_b^{\ 2} \tag{1.21}$$

where g denotes the acceleration of gravity.

If it assumed that the impulsive pressure increases linearly at the start of the collision (t=0) to a maximum value at t=τ, and then reduces to zero for t > τ, the peak value of the impulsive pressure is obtained as:

$$(P_I)_{\max} = \frac{\pi w_0 C_b H_b^{\ 2}}{4g\tau} \tag{1.22}$$

It is seen that for this model the impulsive pressure is inversely proportional to its duration. Thus the impulsive pressure can attain a very large value when the front face of the breaking wave is in the form of a vertical flat plane and collides with the vertical wall over a very short time duration.

However laboratory experiments show that the front face of an impinging wave is always curved and a small amount of air is entrapped at the instant of the collision. As suggested by Bagnold and formulated in a theoretical model by Takahashi and Tanimoto, the entrapped air acts to dampen the impulsive pressure and prevents it from coming abnormally high.

Extension of the Goda formula

Takahashi and Tanimoto [litt tak] extended the Goda formula in order to include the Impulsive wave force on a vertical wall. According to Takahashi and Tanimoto the influence of the impulsive wave force against the construction can be included into the model of Goda by calculating a larger value for the horizontal pressure. The value p_1 (Eq. 1.2) as stated in the formula of Goda is modified with:

$$p_1 = \frac{1}{2}(1+\cos\beta)(\alpha_1 + \alpha * \cos^2\beta)\rho g H_{\max} \tag{1.23}$$

The coefficient α^* in this formula represents the dynamic wave pressure. The value for α^* is denoted by:

$$\alpha^* = \max\{\alpha_2, \alpha_I\} \tag{1.24}$$

$$\alpha_{I0} = \begin{cases} H/d & :H \le 2d \\ 2 & :H > 2d \end{cases} \tag{1.25}$$

$$\alpha_{I1} = \begin{cases} \cos\delta_2/\cosh\delta_1 & :\delta_2 \le 0 \\ 1\big/\left[\cosh\delta_1\sqrt{\cosh\delta_2}\right] & :\delta_2 > 0 \end{cases} \tag{1.26}$$

where

$\alpha_I = \alpha_{I0}\alpha_{I1}$ (1.27)

$\alpha_2 =$ (Eq. 1.6)

and α_{I0} represents effect of wave height on the mound, α_{I1} represents effect of rubble mound shape.

The values for the delta's can be calculated:

$$\delta_1 = \begin{cases} 20\delta_{11} & :\delta_{11} \le 0 \\ 15\delta_{11} & :\delta_{11} > 0 \end{cases} \tag{1.28}$$

$$\delta_2 = \begin{cases} 4.9\delta_{22} & :\delta_{22} \le 0 \\ 3\delta_{22} & :\delta_{22} > 0 \end{cases} \tag{1.29}$$

$$\delta_{11} = 0.93(B_M/L-0.12) + 0.36\{(h-d)/h-0.6\} \tag{1.30}$$

$$\delta_{22} = -0.36(B_M/L-0.12) + 0.93\{(h-d)/h-0.6\} \tag{1.31}$$

where B_M is the berm width and L is the wave length at the breakwater site.

Buoyancy and uplift pressure:
The buoyancy is to be calculated for the displacement volume of the upright section in still water below the design water level, and the uplift pressure acting on the bottom of the upright section is assumed to have a triangular distribution (see Figure 1) with toe pressure p_u given by the equation given below, and with a heel pressure of zero. Both the buoyancy and uplift pressure are assumed to be unaffected by wave overtopping.

$$p_u = \frac{1}{2}(1+\cos\beta)\alpha_1\alpha_3\rho g H_{max} \tag{1.8}$$

Adoption of the wave height H_{max} in the above pressure formulas is based on the principle that a breakwater should be designed to be safe against the single wave with the largest wave pressure among storm waves. In fact the value H_{max} is a probabilistic quantity, but to avoid possible confusion in design, a definite value of $H_{max} = 1.8H_{1/3}$ is recommended in consideration of the performance of many prototype breakwaters as well as with regard to the accuracy of the wave pressure estimation. Certainly there remains the possibility that one or two waves exceeding $1.8H_{1/3}$ will hit the site of the breakwater when storm waves equivalent to the design condition attack. This distance of sliding of an upright section is very small according to Goda, and will not result in failure of the construction.

With the above formulas for the wave pressure, the total horizontal wave pressure and its moment around the heel of the upright section (see Figure 5) can be calculated with the following equations:

$$P = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^* \qquad (1.9)$$

$$M_p = \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h'h_c^* + \frac{1}{6}(p_1 + 2p_4)h_c^{*2} \qquad (1.10)$$

in which

$$p_4 = \begin{cases} p_1(1 - h_c/\eta^*) & :\eta^* > h_c \\ 0 & :\eta^* \le h_c \end{cases} \qquad (1.11)$$

$$h_c^* = \min\{\eta^*, h_c\} \qquad (1.12)$$

The total uplift pressure and its moment around the heel op the upright section are calculated with:

$$U = \frac{1}{2}p_u B \qquad (1.13)$$

$$M_U = \frac{2}{3}UB \qquad (1.14)$$

where B denotes the width of the bottom upright section.

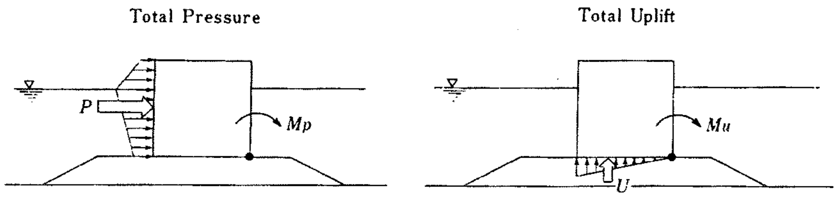


Figure 5 Definition sketch of total pressure and uplift as well as their moments.

H.2 Design of upright sections

Stability condition for an upright section

The upright section of a vertical breakwater must be designed to be safe against sliding and overturning. At the same time the bearing capacity of the rubble mound foundation and the seabed should be examined to ascertain that they remain below the allowable limit. The safety factors against sliding $\gamma_{sliding}$ and overturning $\gamma_{overturning}$ of an upright section under wave action are defined by the following:

$$\gamma_{sliding} = \frac{\mu(W - U)}{p} \qquad (4.15)$$

$$\gamma_{overturning} = \frac{W \cdot t - M_U}{M_P} \qquad (4.16)$$

where W denotes the weight of the upright section per unit extension in still water, μ the coefficient of friction between the upright section and the rubble mound, and t the horizontal distance between the center of gravity and the heel of the upright section.

According to Goda the safety factors against sliding and overturning must not be less than 1.2. The coefficient between concrete and rubble stones is usually taken as μ= 0.6 (Table 1).

Friction layers	μ
Concrete and concrete	0.5
Concrete and base rock	0.5
Concrete and rubble stone	0.6
Rubble stone and rubble stone	0.8

Table 1. Various coefficients of friction μ [litt. takahashi].

H.3 The required bearing capacity of the foundation

The required bearing capacity of the foundation is to be analyzed by means of the methology of foundation engineering for eccentric inclined loads. At sites where the seabed consists of a dense sand layer or soil of good bearing capacity, however a simplified technique of examining the magnitude of the heel pressure is often employed. For this method, it is assumed that a trapezoidal or triangular distribution of bearing capacity exists beneath the bottom of the upright section, and the largest bearing pressure at the heel p_e is calculated as:

$$p_e = \begin{cases} \dfrac{2W_e}{3t_e} & :t_e \leq \dfrac{1}{3}B \\ \dfrac{2W_e}{B}\left(2 - 3\dfrac{t_e}{B}\right) & :t_e > \dfrac{1}{3}B \end{cases} \tag{4.17}$$

in which

$$t_e = \frac{M_e}{W_e}, \qquad M_e = W \cdot t - M_U - M_P, \qquad W_e = W - U \tag{4.18}$$

The bearing capacity at the heel is to be kept below the value of 40 to 50 ton/m^2, but recent breakwater designs are gradually increasing this limit to 60 ton/m^2 or greater, with advancement of breakwater construction sites into deeper water and with increases in the weight of the upright sections.

H.4 Width of the upright section:

H.5 Used parameters in Goda equations and its extension:

B_M	: berm width
B	: caisson width
d	: water depth above the armor layer of the rubble foundation
g	: acceleration of gravity
h	: water depth in front of breakwater
h_c	: the crest elevation of the breakwater above the design breakwater level
h'	: distance from the design water level to the bottom of the upright section
h_b	: water depth at a distance $5 \cdot H_{1/3}$ seaward of the caisson ≈ h
H_{max}	: design wave height
L	: wave length at breakwater site
M_p	: moment around the heel caused by horizontal wave pressure
M_U	: moment around the heel caused by uplifting wave pressure
P	: total horizontal wave pressure on the front of vertical wall
p_i	: wave pressures
p_u	: uplift pressure acting on the bottom of the upright section
T_{max}	: period of the heighest wave
U	: total uplift pressure caused by waves
W	: weight of the upright section per unit extension in still water
α_i, δ_i	: coefficients
β	: angle of wave approach
η*	: elevation to which the wave pressure is exerted
μ	: coefficient of friction between the upright section and the rubble mound

Appendix I:

Transmission model of Goda

The amount of wave transmission is measured by the transmission coefficient which is defined by:

$$K_T = {}^{H_T}\!/_{H_I} \tag{1}$$

where H_I is the incident wave height and H_T is the transmitted wave height.

Transmitted waves are caused by wave transmission through the structure and overtopping, K_{Tt} and K_{to} , with the total transmission coefficient denoted as:

$$K_T = (K_{Tt}^2 + K_{to}^2)^{1/2} \tag{2}$$

Because wave transmission by overtopping waves is produced by waves generated at the lee, which result due to the impact from the fall of the overtopping mass, the transmitted waves have a complicated form with high frequency components. Therefore, in general, not only the wave height but also the wave period of transmitted waves are different from those of incident waves, i.e., the wave period of transmitted waves is generally smaller.

Wave transmission of vertical wall breakwaters is mainly by overtopping, and therefore, the ratio of the breakwaters crest height, h_c to the incident wave height is the principal parameter governing the wave transmission coefficient. Based on regular wave tests, Goda (1969) proposed the following equations to represent the transmission coefficient for vertical breakwaters:

Formulas:

$$K_T = \frac{H_T}{H_i} \qquad \text{for } \alpha = 2.2,\ \beta = \text{see nomograph} \tag{3.a}$$

$$K_T = \left[0.25\cdot\left\{1-\sin\frac{\pi}{2\alpha}\cdot\left(\frac{h_c}{H_i}+\beta\right)\right\}^2 + 0.01\cdot\left(1-\frac{h'}{h}\right)^2\right]^{1/2} \qquad \text{for } \beta - \alpha < h_c/H_i < \alpha - \beta \tag{3.b}$$

$$K_T = 0.1\cdot\left(1-\frac{h'}{h}\right) \qquad \text{for } h_c/H_i \geq \alpha - \beta \tag{3.c}$$

Figure 1 shows a schematized view of the breakwater and the parameters maintained by Goda.

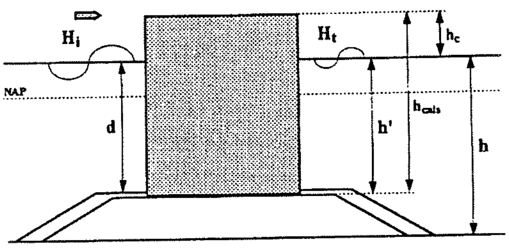


Figure 1. Schematized transmission according to Goda.

The term h' is the distance from the design water level to the bottom of the caisson. According to Goda coefficient α is constant for transmission calculations for caissons and can be taken as $\alpha = 2.2$. The coefficient β can be obtained by using the nomograph in Figure 2 or the following formula:

$$\beta = \left(\frac{d}{h} - 0.19\right) - 0.41\left(\frac{d}{h}\right)^{2.6} \tag{4}$$

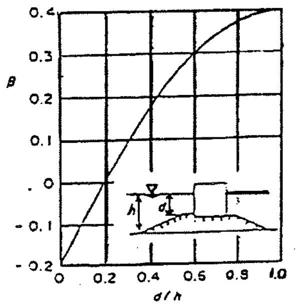


Figure 2 Nomograph for determining β.

Figure 3 shows the transmission coefficient for vertical breakwaters using equation 3.

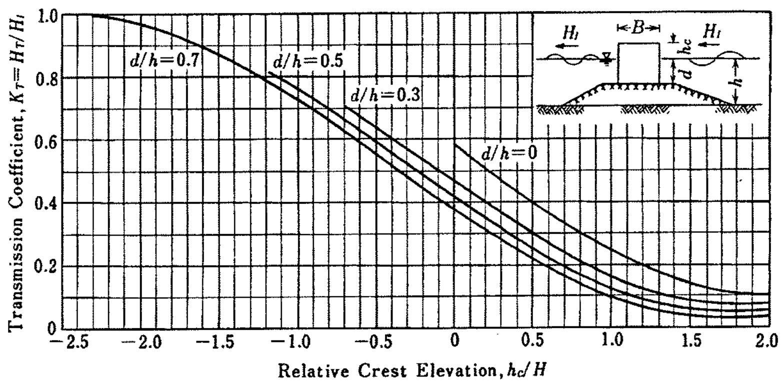


Figure 3 Transmission coefficient for a vertical breakwater (Goda 1969).

Appendix J:

EXCEL computer model Outer Caisson Dimensions O.C.D.

Computer model Outer Caisson Dimensions, (O.C.D.)

by: S. Mann
date: 24-02-99

MODIFIED GODA CALCULATION

INPUT

	Parameter	Value	Dimension	Definition
Wave conditions	$H_{s,sls}$	**4,57**	[m]	*local wave height with return period of 1year*
	HWL_{sls}	**2,45**	[m tov NAP]	*water level with return period of 1year*
	$H_{s,uls}$	**7,43**	[m]	*local wave height with return period of 1000 years*
	HWL_{uls}	**4,40**	[m tov NAP]	*water level with return period of 1000 years*
	$H_{t,sls}$	**0,20**	[m]	*acceptable wave transmission 1/year*
	LWL_{ULS}	-2,05	[m tov NAP]	*design low water level, 1/100 years*
	s	3,50	[%]	*wave steepness*
Breakwater dimensions	b.l.	**-18,00**	[m tov NAP]	*bottom level*
	h_{rm}	**3,25**	[m]	*total rubble mound height (filter & leveling layer included)*
	B_m	5,00	[m]	*berm width*
	θ	0,00	[rad]	*incedence angle of waves*
	α	2,20	[-]	*Goda coefficient*
	L_c	75,00	[m]	*caisson length*
subsoil coef.	φ_{sub}	37,5	[deg.]	*internal angle of friction of the subsoil*
	c_{sub}	0	[kN/m^2]	*cohesion of the subsoil*
	$\rho_{sub,subm}$	11	[kN/m^3]	*underwater weight of the subsoil*
Rubble mound coef.	μ	0,60	[-]	*friction coefficient between concrete and rubble mound*
	φ_{rm}	40,0	[deg.]	*internal angle of friction of the rubble mound*
	c_{rm}	0	[kN/m^2]	*cohesion of the rubble mound*
	$\rho_{rm,subm}$	16,50	[kN/m^3]	*underwater weight of the rubble mound*
Densities	ρ_{water}	10,30	[kN/m^3]	*density of the sea water*
	$\rho_{cs,emerged}$	21,00	[kN/m^3]	*average density of the emerged caisson*
	g	9,81	[m/s^2]	*gravity*
Sea level rise	slr	0,50	[m/century]	*sea level rise*
Safety factor	γ	1,20	[-]	*minimal safety factor*

RESULTS

Outer caisson dimensions:				
Caisson height (sls)	$h_{caisson}$	**23,36**	**[m + NAP]**	*total construction height of caisson*
Caisson width (uls)	**B**	**21,09**	**[m]**	*total construction width of caisson*
Safety factors:				
sliding	$\gamma_{sliding}$	**1,49**	**[-]**	*safety factor*
overturning	$\gamma_{overturning}$	**1,72**	**[-]**	*safety factor*
bearing capacity rm	$\gamma_{bearcap,rm}$	**1,54**	**[-]**	*safety factor*
bearing capacity subsoil	$\gamma_{bearcap,sub}$	**1,21**	**[-]**	*safety factor*

