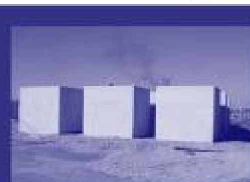
RESEARCH ON COSTS OF ARMOUR UNITS

Breakwaters IJmuiden, The Netherlands

Master's Thesis

Jan Wagner, December 2004



TU Delft
Delft University of Technology

Delta Marine Consultants bv

Research on Costs of Armour Units

Breakwaters IJmuiden, The Netherlands

Final Report for the Master Thesis

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In cooperation with:
Civil Engineering Division of the
Directorate-General for Publics Works and
Water management
Delta Marine Consultants b.v

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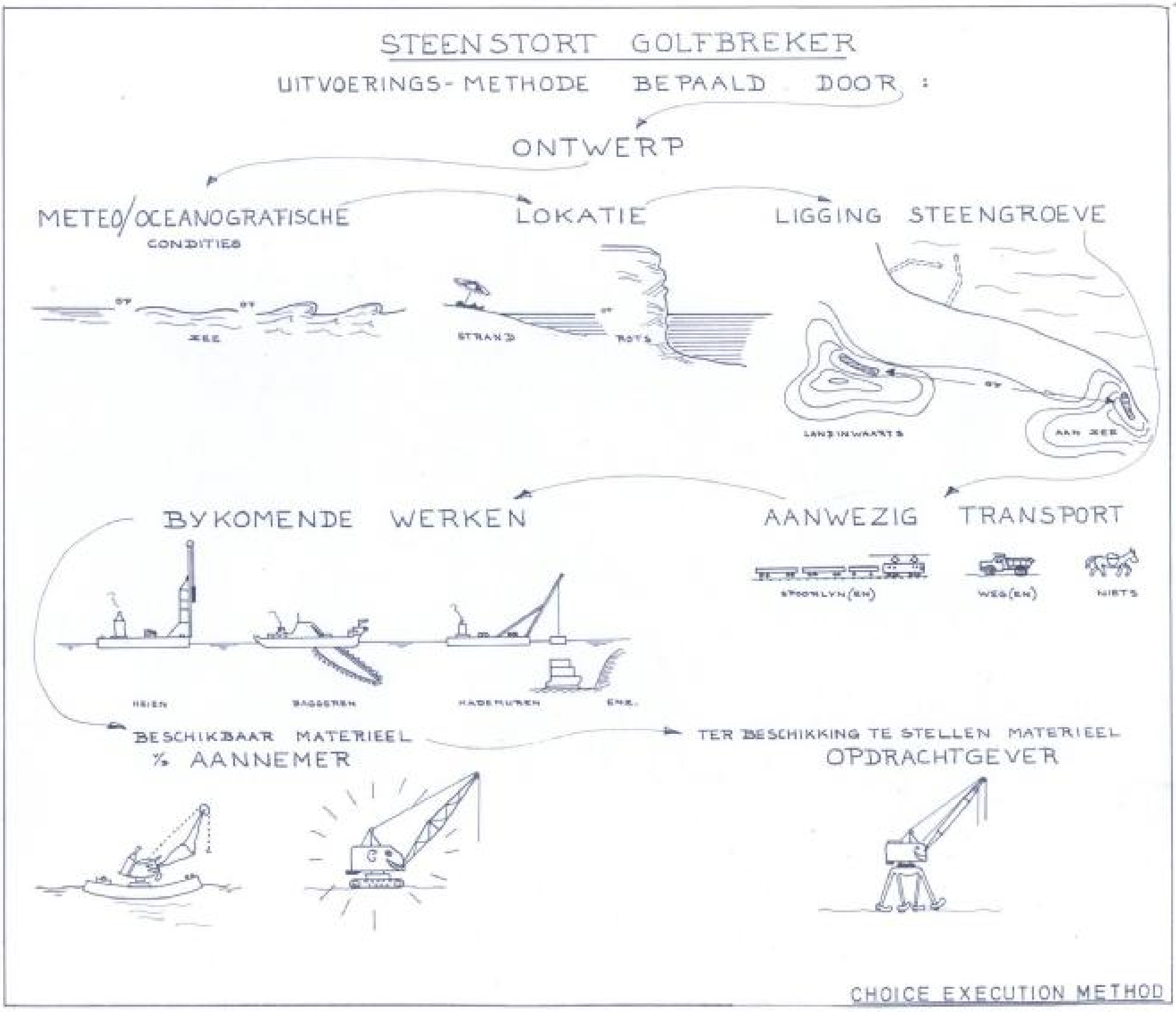

Illustration of Construction methods, February 1966
Royal Netherlands Harbour Works Co. Ltd.

Preface

In order to achieve the degree of Master of Science in Civil Engineering a graduation project has to be executed as final part of the study Civil Engineering. The Master of the study Civil Engineering comprises several specialisations. This Master's Thesis research focuses on breakwaters, which is a subject of the specialisation Coastal Engineering.

This document is the final report of my Master Thesis research. This research is initiated by cooperation between Delft University of Technology, Delta Marine Consultants b.v. and the Civil Engineering Division of the Directorate-General for Publics Works and Water management (Bouwdienst Rijkswaterstaat). The Civil Engineering Division has carried out a research on different solutions for the problems of the breakwaters of IJmuiden. Not only costs have been viewed in this research, but also other criteria. The results of this research are compared with this thesis research in Chapter 7 of this report. The graduation committee consists of the following members:

Prof. dr. ir. M.J.F. Stive	DUT, Civil Engineering, Coastal Engineering (Chairman)
Ir. H.J. Verhagen	DUT, Civil Engineering, Coastal Engineering
Ir. J.S. Reedijk	Delta Marine Consultants b.v.
Ir. J.P.F.M. Janssen	CUD of Directorate-General for Public Works and Water management
Ir. T.H.W. Horstmeier	DUT, Civil Engineering, Construction Processes

This research was carried out under the supervision of Prof. dr. ir. M.J.F. Stive. I would like to thank him for his guidance and advice throughout this research. I would like to thank Henk-Jan Verhagen, Hans Janssen and Bas Reedijk and the others of the Coastal Team of DMC for helping me to understand the design and construction process of breakwaters. I also want to thank Ir. T.H.W. Horstmeier for the discussions and the review work.

I thank my parents, sister and all my friends for their support and the fun times we had during my years in Delft. Last but not least, I want to thank Martine for the pleasant times we had during this year.

Jan Wagner, December 2004

Abstract

Breakwaters are used worldwide to provide shelter from wave action and to protect the coast. Breakwaters diminish the downtime of loading and unloading operations in ports. Breakwaters can be divided into different types: Mound types, monolithic types, composite types and special types.

This thesis research focuses on an overtopped mound type breakwater with an armour layer of concrete armour units. Hydraulic stability and structural integrity are main items of the design process, but costs estimation is merely a small part of the design process. To get a realistic insight in the costs between different armour units, the use of a real case study is necessary for a costs comparison. A double layer of cubes and two interlocking units, Accropode and Xbloc, are chosen for this thesis study.

In The Netherlands the breakwaters of IJmuiden have stability problems with the armour layer and the toe construction. At this moment the Civil Engineering Division of the Directorate-General for Publics Works and Water management (CED) is developing solutions for the long-term. Reinforcing the breakwaters with armour units is one of these solutions. This thesis study is performed simultaneously with the research of the CED.

The objective of this thesis study:

Find the most economical armour unit (Lowest Life Cycle Cost) for the armour layer of the breakwaters of IJmuiden. In every stage of the construction process the three armour units, cubes, Accropodes and Xblocs, will be compared only on costs. This case study has to show the structure of costs of the three armour units in general.

This thesis study is constructed as follows: First the problems of IJmuiden are reviewed and constraints are listed for the new designs of the breakwaters. New designs are required to calculate the construction costs of the armour units. After the total construction costs of the alternatives are compared, the variables of the alternatives are varied in sensitivity analysis

From 1868 till 1876 the old breakwaters of IJmuiden were built as a part of the fairway connection between the North Sea and Amsterdam. In 1956 it was decided that the breakwaters had to be extended to fulfil their functions in the future, because at that time there was a large amount of sediment in the harbour mouth and shipping into the harbour basin was difficult. An armour layer of stone asphalt had been chosen, but during the construction of this layer the problems started. Settlements of the layers took place at both sides of the dam with the consequence that the rigid part above the waterline could not follow the settlements of the part under the waterline, so cracks in the layers were visible. First 17 tonne concrete cubes were placed on the armour layer to increase the strength of this armour layer and later on heavier units (30 and 45 tonne). The placement of the blocks did not solve all the problems, because many of the placed concrete blocks started to fail, like the occurrence of cracks in the surface of the blocks, corner pieces of the concrete blocks dislodged and disintegration of the concrete and some cubes were displaced. In expectation of the optimum maintenance strategy no maintenance has been executed from 1995 and at the moment short term emergency maintenance is necessary.

A breakwater, like any other coastal structure, is planned as a practical measure to solve a specific problem encountered in the coastal zone. In this case the breakwater exists already, but the armour layer requires a reconstruction. A number of stages can be distinguished in the design process:

1. Definition of functions
2. Determination of boundary conditions
3. Creating alternative
4. Geometrical design
5. Final choice of a functional solution

Stage 1 of the design process is clear, because the functions of the breakwater will be the same in the future. The most important design parameters are the significant wave height (H_s=7.5m) and the cross sections of the current breakwaters of IJmuiden. With these boundary conditions alternatives can be created for all three armour units with different kind of density for concrete. Interlocking units are always constructed with normal concrete (2,350 kg/m^3). Two extremes, normal concrete (2,350 kg/m^3) and heavy concrete (3,800 kg/m^3) are chosen for the double layer of cubes. This results in following dimensions of the armour units:

	Accropode	Xbloc	Cube2350	Cube3800
Trunk [m^3]	12	9	20.8	2.2
Units [-]	3,332	3,841	5,719	20,558
Head [m^3]	18	12	29.0	3.4
Units [-]	390	447	644	2,328

Table 1 Overview of the dimensions of armour units on the trunk and the head of the breakwater

To develop a realistic solution for the reconstruction of the breakwater of IJmuiden all construction options are reviewed. The reconstruction alternatives are carried out with waterborne and land-based equipment. The toe and the filter layers are dumped by side stone dumpers and the armour units are placed from the breakwater with a crawler crane. After a multi-criteria analysis the cubes with normal concrete do not form a part of the comparison, because difficulties with units are expected during the production and placement of the units. The remaining three alternatives are compared on costs of three stages: Production, placement and maintenance of armour units. The costs of maintenance of armour units are difficult to quantify, so only maintenance strategies for breakwater are described in this study. Breakwaters with interlocking units are designed damage free, because also minor damage could initiate failure of the total armour layer. An armour layer with cubes can be designed damage free or it can allow minor damage, which can be repaired later on. Monitoring is for both interlocking units as for cubes advisable.

The total costs of the construction consist of the production costs and the placement costs.
The costs, which are made during the production of armour units, are the costs of moulds, storage area, equipment, labour and material cost. The main cost factor of the production of armour units is concrete costs. The price of concrete and the required quantities determine the total concrete costs. The cube alternative has the highest production costs due to the price of high density concrete (250 euro/m^3 concrete) and the double layer (highest quantity of concrete). The Xbloc has the lowest concrete costs, 15 % less than the Accropode alternative.

Placing methods of interlocking units and cubes differ, because interlocking units have to be placed accurately and cubes can be dumped. The placing of interlocking units is done by a normal crawler with a sling or hydraulic clamp and the dumping of cubes is done by a special crane.
The placing rates of the Accropode are known, because since 1980 several breakwaters have been built with Accropodes. The placement rates of different project have a certain range due to the different environmental conditions. For this research the placement rates of 2 comparable projects have been used. In 2003 the Xbloc has been introduced and at the moment no placement rates are available. The placement rates, which are used in this study, are based on a placing experiment with model Xblocs and Accropodes. In this study a placing rate of a factor 1.5 more than the Accropode rates is taken into account. The placing rates of dumping of cubes are based on the experiences with

the construction of the Hook of Holland breakwaters. The placement costs are mainly determined by the duration of the placement operation and price of the cranes. The Xbloc has the lowest placing costs due to the lowest number of placing weeks and the price of the normal crane. The cube alternative has the highest costs due to highest number of placing weeks, because the high number of units, and the expensive crawler crane.

The costs of the cube alternative are very high in comparison with the other two alternatives, so the cost factors of the alternative are varied in a sensitivity analysis. When a normal crane is used instead of a special crane, the cubes have to be placed and the placing rate decreases. This results in a longer placing period, but lower placement costs by the use of normal crane.
The price of high density concrete (3,800 kg/m^3) is the main cause of the high production costs of the cube alternative. When another density, 2,800 kg/m^3, is used the dimensions of the unit increase, but the number of units decreases. However the total concrete quantity of this alternative is higher, the total costs of this alternative are considerably lower than the alternative with 3,800 kg/m^3. A combination of the normal crane and concrete with density 2,800 kg/m^3 is the most economical alternative of all three cube options. Working the whole week including the weekends in the period April – September makes it possible to finish the placement operation of the two interlocking units in one year.

Out of the calculations and the sensibility analysis the conclusions follow:

- Concrete costs form the largest part of the total construction costs, between 73 to 77% of the total costs for the trunk section and between 61 to 81% of the total costs for the head section. It can be concluded that the concrete costs are the most important part of the total costs.
- Applications with high density concrete for cubes are interesting, but expensive
- Knowledge of maintenance and experience with maintenance strategies is limited.
- The outcome of the cost comparison of breakwaters IJmuiden gives the following results, but these costs are only the costs of the armour layer per m^1 breakwater.

	Accropode	**Xbloc**	**Cube3800**
Total cost trunk [euro]	7,250	6,350	20,851
Total cost head [euro]	10,094	8,382	24,492

Table 2 Total costs of the armour layer per m^1 breakwater.

	Accropode [%]	**Xbloc [%]**	**Cube 3800 [%]**
Percentage placement weeks	100	87	123

Table 3 Placement time armour units.

	Accropode [%]	**Xbloc [%]**	**Cube 3800 [%]**
Percentage concrete costs	100	85	290

Table 4 Concrete costs of armour units.

As earlier mentioned the Civil Engineering Division (CED) has reviewed different alternatives for the reconstruction of the breakwaters of IJmuiden. The outcome of both reports differs, because the cube is the less favourable alternative of this report and the most favourable of the CED report. Out of this thesis research follows the following recommendations.

- Placement rates and placement difficulties have to be better documented during the construction of breakwater projects. This information is valuable for the design of new projects.
- An alternative with an armour layer which consists of a single layer of cubes with density 2,800 kg/m^3 has to be worked out. This alternative could be competitive with the interlocking units. The

sensitivity analysed that an alternative with density 2,800 kg/m^3 can be more economical than an alternative with 3,800 kg/m^3.

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1 General Introduction

1.1 Introduction

Breakwaters are used worldwide to provide shelter from wave action, e.g. protected harbours, and to protect the coast. Due to the development of the shipping industry and the dimensions of the ships, breakwaters have become larger and longer, so the wave loads on the construction have also become larger. There are many different types of breakwaters that can be divided into categories according to the structural features:

1. Mound types
2. Monolithic types
3. Composite types
4. Special types

1. Mound types

This type of breakwaters consists of internal graded layers of stones. An outer armour layer composed of quarry material or concrete units, will resist the wave attack, but allows high wave energy transmission. Graded layers below the armour layers absorb wave energy and prevent the finer material in the foundation from being washed out. This type is applicable for a wide range of water depths, suitable for nearly all foundations and is readily repairable. A disadvantage of this construction is the rapid increase of material with increasing depth.

2. Monolithic types

Monolithic types of breakwaters have a cross section designed in such a way that the structure acts as one solid block. In practice, one may think of a caisson, a block wall, or a masonry structure. Monolithic structures require a solid foundation that can cope with high and often dynamic loads. An assessment of the reflection of wave energy and of scour at the toe of the structure has to be made.

3. Composite types

Composite types of breakwaters combine a monolithic element with a berm composed of loose element or monolithic element in a mound.

4. Special types

Besides the traditional breakwaters, there are also special structures available:

- Floating breakwaters;
- Pile breakwaters;
- Breakwaters composed of geotubes filled with sand.

All these types of breakwaters can be overtopped or non-overtopped. Overtopping is defined as the quantity of water passing over the crest of a structure per unit of time. Overtopped breakwaters only allow larger waves to wash across the crest. Crest elevation determines the amount of wave overtopping. The optimum crest elevation is determined by the minimum height, which provides the required protection. Non-overtopped breakwaters always prevent wave energy passing across the crest.

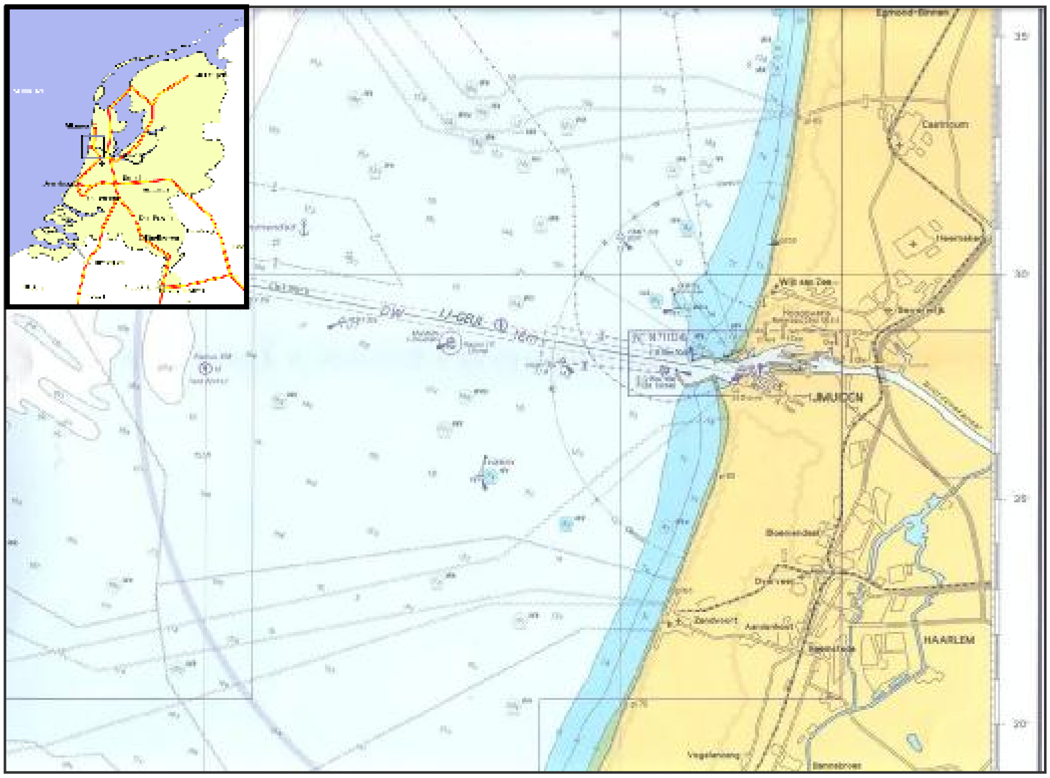


Figure 1-1 Nautical map of IJmuiden

This research focuses on an overtopped mound type breakwater with an armour layer of concrete armour units. Hydraulic stability and structural integrity are main items of the design process, but costs estimation is merely a small part of the design process. The costs of the different armour units are based on recent breakwater projects, but a detailed understanding of the costs is lacking, so the influence of costs on the choice of a design is marginal. The use of a real case study for a costs comparison is necessary, to get a realistic insight in the costs between the different armour units.

The present breakwaters of IJmuiden have stability problems with the armour layer and the toe construction. The Civil Engineering Division is developing solutions for the long-term at this moment. Reinforcing the breakwaters with armour units is one of these solutions, so this project can serve as a case study for a costs comparison.