CALCULATIONS

wave periods:	$T_{p,sls}$	9,14	[s]	*peak period, sls*	$T_p=(2\pi H_s/s_p g)^{1/2}$
	$T_{z,sls}$	6,40	[s]	*zero crossing period, sls*	$T_z=0.7T_p$
	$T_{s,sls}$	8,69	[s]	*significant wave period, sls*	$T_s=0.95T_p$
	$T_{p,uls}$	11,66	[s]	*peak period, uls*	$T_p=(2\pi H_s/s_p g)^{1/2}$
	$T_{z,uls}$	8,16	[s]	*zero crossing period, uls*	$T_z=0.7T_p$
	$T_{s,uls}$	11,08	[s]	*significant wave period, uls*	$T_s=0.95T_p$
water heights:	h_{sls}	20,95	[m]	*water depth seawards of caisson*	$h_{sls}=$ -b.l.+HWL_{sls}+slr
	h_{uls}	22,90	[m]	*water depth seawards of caisson*	$h_{uls}=$ -b.l.+HWL_{uls}+slr
	$h_{uls,low}$	15,95	[m]	*water depth seawards of caisson*	$h_{uls,low}=$-b.l.+LWL_{uls}
	d_{sls}	17,70	[m]	*water depth above rm (land side)*	$d_{sls}=h_{sls}-h_{rm}$
	d_{uls}	19,65	[m]	*water depth above rm (land side)*	$d_{uls}=h_{uls}-h_{rm}$
	$d_{uls,low}$	12,70	[m]	*water depth above rm*	$d_{uls,low}=h_{sls,low}-h_{rm}$
	h'_{sls}	17,70	[m]	*water depth above rm (landside)*	
	$h_{b,uls}$	22,90	[m]	*water depth at distance 5 $H_{1/3}$ seaward of caisson*	$h_{b,uls}=h_{uls}$
	h'_{uls}	22,90	[m]	*water depth land wards of caisson*	$h'_{uls}=h_{uls}$
	$h_{b,uls,low}$	15,95	[m]	*water depth at distance 5 $H_{1/3}$ seaward of caisson*	$h_{b,uls,low}=h_{uls,low}$
	$h'_{uls,low}$	15,95	[m]	*water depth land wards of caisson*	$h'_{uls,low}=h_{uls,low}$
wave lengths:	$L_{0,sls}$	117,84	[m]	*deep water wave length*	$L_{0,sls}=gT_s^2/2\pi$
	$L_{0,uls}$	191,59	[m]	*deep water wave length*	$L_{0,uls}=gT_s^2/2\pi$
	L_{sls}	101,47	[m]	*local water depth wave length*	$L_{sls}=L_0\tanh kh_{sls}$ with:$k=2\pi/L_{sls}$ (iteratie)
	L_{uls}	145,15	[m]	*local water depth wave length*	$L_{uls}=L_0\tanh khu_{ls}$ with:$k=2\pi/L_{uls}$ (iteratie)
	$L_{uls,low}$	126,39	[m]	*local water depth wave length*	$L_{uls}=L_0\tanh kh_{uls,low}$ with:$k=2p/L_{uls}$ (iteratie)
wave heights:	H_D	13,37	[m]	*Design wave height according to Goda*	$H_D=1.8H_{s,uls}$
Crest level (sls):	hcrest	8,61	[m tov NAP]	*crest elevation level above design water level*	h_{crest}=slr+$HWL_{sls}+h_c$
	$K_{T,sls}$	0,04	[-]	*transmission coefficient*	$K_T=H_T/H_I$
	h_c	5,66	[m]	*Goda transmission height*	....
	$h_c/H_{I,sls}$	1,24	<=1,25?	*voorwaarde!*	
	β	0,390	[-]	*Goda coefficient*	

Wave forces:

Horizontal wave force,uls,HWL

δ	δ_{11}	-0,24	[-]	*Goda coefficient*	$\delta_{11}=0{,}93(B_m/L-0{,}12)+0{,}36((h-d)/h-0{,}6)$
	δ_{22}	-0,40	[-]	*Goda coefficient*	$\delta_{22}=-0{,}36(B_m/L-0{,}12)+0{,}93((h-d)/h-0{,}6)$
	δ_1	-4,89	[-]	*Goda coefficient*	$\delta_1=20\delta_{11}$ als $\delta_{11}<=0$ en $\delta_1=15\delta_{11}$ als $\delta_{11}>0$
	δ_2	-1,94	[-]	*Goda coefficient*	$\delta_2=4{,}9\delta_{22}$ als $\delta_{22}<=0$ en $\delta_2=3\delta_{22}$ als $\delta22>0$
α	α_1	0,75	[-]	*Goda coefficient*	$\alpha_1=0{,}6+0{,}5((4\pi h/L_{uls})/(\sinh(4\pi h/L_{uls})))^2$
	α_2	0,03	[-]	*Goda coefficient*	$\alpha_2=\min\{((h_b-d)/3h_b)(H_{max}/d);(2d/H_{max})\}$
	α_3	0,65	[-]	*Goda coefficient*	$\alpha_3=1-h'/h[1-1/\cosh(2\pi h/L_{uls})]$
	α^*	0,03	[-]	*Goda coefficient*	$\alpha^*=\max\{\alpha_2,\alpha_I\}$
	α_{I0}	0,38	[-]	*Goda coefficient*	$\alpha_{I0}=H_{s,uls}/d_{uls}$ als $H_{s,uls}<=2d_{uls}$ en $\alpha_{I0}=2$ als $H_{s,uls}>2d_{uls}$
	α_{I1}	-0,01	[-]	*Goda coefficient*	$\alpha_{I1}=\cos\delta_2/\cosh\delta_1$ als $\delta_2<=0$ en $1/(\cosh\delta_1{*}(\cosh\delta_2)^{1/2})$ als $\delta_2>0$
	α_I	0,00	[-]	*Goda coefficient*	$\alpha_I=\alpha_{I0}\alpha_{I1}$
pressure coeff.	θ	0,00	[rad]	*incedence angle of waves*	
	η^*	10,78	[m]	*elevation to which the wave pressure is exerted*	$\eta^*=0{,}75(1+\cos\beta)H_{max}$
	h_c^*	5,66	[m]	*minimum of η^* and hc*	$h_c^*=\min\{\eta^*,h_c\}$
	p1	108.427	[N/m^2]	*pressure*	$p_1=0{,}5(1+\cos\beta)(\alpha_1+\alpha^*\cos^2\beta)\rho_{water}{*}H_D$
	p2	70.735	[N/m^2]	*pressure*	$p_2=p_1/\cosh(2\pi h/L_{uls})$
	p3	70.735	[N/m^2]	*pressure*	$p_3=\alpha_3 p_1$
	p4	51.475	[N/m^2]	*pressure*	$p_4=p_1(1-h_c/\eta^*)$ als $\eta^*>h_c$ and 0 als $\eta^*<=h_c$
	p_U	67.841	[N/m^2]	*pressure*	$p_u=0{,}5{*}(1+\cos\beta)\alpha_1\alpha_3\rho_{water}{*}H_D$
	P	2.504.126	[N/m^1]	*horizontal force on caisson per unit length caisson*	$P=0{,}5(p_1+p_3)h'_{uls}+0{,}5(p_1+p_4)h_c^*$
	M_P	36.632.699	[Nm/m^1]	*moment around heel of caisson resulting from horiz. force*	$M_P=(1/6){*}(2p_1+p_3)h'^2_{uls}+0{,}5(p_1+p_4)h'_{uls}h_c^*+(1/6)(p_1+2p_4)h_c^{*2}$

Wave forces:

Horizontal wave force,uls,LWL

δ	δ_{11}	-2,78	[-]	*Goda coefficient*	$\delta_{11}= 0,93(B_m/L_{uls,low}-0,12)+0,36((h_{uls,low}-d_{uls,low})/h_{uls,low}-0,6)$
	δ_{22}	-0,55	[-]	*Goda coefficient*	$\delta_{22}= -0,36(B_m/L_{uls,low}-0,12)+0,93((h_{uls,low}-d_{uls,low})/h_{uls,low}-0,6)$
	δ_1	-55,67	[-]	*Goda coefficient*	$\delta_1=20\delta_{11}$ als $\delta_{11}<=0$ en $\delta_1 = 15\delta_{11}$ als $\delta_{11}>0$
	δ_2	-2,69	[-]	*Goda coefficient*	$\delta_2=4,9\delta_{22}$ als $\delta_{22}<=0$ en $\delta_2 = 3\delta_{22}$ als $\delta 22>0$
α	α_1	0,75	[-]	*Goda coefficient*	$\alpha_1=0,6+0,5((4\pi h/L_{uls})/(\sinh(4\pi h/L_{uls})))^2$
	α_2	0,03	[-]	*Goda coefficient*	$\alpha_2=\min\{((h_b-d)/3h_b)(H_{max}/d);(2d/H_{max})\}$
	α_3	0,65	[-]	*Goda coefficient*	$\alpha_3=1-h'/h[1-1/\cosh(2\pi h/L_{uls})]$
	α^*	0,03	[-]	*Goda coefficient*	$\alpha^*=\max\{\alpha_2, \alpha_I\}$
	α_{I0}	0,38	[-]	*Goda coefficient*	$\alpha_{I0}=H_{s,uls}/d_{uls}$ als $H_{s,uls}<=2d_{uls}$ en $\alpha_{I0}=2$ als $H_{s,uls}>2d_{uls}$
	α_{I1}	0,00	[-]	*Goda coefficient*	$\alpha_{I1}=\cos\delta_2/\cosh\delta_1$ als $\delta_2<=0$ en $1/(\cosh\delta_1{*}(\cosh\delta_2)^{1/2})$ als $\delta_2>0$
	α_I	0,00	[-]	*Goda coefficient*	$\alpha_I=\alpha_{I0}\,\alpha_{I1}$
pressure coeff.	θ	0,00	[rad]	*incedence angle of waves*	
	η^*	10,78	[m]	*elevation to which the wave pressure is exerted*	$\eta^*=0,75(1+\cos\beta)H_{max}$
	h_c^*	5,66	[m]	*minimum of η^* and hc*	$h_c^*=\min\{\eta^*,h_c\}$
	p1	108.427	[N/m²]	*pressure*	$p_1=0,5(1+\cos\beta)(\alpha_1+\alpha^*\cos^2\beta)\rho_{water}{*}H_D$
	p2	108.427	[N/m²]	*pressure*	$p_2=p_1/\cosh(2\pi h/L_{uls})$
	p3	70.735	[N/m²]	*pressure*	$p_3=\alpha_3 p_1$
	p4	51.475	[N/m²]	*pressure*	$p_4=p_1(1-h_c/\eta^*)$ als $\eta^*>h_c$ and 0 als $\eta^*<=h_c$
	p_u	555.635.667	[N/m²]	*pressure*	$p_u=0,5{*}(1+\cos\beta)\alpha_1\alpha_3\rho_{water}{*}H_D$
	P	455.607	[N/m¹]	*horizontal force on caisson per unit length caisson*	$P=0,5(p_1+p_3)h'_{uls}+0,5(p_1+p_4)h_c^*$
	M_P	1.144.220	[Nm/m¹]	*moment around heel of caisson resulting from horiz. force*	$M_P=(1/6){*}(2p_1+p_3)h'^2_{uls}+0,5(p_1+p_4)h'_{uls}h_c^*+(1/6)(p_1+2p_4)h_c^{*2}$

Bearing capacity:

Rubble mound

	M'_U	2515669	[Nm]	*moment around centre of caisson resulting from wave uplift pressure*	$M'_U=UB/6$
	V	6874319	[N]	*resulting vertical force of caissonon rubble mound foundation*	$V=W_{LWL}-U$
	B_e	9,70	[m]	*effective width of caisson*	$B_e=B-2e$
	e	5,69	[m]	*excentricity of the total force on the caisson*	$e=(M_P+M'_U)/V$
	t_{rm}	258030	[N/m]	*coefficient for the direction of the forces*	$t_{rm}=P/B_e$
	p_{rm}	708343	[N/m]	*coefficient for the direction of the forces*	$p_{rm}=V/B_e$
	$i_{c,rm}$	0,57	[-]	*coefficient for the direction of the forces*	$i_{c,rm}=1-t_{rm}/(c_{rm}+p_{rm}{*}\tan\varphi_{rm})$
	$i_{q,rm}$	0,32	[-]	*coefficient for the direction of the forces*	$i_{q,rm}=i_{c,rm}^2$
	$i_{\gamma,rm}$	0,18	[-]	*coefficient for the direction of the forces*	$i_{\gamma,rm}=i_{c,rm}^3$
	t<c?				
	$s_{c,rm}$		[-]	*shape coefficient*	
	$s_{q,rm}$		[-]	*shape coefficient*	$s_{q,rm}=1+\sin(\varphi_{rm} * B_e/L_c)$
	$s_{\gamma,rm}$	0,95	[-]	*shape coefficient*	$s_{\gamma,rm}=1-0,4{*}B_e/L_c$
	$N_{c,rm}$		[-]	*shape coefficient*	
	$N_{q,rm}$	64,20	[-]	*shape coefficient*	$N_{q,rm}=(1+\sin\varphi_{rm}){*}e^{\pi\tan\varphi rm}/(1-\sin\varphi_{rm})$
	$N_{\gamma,rm}$	79,54	[-]	*shape coefficient*	$N_{\gamma,rm}=1,5{*}(N_{q,rm}-1)\tan\varphi_{rm}$
	$P_{bear,rm}$	1094242	[N/m²]	*bearing capacity of the rubble mound*	$p_{bear,rm}=\ldots+\ldots+s_\gamma i_\gamma N_\gamma{*}0,5\rho_{rm,subm}B_e$
	p_c	708343	[N/m²]	*average ground pressure from caisson on effective section rubble mound*	$p_c=V/B_e$
	γ_{rm}	1,54	[-]	*safety factor of the rubble mound bearing capacity*	$\gamma_{rm}=P_{bear,rm}/P_c$

Bearing capacity:

Subsoil	V_{sub}	7.394.739	[N]	*resulting vertical force of caisson and rubble mound foundation*	$V_{sub}=V+\rho_{rm,subm}*h_{rm}*B_e$
	M_{sub}	8.138.411	[N/m]	*moment around centre of effective rubble mound width*	$M_{sub}=P*h_{rm}$
	e_{sub}	1,10	[m]	*excentricity of the total force on the rubble mound foundation*	$e_{sub}=M_{sub}/V_{sub}$
	B_{sub}	16,20	[m]	width of subsoil section	$B_{sub}=B_e+2h_{rm}*\tan 45^{\circ}$
	$B_{e,sub}$	14,00	[m]	effective width of subsoil section	$B_{e,sub}=B_{sub}-2e_{sub}$
	$p_{rm,sub}$	528.058	[N]	*average ground pressure from rubble mound foundation on effective section subsoil*	$p_{rm,sub}=V_{sub}/B_{e,sub}$
	t_{sub}	178.819	[N/m]	*coefficient for the direction of the forces*	$t_{sub}=P/B_{e,sub}$
	p_{sub}	528.058	[N/m]	*coefficient for the direction of the forces*	$p_{sub}=p_{rm,sub}$
	$i_{c,sub}$	0,56	[-]	*coefficient for the direction of the forces*	$i_{c,sub}=1-t_{sub}/(c_{sub}+p_{sub}*\tan\varphi_{sub})$
	$i_{q,sub}$	0,31	[-]	*coefficient for the direction of the forces*	$i_{q,sub}=i_{c,sub}^2$
	$i_{\gamma,sub}$	0,17	[-]	*coefficient for the direction of the forces*	$i_{\gamma,sub}=i_{c,sub}^3$
	$s_{c,sub}$		[-]	*shape coefficient*	$s_{c,sub}=$
	$s_{q,sub}$		[-]	*shape coefficient*	$s_{q,sub}=1+\sin(\varphi_{sub}*B_{e,sub}/L_c)$
	$s_{\gamma,sub}$	0,93	[-]	*shape coefficient*	$s_{\gamma,sub}=1-0,4*B_{e,sub}/L_c$
	$N_{q,sub}$	45,81	[-]	*shape coefficient*	$N_{q,sub}=(1+\sin\varphi_{sub})*e^{\pi\tan\varphi sub}/(1-\sin\varphi_{sub})$
	$N_{\gamma,sub}$	51,58	[-]	*shape coefficient*	$N_{\gamma,sub}=1,5*(N_{q,sub}-1)\tan\varphi_{sub}$
	$p_{bear,sub}$	640.979	[N/m^2]	*bearing capacity of the subsoil*	$p_{bear,sub}=...+...+s_{\gamma,sub}i_{\gamma,sub}N_{\gamma,sub}*0,5\rho_{sub,subm}B_{e,sub}$
	$\gamma_{bearcap,sub}$	1,21	[-]	*safety factor of the bearing capacity of the subsoil*	$\gamma_{bearcap,sub}=p_{bear,sub}/p_{rm,sub}$

Stability:

Sliding and overturning	B	21,09	[m]	*caisson width*	
	U	715.541	[N/m]	*total uplift pressurefrom wave forces*	$U=0,5p_UB$
	M_U	10.062.674	[Nm/m]	*moment around heel of caisson resulting from uplift pressure*	$M_U=(2/3)*UB$
	W	6.943.629	[N/m]	*weight of caisson in design water level per unit length caisson*	$W=[d_{uls}(\rho_{cs,emerged}-\rho_{water})+(h_{caisson}-d_{uls})(\rho_{cs,emerged})]B$
	W_{LWL}	7.589.860	[N/m]	*max weight of caisson in design water level per unit length caisson(LWL)*	$W_{LWL}=[d_{uls,lwl}(\rho_{cs,emerged}-\rho_{water})+(h_{caisson}-d_{uls,lwl})(\rho_{cs,emerged})]B$
	M_W	73.236.326	[Nm/m]	*moment around heel of caisson resulting from own weight of caisson*	$M_W=W*B/2$

SAFETY FACTORS

sliding	$\gamma_{sliding}$	1,49	[-]	*safety factor against sliding*	$\gamma_{sliding}=\mu(W-U)/P >=1,2$
overturning	$\gamma_{overturning}$	1,72	[-]	*safety factor against overturning*	$\gamma_{overturning}=(M_W-M_U)/M_P >=1,2$
bearing capacity rm	$\gamma_{bearingcap,rm}$	1,54	[-]	*safety factor bearing capacity rubble mound*	$\gamma_{bearingcap,rm}=P_{rm}/P_c >=1,2$
bearing capacity subsoil	$\gamma_{bearingcap,ss}$	1,21	[-]	*safety factor bearing capacity subsoil*	$\gamma_{bearingcap,ss*} = p_{bear,sub}/p_{rm,sub} >=1,2$

Appendix K:
Calculation of the concrete and steel dimensions

K.1 Forces on the cell elements

The caisson has been schematised into the following elements:

- Outer walls : plate clamped on three sides (ingeklemd)
- Inner walls : plate clamped on three sides (ingeklemd)
- Floor : plate clamped on four sides (ingeklemd)

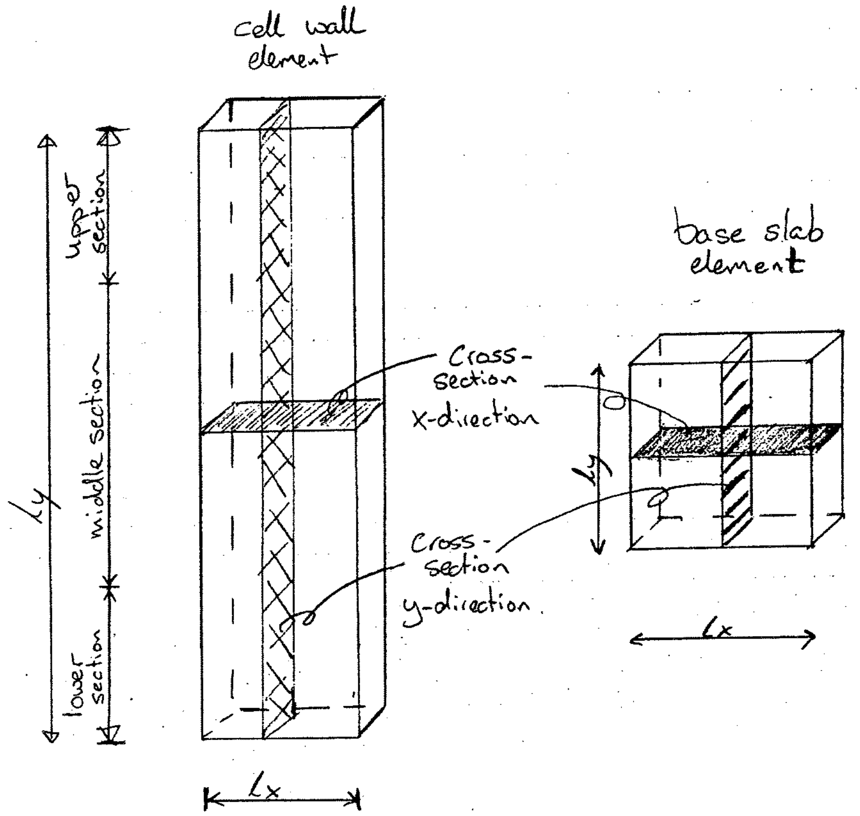


Figure 1. Caisson elements.

K.1.1 Forces on the outer wall

As the forces on the caisson are not constant of the entire shaft, the shaft will be devided into 3 sections:

1. Upper section
2. Middle section
3. Lower section

A schematised presentation of the horizontal (x-direction) and vertical (y-direction) forces on the caisson shaft are presented in the following figure. The forces used for the calculations of the outer wall can be read form this figure.

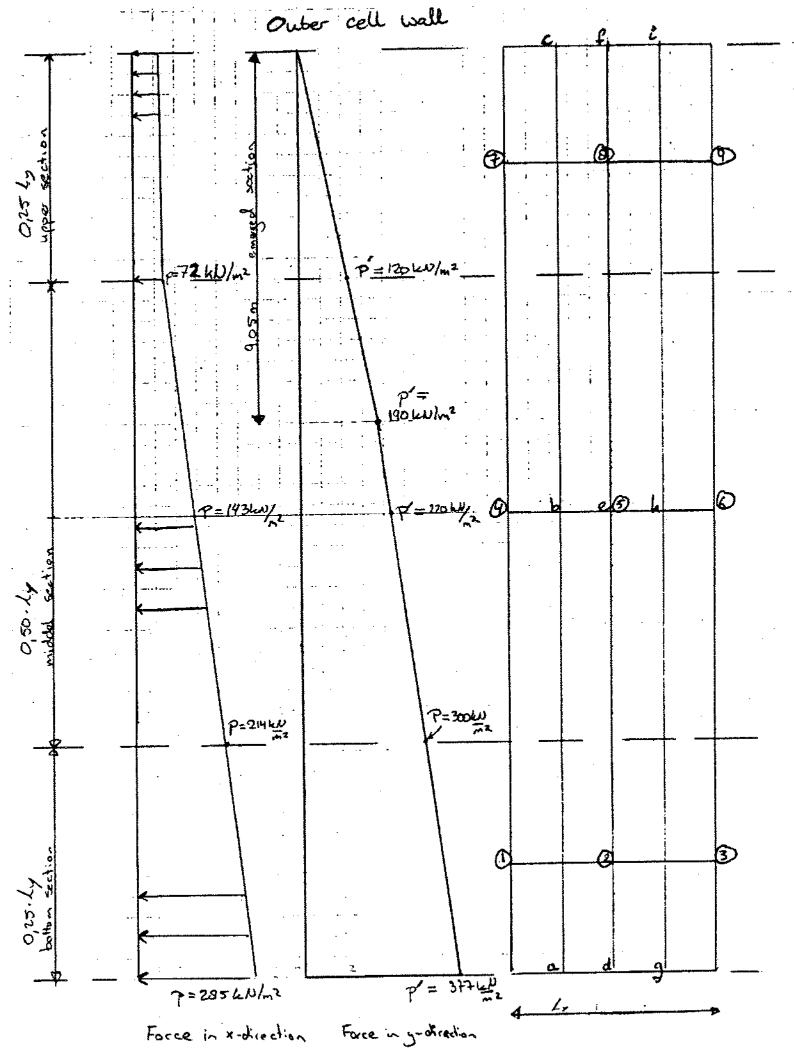


Figure 2. Pressure on the outer caisson wall.

Tension forces resulting from p_d in the outer wall due to vertical ground pressure:

Upper section	x-direction (7,8,9)	$N_{xx} = p\,[kN/m^2] \cdot w_{cell}$	= 431 kN/m
	y-direction (c,f,i)	$N_{yy} = p'[kN/m^2] \cdot w_{cell}$	= 135 kN/m
Middle section	x-direction (4,5,6)	$N_{xx} = p\,[kN/m^2] \cdot w_{cell}$	= 1250 kN/m
	y-direction (b,e,h)	$N_{yy} = p'[kN/m^2] \cdot w_{cell}$	= 405 kN/m
Lower section	x-direction (1,2,3)	$N_{xx} = p\,[kN/m^2] \cdot w_{cell}$	= 1425 kN/m
	y-direction (a,d,g	$N_{yy} = p'[kN/m^2] \cdot w_{cell}$	= 540 kN/m

K.1.2 Forces on the floor

As can be read from Figure 2 the vertical force on the floor due to the ground pressure is p_{floor}= 377 kN/m^2. This value is used to calculate the bending moments of the cell elements of the floor.

Tension in the floor cell elements due to horizontal ground pressure (x- and y-direction are identical):
($p_{floor} \cdot A_{cell}$) / contour cell = (377[kN/m^2] ·5[m]·5[m]) / (4·5[m])= 500 kN/m.

$N_{xx} = N_{yy} = 500$ kN/m

K.2 Calculation of the bending moments in the plates

After the construction has been schematised and the forces on the cell walls have been inventoried, the bending moments in the plates are calculated. This can either be done with the "GTB tables" or the document "Platten", lit.[...]. The relevant tables from these documents are presented in the end of this appendix.

GTB makes use of the following formulas:
$m_{xx} = 0.001[-]\cdot p_d[kN/m^2]\cdot l_x^2\ [m^2]\cdot f_{GTB,x}$
$m_{yy} = 0.001[-]\cdot p_d[kN/m^2]\cdot l_x^2\ [m^2]\cdot f_{GTB,y}$
with:
p_d = perpendicular force on the caisson, values from Figure 2
$l_{x,wall}$ = 5.00m
$l_{y,wall}$ = 22.50m
$l_{x,floorl}$ = 5.00m
$l_{y,floor}$ = 5.00m
$f_{GTB,x}$ and $f_{GTB,y}$ can be read from the GTB tables.

This leads to the following results[1]:

axis	section	bending moment in [kN]	tension force[2] in [kN]	
xx	upper	m_{xx} (7,9) = -152	N_{xx} (7,9)= 431	Decisive
		m_{xx} (8) = +76	N_{xx} (8)= 431	-
	middle	m_{xx} (4,6) = -482	N_{xx} (4,6) = 1250	Decisive
		m_{xx} (5) = +252	N_{xx} (5) = 1250	-
	lower	m_{xx} (1,3) = -442	N_{xx} (1,3) = 1425	Decisive
		m_{xx} (4) = +221	N_{xx} (4) = 1425	-

Table 1. Design forces on caisson shaft, x-direction.

axis	section	bending moment in [kN]	tension force in [kN]	
yy	upper	m_{yy}(c,i) = -27	N_{yy} (c,i)= 135	-
		m_{yy}(f) = -22	N_{yy} (f)= 135	-
	middle	m_{yy}(b,h) = +81	N_{yy} (b,h) = 405	-
		m_{yy} (e) = +81	N_{yy} (e) = 405	-
	lower	m_{yy} (a,g) = -107	N_{yy} (a,g) = 540	-
		m_{yy} (d) = -292	N_{yy} (d) = 540	Decisive

Table 2. Design forces on caisson shaft, y-direction.

Based upon the calculation method discussed above, the values of Table 2 are used as input tension forces and bending moments for the Excel program RCA.

[1] In order to verify these results, the same calculations have been made with "Platte". Platte makes use of the following formulas:
$K = p_d \cdot l_x \cdot l_y$ and M=K/m
where: p_d is the perpendicular force on the caisson, l_x and l_y represent the element dimensions, and m can be read from the Platte tables. The results of the bending moments are approximately 10% larger than those calculated according to the GTB.

[2] Tension force due to ground pressure.

K.3 Material characteristics

Concrete
Applied concrete: B45

Outer steel reinforcement
Applied steel: FeB 500 N/mm^2

Prestressed steel reinforcement
Applied prestressed steel: FeP 1860 N/mm^2

K.4 Output Excel program RCA

The design methodology applied in RCA is summarised in the diagram presented in the end of this section

The following pages contain the results of the Excel program RCA.

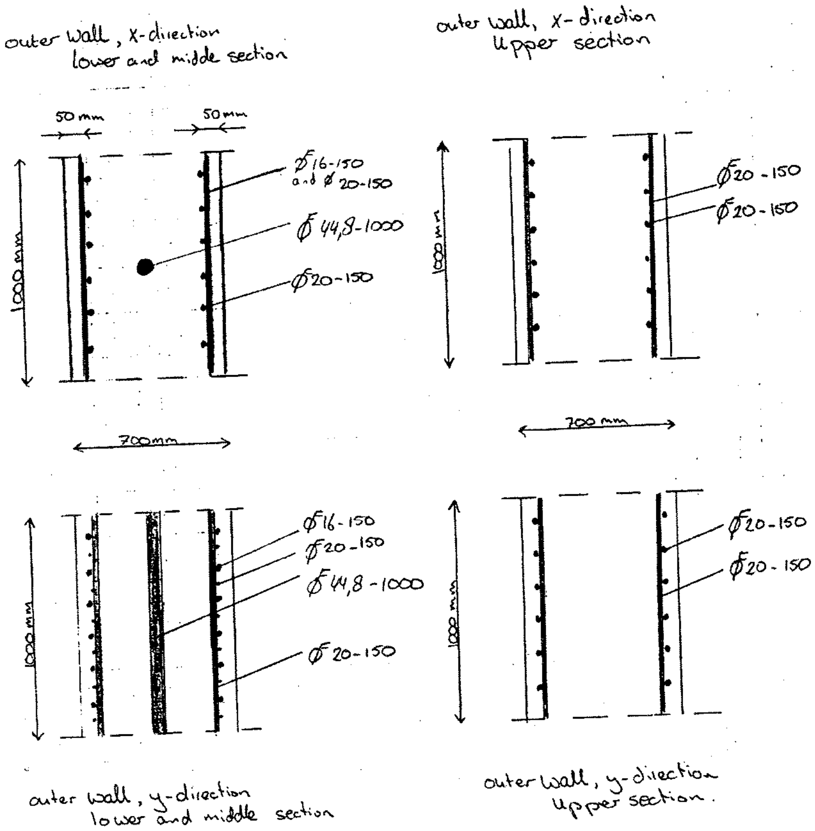
With the EXCEL program RCA the following dimensions have been determined for the caisson dimensions:

element	**construction height**	**concrete thickness (B45)**	**outer steel reinforcement (FeB 500)**	**prestressing steel reinforcement (FeP 1860)**
Outer cell wall, x-direction				
• lower section	5.47m	700mm	Ø20-150	16x Ø12.9-1000
• middle section	10.93m	700mm	Ø20-150	16x Ø12.9-1000
• upper section	5.47m	700mm	Ø20-150	-
Outer cell wall, y-direction				
• lower section	5.47m	700mm	Ø20-150 + Ø15-150	-
• middle section	10.93m	700mm	Ø20-150 + Ø15-150	-
• upper section	5.47m	700mm	Ø20-150	-
Inner cell wall, x direction				
• all sections	21.86m	500mm	Ø20-150	-
Inner cell wall				
• all sections	21.86m	500mm	Ø20-150	-
Floor, x-direction = y-direction	1.00m	1000mm	Ø25-150	-

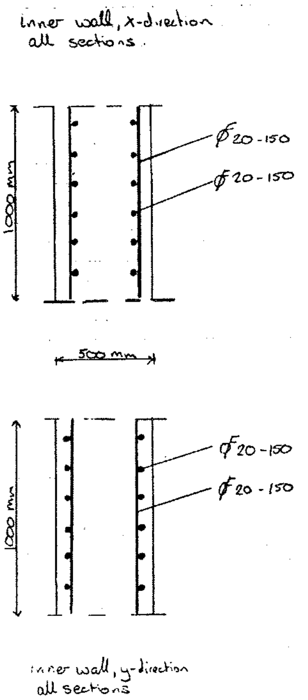
Table 3. Overview concrete and steel dimensions.

K.5 Technical drawings

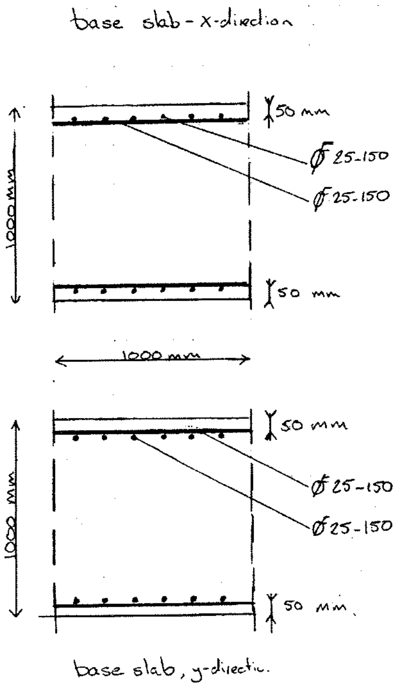
K.5.1 Outer wall



K.5.2 Inner wall



K.5.3 Base slab



(3a)

Controleberekening wapening (inclusief voorspanning) m.b.v. Moment en Normaalkracht Datum: 21-12-98

Project: caisson X-richting
Snede: wand
Belastingcombinatie: gronddruk snede 1,3 - Lower section

Invoer

Dimensies betondoorsnede: 21,700

Hoogte =	**700**	mm.
Breedte =	**1000**	mm.

Krachten N = - (druk); N = + (trek); M = + :

M_d =	**575**	kNm.	γ_M =	**1,300**	M_{rep} =	442,31	kNm.	
N_d =	**-500**	kN.	γ_N =	**1,000**	N_{rep} =	-500,00	kN.	(druk)

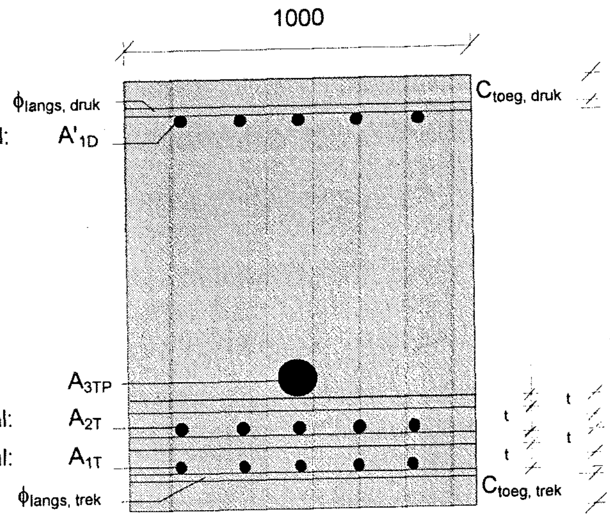
Mu = 1222,81 kNm. > M_d = 575,00 kNm. **Trekwapening accoord; controleer scheur**

Betongegevens:

B:	**45**		
f'_b =	27,0	N/mm^2	(= f'_{brep})
ε'_{bpl} =	**1,750**	‰	
ε'_{bu} =	**3,50**	‰	
E'_b =	15428,57143	N/mm^2	(incl. ϕ)
E'_b =	33500	N/mm^2	(excl. ϕ)
f_{bm} =	3,3	N/mm^2	

Staalgegevens:

FeB:	**500**	N/mm^2	(= f_{srep})
γ_f =	**1,15**		
f_s =	435	N/mm^2	
ε_{spl} =	**2,175**	‰	
f'_s =	435	N/mm^2	
ε_{su} =	3,25	%	
E_s =	199900	N/mm^2	



Voorspangegevens:

FeP:	**1860**	N/mm^2	(= f_{purep})
f_{pu} =	1691	N/mm^2	
ε_{ppl} =	8,032	‰	(bij 0,95 f_{pu})
ε_{pu} =	**3,5**	%	
E_s =	**200000**	N/mm^2	

Niet accoord

Wapening:

	Diam. (ϕ): (mm.)	H.o.h.: (mm.)	Diam. (ϕ): (mm.)	H.o.h.: (mm.)	Opp. (A): (mm^2)		
A_{1T} =	**20,00**	**150**	**0**	**0**	2094,40	A_{totaal} (trekzijde)	Rondstaal
A_{2T} =	**0**	**0**	**0**	**0**	0,00	A_{totaal} (trekzijde)	Rondstaal
A_{3TP} =	**44,80**	**1000**	**0**	**0**	1576	A_{totaal} (trekzijde)	Voorspann
A'_{1D} =	**20**	**150**	**0**	**0**	2094,40	A_{totaal} (drukzijde)	Rondstaal

* Met voorspanning (incl. A3TP):
Afstand a_{gem} vanaf meest getrokken vezel: 193,66 mm.
Totaal wapeningsperc. trekzijde (ω_{tot}): 0,725 % bij A_{tot} trekzijde: 3670,72 mm^2

* Zonder voorspanning (excl. A3TP):
Afstand a_{gem} vanaf meest getrokken vezel: 76,00 mm.
Totaal wapeningsperc. trekzijde (ω_{tot}): 0,336 % bij A_{tot} trekzijde: 2094,40 mm^2

K.6

36

$C_{toeg,\ trek}$ =	**50** mm.	$\phi_{langs,\ trek}$ =	**16** mm.	t =	**43** mm.	(trekzijde)
$C_{toeg,\ druk}$ =	**50** mm.	$\phi_{langs,\ druk}$ =	**16** mm.			(drukzijde)

K.7

Dekking + gemiddelde zwaartepuntsafstand van de betreffende laag: Datum:

a_{1T} =	76,00	mm.	(Afstand van uiterste getrokken vezel tot 1e laag trekwapening)
a_{2T} =	0,00	mm.	(Afstand van uiterste getrokken vezel tot 2e laag trekwapening)
a_{3TP} =	**350,00**	mm.	(Afstand van uiterste getrokken vezel tot 1e laag voorspanwapening
a'_{1D} =	76,00	mm.	(Afstand van uiterste gedrukte vezel tot 1e laag drukwapening)

Berekening X en epsilon bij gegeven M_{rep} en N_{rep}:

								Controle t
X =	210,6348	mm.	ΣH =	0,00000	kN.			**X=**
ε'_b =	0,587333	‰	M_{rep} =	442,308	kNm. =	442,3077	kNm.	**ε'_b =**
ε_b =	1,364543	‰						**ε_b =**
								ΣH =
								M_{rep} =

Drukwapening in 1e laag:

ε'_b =	0,375415146	‰	Op: 76,00 mm. vanaf meest gedrukte vezel
σ'_{s1D} =	75,046	N/mm^2	
N'_{s1D} =	157,175	kN.	
M'_{s1D} =	86,132	kNm.	