1.2 Problem Analysis

During the design process of a breakwater several armour units are compared on structural and hydraulic stability. Cost calculations are made on unit prices and are not very detailed and a clear cost comparison of the construction costs of armour units is lacking. Other projects which have been performed already, serve as reference for the estimation of the construction costs. Knowledge about the life cycle costs (including maintenance costs) is also lacking. During the decision-making for a new design also non-technical, like experience with particular armour units, preference for national elements (e.g. the French government prefers Accropodes, because these are developed by a French company). In 2003 a new interlocking armour unit is introduced by Delta Marine Consultants, the Xbloc. A comparison on costs between this new armour unit and two other worldwide used elements, the concrete cube and the Accropode could add more information into the design, so a well considered choice can be made between the several armour units. To make a realistic comparison the reconstruction of the breakwaters of IJmuiden is chosen as case study. The situation of IJmuiden is exemplary for the situation of the ports in the Western world: shortage of space, high labour costs for construction and a lot of navigation, which will be influenced by the construction works.

Problem Definition of Thesis

At the moment the role of costs is underexposed during the design process. Costs of armour units are based on unit prices and not on Life Cycle Costs. A clear overview of all costs concerning the design, construction and maintenance is lacking.

For this research 3 armour units are chosen, concrete cubes, Accropodes and Xblocs and as case study is chosen the reinforcement of the armour layer of the breakwaters of IJmuiden. A double layer of cubes is used in this study.

Objective of Thesis

Find the most economical armour unit (Lowest Life Cycle Cost) for the armour layer of the breakwaters of IJmuiden. In every stage of the construction process the three armour units, cubes, Accropodes and Xblocs, will be compared only on costs. This case study has to show the structure of costs of the three armour units in general.

1.3 Plan of Research

Due to the available time, the study will concentrate on a section of the southern breakwater. From the head of the breakwater to section 2400 the part which is most exposed to storm conditions, this part of the breakwater is outlined in Figure 1-2. A cross section of the head and one cross section from the trunk will be studied. The most severe damage occurred at the outer slope, so the inner slope will be out of the scope of this research. Drawings of the head and trunk section are given in Appendix II.

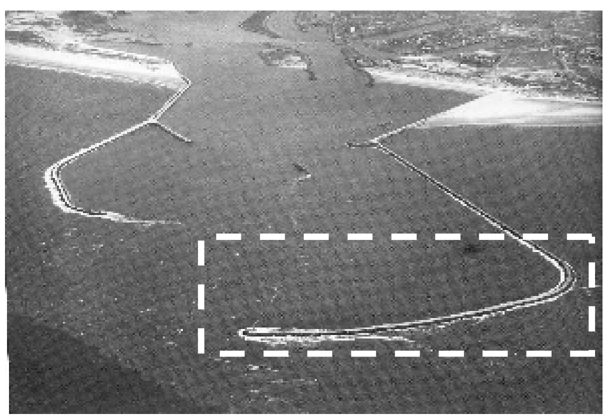


Figure 1-2 Project area

This thesis research is divided into three parts. In the first part an introduction is given about the thesis subject and information is given about breakwaters in general. The second part focuses on the case of the breakwaters of IJmuiden. The history and the problems of those breakwaters are described in this second part. New designs will be made for the comparison study between the three different armour units. The last step of this second part is the description of the execution phase and the calculation of the total costs. The third part uses the outcome of the research for conclusions specified on IJmuiden, but also general conclusions. This part holds also a sensitivity analysis, in which parameters and scenarios can be varied. Figure 1-3 shows the structure of this case study.

This report is organised as follows: Chapter 1 holds the problem description of this study and the main objectives. Chapter 2 describes breakwaters in general and all materials, parts, execution methods, equipment and also maintenance is listed in this chapter. The third chapter treats the history of the breakwaters of IJmuiden and the current problems and proposed solutions. Chapter 4 holds the designs with the three different armour units. In Chapter 5 the construction methods for all three armour units are described in detail and the total costs are calculated. In Chapter 6 a sensitivity analysis is worked out. The last chapter, Chapter 7, provides the conclusions and recommendations about the costs of the three armour units.

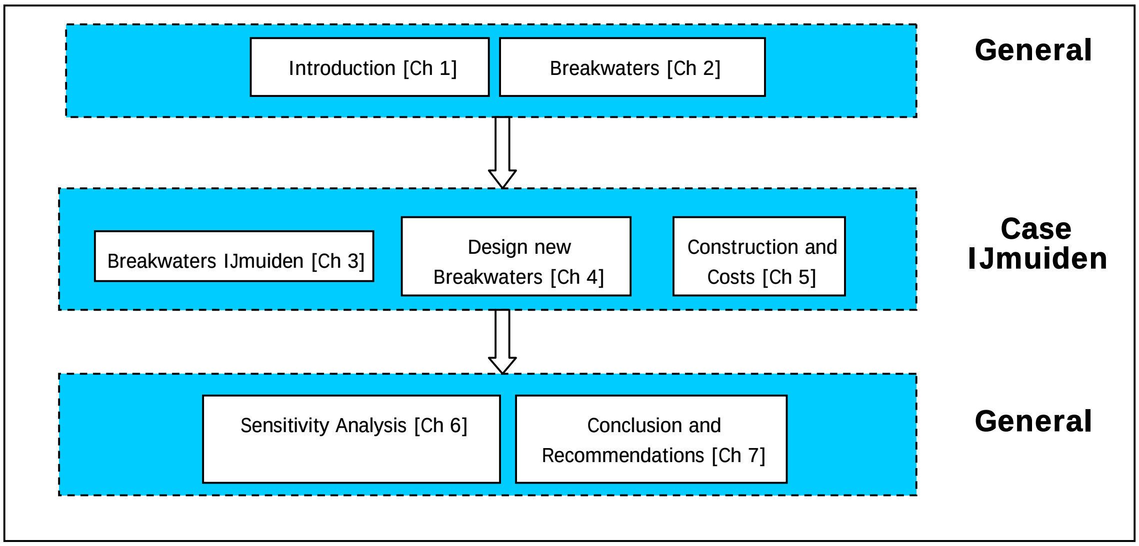


Figure 1-3 Organisation of the case study

2 Breakwaters in General

2.1 Introduction

This thesis research focuses on an overtopped mound type breakwater with an armour layer of concrete armour units. This type is composed of several parts of different materials, see Figure 2-1.

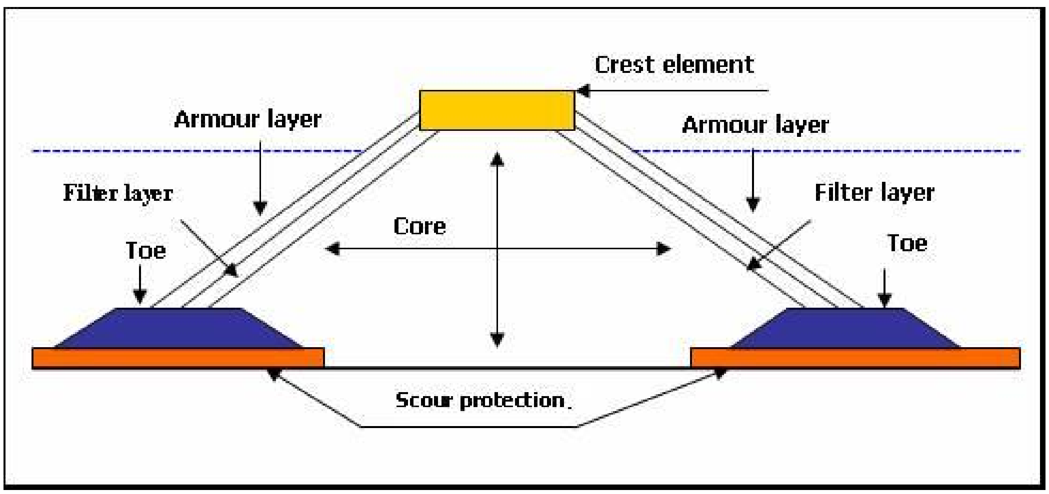


Figure 2-1 Cross-section of a typical breakwater

- **Scour protection**
  The base of the breakwater consists of soil protection to prevent scour in front of the structure, which could initiate geotechnical instability. The soil protection can be a granular filter or a geotextile filter.

- **Toe**
  The toe structure lends stability to the armour layer of the breakwater. The toe structure can be composed of quarry material or of a few armour units.

- **Core**
  The core forms the largest part of the breakwater and it consists of fine quarry material, which can be dumped easily by vessels

- **Filter layer**
  The filter layers form the transition between the core and the armour layer. The number and grading of the filter layers depend on the dimensions of the core material and the armour units. The breakwater is constructed following the filter rules.

- **Armour layer**
  The armour layer is the most important part of the breakwater, because it protects the total structure and it takes care that the breakwater fulfils its functions. The armour layer has to withstand wave forces and internal forces. In cooperation with the filter layers the armour layer prevents wash out of core material.

- **Crest element**
  The crest element is the top element from which construction and maintenance activities can be executed.

Chapter 1 gave a problem analysis, from which a problem definition and objective of this thesis research were listed. This chapter gives an overview about the aspect of breakwaters, design, construction and maintenance. In Section 2.2 the materials, which are used for breakwaters, are described. These materials are quarry stone, concrete armour units and other. Section 2.3 holds a description of the construction of breakwaters, in which all methods and equipment are treated. Environmental conditions play also an important role during the construction and the right working conditions are discussed in this section. The last section deals with maintenance of coastal structures, e.g. breakwaters but also failure mechanisms of the whole armour layer and individual units are treated. The next chapter gives a comprehensive description of the history of the breakwaters IJmuiden and holds the constraints for a new design.

2.2 Construction Materials of Breakwaters

2.2.1 Introduction

Coastal structures are constructed out of different kind of materials. Quarry stone has always been used for the construction of large structures, because it is a cheap construction material and it is easily obtainable. Quarry stone has limited dimensions, by which the demand for concrete armour units started. Concrete is also used for crest element and other parts of the breakwater.

2.2.2 Quarry stone

Rock can be divided into three primary groups (see Table 2-1), depending on their mode of formation:

1. **Igneous rock**
   Rock, which are formed by the crystallisation and solidification of molten silicate magma.