Trekwapening in 1e laag:

ε_b =	1,152625451	‰	Op: 76,00 mm. vanaf meest getrokken vezel
σ_{s1T} =	230,410	N/mm^2	
N_{s1T} =	482,5693352	kN.	
M_{s1T} =	0	kNm.	

Trekwapening in 2e laag:

ε_b =	-----------	‰	Op: 0,00 mm. vanaf meest getrokken vezel
σ_{s2T} =	0,000	N/mm^2	Geen trekwapening in 2e laag aanwezig
N_{s2T} =	0,000	kN.	
M_{s2T} =	0,000	kNm.	

Voorspanning in 3e laag (= 1e laag voorspanning):

ε_b =	0,388605152	‰	Op: 350,00 mm. vanaf meest getrokken vezel
σ_{s3TP} =	81,812	N/mm^2	
N_{s3TP} =	128,962	kN.	
M_{s3TP} =	35,336	kNm.	

Betondrukkracht:

ε'_b =	0,587333	‰	Op: 0,00 mm. vanaf meest gedrukte vezel
σ'_b =	9,062	N/mm^2	
N'_b =	954,3561243	kN.	Op: 70,21161 mm. vanaf meest gedrukte vezel
M_b =	528,511	kNm.	

Normaalkracht:

N_g =	- 500	kN.	Op: midden betondoorsnede
M_N =	137,000	kNm.	

$EI_{gebruik}$ =	158624,5	kNm^2 =	0,166 * $EI_{ongescheurd}$
M_{rep} =	442,3077	kNm.	
κ =	0,002788	‰	
$EI_{ongescheurd}$ =	957542	kNm^2	

K 8

3d

Controle op scheurvorming (NEN 6720, art. 8.7) Datum:

σ_b = 4,70 N/mm^2 > fbm = 3,3 N/mm^2 dus volledig ontwikkeld scheurpatroon

** Volledig ontwikkeld scheurpatroon (NEN 6720, art. 8.7.2):*

a.

Milieukl.:	**4**	Met voorspanstaal? ja/nee:	**nee**	
k_1 =	2500			
ξ =	**1**			
σ_s =	230,410	N/mm^2		
$\Delta\sigma_s$ =	**0,000**	N/mm^2		
$\sigma_s + \Delta\sigma_s$ =	230,410	N/mm2 <	435 N/mm^2	Accoord
C/C_{min} =	**2,2**	wordt:	2	
$\phi_{gem.}$ =	24,690	mm.		
ϕ_{km} =	21,700	mm. <	24,690 mm.	Niet accoord

b.

Milieukl.:	4	(Zonder voorspanstaal)		
k_2 =	500			
m_1 =	1,75			
s =	162,89	mm. >	150 mm.	Accoord

S zelf bepalen bij toepassing van staafbundels!

** Onvolledig ontwikkeld scheurpatroon (NEN 6720, art. 8.7.3):*

Milieukl.:	**4**	Met voorspanstaal? ja/nee:	**nee**	
k_3 =	40000			
ξ =	**1**			
σ_s =	230,410	N/mm^2		
σ_{sr} =	**0,000**	N/mm^2		
$\sigma_s + \sigma_{sr}$ =	230,410	N/mm2 <	435 N/mm^2	Accoord
C/C_{min} =	**1,666**	wordt:	1,666	
$\phi_{gem.}$ =	24,690	mm.		
ϕ_{km} =	56,487	mm. >	24,690 mm.	Accoord
ϕ_{km} <	50	en ϕ_{km} >	10,85 mm.	Niet accoord
	14,00	128		

Berekening Xu, epsilon, Mu (breukmoment) en EI bij gegeven N_{rep}:

X_u =	132,6862	mm.			
ΣH =	0,0000	kN.			
M_u =	1222,81	kNm. >	M_d = 575,00	kNm.	Accoord
ε'_b =	3,5	‰			
ε'_{1D} =	1,495	%o <	2,175	%o	
ε_b =	14,965	‰			
ε_{1T} =	12,960	%o >	2,175	%o	
ε_{2T} =	-----------				
ε_{1TP} =	5,732	%o <	8,032	%o	
N'_b =	2686,90	kN.			
N_{s1T} =	910,61	kN.			
N_{s2T} =	0,00	kN.			
N_{s1TP} =	1902,31	kN.			
N'_{s1D} =	626,024	kN.			

EI_{breuk} =	46356,99	kNm^2 =	0,048 * $EI_{ongescheurd}$
M_u =	1222,806	kNm.	
κ =	0,026378	‰	
$EI_{ongescheurd}$ =	957541,7	kNm^2	

K9

32

Controle hoogte betondrukzone (NEN 6720, art. 8.1.3) Datum:

N'_d =	1890,00	kN. >	500,00	kN.	(druk)	Accoord
β =	0					
$k_{x\ max}$ =	0,53488					
x_u <	271	mm. >	133	mm.		Accoord

Minimum wapeningspercentage hoofdwapening (NEN 6720, art. 9.9.2.1)

Algemeen:

f_s =	435	N/mm^2
f_{bm} =	3,3	N/mm^2

Bij druk:

$\omega_{0\ min}$ =	0,21	%	Exclusief normaalkracht				
η =	9,857						
$M_d = M_r$ =	419,898	kNm.					
N_d =	-365,129	kN.	drukkracht				
$\omega_{0\ min}$ =	0,18	%	Inclusief normaalkracht	<	0,725	%	Accoord
N_{sT} =	533,886	kN.	trekkracht				
N'_b =	-899,015	kN.	drukkracht				

Bij trek

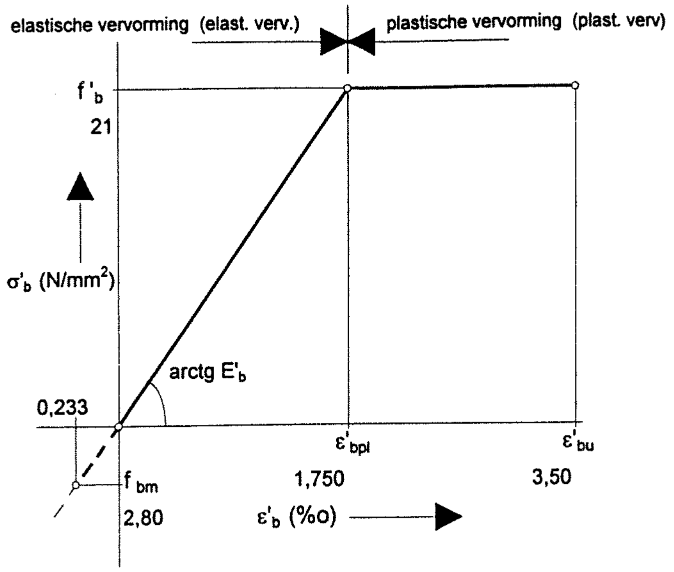
$\omega_{0\ min}$ =	0,21	%	Exclusief normaalkracht				
η =	9,857						
$M_d = M_r$ =	342,549	kNm.					
N_d =	297,868	kN.	trekkracht				
$\omega_{0\ min}$ =	0,24	%	Inclusief normaalkracht	<	0,725	%	Accoord
N_{sT} =	715,885	kN.	trekkracht				
N'_b =	-418,017	kN.	drukkracht				

Maximum wapeningspercentage hoofdwapening (GTB tabel 11.2.b)

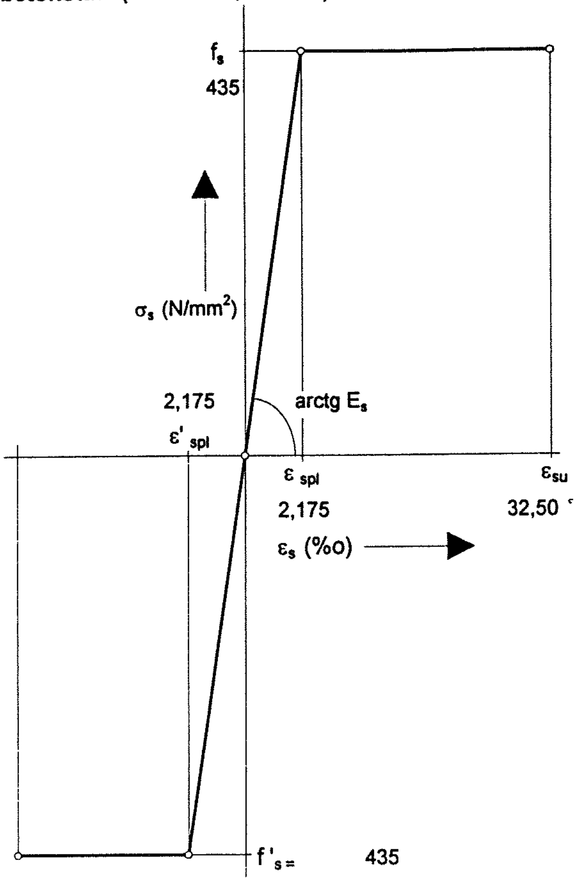
d_{gem} =	506	mm.			
$\omega_{0\ max}$ =	2,491	%			
$\omega_{0toegepast}$ =	0,725	% <	2,491	%	Accoord

K 10

Spanning-rek-diagrammen van beton (NEN 6720, art. 6.1) en betonstaal (NEN 6720, art. 6.2)

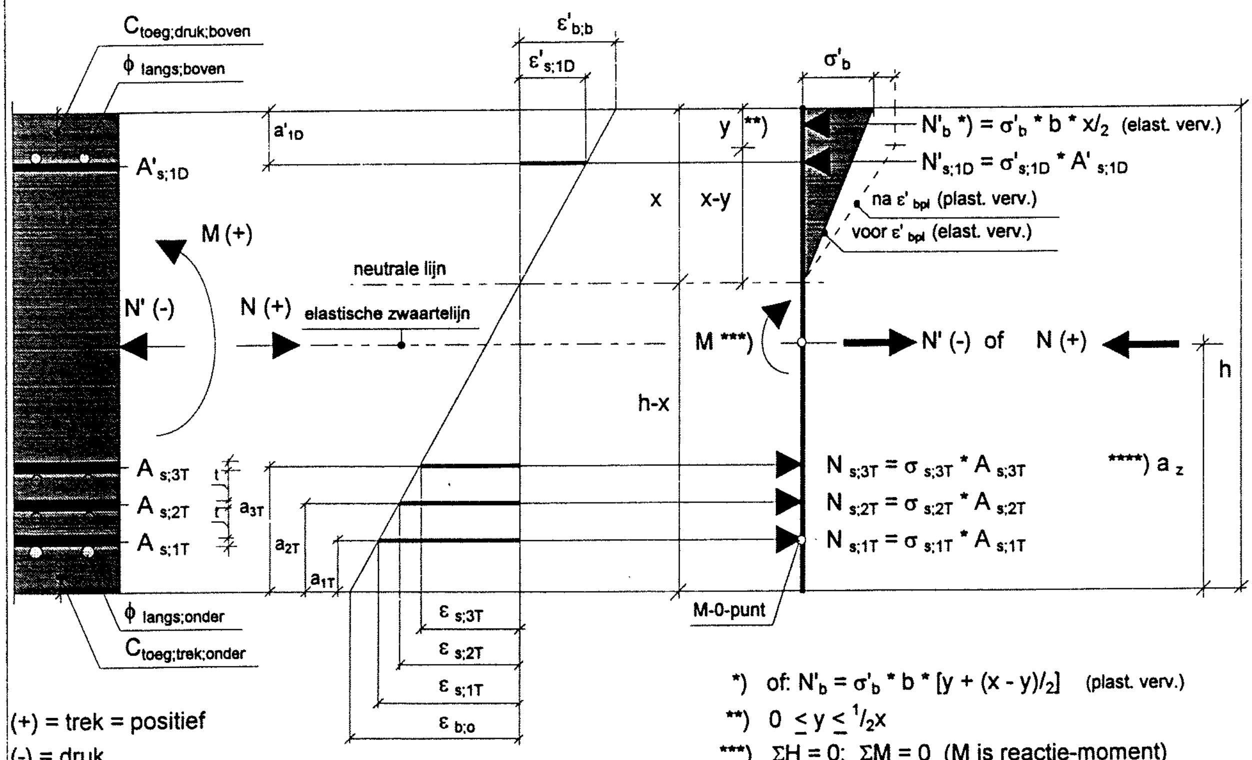


Toegepast σ – ε – diagram van beton B - 35



Toegepast σ – ε – diagram van betonstaal FeB - 500

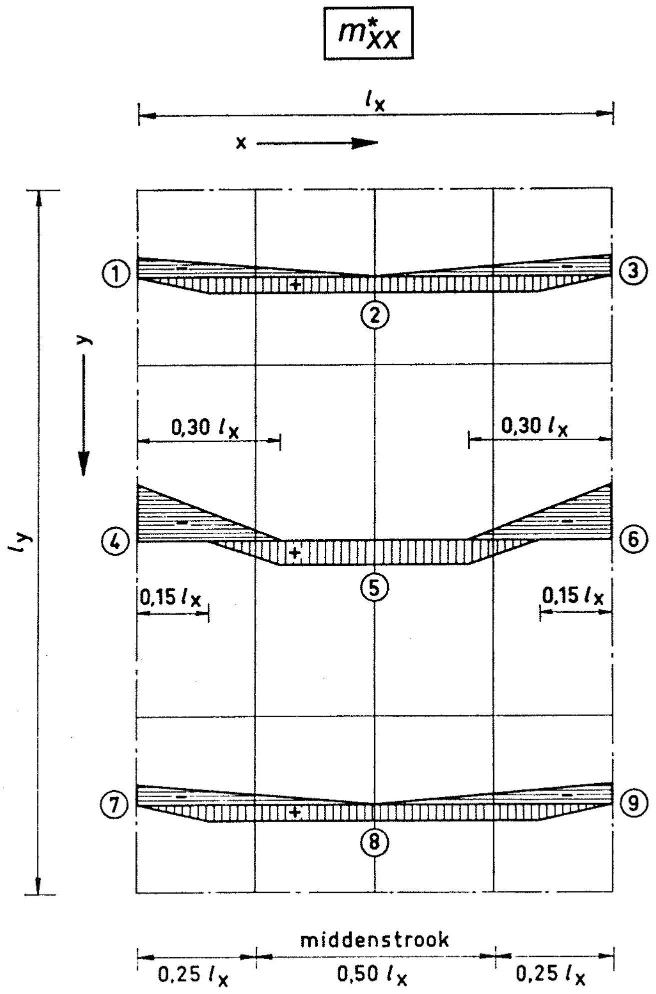
Principe-notatie van grootheden, rekken, spanningen en krachten (*doorsnede gedeeltelijk onder druk*)