2. **Sedimentary rock**
   Rock, which is formed by the sedimentation and subsequent lithification of mineral grains, either under water or, more rarely, on an ancient land surface.

3. **Metamorphic rock**
   Rock, formed by the effect of heat and pressure on igneous or sedimentary rock for geological period of time with the consequent development of new minerals and textures within the pre-existing rock.

Igneous rock			**Sedimentary rock**			**Metamorfic rock**
Granite		Porfierite	Limestone	Limestone	Limestone	Gneiss
Grabbo		Basalt	Sandstone			Eklogite
Syenite		Basalt lava	Grauwacke			Kwarsite
Diorite		Basanite				

Table 2-1 Quarry stone used in hydraulic engineering [CUR 192 (1998)]

Different steps have to be taken before the quarry stone ends as construction material in a breakwater. When a quarry has been chosen, activities will start in the quarry. After the blasting quarry stone can go to the production or from the storage or directly be placed into vessels (Figure 2-2). This process is extensively described in following steps:

1. Extraction of quarry material;
2. Handling;
3. Transport;
4. Placement.

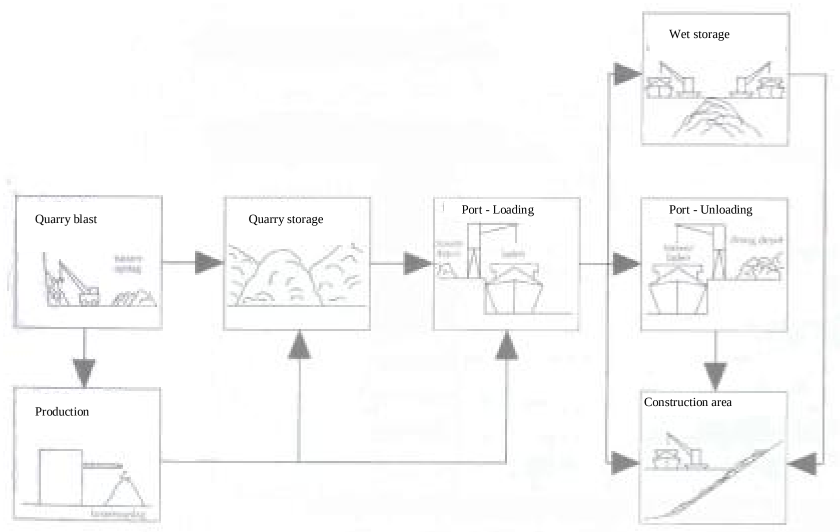


Figure 2-2 Quarry stone from the quarry to the construction

1. Extraction of quarry material

Quarries are positioned through whole Europe, but only the quarries in Great Britain, Scandinavia, Belgium, France and Germany are useful for the hydraulic works in Holland (see Figure 2-3). Other quarries in Europe, Canada and South-America are not competitive due the high transport costs. The quarry has to be in the neighbourhood of water, because the costs of road transport are very high. Rail transport is another option, for example the Betuwe rail connexion between Holland and Germany.

Marine gravels are a suitable source of materials for coastal and shoreline structures. Existing maps and charts should be used as a starting point to locate gravels and other materials. Optimum depths of deposit below the water surface lie between 10 and 30m. Depths of 40m are possible for some dredgers, but are not preferred, as production is low and costs are high. Also a wide area a few metres thick is most economically dredged because each load can be dredged without much manoeuvring, but conflicts with ecological requirements to minimise the area of seabed destroyed. For gravel production two principal types of dredgers are used: cutter suction dredgers and trailer hopper suction dredgers. Sea-going suction hopper trailer dredgers have been built up to 20,000-tonne capacity and are suited to high-volume contracts. Rock can be extracted by different ways.

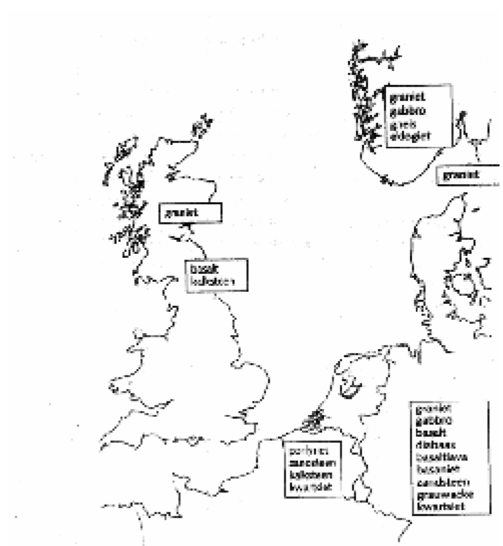


Figure 2-3 Quarries which can deliver material for Dutch coastal works

1. **Aggregate quarries**
   In aggregate quarries, blasting and other processes are aimed at achieving the optimum production of material of around 40mm and finer. Blast design will be aimed at achieving maximum reduction in size of material and minimal production of sizes suitable for armour stone. However, there will be a percentage of material from a blast that is too large for the primary crusher, and will either require further processing before crushing or be sold.

2. **Dimension stone quarries**
   These quarries produce large blocks of stone for masonry or monumental works. Production methods may exclude blasting. All small percentage of the stones, which is not suitable for its purpose, can be used for coastal engineering, if they have been carefully inspected.

3. **Dedicated armour stone quarries**
   There are a few permanent quarries that specialise in the production of stone for coastal engineering. In either case there is a need to maximise fractions of usable material in order to minimise costs and wastage. For quarries serving more than one contract the standardisation of specifications will aid the maintenance of stockpiles. For the permanent quarries, stockpiles are likely to exist and lead times could be low.

Blast design is a significant factor in securing desired fragmentation. However, it is unlikely that an aggregate quarry will consider changing its blasting pattern to suit a particular contract unless a significant proportion of the blasted material is required. Quarries may be prepared to set aside one particular area of the quarry of special material. Blast design largely depends on the skill and experience of the people involved. However, the sizes and proportions of rock particles produced will depend on the following factors:

1. The geological characteristics of the rock;
2. Properties and detonation methods of the explosives used;
3. The blast design (configuration and drilling pattern).

Selection process will differ, depending on whether material is selected from a blast pile or from a stock of oversize stones. Blast pile selection is likely to be made by using large machinery. The precise selection is made difficult by most of the stone being obscured by other material. Generally, the selection is done by eye against example blocks. Therefore it best to specify a wide range in weight for stones, ideally following the class limit grading system, see Table 2-2.

Grading	Class designation (kg)
Light grading	10-60 kg
	60-300 kg
	10-200 kg
Heavy grading	300-1,000 kg
	1,000-3,000 kg
	3,000-6,000 kg
	6,000-10,000 kg
	10,000-15,000 kg

Table 2-2 Grading of quarry stone; (EN 13381)

In general, stocking and re-handling of armour stones is considered to have high costs. However, there are benefits in placing equal sizes together and stocking them in a manner to aid re-handling. But there are also disadvantages:

- Breakage and rounding of stones;
- Mixture with other grading, other sorts of stones;
- Segregation.

2. Handling of quarry material

A variety of handling equipment exists, and its availability will depend on type of production unit, location and economic factors. The following equipment can be used:

- **Conveyor belt**
  The conveyor belt is used for transport of fine and light grading between fixed places and for the loading of ships (sees Figure 2-4).

- **Wheeled loaders**
  Suitable for handling lighter grading, but is not ideal for individual stones.

- **Hydraulic excavator**
  A back hoe or a front shovel can be used as a hydraulic excavator.

- **Grabs**
  Grabs may be fitted to either cranes or hydraulic machines. There are a number of designs of grabs or grapples.

- **Cranes**
  Cranes can be used with a variety of attachments including grabs.

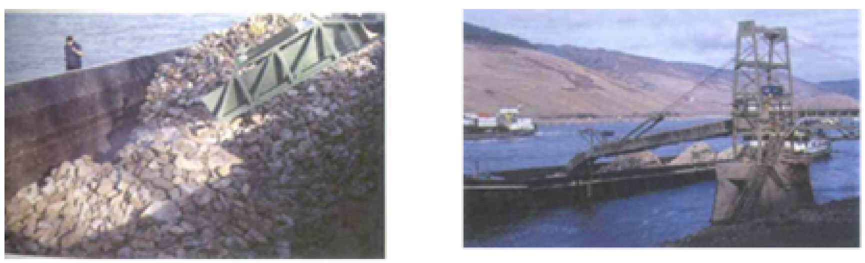


Figure 2-4 Loading of ship by a conveyor belt

3. Transport of quarry material

Three methods are distinguished to transport the quarry stone to the construction site:

1. **Road transport**
   Steel-bodied tippers can be used for the public way transport and dump trucks can be used in the quarry. Flat-bed Lorries can be used for large individual stones, which have to be secured.

2. **Water transport**
   Ships and barges are used for the inland water transport. Conventional coasters or barges of between 2,000 and 3,000 tonnes and draught 3-5m may carry stones up to approximately 6 tonnes in weight and they can be used for sea transport. Eyebolts, hooks and split boxes can be used for the unloading of quarry material. Drive on and off pontoon can be used for carrying all sizes of stone. A variety of pontoons are available, including sea-going versions and some are self-propelled. Specially strengthened pontoon barges can be used of up to 20,000 tonnes, with a draught of up to 8m.

3. **Train transport**
   Special adaptations have to be made to make the construction area accessible for train transport. Flat wagons can be used for the transportation of individual stones and box wagons for material up to 100mm in size due to difficulties with unloading.

The placement of quarry stone, step 4, is described in Section 2.3.

2.2.3 Armour units

In ancient times harbours were constructed in sheltered locations as river mouths, bays and areas sheltered by islands where the wave climate is calm. In time nautical trade developed and more harbours were built. Ports were constructed in less sheltered locations where port operations were hindered by the wave climate. In order to reduce the waves entering the harbour constructions were made of wood, stone or even concrete.

As ships became bigger, the required water depth in the ports increased to cope with the increased draught. The locations of the harbours therefore shifted seaward which resulted in increased wave exposure. In order to withstand these high waves breakwaters were built that consisted of an outer layer (armour layer) of large heavy rocks. However, the application of rock is limited as the maximum rock size is limited and in some parts of the world no large size rock or good rock quality was available. Therefore, for locations where a rock armour layer was impossible or very expensive, concrete armour units were developed. Nowadays most of the major ports in the world are protected using breakwaters with concrete armour units. These armour units are not reinforced, because the sea climate makes reinforcement difficult and expensive. This section treats only the concrete armour units, which are used in this thesis study. Besides the description of the armour units, this section also holds the production process of these units.

Cube

Before World War II the only armour units which were used were cubes. In cube armour layers (Figure 2-5), the stability against the wave action is derived from the weight of the armour units, a gravity block. Each block on the slope must be sufficiently heavy to withstand the wave forces. In the Netherlands the breakwaters of Hook of Holland and Scheveningen have been built with a double layer of concrete cubes. The moulds are simple and handling and storage is also quite simple. Nowadays cubes are used in single and double layers.

Figure 2-5 Concrete cubes

The breakwaters of IJmuiden have a single layer of cube armour, but the original breakwater is not designed with concrete armour units. The new armour layer of concrete cubes was a part of an emergency plan. After storm conditions these concrete cubes had been placed to enforce the breakwater, because the old design did not suffice.

Accropode

After 1950 a new stabilising mechanism was developed, armour units, which were interlocking. Because of the more slender shapes, these armour units not only use the weight of an individual element, but also of the surrounding elements. This leads to a higher hydraulic stability of the units.

Figure 2-6 Accropode

Apart from the interlocking feature, another advantage compared to the cube was the higher porosity of the armour layer, which is required for wave energy absorption, and reduction of wave run-up. The first interlocking unit on the market was the Tetrapode (1950). In 1980 the French engineering company Sogreah introduced the Accropode (Figure 2-6). As this unit was the first single layer armour unit, significant costs savings were made compared with the double layer units.

Xbloc

Delta Marine Consultants b.v., a Dutch engineering firm, started the development of the Xbloc in 2001. The Xbloc (Figure 2-7) is a single layer armour unit with a high hydraulic stability and a high structural stability. The element is easy to produce, to handle and to store.

Figure 2-7 Xbloc

Most of the breakwater construction projects of the last decades had a production and storage area in the neighbourhood of the construction site. This limits the transport costs.

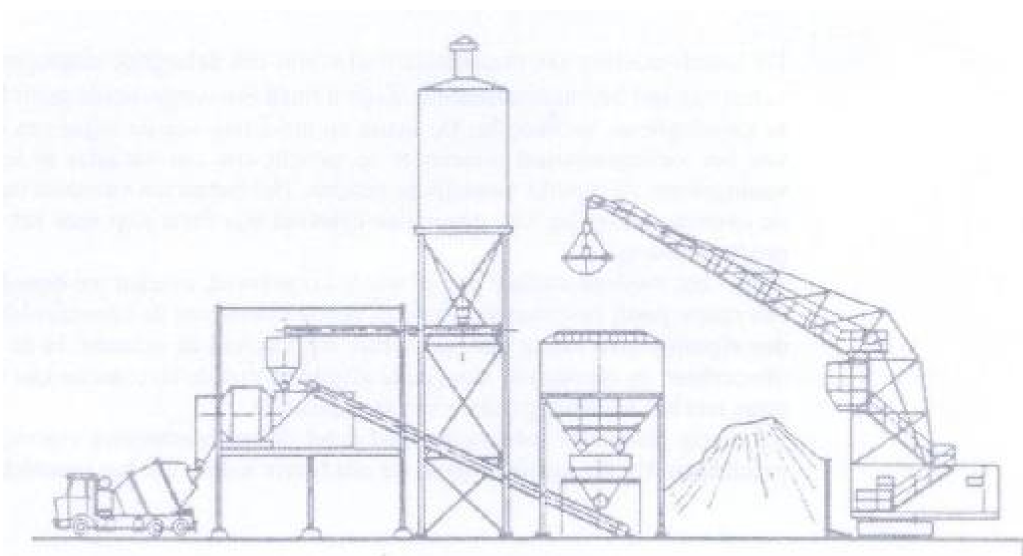


Figure 2-8 Lay-out of the batch plant

A concrete factory and other equipment are placed on the casting area. Also quality certificates and environmental licences are required. In densely populated areas, where land is scarce, pre-casting of armour units could be an option. This area is used for the storage of quarry material, the production lines of armour units. Availability of concrete factory with corresponding storage for aggregates depends on the price of concrete and construction permits. A batch plant on site is interesting when large quantities are required. The daily quantities determine the dimensions of storage of the aggregate materials, cement silos, cement mixers. The different aggregates have to be transported separately to prevent mixing during the transport. Also pollution of the aggregate has to be prevented. Coarse materials have to be stored in layers and the dump height has to be modest. During transport and storage of cement contact with moisture has to be prevented. All aggregate and cement are weighed and after the weighing process they can be mixed in the mixers. The whole process is computer-controlled.

Figure 2-9 Pouring of concrete in mould.

On the construction yard there are also workshops, stores, office accommodation and space for trucks and other equipment. For the vessels and sometimes for floating cranes a construction harbour and loading and unloading equipment are required.

The following consecutive activities make up the production process:

Step number	Activities
1.	Preparation of moulds for production process
2.	Pouring of concrete in moulds (several options for equipment)
3.	Hardening of concrete
4.	Striking the mould
5.	Numbering the units (production date and number)
6.	Transport from production line to storage area by shovel or gantry crane
7.	Up to 28 days of hardening after the casting on storage area.

Table 2-3 Casting process

Minor defects shall be repaired immediately after striking. The mortars for these repairs shall be at least of the same quality as the armour unit itself (strength and durability). The construction shall include measures to prevent shrinkage and ensure adherence.

The forms have to support the loading of the concrete mixture in the first part of the hardening process. A concrete pump or a bucket can be used for the pouring of concrete and than the concrete has to be compacted mechanically. In summer the concrete reaches enough strength after hardening of one day. When the evaporation of water in the mixture goes too fast, the chemical reaction will delay or stop, so evaporation of water has to be prevented.

Measures to prevent evaporation of water:

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

Figure 2-10 Gantry crane

During the winter period the units have to stay 2 days in the moulds, because the hardening goes slower due to lower temperatures. In winter special attention is required for the hardening process when the freezing point is reached, because the chemical reaction will stop and the quality of the concrete will not satisfy anymore. Measures to prevent freezing of the mixture are, e.g. heated concrete mixture or covering the units with plastic sails and steam.

2.3 Construction Methods of Breakwaters

2.3.1 Introduction

The choice of construction method depends on different factors:

- Dimensions of the breakwater (design);
- Environmental conditions;
- Bathymetry;
- Accessibility of breakwater;
- Availability of equipment.

Within these boundary conditions a choice can be made between different types of construction methods and types of equipment. Production rates and fitness of equipment play an important role in the decision-making process. A breakwater can be built by several methods; a distinction can be made between land-based and waterborne operations.

1. **Land-based operations**
   Cranes are positioned on top of the construction and trucks or pontoons supply them with quarry material or armour units. The cranes can be movable (on rails or on tracks) or non-movable. When the crane has built a new part of the breakwater, it moves to the end of the construction and starts again. For this construction method, the crest has to be high enough and the construction has to be wide enough to drive with two trucks. When a breakwater is too long, the transport time over the breakwater will increase and this option will be uneconomical.

2. **Waterborne operations**
   Low-crested breakwaters are usually constructed from sea, because land cranes can not work from the crest. This method depends on environmental conditions.

Combinations of the above mentioned methods are frequently used in coastal engineering. First the lower part of the breakwater is constructed by vessels and the upper part is built by land cranes.

This section gives an extensive overview of equipment, which is frequently used for construction of breakwaters. Environmental conditions influence the construction of breakwaters, because they can restrict the workability of equipment. All environmental conditions are described in this section.

2.3.2 Equipment

1. Land-based equipment

Cranes are positioned on the breakwater and usually dump trucks and flatbed trailers (Figure 2-11) transport the construction materials. Barges are sometimes used for this transport. Wheel loaders, backhoe cranes and bulldozers are used on the production and storage area of a project. Concrete mixers are used for the pouring of the moulds of the element.

Figure 2-11 Flat bed lorry

Figure 2-12 Left: Crane topping of to reach outside berm Right: Orange peel grab

The production capacity of the cranes is determined by the dimensions of the armour unit or the volume of stone which can be lifted by the crane and by the rotation and lifting speed.
For cyclic placement of relatively small stone quantities backhoes can be used, in which case cranes can be equipped with a clamshell or with orange-peel grab, see Figure 2-12.

A number of options are available for individual lifting of armour units (sometimes provided with eyebolts) which will be dependent on the armour unit itself and the handling required: chains, dogs, chains in slings, wire-rope slings (see Figure 2-13), clamps. Divers are required for inspection of placing of interlocking armour units under water and in the breakers zone.

Figure 2-13 Placing of Accropodes with sling on Fregate Island; Photograph DMC

2. Floating equipment

Working vessels and other floating structures have to be seaworthy, when they are working on a breakwater. A successful dumping operation realises the design layer thickness with a minimum rate of stones dumped. The dumping process and its result depends on the type of equipment used, water depth, current velocity and stone characteristics. Several types of self-unloading barges can be used:

- Flat deck and tilt barges;
- Bottom door barges;
- Split barges;
- Side unloading vessels;
- Crane ship and floating crane;
- Pontoons.

Flat deck and tilt barges

Flat deck barges and tilt barges are propelled by tugs. The quarry material is dumped from the flat deck barge by a bulldozer (see Figure 2-14). The tilt barge tilts its deck and so the quarry material can be dumped.

Figure 2-14 Flat deck barge

Bottom door barge

The bottom door barge can only be used for fine gradations, because the mechanism of the doors is vulnerable for the loading and unloading of quarry stone. Arching can be a problem during the unloading of quarry stone. The doors of a bottom door barge are located in the keel, so during opening they are positioned under the barge. To prevent problems with the draft of the barge, the door can be positioned higher in the barge; it is called an "oplosser".