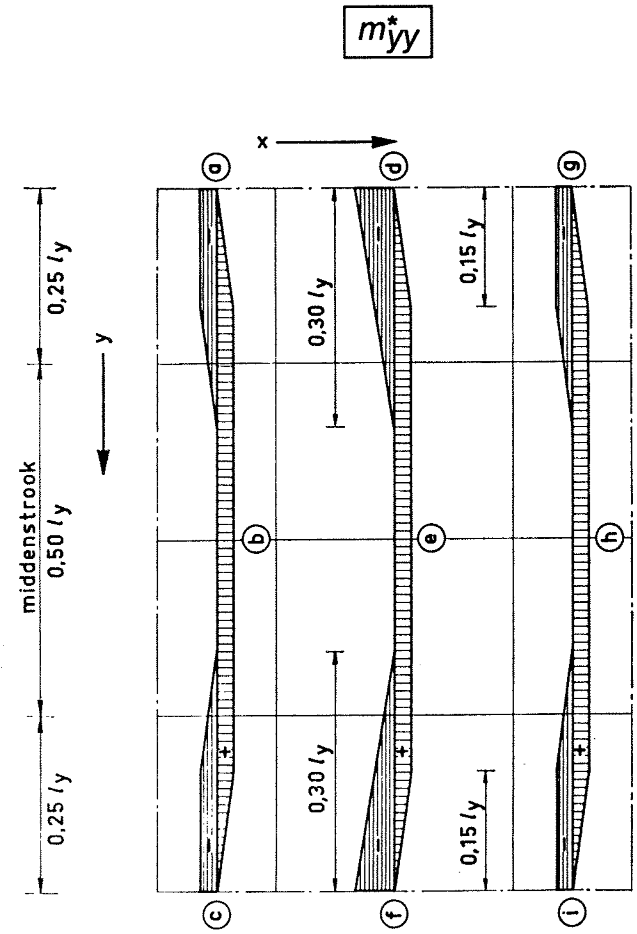


K.11

Wapeningsmomenten per lengte onder gelijkmatig verdeelde belasting

geschematiseerd verloop van wapeningsmomenten





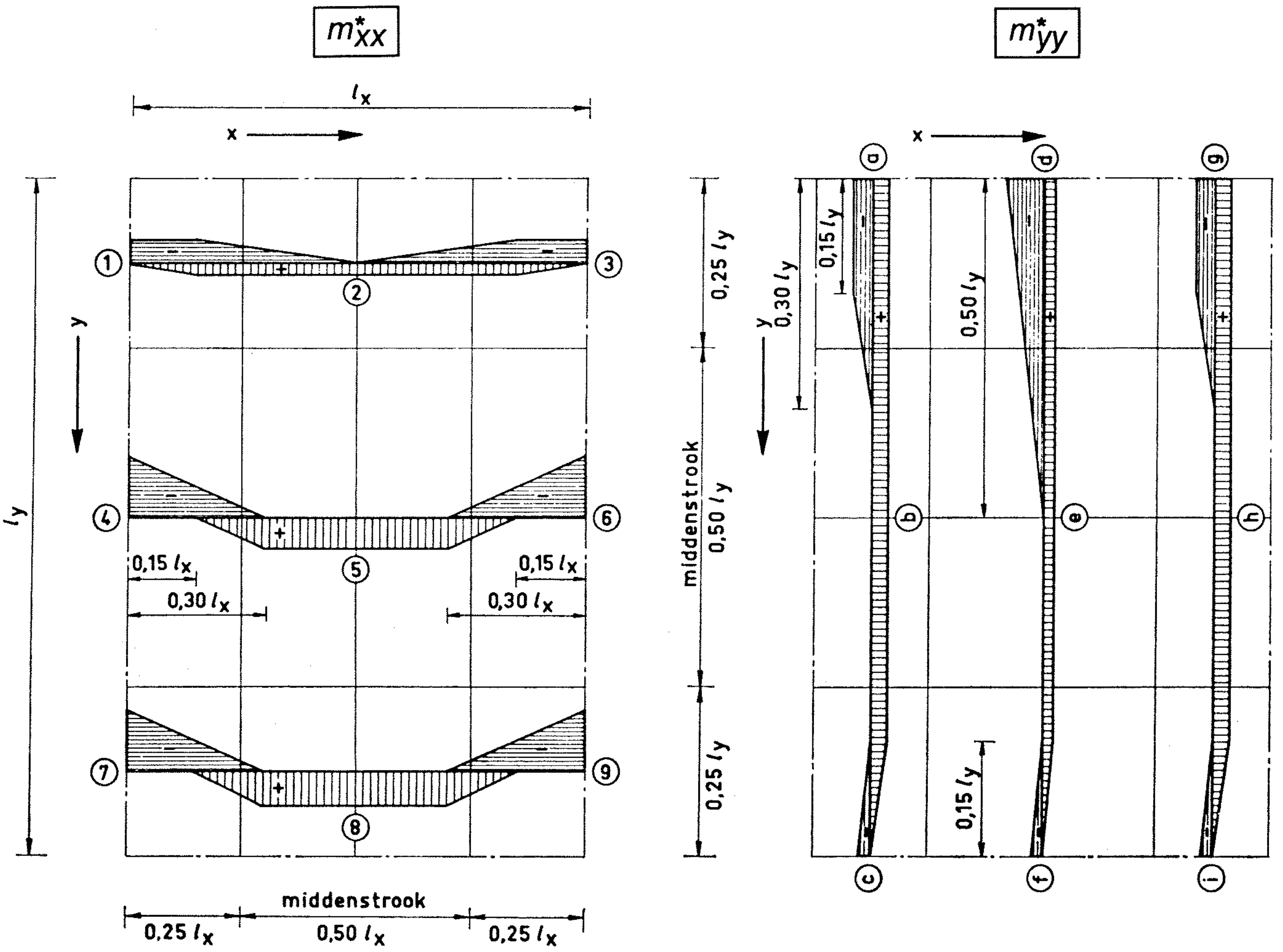
coëfficiënten voor wapeningsmomenten																		
	$m^*_{xx} = 0{,}001 p_d l_x^2 \times$									$m^*_{yy} = 0{,}001 p_d l_x^2 \times$								
l_y/l_x	1	2	3	4	5	6	7	8	9	a	b	c	d	e	f	g	h	i
1,0	−14	+11	−14	−44	+17	−44	−14	+11	−14	−14	+11	−14	−44	+17	−44	−14	+11	−14
1,2	−19	+13	−19	−56	+24	−56	−19	+13	−19	−15	+12	−15	−47	+17	−47	−15	+12	−15
1,4	−23	+15	−23	−65	+29	−65	−23	+15	−23	−15	+12	−15	−47	+16	−47	−15	+12	−15
1,6	−27	+17	−27	−71	+32	−71	−27	+17	−27	−15	+13	−15	−47	+15	−47	−15	+13	−15
1,8	−31	+18	−31	−75	+35	−75	−31	+18	−31	−15	+13	−15	−47	+15	−47	−15	+13	−15
2,0	−34	+19	−34	−78	+37	−78	−34	+19	−34	−15	+13	−15	−46	+15	−46	−15	+13	−15
2,5	−41	+20	−41	−81	+40	−81	−41	+20	−41	−15	+13	−15	−45	+15	−45	−15	+13	−15
3,0	−47	+23	−47	−83	+41	−83	−47	+23	−47	−15	+14	−15	−44	+15	−44	−15	+14	−15

Voor $l_y/l_x = 1$ mag voor het momentenverloop in y-richting ook het momentenverloop in x-richting worden aangehouden.

1/12

Wapeningsmomenten per lengte onder gelijkmatig verdeelde belasting

geschematiseerd verloop van wapeningsmomenten



	coëfficiënten voor wapeningsmomenten																	
	$m^*_{xx} = 0{,}001 p_d l_x^2 \times$									$m^*_{yy} = 0{,}001 p_d l_x^2 \times$								
l_y/l_x	1	2	3	4	5	6	7	8	9	a	b	c	d	e	f	g	h	i
1,0	−15	+13	−15	−63	+30	−63	−83	+40	−83	±15	+15	−3	−48	+14	−2	±15	+15	−3
1,2	−19	+14	−19	−69	+33	−69	−84	+42	−84	±15	+15	−2	−48	+15	−2	±15	+15	−2
1,4	−23	+15	−23	−74	+35	−74	−84	+42	−84	±15	+15	−2	−47	+15	−2	±15	+15	−2
1,6	−27	+17	−27	−77	+37	−77	−84	+42	−84	±15	+15	−2	−47	+15	−2	±15	+15	−2
1,8	−31	+18	−31	−79	+39	−79	−84	+42	−84	±15	+15	−3	−47	+15	−2	±15	+15	−3
2,0	−34	+19	−34	−80	+40	−80	−84	+42	−84	±15	+15	−4	−46	+15	−3	±15	+15	−4
2,5	−41	+20	−41	−82	+41	−82	−84	+42	−84	±15	+15	−5	−45	+15	−4	±15	+15	−5
3,0	−47	+23	−47	−83	+42	−83	−84	+42	−84	±15	+15	−8	−44	+15	−6	±15	+15	−8

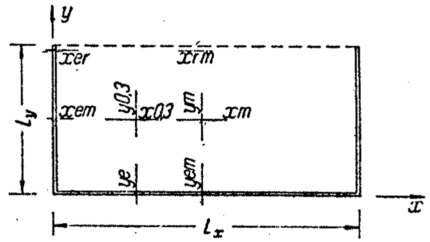
K.13

Pl Nr. III/1/a

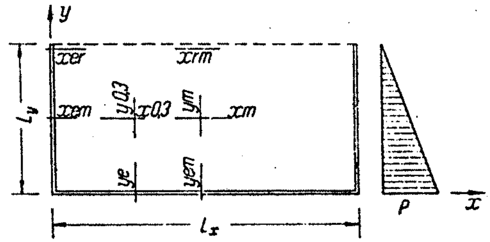
Gleichlast p

$K = p \cdot L_x \cdot L_y$ $\qquad M = \frac{K}{m}$ $\qquad w = \frac{K \cdot L_x^2}{k_w \cdot N}$

$\frac{L_y}{L_x}$	m_{xer}	m_{xem}	m_{xrm}	m_{xm}	$m_{x0,3}$	m_{yem}	m_{ye}	m_{ym}	max m_y	$\frac{y}{L_y}$	$m_{y0,3}$	max $m_{y0,3}$	$\frac{y}{L_y}$	k_w
0,25	−8,60	−27,2	77,0	225	174	−9,64	−10,6	−64,3	415	0,88	−84,?	355	0,88	690
0,3	−7,57	−23,0	46,4	126	120	−9,03	−10,7	−88,9	205	0,80	− 116	205	0,80	490
0,4	−6,77	−18,1	26,7	63,6	80,0	−9,02	−10,8	− 670	108	0,75	− 770	14	0,75	330
0,5	−6,86	−15,8	21,3	45,6	65,8	−9,86	−12,0	175	83,4	0,70	235	119	0,70	280
0,6	−7,42	−14,6	19,8	38,6	59,9	−11,1	−13,7	105	80,0	0,65	140	111	0,65	270
0,7	−8,24	−14,3	19,7	35,6	57,1	−12,6	−15,7	91,2	83,5	0,60	119	114	0,60	280
0,8	−9,27	−14,3	20,9	34,2	56,5	−14,2	−17,9	91,7	90,9	0,55	122	119	0,55	300
0,9	−10,4	−14,5	22,4	34,0	57,0	−15,9	−20,2	99,5	99,5	0,50	135	135	0,50	330
1,0	−11,6	−15,0	24,3	34,3	58,2	−17,6	−22,8	113	109	0,45	155	145	0,45	350
1,1	−12,8	−15,6	26,4	34,9	60,0	−19,3	−25,6	130	119	0,40	180	160	0,40	380
1,2	−14,1	−16,2	28,6	35,8	62,2	−21,1	−28,4	155	130	0,35	220	175	0,35	410
1,3	−15,3	−17,0	31,0	37,0	64,9	−22,9	−31,3	190	140	0,32	270	190	0,32	450
1,4	−16,6	−17,9	33,4	38,2	67,8	−24,5	−34,5	235	150	0,30	345	205	0,30	490
1,5	−17,8	−18,8	35,8	39,8	71,0	−26,3	−38,2	295	160	0,28	435	215	0,28	530
$\frac{x}{L_x}$	0; 1,0	0; 1,0	0,5	0,5	0,3; 0,7	0,5	0,3; 0,7	0,5	0,5		0,3; 0,7	0,3; 0,7		0,5
$\frac{y}{L_y}$	1,0	0,5	1,0	0,5	0,5	0	0	0,5	veränderlich		0,5	veränderlich		1,0



max $M_{xr} = M_{xrm}$



Platte Nr. III/1/b

Dreiecklast $p\,\Delta$

$K = \frac{p \cdot L_x \cdot L_y}{2}$ $\qquad M = \frac{K}{m}$ $\qquad w = \frac{K \cdot L_x^2}{k_w \cdot N}$

max $M_{xr} = M_{xrm}$

$\frac{L_y}{L_x}$	m_{xer}	m_{xem}	min m_{xe}	$\frac{y}{L_y}$	m_{xrm}	m_{xm}	max m_x	$\frac{y}{L_y}$	$m_{x0,3}$	m_{yem}	m_{ye}	m_{ym}	max m_y	$\frac{y}{L_y}$	min m_y	$\frac{y}{L_y}$	$m_{y0,3}$	max $m_{y0,3}$	$\frac{y}{L_y}$	k_w
0,25	−17,7	−36,5			140	430	140	1,00	300	−13,8	−15,5	−485	310	0,75	—	—	∞	300	0,75	1350
0,30	−15,7	−30,7			85,4	230	85,4	1,00	200	−12,6	−14,1	695	170	0,70	—	—	385	190	0,70	900
0,40	−14,3	−23,9	m_{xer}		48,8	109	48,8	1,00	125	−11,7	−14,0	114	91,0	0,65	—	—	123	110	0,63	610
0,50	−15,0	−20,3			38,9	74,2	38,9	1,00	96,1	−11,7	−14,3	70,8	69,0	0,60	—	—	84,4	83,4	0,56	530
0,60	−17,0	−18,3			36,6	59,2	36,6	1,00	82,0	−12,0	−15,0	59,1	59,1	0,50	—	—	73,3	73,3	0,50	520
0,70	−20,2	−17,2	−16,8	0,60	37,9	51,3	37,9	1,00	74,6	−12,5	−15,8	56,5	56,4	0,47	—	—	72,0	70,5	0,46	560
0,80	−24,6	−16,7	−16,6	0,55	41,5	46,7	40,1	0,80	70,0	−13,0	−16,6	58,6	54,0	0,45	—	—	76,4	70,5	0,42	630
0,90	−30,2	−16,5	−16,5	0,50	47,0	43,9	41,0	0,70	67,5	−13,5	−17,3	63,7	56,9	0,40	−2540	0,90	85,5	72,5	0,38	730
1,00	−37,1	−16,6	−16,5	0,47	54,1	42,2	41,0	0,60	66,5	−14,2	−18,5	72,0	58,8	0,35	− 875	0,90	98,3	75,8	0,35	850
1,10	−45,1	−16,9	−16,8	0,45	63,1	41,3	41,0	0,55	66,7	−14,9	−19,6	83,5	62,5	0,32	− 610	0,90	118	80,8	0,32	990
1,20	−54,1	−17,3	−17,1	0,43	73,8	40,9	40,9	0,50	67,5	−15,6	−20,4	99,0	66,6	0,31	− 525	0,90	140	85,0	0,30	1150
1,30	−64,0	−17,8	−17,4	0,40	86,1	41,1	40,6	0,47	69,0	−16,3	−21,4	119	71,4	0,30	− 495	0,90	170	91,7	0,28	1350
1,40	−74,6	−18,4	−17,7	0,38	100	41,6	41,2	0,45	71,1	−17,0	−22,3	145	76,2	0,30	− 495	0,90	205	100	0,26	1600
1,50	−85,7	−19,2	−18,1	0,36	115	42,4	41,7	0,42	73,6	−17,7	−23,6	180	80,5	0,28	− 515	0,90	265	111	0,25	1850
$\frac{x}{L_x}$	0; 1,0	0; 1,0	0; 1,0		0,5	0,5	0,5		0,3; 0,7	0,5	0,3; 0,7	0,5	0,5		0,5		0,3; 0,7	0,3; 0,7		0,5
$\frac{y}{L_y}$	1,0	0,5	veränderlich		1,0	0,5	veränderlich		0,5	0	0	0,5	veränderlich		veränderlich		0,5	veränderlich		1,0

Appendix L:

Caisson Stability

Stability
The design of large concrete elements which are transported floating must ensure the elements are stable during transportation and placement [litt f9].
Forces from tugs, wave motions, wind and other forces can induce unstable elements to capsize. Therefore elements must be designed in such a manner that when external forces cause the element to tilt, a righting moment is induced which forces the element upright.
In Figure 1 the center points which are of importance for stability calculations are illustrated.

B is the center of buoyancy, the center point of the uplifting force when the element is in position of rest. This is when the symmetric axis of the element is vertical. *B* is the center of gravity of the displaced water. With a box-shaped caisson, *B* is halfway between the water-level and the caisson bottom.
G is the center of gravity of the caisson. If ballast or water is applied in the caisson to make it sink deeper, this has to be included in the calculation of *G*.
M is the metacenter, this is the point where the uplifting force intersects the symmetric axis, z-axis, when the element is tilted.

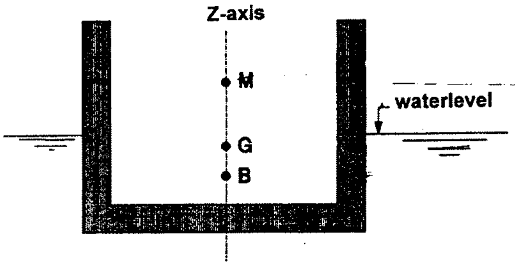


Figure 1. Center points of a floating caisson.

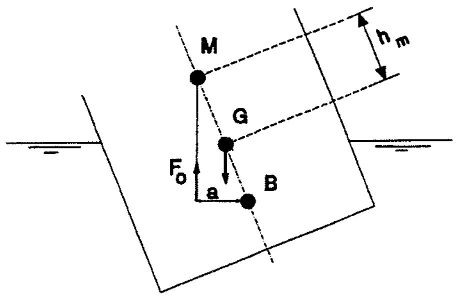


Figure 2. Metacentre.

For static stability *M* must be above *G*: line *GM*, also referred to as metacenter height, h_m, must be positive.
In Figure 3 shows a caisson which tipped at an angle. The element with a width of d*x* which is forced under water resulting from the angle φ (shaded in the figure), has an uplift force of d*F* with $dF = \varphi \cdot x \cdot y \cdot dx \cdot \rho_{water} \cdot g$ for an angle smaller than 10°. To point O this results in an uplifting moment of $dM = x \cdot dF$. For the entire caisson width this results in a righting moment of:

$$M = \int_{-\frac{1}{2}B}^{+\frac{1}{2}B} \varphi \cdot x^2 \cdot y \cdot dx \cdot \rho \cdot g = \varphi \cdot \rho \cdot g \cdot I \qquad \text{[N/m]}$$

Here *I* is the moment of inertia of the underwater surface area of the caisson relative to the y-axis.

$$I = \frac{1}{12} B \cdot Y^3 \qquad [\text{m}^4]$$

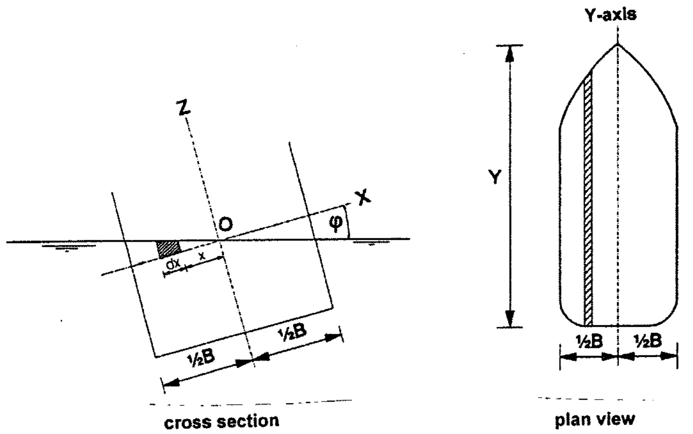


Figure 3. Tilted element.

In the upright position of rest the center-point of the uplifting force F_O is in B. As a result of the angle of tipping φ, the line of action of F_O translates over a distance a:

$$a = \frac{M}{F_O} = \frac{\varphi \cdot \rho \cdot g \cdot I}{\rho \cdot g \cdot V} = \frac{\varphi \cdot I}{V} \qquad \text{[m]}$$

V is the volume of the displaced water which is equal to the submerged part of the caisson.
The distance between the center of buoyancy B and metacenter M is:

$$BM = \frac{a}{\varphi} = \frac{I}{V} \qquad \text{[m]}$$

When M is above G then a righting moment is induced:

$$F_O \cdot h_m \cdot \varphi = \rho \cdot g \cdot V \cdot h_m \cdot \varphi \qquad \text{[N/m]}$$

Usually h_m>0.50 m is maintained as a requirement for stability. If M is below G, then the caisson will capsize.

If the caisson is unstable, the design must be adapted or extra provisions must be installed.
Changes of the design:

- widening of caisson, resulting in a larger value for I;
- increasing the weight of the floor results in the lowering of G and increase of B as the caisson sinks deeper, application of this measure also results in an increase of V leading to a lower value of BM, however this effect is smaller than the other effects;

Extra provisions:

- placement of extra ballast on the caisson floor during transportation;
- placement of stability pontoons, thus increasing I (Figure 4);

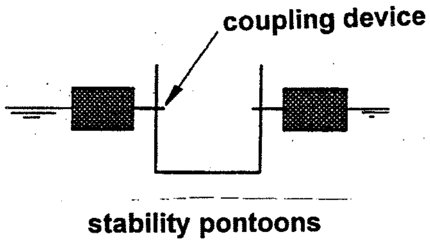


Figure 4. Extra provisions for stability.

Stability during placement

The floating caissons usually acquire stability from their large value for I.

Effect of water ballast on stability

The use of water as ballast material has the advantage that it can easily be pumped in and out of the caisson. However it has a negative effect on the stability (Figure 5).

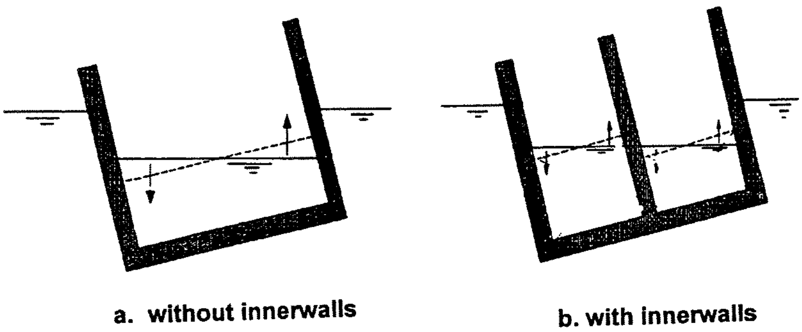


Figure 5. Effect of waterballast.

When the caisson tilts to the left, the depth of the ballast water increases at that side whilst on the right side the depth of the ballast water decreases. This results in a moment in the direction the caisson is tilted and works against the righting moment.
Application of innerwalls reduces the moment caused by the ballast water.

The influence of the ballast water on the stability can be calculated in the moment of inertia:

$$I = I_u - \sum I_i$$

with:
I_u = moment of inertia of the watercrossed surface area;
I_i = moment of inertia of the ballast-water surface area as opposed to the gravity line of the compartment;

The larger the number of compartments, the more the increase in I and therefore stability.

The innerwalls are not only effective for stability in the width direction of the caisson, also in longitudinal direction cells increase the stability.

Other advantages of the innerwalls:

- smaller spans of the outer walls and floor resulting in smaller transverse forces and bending moments;
- trimming of the caisson during transport and positioning can be done by filling the individual compartments with water;

The stability problems mentioned above can be prevented by using inert ballast material instead of water ballast.

Besides the static stability discussed above, also the dynamic stability of the caisson must be verified in the case the transport and placement of the caisson takes place in heavy seas. To prevent the caisson from opslingeren, the natural frequency of the caisson must be significantly higher than that of the waves or swell.

The natural frequency of a floating caisson disregarding the hydrodynamic mass, ballast water, is:

$$T_0 = \frac{2\pi \cdot j}{\sqrt{h_m \cdot g}}$$

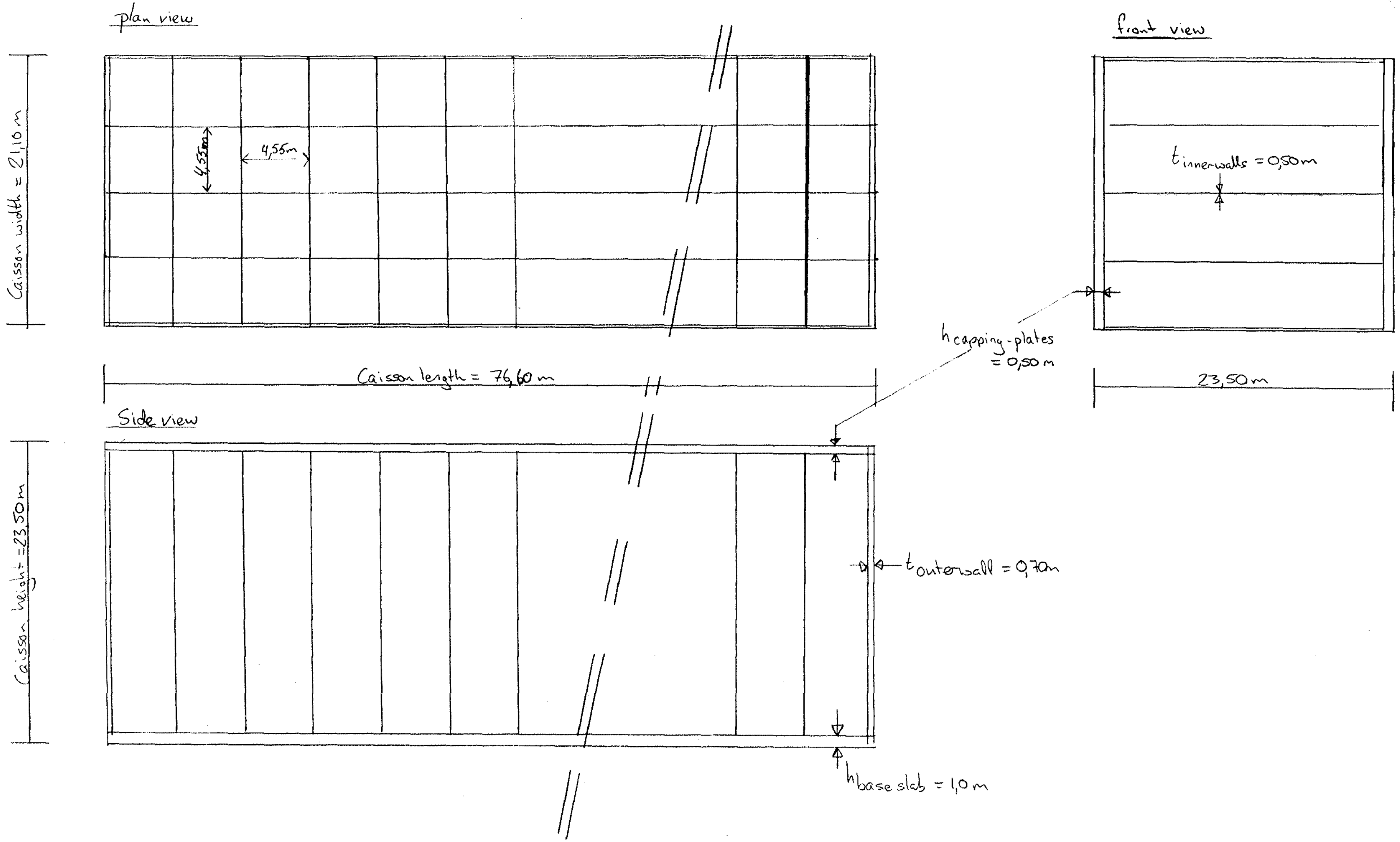
with:

T_0=natural frequency [s]	[s]
j= polaire-massa-traagheids-straal of the caisson	[m]
h_m= metacenter height	[m]
g = gravity force	[m/s^2]

A low natural frequency is achieved by a large traagheidsstraal. A high metacenter height which is favorable for static stability, reduces the natural frequency.

Appendix M:

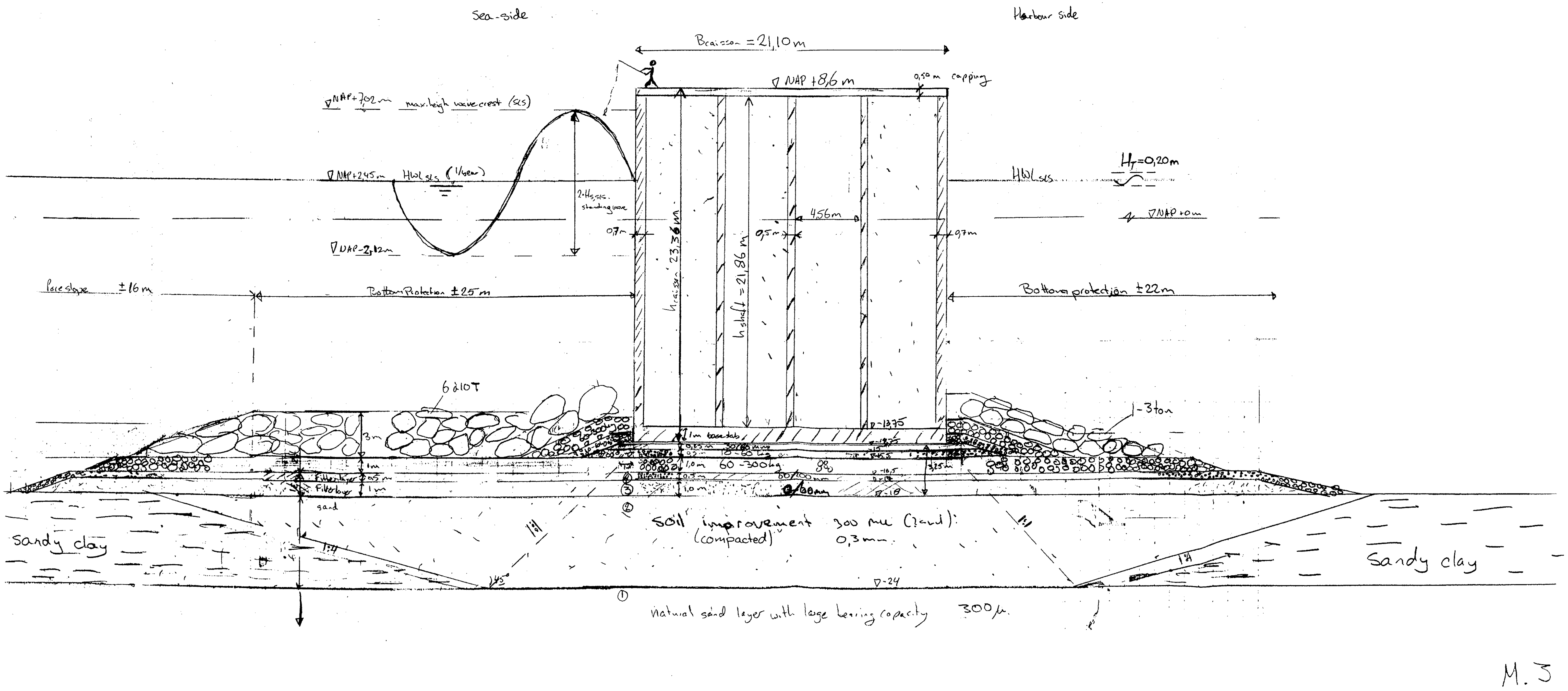
Technical drawings



Caisson	scale 1:300
Sketch of caisson MV2	
4 cells in width direction	
15 cells in length direction.	

M2.

Cross section Maasvlakte 2 caisson breakwater:



Appendix N: Maasvlakte 2 Project

Abstract

This appendix contains background information in order to become familiar with the assumptions made concerning the Maasvlakte 2 caisson breakwater. The most important conclusions of several studies analysing the Maasvlakte 2 construction site relevant to this thesis can be summarised as the following:

In the near future the mainport Rotterdam will have insufficient space to expand in order to compete with other large ports. According to the Harbour plan the following terrain is required for harbour and industrial means:

- 1000 ha by the year 2010;
- an additional 1000 ha by the year 2030 (total);
- 750 ha is required for the natural ecosystem and recreational purposes;

The project group examined possible contours which could meet the required demands of the Maasvlakte 2. It concluded the Northern alternative was the most suited alternative based on the following aspects:

- the most environmentally friendly solution;
  - -maximum access of currents of all the alternatives
  - -limited skyline pollution
- good possibilities for phased execution based on the (innovative) concept of a reusable caisson breakwater;

The basic assumption of the Maasvlakte 2 alternative is to construct the terrain ground of sand. Artificial dunes will serve as sea-defence works for protection against inundation. A breakwater must be placed before the most heavily exposed section of the coastline to break the waves and limit their impact on the sea defence works reduce erosion of the terrain and prevent wave hinder to shipping. Due to the phased construction of the Maasvlakte 2 the breakwater must be able to be reused without any major problem.

This study focuses on the design of a caisson breakwater constructed in floating condition as an alternative for the breakwater required for the Maasvlakte 2. As there is much discussion concerning the exact contours of the Maasvlakte 2, the following assumptions are made concerning the demands of the breakwater design:

• H_T = 0.20 m (SLS)	Acceptable maximum wave height in harbour basin, return period 1/year;
• Design period = 100 years	Time period for which the breakwater must fulfil its function;
• P(SLS) = 1/year	The design storm for serviceability limit state is the storm with a return period of one year;
• P(ULS) = 0.001/year	The design storm for ultimate limit state is the storm with a return period of 1000 years;
• P(insufficient keel clearance) = 0.01	Design level for which sufficient keel clearance must be maintained for all ships in the harbour basin is the lower-low-water level with a return period of 100 years;
• $L_{breakwater}$ = 4.000 m	Total length of the caisson breakwater;
• $b.l._{phase\ 1}$ = NAP -18.0m. • $b.l._{phase\ 2}$ = NAP -20.0m.	Design bottom level during construction phase 1 of the Maasvlakte 2; Design bottom level during construction phase 2 of the Maasvlakte 2;
• $T_{construction}$ = 4 years	Available construction time for breakwater;
• Reusable caisson	The caissons must be able to be reused after a time period of several years, without major problems or costs;

Table 1. Overview design criteria stated by the mainport Rotterdam.

The following time schedule has been set for the Maasvlakte 2 realisation:

date:	activity:
± 2001	Governmental discussion and juridical procedures of Maasvlakte 2 construction phase 1.
2003	Decision of Maasvlakte 2 phase 1 layout and construction.
2006	Award of contract, begin of construction phase 1.
2010	Completion of phase 1 (10 km^2).
± 2020	Governmental discussion and juridical procedures of Maasvlakte 2 construction phase 2.
2023	Decision of Maasvlakte 2 phase 2 layout and construction.
2026	Award of contract, begin of construction phase 2.
2030	Completion of phase 2 (10 km^2).

Table of contents Appendix N.

1. Introduction

The Netherlands has always been an international oriented country. Its excellent geographical location in the North Sea and at the mouth of the Rhine, Maas and Scheldt rivers has led to extensive harbor developments. Therefore The Netherlands is now one of the main gateways of Europe for goods from and to other continents.

Since the second World War several large works have been executed in the Dutch coastal region. The floods of 1953 led to the construction of the Delta works, and in the 1960's and 1970's terrain for harbor extension was gained at the northern side of the Delta-area, the harbor area Europoort/ Botlek and the Maasvlakte 1. As a consequence of these works, significant changes have taken place in the coastal ecosystem of the Delta-area. To ensure safe terrain for work and industries, a series of dams and dikes has been constructed resulting in destruction of large amounts of nature. However the natural ecosystem has adapted to the changed conditions and a new ecosystem has come to life, the Voordelta-area and its dunes.

The economical effects of the mainport Rotterdam are beneficial for the whole of the Netherlands. A network of activities has developed in the surroundings of the mainport which contributes almost 10% to the national income.