Split barge

The split barge is appropriate to process the gradation quarry stone till 1,000-3,000 kg (see Figure 2-15). The bottom door barge and the split barge are used for dumping of large amounts of quarry stone in a short time, less than a minute, on a limited area. To control the dumping of quarry stone, the width of the opening can be adjusted, but can not be too small due the danger of arching. During the dumping of large quantities the quarry stone does not segregate, because it falls as one unity. The displacement of water, induced by the dumping, can initiate currents which can displace sand or other material.

Figure 2-15 Split barge

Side unloading vessel
The side unloading vessel is specially designed for quarry stone and is appropriate for every standard grading, see Figure 2-16. The vessel dumps the quarry material by moving hydraulic movable slides and chain dumpers dump their cargo by a net of chains, which move slowly along the deck. Chain dumpers are not appropriate for the dumping of heavy grading, because the strength of the chains is limited. The dumping process of the side unloading is gradually, so the separate stones can fall with different velocities and the whole dump can segregate.

Figure 2-16 Side dumping vessels; Photograph Royal Boskalis Westminister

Crane ship and floating crane
The crane ship and the floating crane can both handle all standard grading (Figure 2-17 and Figure 2-18).

Figure 2-17 Left: Crawler crane on rails. Right: Back hoe on ship; Photograph L. Paans &Zonen

Figure 2-18 Crawler crane on pontoon; Photograph Stemat

Pontoon

Pontoons can handle all standard grading and they are available in different sizes. A dose pontoon is divided in different compartments and every compartment has its own slide, so an equal layer can be dumped (Figure 2-19).
A stone pontoon is a normal pontoon where the cargo is protected by steel walls to prevent loss of quarry stone. A wheel loader can unload the quarry stone into the construction.

Figure 2-19 Dose pontoon; Photograph L. Paans & Zonen

3. Jack-up platform

A Jack-up platform (Figure 2-20) can be used in deep water when the hydraulic conditions are rough. A Jack-up platform moves by moving his piles and is less vulnerable for wave attack, because the topside can be lifted above the waves. Elevating during high waves is not possible.

Figure 2-20 Two jack-up platforms

2.3.3 Environmental conditions

Extreme hydraulic, meteorological and geotechnical conditions prevent continuously working of a contractor. The amount of workable days per year diminishes considerably by these events. To determine the workability of the equipment the wave climate and the meteorological data of the last years have to be investigated.

Geotechnical data
A geotechnical research is recommended, because settling of the sea bottom and bearing failure can cause dangerous situations, like the displacement of equipment or the construction itself. Also settlements of the construction itself can cause instability of the construction.

Meteorological data
Large temperature differences and rainfall can influence the hardening process of the concrete armour units badly; so proper meteorological data are necessary to take precautions. During the construction it is important to have the latest information about the weather conditions. A weather prediction system on the site is valuable for the continuing of the construction. In Holland the KNMI and Meteoconsult provide daily weather forecasts.

Hydraulic and nautical data
A distinction can be made between land-based operations and waterborne operations.
For land-based operations (e.g. using dump trucks) the elevation of working area and haul track is important. This elevation should preferably be 1.5m above water level; to avoid problems due to possible wave induced splash and spray. In a marine environment, a tidal analysis will reveal the effective operational time. For waterborne operations a distinction can be made between operational site conditions in general and in more detail the effect of environmental conditions on stone placements.

1. **Operational site conditions**
   Operation site conditions are divided into the following aspects:

   Current, waves and wind conditions
   Dumping is preferable carried out around slack tide. Positioning is achieved by anchoring system or dynamic positioning system. When dumping in a (tidal) current, apart from displacement and spreading in the direction of current, also a significant effect on segregation may be expected. For common types of equipment current should not exceed maximum values of 1.5 to 2.0 m/s. Downtime caused by waves and wind is mainly determined by their influence on the positioning accuracy of the stone dumping vessel and through this on the accuracy of stone placement rather then on operational limitations of the equipment.

   Available water depth
   The maximum height for dumping of material is determined by three criteria:
   1. The maximum draught of barges or pontoons, plus a safety clearance for heave.
   2. The available manoeuvring space and presence of structures may restrict the use of floating equipment.
   3. The loss of material; especially for winter construction, this loss, occurring during typical winter storms, should be limited to acceptable values.

   Seasonal influences
   Construction is not possible during the winter season with severe wave conditions. In case the construction period is several years, the construction has to be protected by a temporary armour layer during the winter season.

   Shipping
   During the construction period the construction vessels may not hinder the shipping.

   Visibility
   Although modern radar and positioning systems usually enable stone placement to proceed at reduced visibility, operations in confined areas (entrance channel) and close to structures will be very risky during fog.

2. Effect on stone placement

Dumping usually aims at placement of a certain amount of stones under a specified profile and at a prescribed position on the bed. The amount can be specified as a mass (kg/m^2) or as a layer thickness (m). The accuracy of dumping is expressed in terms of differences between the target and as-dumped profiles. Apart from the type of equipment, the accuracy depends on:

- Waves and wind;
- Water depth;
- Current velocities.

Waves and wind

Generally wind waves should not exceed a height of H_s = 1 to 1.5m, whereas under swell conditions wave height beyond H_s = 0.5m can already impose restrictions on the dumping. When working with barge-mounted cranes, the maximum wave height is limited by the ringer mechanisms and derricks. Cranes are normally not designed to take any lateral forces such as caused by swinging loads due to barge motions and for this reason maximum allowable tilts should not exceed 1 degree.

Water depth

The spreading width of the stone mass on the sea bed is affected by the water depth. The behaviour of the falling stones during the dumping process is different for direct dumping and controlled placement. After direct dumping the stone will act as one bulk and when it hits the bottom it will spread all over the bottom, leaving a minor quantity at the desired spot.
With side-dumping vessels and cranes a more accurate placement can be achieved by controlled placement.

Current

Currents also determine the position of dumping, because currents change the fall pattern of dumped stones and segregation occurs.

2.4 Failure Mechanisms of Breakwaters

2.4.1 Introduction

A distinction can be made between total failure of the structure, Ultimate Limit State, or the situation when the structure does not fulfil his functions, Serviceability Limit State, e.g. more downtime in the harbour basin. Also a distinction can be made between failure of the total breakwater and failure of individual units.

2.4.2 Failure of breakwaters

Failure can be simply defined as the exceedence of a predefined limit state, which occurs when the loading exceeds the strength. When this exceedence occurs, a failure response of the structure (or parts of it) can be defined. Failure can occur both during construction and the lifetime of the breakwater. Typical loadings and responses for rock are wave height and displacement relative to the as-built position. Both loading and response are functions of time. The limit when the breakwater collapse is called the Ultimate Limit State (ULS). The limit for insufficient functioning of the breakwater is called Serviceability Limit State (SLS).

A breakwater consists of different parts, which could iniate individually failure of the whole breakwater. Failure modes of breakwaters:

1. Failure of armour layer;
2. Failure of toe construction;
3. Failure of inner slope of the breakwater;
4. Failure of subsoil (geotechnical failure).

Besides these specific failure modes, there are also overall failure modes:

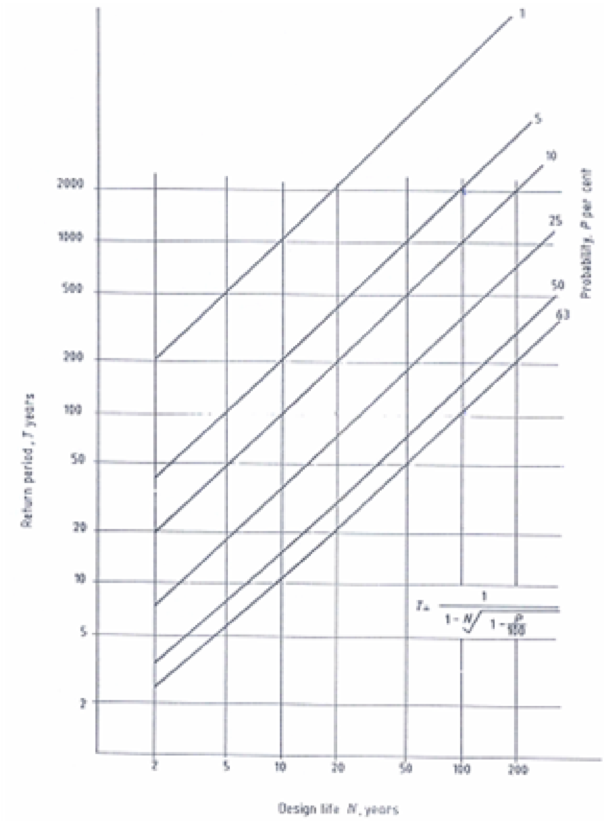
1. Design based on wrong data

- Insufficient data;
- Errors in measurements;
- Improper data handling and processing;
- Modelling (wrong input data).

2. Construction errors

Breakwaters are designed to withstand extreme waves caused by storm events that occur during the design lifetime of the structure. The design wave height is derived from past storm events and can be exceeded during an extreme storm event within the lifetime of the breakwater. Exceedance of design waves might cause damage to the breakwater; however a severe damage or complete failure shall be prevented. Therefore, the design formula should contain a safety margin to decrease the effect of an underestimated wave height.

Nowadays in breakwater design, a 1 in 100 year storm is adapted, which gives a certain H_s.
The British Standard even advises a once in 50 year storm. The result of the design life of 100 years and the return period of 100 years is a probability of failure of 63 %, Figure 2-21. The chance that the design H_s is reached during the lifetime of a breakwater is very high (63 %). Special attention is required for the conversion of wave data on deep water to wave data near the breakwater.



T = return period of a particular wave condition in years
P = probability of a particular extreme wave condition occurring during design life N years.

Figure 2-21 Relationship between design life, return period and probability of exceedence

The three armour units have their own damage function. The 2-layer cubes and the interlocking units differ in damage development during storm conditions. Damage of 2-layer cubes can be tolerated, because it can be repaired quickly, but damage to interlocking armour layer can not be tolerated. In text below first the damage development of quarry stone will be treated and subsequently the damage development of cubes and interlocking units.