The two systems described above, the natural ecosystem and the mainport Rotterdam, both have their own dynamics, which can not be separated. The developments in the mainport Rotterdam are so successful that in the near future the will be a shortage of terrain for accommodation of the harbor and industrial activities. Space for these economical developments could be found in further extension of the mainport Rotterdam by once again creating new terrain in sea in the northern part of the Voordelta. A strategic planning must bring a solution beneficial to both the natural ecosystem and mainport Rotterdam system.

The project to expand the mainport Rotterdam is very extensive, and therefore several project groups have been installed, each examining a different aspect:

- the project group *'Harbor Plan'* which examined the required amount of terrain to accommodate future harbor and industrial activities, and the time period with which the terrain is required;
- the project group *'Contour search directions'*, which examined possible locations suited for extension of the mainport Rotterdam in seaward direction;
- the project group *'Building blocks'*, which examined several constructional alternatives for a seaward extension of the mainport Rotterdam;

Based on the results of these project groups, a contour is selected in this thesis which:

- satisfies the required amount of surface area for the industrial and harbor terrain;
- satisfies the required amount of surface area for the natural ecosystem;
- limits the impact on the natural ecosystem as much as possible;
- is suited for a phased execution of the MV2 in combination with a reusable breakwater;

Of the selected contour the layout, the phased construction and a global indication of the costs will be examined.

2. Harbour Plan

The project group Harbor Plan was installed in 1994 with the goal to investigate the possibilities of expansion of mainport Rotterdam on the Maasvlakte 2. Participants of this group are Municipal Harbor Company Rotterdam and the Ministry of Transport, Public Works and Watermanagement of the Dutch government. The project group investigated alternatives for the Maasvlakte 2, paying special attention to technical, morphological, environmental and ecological aspects.

According to the Harbor-plan 1000 ha. *(1 ha = 100 x 100 m^2)* harbor and industrial terrain is needed by the year 2010, and by the year 2025 to 2035 a total of 2000 ha. is needed. To maintain balance with the natural ecosystem 750 ha. is required.

There has been much discussion concerning the exact amount of required surface area. Due to constantly changing political and social views the exact amount of surface area for harbor and industrial terrain may change in the future. Also morphological, infrastructural and environmental results from earlier phases may lead to changes of the initial plans. By creating the Maasvlakte 2 in several phases, a more flexible planning can be maintained, which can be adapted to changed views. Another aspect which pleads for a phased realization of this project concerns the extent of the work to be made and the large investments which are needed. As the total required terrain is not needed at once, it can be delivered in phases. This way the investments can be spread.

3. Project group Contour search directions

In may 1995 another project group was installed to examine possible contours for the Maasvlakte 2. A first study indicated that extension of the mainport Rotterdam in eastern direction was problematic due to the urban area. The extra activities would lead to too much pressure in the Rotterdam area and furthermore the great depth of the access channel to the quays of the sea-going vessels would cause problems with salt intrusion far land inwards from the sea at high tide. Extension in northern direction was problematic due to valuable agriculture en horticulture activities of the Westland and extension in southern direction raised much resistance due to the recreational value of the Brielse Maas.

The only location which remained suited for further extension of the Mainport Rotterdam for harbor and industrial terrain (2000 ha.) was the area between the Euro/ Maasgeul and (imaginable) demarcation line in south-south-western direction as seen from the Slufter (Figure 1). The space required for extension of nature and recreation (750 ha.) is wider, and stretches from the Euro/ Maasgeul to the coast of the island of Goeree, including the dune-coast of Voorne and Goeree.

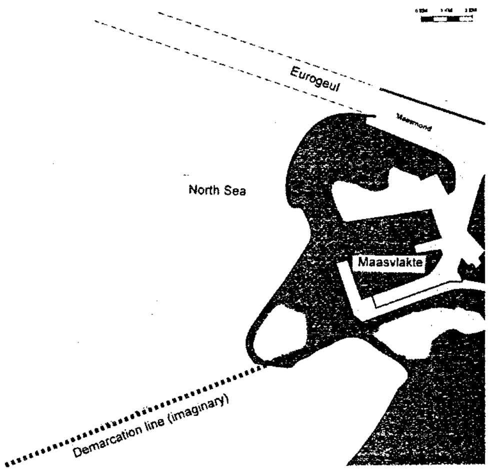


Figure 1 Demarcation line and plan-area of the Maasvlakte 2 [litt]

To generate the possible contours (roughly) the project team made an inventory of the known relevant aspects which had to be taken into account and made a series of assumptions. The assumptions were divided into two categories, constants and variables. The two main systems which must be accommodated into the possible contours are the ecological system and the mainport system.

3.1 Assumptions of the Maasvlakte 2 contours

Constant assumptions:

The area of study:

- This is the area for which the impact of the construction of the Maasvlakte 2 is to be studied.

Nature and recreational area:

- In total 750 ha. terrain must be constructed for nature and recreational purposes physically connected to the harbor and industrial terrain of the MV2.

- The realization of this nature and recreation terrain will primarily not be located south of the demarcation line.
- At the boundaries of the harbor and industrial terrain ecological development can take place, even if it is south of the demarcation line.
- Further study will indicate the optimal integration of the two systems.
- The recreational terrain must not be located in left over spaces, but placed at proper sites.
- Recreation may not be hindered by economical activities, avoiding dangerous situations plays a major role in this aspect.
- If necessary, a zone must be installed to separate industrial and recreational activities.

Variable assumptions:

Phasing of the construction:

- Each possible contour must be suited for phased construction of the Maasvlakte 2. This with respect to both the coastal defense works as the sand filling.

Disclosure for sea going vessels:

- The design must possess sufficient flexibility to adapt to changed demands of terrain functions, therefore a direct connection of the terrain to the Maasgeul or Eurogeul must be possible if the demanded in the course of time.

Hinterland connections:

- The MV2 must be accessible for all types of transport (inland shipping, road, railway, pipeline).

Type of coastal protection:

- The nature of the coastal protection mainly consists of the difference between a soft or hard shoreline protection (sand or dams and caissons). In some sections a combination of these types of shore protection can be used.

Elevation level of the MV2:

Two possibilities are examined:

- high-lying terrain;
- partial construction as low-lying terrain, (polder);

Other assumptions

The dimensions and contour of the Maasvlakte 2 will determine the effects upon the surrounding coastal area. Both the northern and southern boundaries are of influence. When the southern boundary is chosen in such a manner that the Brielscgat and Slufter are in the lee of the MV2, the water body by the coast of Goeree will be filled and drained by currents through trenches, reducing the tidal influence of this area. When the southern boundary is chosen in such a manner that it is more in line with the coast of Voorne, the tide will be able to enter this water body more freely which is more favorable from an ecological point of view. The northern boundary is more of nautical and technical influence on the shipping lane.

In the first examinations of the possible contours the morphological consequences are not taken into account.

The choice of a hard or soft shore protection (dams and caissons or sand) depends on the local water depth, wave attack, and the orientation of the shore protection works opposed to the coastline which is in natural equilibrium. With a contour close and parallel to the western Slufter beach, the preference will tend to a soft protection. With an increase of the water depth and an orientation not parallel to the western Slufter beach, a sea dike offers advantages. With water depths of 15 m or more, or if the coastal defense works are to be used as quays in the future, caissons might be a good option for hard shore protection works. Also in the case the sea wall is to be reused, caissons might be a good option.

3.2 Search directions for possible contours

Based on these assumptions the project group generated four search directions suited for the contours of the Maasvlakte 2. Appendix A presents a plan view of these contours.

1. A contour with the harbor, industrial, natural and recreational area between the present Maasvlakte coastline and the demarcation line, see *Parallel coastline* Appendix A, Figure 1.

2. A contour with the harbor, industrial, natural and recreational area between the present Maasvlakte coastline and the imaginary extended Voornse coast, see *Northern development* Appendix A, Figure 2.

3. A contour which locates the first development phase of the harbor and industrial area on the coast and further development on an island in front of the coast. The natural and recreational area can be realized on the coast or as a row of small islands leading to the large harbor and industry island, see *Island development* Appendix A, Figure 3.

4. A contour similar to the orientation of the local islands of Dutch Flanders, see *Southern development* Appendix A, Figure 4.

For further evaluation of these contours special attention has been paid to the following aspects:

Costs:
- Construction;
- Maintenance;

Harbor and Industry:
- Possibility to extend;
- Flexibility;
- Phased construction possibilities;

Nature and recreation:
- Natural environment;
- Diversity;
- International wildlife migration routes;
- Recreational possibilities;

Landscape:
- Consequences of contours;

Effects on the surroundings:
- Morphologic;
- Tides and currents;
- Environmental;
- Traffic and transport;
- Effects on the current nature;

Three of the main aspects which are focused upon here are the costs of the construction, the possibility for phased execution and the effects on the natural ecosystem. The other listed aspects are important issues for further study.

3.3 Evaluation of the search directions

Base case

In order to have an indication of the costs of the Maasvlakte 2 the project group and several consultants set up a base case. The costs of this base case are founded upon the following aspects:
- coastal defense works (sea defense works and breakwater);
- terrain elevation;

- harbor and terrain facilities;
- project preparation and research costs;
- direct loss of income resulting from the construction of the Maasvlakte 2;
- connection of the Maasvlakte 1 infrastructure to the Maasvlakte 2;

The costs of the buildingstones terrain, sea defense works and breakwater are based upon the document 'Buildingblocks', written by another project group. This document is discussed in section 2.4.

The design of this base case was founded on the following assumptions:
- a total surface area of 2000 ha.;
- hard shore protection works;
- terrain height NAP + 5 m;
- surface area for nature and recreation of 750 ha.;
- harbor entrance for see-going vessels on northern side of terrain;
- construction of main infrastructure;
- lengthening of pipelines currently connected to shoreline;
- connections to the Maasvlakte 1 infrastructure;

The nominal costs of the base case design were calculated to be *f* 6.6 billion Dutch Guilders. Due to the roughness of the design in this phase, a bandwidth of 21% was maintained for variation of the costs. Taking this into account the costs of the base case are set in the order from 5.2 to 8 billion Dutch guilders. The fact must be emphasized that these costs merely form a first indicative number which can be used for a first comparison of the alternatives.

Unlikely contour search directions:
First investigations of the contour search directions *Island alternative* and *Parallel alternative* indicated several negative aspects of these search directions.

The Island search direction turned out to be significantly more expensive than the other alternatives, order 1 to 3 billion Dutch guilders. Based on this large difference in costs and also taking into account the vulnerability of the infrastructure system to access an island (tunnels, bridges, pipelines), this contour alternative was further disregarded. Also an island is not accessible for inland shipping under all conditions, which is unacceptable.

The Parallel search direction results in an extreme high investment of the Maasgeul breakwater in order to build parallel to the coastline and limit the negative morphological impact. Therefore this search direction was further disregarded.

Further investigation of the left over search directions led to four possible lay-outs which were suited for a phased construction of the Maasvlakte 2, these are discussed in the next section.

Likely contour search directions

Southern alternative:

- A contour with minimum construction costs within the southern contour lines.
- Costs: *f*6,6 billion guilders.

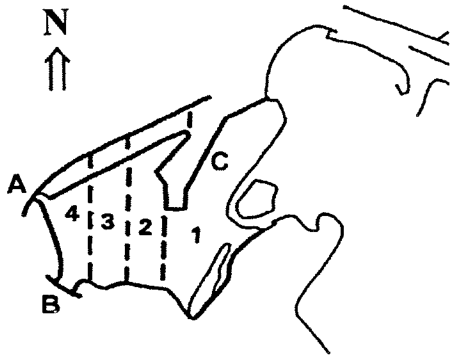


Figure 2 Southern Alternative-construction phases.

The northern barrier of this alternative consists of a hard shore protection (caissons) which extends into the ocean. During each consecutive phase, the breakwater must be extended in south-western direction. The shore protection at the north western point, section A, is moved seawards each phase over a length of approximately 1500m.

Section B at the south western point of the Maasvlakte 2 is also constructed of a hard shore protection. As the hydraulic conditions from south western direction are not as rough as those from northwestern direction, section B can probably be executed as a soft protection (sand beach) during the first phases. Only in the final stage section B will be constructed as a hard shore protection. The wave attack on section B requires further investigation. The section between A and B may be executed as a sand beach.

Central south alternative:

- This alternative offers maximum access to currents and tidal effects within the *southern* contour lines.
- Costs: *f* 7,7 billion guilders.

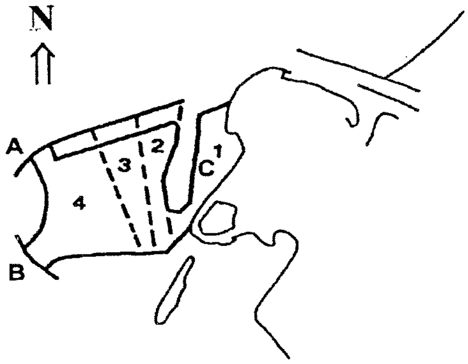


Figure 3 Central south alternative construction phases

The northern barrier of this alternative is constructed in a similar manner as the southern alternative and the consecutive phases will also be constructed in seaward direction. In this situation the southern barrier B is perpendicular to the direction of the phased construction and has a length of 2500 m. As with the southern alternative the necessity of a hard shore protection for section B during the first phases must be examined.
Taking into account the local shallow waters it is questionable weather it is feasible to construct this shore protection of caissons.

Section C consists of a hard shore protection which must be constructed during phase 1. A 1800 m long breakwater will extend seaward in line with the present Southern-dam of the Maasvlakte 1, and 5000 m breakwater will be constructed in southern direction situated to the west of the future Maasvlakte 2.

If the hard shore protection of section C is constructed of caissons, it may serve as quay wall in later phases, however the local shallow waters, NAP -15 m to NAP - 5 m are not ideal to construct with caissons, and measures (dredging) will have to be taken.

Central north alternative:

- This alternative is situated 'behind' the Maasvlakte 1 as much as possible, and interferes minimally with the ocean currents.
- Costs: *f*8,8 billion guilders.

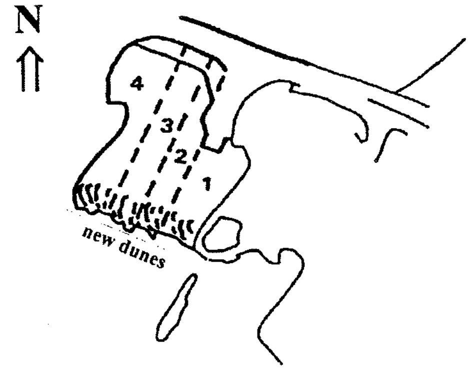


Figure 4 Central north alternative-construction phases.

From a navigational point of view the northern breakwater must extend at least 3500 m seaward from the harbor entrance and therefore the complete breakwater must be directly constructed during the first phase. The terrain itself is constructed in phases behind the breakwater. This alternative does not offer any possibilities for reuse of the hard shore protection in combination with a phased execution of the MV2 and is therefore not interesting for construction with reusable caissons.

Northern alternative

- This alternative is situated 'behind' the Maasvlakte 1 and offers maximum accessibility of ocean currents and tidal effects to the coasts of Goeree and Voorne. Also this alternative results in the least skyline pollution.
- Costs: *f*10,[illegible] billion guilders.

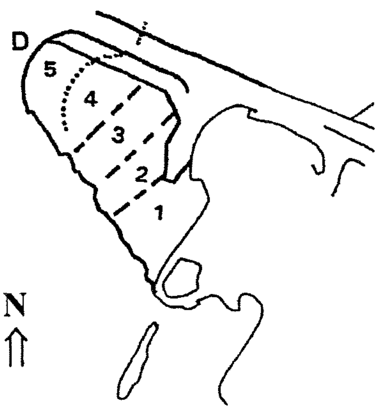


Figure 5 Northern alternative construction phases

Similar to the center alternative the minimum length of the northern barrier is 3500 m in the first phase. The construction phases will be in north western direction and therefore section D, executed as a hard shore protection, must be moved in each phase. In the final phase the breakwater will extend 2000 m further seaward than during the first phase. The length of section D is approximately 5000 m. The straight section of the hard shore protection parallel to the Maasgeul will be extended during the construction of the phases. This alternative offers interesting possibilities for phased execution using a hard reusable shore protection.

4. Building blocks[1]

The project group building blocks studied the costs of the building blocks of the Maasvlakte 2. This was done in such a manner that the costs could be calculated independent of the exact contour line. The building blocks which were examined were:

- the terrain;
- the sea defense works;
- the breakwater;

4.1 Innovative Aspect

The project Maasvlakte 2 is a water engineering works which is so extensive that it offers possibilities for innovative techniques in this field of civil engineering. Therefore many types of less obvious constructional alternatives were considered. However the first indicative studies executed by the project group Maasvlakte 2 indicate that less traditional solutions are significantly more expensive than the more traditional solutions (soft sea defense works, sea dike, rubble mound breakwater), and therefore seam unrealistic. An exception is the caisson-breakwater, which is internationally (especially in Japan) constructed on large scale. A caisson breakwater seams a very feasible solution for the Maasvlakte 2. Other aspects which can be approached in an innovative manner are the use of materials (e.g. plastics), an 'intelligent' flexible and phased execution of the project, and combinations of the functions. For example by constructing the breakwater in such a manner that it can also fulfill the function of quay wall, or by maintaining such a phased execution that the caisson breakwater of an early phase can be used as quay wall in later phases.
Also the combination of soft shore protection works, the development of nature and recreational values, and hard shore protection works offer interesting possibilities.

This thesis focuses on the possibilities of a caisson-breakwater which can easily be repositioned during the consecutive construction phases.

4.2 Building block: terrain

The terrain must have no hinder due to high water levels or erosion under design conditions. The elevation level of the terrain is set at NAP + 5.0 m. Considered options for the terrain were:

- sand;
- a floating construction;
- a construction consisting of large elements;
- a construction on piles;

The project group MV2 studied construction costs of the different types of terrain:
A note must be made here that the construction costs merely form an indicative base in order to compare the different constructional alternatives with one another. More specific calculations must be made in a later stadium. The costs in the table are the construction costs for a segment of terrain of 1 x 1 km^2 ($=1.10^6$ m^2), calculated for a certain depth of the terrain site.

[1] The costs presented in this section have been taken from the document Building blocks[litt]

Bottom level[2] [m - NAP]	Costs of constructional alternative in Hfl x 10^6					
	Sand	Floating construction (q=40 kN/m^2)		Large elements	Construction on piles	
	(q=40 to 150 kN/m^2)	terrain height NAP +5m	constant freeboard of 3 m	(q=40 to 150 kN/m^2)	(q=40 kN/m^2)	(q=150 kN/m^2)
5	50	-	-	-	-	-
10	70	-	-	8.500	1.700	3.900
>11.9	-	-	8.600	-	-	-
15	80	-	8.600	9.500	1.800	4.200
>15.3	-	10.800	-	-	-	-
20	90	10.800	8.600	10.500	1.900	4.600

Table 2 Overview construction costs for a segment of terrain of 1.10^5 m^2.

The conclusion was that it was by far the cheapest to construct the terrain of sand, even with underground conditions of such poor quality that it must first be excavated in order to replace it with material of better quality. Therefore this study further assumes that the terrain is constructed of sand.

4.3 Building block: sea-defence works

The primary function of the sea defence works is to protect the terrain behind it against floods due to high water levels, water hinder caused by waves and erosion of the terrain. Options considered for the heavily exposed *northern* sea-defense works were:

- artificial dunes;
- artificial dunes combined with hard constructions;
- a sea dike;
- caissons;
- a block wall;
- a retaining wall;

For the less heavily loaded *southern* sea-defense works considered options were:

- a wide beach with regular beach nourishment;
- artificial dunes;
- a sea dike;

These sea defense works can primarily be subdivided into two categories, *hard* and *soft* shore protection works. The soft shore protection works are constructed of sand which is basically not protected by hard constructions. These soft shore protection works (*wide beach with regular nourishment and artificial dunes*) are flexible and adapt to changed conditions (hydraulic forces, wind). Hard shore protection works can be subdivided into constructions with a slope (*sea dike*) and vertical constructions (*caisson, block wall or retaining wall*). A sea dike mainly consists of sand; the outer layer of the dike however consists of an armor layer to protect it from wave attack.

The conclusion of the alternatives for the *northern* sea-defense works was that a block wall caused difficulties with the design and the construction. Furthermore the costs of a block wall as sea-defense works were calculated to be so high that no further analysis was made regarding this option.

The costs of the caisson and block wall alternative are so high that basically use of such constructions are not obvious. However under certain conditions, for example if the sea defense works also function as quay wall or if there is only little space available the option caisson or large elements placed on a caisson foundation may offer a solution.

Artificial dunes offer the cheapest solution, even though there is a large uncertainty with respect to the equilibrium bottom profile and loss of sand due to longshore transport. Even if conditions demand the sand profile to be constructed under a very low angle (1:50) under NAP -5.0 m and, even if sand loss is high resulting from longshore transport, this alternative is significantly cheaper than caisson or block

[2] Due to the different constructional alternatives it is not possible to calculate the costs of the terrain for exactly the same bottom level.

wall solutions. An aspect which may cause problems is sedimentation of the Euro-Maasgeul shipping lane, resulting in high maintenance costs for dredging.

A sea dike construction is advantageous because it uses less space and sand than the other soft solutions. Also it has the least uncertainties concerning unexpected changes of the bottom profile due to changed currents and wave patterns which may lead to exceeding the costs. A sea dike was concluded to be the cheapest solution.

The conclusions of the *southern* alternatives of the sea defense works were that soft options (*beach and artificial dunes*) offer advantages for the aspects 'flexibility' and 'recreation/ nature' as opposed to hard options (*sea dike*) for the sea defense works.

The costs of the different types of sea-defense works are presented in Table 3 and Table 4.

Building block-alternative	Bottom level [m - NAP]			
	5	10	15	20
Artificial dunes	90	130	190	290
• Southern dam alternative	-	-	400	-
• alternative with hanging beach	-	-	230	-
• alternative with rubble mound breakwater	-	-	190	-
Sea dike	-	220	270	320
Caisson[3]	-	470	570	650
Block wall	-	730	780	860
Retaining wall	-	880	1300	1800

Table 3 Overview costs of *north-western sea-defense works* (in *f*millions and for a 4 km length).

Buildingstone-alternative	Bottom level [m - NAP]			
	5	10	15	20
Wide beach with regular beach nourishment	80	120	160	210
Artificial dunes	60	100	130	180
Sea dike	-	-	160	-

Table 4 Overview costs of southern *sea-defense works* (in *f*millions and for a 4 km length).

4.4 Building block: breakwater

The Maasvlakte 2 breakwater must fulfill the following functions during its lifetime:

- wave reduction for shipping;
- guiding of the currents;
- protection of the shipping lane from sedimentation;
- visual guidance fore shipping;
- protection of moored ships during transshipment of goods;
- protection of the sea defense works against erosion;
- reduction of the wave forces on the terrain and sea defense works;

Analyzed options for the breakwater were:

- floating breakwater;
- rubble mound;
- pile row;
- caisson;
- block wall;
- large elements placed on caisson foundation;
- inflatable weir construction;

[3] These costs include the costs of the rubble mound foundation of the caisson and the construction dock for the caissons

First indicative calculations showed that a floating breakwater could not meet the demands of wave transmission (see section *Caisson height* and Appendix *Transmission model of Goda*), and therefore this alternative has not been further investigated. Calculations of the weir construction showed this alternative was significantly more expensive alternative, and therefore this alternative also wasn't examined further.

The project group MV2 studied construction costs of the different types of breakwaters:
The costs of the different constructional alternatives for the breakwater were based on the assumption of a breakwater length of 4 km. and with an acceptable wave transmission K_T of 10 %. A note must be made that these values are purely indicative.

Constructional alternative:	Bottom level [m - NAP]		
	-10m	-15m	-20m
Rubble mound	370	450	630
Pile row	-	1.040	-
Caisson[4]	360	440	530
Large elements on caisson foundation	-	-	-
Block wall	1.480	1.570	1.660

Table 5 Overview of breakwater costs (in *f* millions for a section of 4 km with $K_{T,max}$ = 10 %).

Conclusions concerning costs of the breakwater alternatives were that the alternatives rubble mound and caisson are significantly cheaper than other alternatives, and for bottom levels lower than NAP-20 m the caisson breakwater is considerably cheaper than the rubble mound breakwater. The alternatives pile row and block wall are so expensive that they are not further analyzed.

A caisson breakwater offers interesting possibilities regarding the phased construction of the Maasvlakte 2. Also the potential use of the (initial) breakwater as quay wall during a later construction phase might offer interesting possibilities.

5. Evaluation of alternatives

Based on the results of the project groups discussed in the previous sections, a contour is selected in this thesis which:

- satisfies the required amount of surface area for the industrial and harbor terrain;
- satisfies the required amount of surface area for the natural ecosystem;
- limits the impact on the natural ecosystem as much as possible;
- is suited for a phased execution of the MV2 in combination with a reusable breakwater;

The layout, the phased construction and a global indication of the costs will be examined for this contour.

Each alternative of the Maasvlakte 2 has its specific length and layout of the hard shore protection and possibilities to reposition the breakwater during the consecutive construction phases. Based on the innovative concept for the constructional alternatives for the Maasvlakte 2, the focus of this thesis will be in the direction of an alternative which offers interesting possibilities for a hard shore protection which is reusable.

The southern and central south alternative:
Repositioning of the hard shore protection of section A and possibly B. It is not very likely that the northern breakwater will be constructed in this manner; a better and more realistic constructional option would be to extend the breakwater in a straight line seawards in each phase. This would limit the possible reuse of a caisson breakwater. An important aspect which must be further examined is the effect on the sea currents and morphological effects of this design.

[4] These costs include the costs of the rubble mound foundation of the caisson and the construction dock for the caissons

Another questionable point is weather a hard shore protection is essential for section B. Also the shallow water near the Maasvlakte 1 contour is not ideal for a reusable caisson breakwater. Based on these aspects the southern and central south alternative are not further discussed.

Central south alternative
Design of the hard shore protection of section C during the first phase which can serve as quay wall in later phases. This alternative offers interesting possibilities for construction with reusable caissons.

Central north alternative

Northern alternative
Repositioning of the hard shore protection of section D during the consecutive construction phases. During the first phase the northern breakwater must extend 3500 m seaward from the present harbor entrance. This hard shore protection will directly be placed at its final destination. The rounded part of the breakwater with a length of approximately 4000 m must be repositioned northward in each phase.

This alternative offers the most interesting possibilities for construction with a reusable hard shore protection in the form of caissons, and therefore will be the basis for this thesis.

5.1 Northern alternative of the Maasvlakte 2

In this study it is assumed that the Maasvlakte 2 Northern alternative will be constructed in 2 phases,

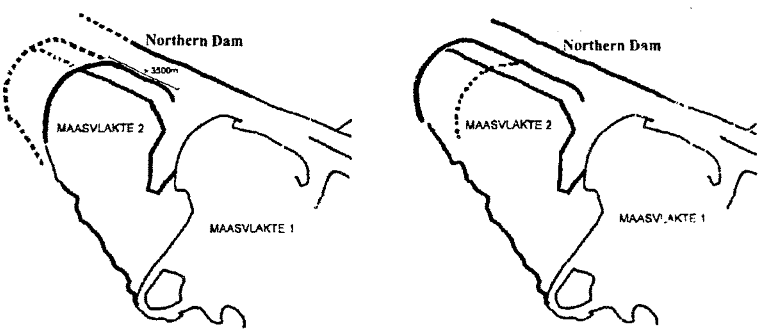


Figure 6 First and final construction phase of the Maasvlakte 2.

Construction phase 1 of the Maasvlakte 2:
During the first construction phase of the Maasvlakte 2, 1000 ha. (10 km^2) terrain must be constructed by the year 2010. With regard to the high currents which occur frequently in the shipping lane the Northern Dam and the breakwater must provide at least 3500 m of calm water for connection of tugs to the incoming sea vessels (geographical boundary conditions). This stretch of breakwater will directly be placed in its final position and will not have to be replaced during the final phase. The rest of the Maasvlakte 2 contour requires a hard shore protection for the northern and western side of the terrain. The length of this section, which will have to be repositioned in the final construction phase, is approximately 4000 m. The shore protection on the south western side of the Maasvlakte 2 (length: 7000 m), can be executed as a soft shore protection.

Buildingstone	section	costs building stone	costs section
terrain	18.5 km^2 (=18.5 $\cdot 10^6$ m^2)	*f* 80$\cdot 10^6$ /km^2 (=*f* 80.0 /m^2)	*f* 1480$\cdot 10^6$
north western sea-defense works • caissons (quay) • artificial dunes (in combination with caisson breakwater) south sea-defense works • artificial dunes	 8750 m 2500 m 7000 m	 *f* 142500 /m *f* 47500 /m *f* 32500 /m	 *f* 1246.9 $\cdot 10^6$ *f* 118.8 $\cdot 10^6$ *f* 227.5 $\cdot 10^6$
breakwater • Northern Dam caissons[5] • MV2 caissons south of Eurogeul[3] • Caissons of north western section which are to be reused	 7000 m 3500 m 4000 m	 *f* 102000 /m *f* 102000 /m *f* 102000 /m	 *f* 714 $\cdot 10^6$ *f* 357 $\cdot 10^6$ *f* 408 $\cdot 10^6$
Total costs construction phase 1	-	-	*f* 4552.2 $\cdot 10^6$

Table 6 Overview construction costs phase 1 MV2, for average depth NAP -15m.

Construction phase 2 of the Maasvlakte 2:
By the year 2030, for the second phase, once again 1000 ha. industrial terrain must be delivered. In total for the first and second phase 2000 ha. of terrain will be created for harbor and industrial purposes, and 750 ha for the natural ecosystem and recreation. During this phase 3500 m of the northern breakwater caissons are already in position from the first phase, 4000 m of western breakwater caissons (sections 1 and 2) will have to be repositioned to the contour lines of the second phase and 2000 m of new caissons must be constructed to protect the remaining coastline.
The artificial dunes in combination with hard shore protection must be moved 1500 m in western direction.
The (soft) southern sea-defense works, artificial dunes, will have to be moved approximately 1 km seawards, and have a length of 7500 m.
In total during this phase 4000 m of caissons will have to be repositioned and 2000 m of caissons will have to be constructed. As in the first construction phase the shore protection works on the south western side of the Maasvlakte 2 can be executed as a soft beach (length: 8250 m).

Buildingstone	section	costs building stone	costs section
terrain	11.5 km^2 (=11.5 $\cdot 10^6$ m^2)	*f* 80$\cdot 10^6$ /km^2 (=*f* 80.0 /m^2)	*f* 920$\cdot 10^6$
north western sea-defense works • caissons (quay) • artificial dunes (in combination with caisson breakwater) south sea-defense works • artificial dunes	 1350 m 3000 m 8250 m	 *f* 142500 /m *f* 47500 /m *f* 32500 /m	 *f* 192.4$\cdot 10^6$ *f* 142.5$\cdot 10^6$ *f* 268.1$\cdot 10^6$
breakwater • Caissons of north western section which are to be repositioned[6] • Caissons for north western section to be newly constructed	 4000 m 2000 m	 *f* 102000 /m *f* 102000 /m	 *f* 408$\cdot 10^6$ *f* 204$\cdot 10^6$
Total costs construction phase 2	-	-	*f* 2135 $\cdot 10^6$

Table 7 Overview construction costs phase 2 MV2, for average depth NAP -15m.

[5] These caissons are directly positioned at their final location and will not be repositioned during the second phase.
[6] First assumption is that costs of reuse of the caissons is the same as construction of new caissons.

The total construction costs of the MV2 building blocks are approximately *f* 6.7 billion.

The delay of the second construction phase of the MV2, costs *f* 2.1 billion, for 20 years and based on an interest rate of 8% and an inflation of 1% per year, will amount to *f* 1.6 billion saved expenses[7].

If the total Maasvlakte is built at once, costs are:

Buildingstone	section	costs building stone	costs section
terrain[8]	30 km^2 (=30 $\cdot 10^6$ m^2)	*f* $80 \cdot 10^6$ /km^2 (=*f* 80.0 /m^2)	*f* $2400 \cdot 10^6$
north western sea-defense works • caissons (quay) • artificial dunes (in combination with caisson breakwater) south sea-defense works • artificial dunes	 10100 m 3000 m 8250 m	 *f* 142500 /m *f* 47500 /m *f* 32500 /m	 *f* $1439.3 \cdot 10^6$ *f* $142.5 \cdot 10^6$ *f* $268.1 \cdot 10^6$
breakwater • Northern Dam • Caissons of north western	 7000 m 9500 m	 *f* 102000 /m *f* 102000 /m	 *f* $714 \cdot 10^6$ *f* $969 \cdot 10^6$
Total costs construction phase 2	-	-	*f* $5932.9 \cdot 10^6$

Table 8 Overview construction costs total MV2 at once, for average depth NAP -15m.

The total construction costs of the MV2 building blocks if the complete Maasvlakte 2 is constructed at once are *f* 5.9 billion, which is *f* 0.8 billion cheaper than with phased construction. However this is only half of the saved expenses when the second construction phase is postponed with 20 years.

5.2 Construction time of the Maasvlakte 2

The following time schedule has been set for the Maasvlakte 2 realisation:

date:	activity:
± 2001	Governmental discussion and juridical procedures of Maasvlakte 2 construction phase 1.
2003	Decision of Maasvlakte 2 phase 1 layout and construction.
2006	Award of contract, begin of construction phase 1.
2010	Completion of phase 1 (10 km^2).
± 2020	Governmental discussion and juridical procedures of Maasvlakte 2 construction phase 2.
2023	Decision of Maasvlakte 2 phase 2 layout and construction.
2026	Award of contract, begin of construction phase 2.
2030	Completion of phase 2 (10 km^2).

Table 9 Time schedule of Maasvlakte 2.

[7] Saved expenses: $2135.10^6 - \dfrac{2135.10^6 \cdot (1.01)^{20}}{(1.08)^{20}} = 1576.10^6 \approx 1.6.10^9$

[8] The surface area of the terrain refers to the required surface area of the harbor and industrial terrain and also to the terrain.

6. Conclusions

There is still very much discussion concerning the design of the Maasvlakte 2. Due to the uncertainties of the exact required amount of terrain there is much discussion of the contours. In order to design a caisson breakwater for the Maasvlakte 2 several conclusions will be drawn from the numerous studies and assumptions will be made for this study.

In the near future the Mainport Rotterdam will have insufficient space to expand in order to compete with other large ports. According to the Harbour plan the following terrain is required for harbour and industrial means:

- 1000 ha by the year 2010;
- an additional 1000 ha by the year 2030 (total);
- 750 ha is required for the natural ecosystem and recreational purposes;

The project group examined possible contours which could meet the required demands of the Maasvlakte 2. It concluded the Northern alternative was the most suited alternative based on the following aspects:

- the most environmentally friendly solution;
    -maximum access of currents of all the alternatives
    -limited skyline pollution
- good possibilities for phased execution based on the (innovative) concept of a reusable caisson breakwater;

The basic assumption of the Maasvlakte 2 alternative is to construct the terrain ground of sand. Artificial dunes will serve as sea-defense works for protection against inundation. A breakwater must be placed before the most heavily exposed section of the coastline to break the waves and limit their impact on the sea defense works reduce erosion of the terrain and prevent wave hinder to shipping. Due to the phased construction of the Maasvlakte 2 the breakwater must be able to be reused without any major problem.

This study focuses on the design of a caisson breakwater constructed in floating condition as an alternative for the breakwater required for the Maasvlakte 2.