Quarry stone

The traditional multi-layered breakwater and the berm breakwater, which both totally consist of rock, allow a damage of 5 %. Van der Meer has defined a clear definition of damage. In his PhD thesis at Delft University, Van der Meer [1988] succeeded in presenting an approach on irregular waves, which has been accepted worldwide.

Initially, this was expressed by the parameter in which:

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

A = the erosion area in a cross section in [m²]

$$D_{n50} = \left(\frac{W_{50}}{g r_r}\right)^{\frac{1}{3}} = \left(\frac{M_{50}}{r_r}\right)^{\frac{1}{3}} \quad [m]$$

W_{50} = mean weight of armour stones [N]
r_r = density of armour stone [kg/m^3]

The erosion of area A (see Figure 2-22) is partly caused by settlement of the rock profile and partly by removal of stones that have lost stability. Since in the formula for S the erosion area is divided by the area of the armour stone, the damage S represents the number of stones removed from the cross-section, at least when permeability/porosity and shape are not taken into account. In practice, the actual number of stones removed from a D_{n50} wide strip is between 0.7 and 1.0 times S.

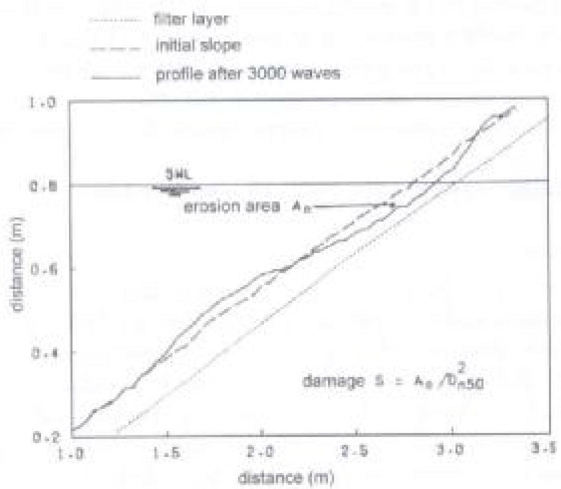


Figure 2-22 Damage (S) based on erosion area (A)

Damage	Damage value S
Initial damage (needs no repair)	2
Intermediate damage (needs repair)	3-5
Failure (core exposed)	8

Table 2-4 Damage development with slope 1:1.5

Concrete elements

All formulae for concrete units except the Accropode and Xblocs are based on a slope of 1:1.5.

Since the mechanical strength of concrete may play a role, it is useful to distinguish damage due to actually displaced units. A value N is defined, which is the number of units displaced from one strip of the breakwater with a width of D_{n50}. The relation with S is established by the permeability/porosity. When the number of displaced units is counted, the settlement of the mound is omitted from the considerations of damage. The number N is often used when studying the stability of armour layers consisting of concrete units.

Van der Meer chose to express the stability in terms of $Hs/\Delta D_{n50}$ and then investigated the influence of several parameters that he considered relevant. Table 2-5 gives an overview of damage values.

<u>Cubes</u>

$$H_s / \Delta D_n = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \qquad H_s / \Delta D_n = \left(6.7 \frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} - 0.5$$

N = number of waves
N_{od} = number of displaced blocks
N_{or} = number of rocking blocks
N_{omov} = N_{od} + N_{or}
s_{om} = $2p / g * H_s / T_m^{\;2}$ (fictitious wave steepness)
T_m = mean period
Δ = relative mass density (ρ_a -ρ)/ρ where ρ_a is the mass density of stone and ρ is the mass density of water

Damage development	N_{od}
Initial damage (needs no repair)	0.0 - 0.5
Intermediate damage (needs repair)	0.5 - 1.5
Failure (core exposed)	>2

Table 2-5 N_{od} value

Interlocking armour units

Damage to interlocking units, Accropode and Xbloc, is not tolerated, because damage to the armour layer can initiate rapid failure of the total armour layer. The optimal slope for both units is 1:1.33. Van der Meer formula does not take into account the storm duration and the wave period for Accropodes and Xblocs. The values of Van der Meer are not used by Sogreah and DMC, because the values of Van der Meer are believed to be too positive.

Accropode

Design value $\frac{H_s}{\Delta D_n} = 2.6$

Start of damage $\frac{H_s}{\Delta D_n} = 3.25$

Failure (core exposed) $\frac{H_s}{\Delta D_n} = 3.8$

Xbloc

[Significant] design wave height, H_d	**Effect on Xbloc slope**
1.0* H_d	Slope is completely stable
>1.1* H_d	Start of rocking
>1.25* H_d	Start of damage [1 or more units displace]
>1.3* H_d	Continues damage [further units displace]
>1.4* H_d	Start of progressive failure

Table 2-6 Limiting wave conditions for design purpose

This means that when the wave in the design of an Xbloc armour layer is exceeded up to 25 % only start of damage may occur. Failure of an Xbloc armour slope will only occur when the design wave height is exceeded with more than 40 %. The design criterion for start of damage is similar to the criterion used in the model tests of construction with Accropode armour units.

Design value for Xbloc: $\frac{H_s}{\Delta D_n} = 2.8$

Start of damage $\frac{H_s}{\Delta D_n} = 3.5$

Start of failure $\frac{H_s}{\Delta D_n} = 3.9$

2.4.3 Failure of individual armour units

Besides the failure of the total structure, individual armour units can also fail by different mechanisms. Two important failure mechanisms, rocking of armour units and a temperature gradient in the concrete, are described here.

1. Rocking

Although the size of the concrete armour units seems unlimited, there is a limitation in practice. Since the units are in principle not reinforced, to avoid corrosion problems, the structural integrity depends largely on the tensile strength of the concrete. Increasing the linear size of the units leads to an increase of mass and forces proportional to the third power of the size. The cross-sectional area that provides the structural strength increases, however with the square of the size only. Since the strength is constant, a larger block becomes more and more vulnerable to structural damage when the actual tension exceeds the available tensile strength. Rocking, the movement of units against each other during wave attack can start the failure of the armour layer.

2. Temperature gradient in the concrete

During the hardening of the units a lot of heat is generated, because of the large mass of the units. The heat is concentrated in the heart of the units and when the mould is removed, the surface of the units can cool rapidly, which leads to large tensions in the fresh concrete. In many cases these temperature gradients initiate cracks. It can be easily be demonstrated that such cracks have a large influence on eventual strength of the unit and thus on breakage of units during handling or during exposure to high wave loads. The problem of temperature gradients plays a very dominant role in places with a strong wind and a low humidity (cooling of the surface) and in regions with a large range between daily maximum and minimum temperatures. The latter occurs in areas with a desert climate. This problem can be tackled by the following measures:

- Reduction of the cement content;
- Use of low heat cement;
- Use slower hardening cement;
- Insulation of the units after removal of the moulds;
- Cooling of units.

2.5 Total Costs of Breakwaters

2.5.1 Introduction

The coastal structures are expensive, because they have to be designed robust to the environmental conditions. Costs are not only made during the construction, but also during the lifetime of those structures maintenance/operation costs are paid. First this section describes the construction costs of a breakwater

2.5.2 Construction costs

The costs of the construction are a function of the total construction time and can be divided in direct and indirect cost.

1. **Direct costs**
   All costs which are directly related with the construction of the structure, like costs of equipment and material. The direct costs consist of three parts:

   - **Constant costs**
     Material costs are determined by the market prices and the total quantities of material. A variation in both quantities and prices occurs. Design drawings can provide information about the theoretical required quantities of material. In practice more material is required. Causes of this difference between practice and theory:

     1. Uncertainties of the position of the sea bottom;
     2. Settlements of the soil;
     3. Settlement of the quarry material;
     4. Dumping of material outside the design profile;
     5. Loss of material during transfer and transport;

     Also the prices vary due to location of the construction site, transport and market situation.

   - **Mobilisation and demobilisation costs**
     Before the construction of a breakwater starts equipment, cranes, vessels and other, have to be mobilised to the construction area. At the end of the work period all equipment is demobilised from the breakwater.

   - **Time-related costs**
     The costs of labour, fuel and equipment are time related, so their total costs can be calculated by multiplying the construction time and the price of the equipment. To make a good estimate of the construction time a probability density function has to be drawn up. The mean value and the standard deviation of the production rates of different construction methods determine the construction time. Different scenarios, optimistic, pessimistic and the most realistic are considered to make a proper estimate of the construction time. The number of unworkable days makes this function discontinuous. The construction can be delayed due to several factors and delay causes extra costs. When the construction time exceeds contract construction time penalties have to be paid and to diminish the delay time, more equipment and labour have to be involved.

     Not only the construction time is stochastic, but also the equipment costs are stochastic. The variance in equipment is determined by:

     1. Depreciation and interest;
     2. Maintenance and repair;
     3. Fuel and lubricants;
     4. Wages and insurances.

2. Indirect costs

Indirect costs are the costs, which are not directly related to the construction of the structure or a part of it, like offices and other staff. These costs are divided over the construction projects, which a contractor executes.

2.5.3 Maintenance costs

Recently constructed breakwaters have used a value of 2% of the initial construction costs for the annual maintenance costs. This value is acceptable for a preliminary design. Maintenance costs are determined by two parts of the maintenance programme:

1. Monitoring

First inspection and monitoring of environmental conditions and state of the structure are executed. Subsequently appraisal of monitoring data predict structural deterioration and to assess compliance of performance with pre-determined standards, such as service level and planned lifetime.

2. Maintenance

Maintenance of a breakwater is the repair or replacement of components of a structure which life is assessed to be less than the overall structure.

An example of a maintenance cycle is given in the figure below. First this section will deal with different kind of maintenance strategies and subsequently different types of monitoring are described.

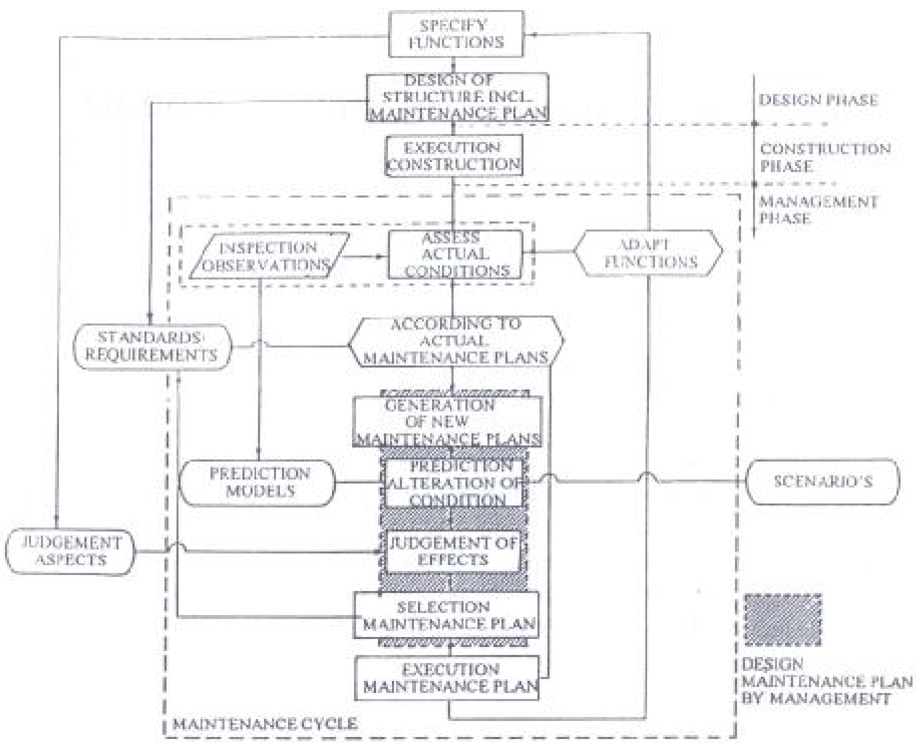


Figure 2-23 Maintenance cycle; lifetime stages of a coastal defence work

Maintenance strategies

Before a maintenance programme can be drawn up, failure limits of the structure have to be known. A distinction can be made between total failure of the structure (Ultimate Limit State) or the situation when the structure does not fulfil his functions (Serviceability Limit State), e.g. more downtime in the harbour basin.

Globally two types of maintenance are adapted in practice:

1. **Corrective maintenance after failure**
   No cost of preventive maintenance and high cost of corrective maintenance

2. **Preventive maintenance before failure.**
   Frequent maintenance means high cost of preventive maintenance and low cost of corrective maintenance

The smaller the uncertainty of the deterioration is, the smaller the uncertainty of time of failure. At that moment the preventive maintenance can be planned better. The most optimal intervention interval is a bit smaller than the expected lifetime, when there is almost no uncertainty of the deterioration of the structure. With a high inspection frequency an intervention level close to the failure limit can be adapted and with a low inspection frequency an intervention level far from the failure limit. When inspections are used in cost analysis, a more extensive model has to be used. Preventive maintenance is favourable, because the costs of corrective maintenance (due to the costs of the consequences of failure) are much higher than the costs of preventive maintenance. Preventive maintenance has to be carried out, when the state of the structure drops below a strength level, the preventive intervention level. To minimise the maintenance costs, different maintenance strategies can be chosen.

1. **Failure based maintenance**
   Repair is only undertaken if the structure or a part has failed. This type of maintenance is advisable if the consequences of failure are very limited.

2. **Time based maintenance**
   This type assumes that the structural state deteriorates according to a known function of time. Repair is applied after a certain time has elapsed. Time based inspection concerns the simplest approach, where inspections are performed at regular intervals. The rational minimum interval, based on the changing of the season, is 6 or 12 months. An inspection is planned some time before the state is predicted to reach a certain minimum value. If the inspection confirms the prediction, repair will be planned.

3. **Load-based maintenance**
   Structural deterioration is caused by heavy loading (e.g. storms). Loading has to be monitored and repair will be carried out after a certain number of heavy loadings took place. The monitoring of the environmental loading conditions should be performed on a continuous basis. In some cases the detailed measurements may be limited to severe conditions, to reduce the amount of data to manageable quantities.

4. **State-based maintenance**
   This type depends on the inspection of the physical state of the structure. Three situations can be viewed (see Figure 2-24):

   1. Do nothing, because the condition of the structure does not drop below the preventive intervention level (action-limit);
   2. Preventive maintenance have to be carried out, because the condition of the structure drops below the preventive intervention level (action-limit), but not the failure level;
   3. Corrective maintenance has to be carried out, because failure has taken place.

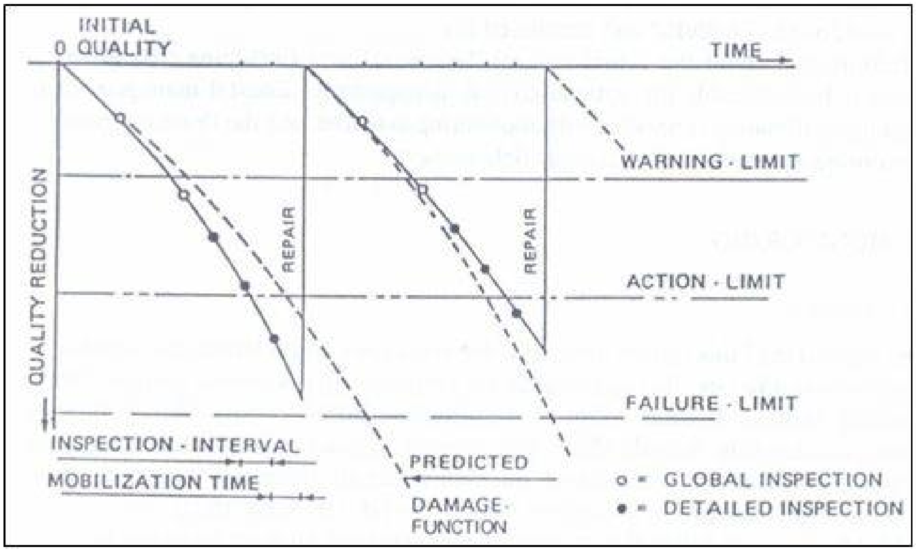


Figure 2-24 Preventive condition-based maintenance

Preventive condition-based maintenance strategy distinguishes three limits of quality levels of the condition parameters. The margin between the action limit and the failure limit will depend on the inspection frequency and the mobilisation time for the construction of the repair measures. Finally it should be realised that the interval between inspections may increase during the lifetime of the structure and the structure's degressive deterioration process.
On the other hand, decreasing intervals between inspections may be required in case of progressively reduced resistance to failure as it degrades with time.

From this classification of maintenance strategies it may be clear that the choice of the maintenance strategy (see Figure 2-25) depends on:

- Predictability of the structural deterioration (SLS);
- Cost of inspection and monitoring;
- Cost of repair;
- Consequences of failure;
- Availability of methods to measure the physical state accurately.

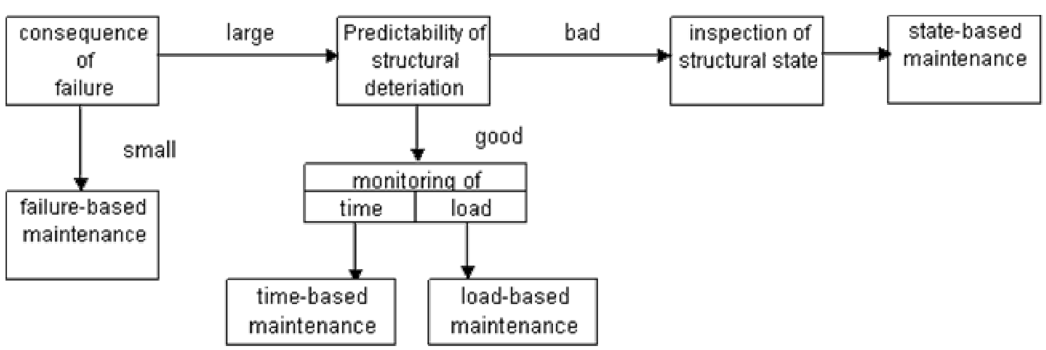


Figure 2-25 Selection of maintenance strategy

Looking at only the structure itself, maintenance can be regarded as the repair and replacement of the component elements or of complete sections of the structure, which have failed. Most economic maintenance strategy is determined by the optimal balance between the costs of preventive maintenance and the costs of corrective maintenance. Inspection strategy should provide information on what (which damage pattern characteristics), where (location and depth), when (how often), how (which measuring method) and by whom (which organisation) the data should be collected and evaluated.

Monitoring

Monitoring of a breakwater starts even before the construction and it will go on during and after construction. These activities include documentation of the situation before construction (reference state), construction records and periodic post-construction inspections. After construction topographic and hydro graphic survey should be carried out at quarterly intervals and also after storms. The monitoring should include periodic inspection of the structure as well. The monitoring program should consider not only data collection, but also analysis methods and associated costs once the data are obtained. Different types of monitoring are possible:

1. Monitoring methods of the structural state of the breakwater

- **Crane and ball survey**
  A mobile crane with sufficient reach is used for the crane and ball survey. The survey is carried out by positioning at each station with the boom perpendicular to the breakwater. The profile of the armour units can then be measured by suspending the ball from the boom by a calibrated staff. A DGPS satellite system is mounted on the top of the boom to fix the positions at which the levels were taken (see Figure 2-29).

- **Visual inspection**
  Visual inspection is done by climbing over the slope with armour units during low spring tide and small waves. The measured positions are then related with the ball survey lines

- **Diving inspection**
  A diving inspection is undertaken by a diver in the water and one assistant on the breakwater to record the position and state of the underwater part of the breakwater. The diver swims along the toe of the breakwater from the roundhead along the outer slope.

Figure 2-26 Inspection by diver with light and camera

  Poor visibility can reduce the effectiveness of the breakwater diving survey to counting broken units and recording those units which had moved away from the breakwater toe.

- **Side scan sonar**
  Side scan sonar is a sonar device that can look sideways. This can be compared with radar, but uses ultrasound echoes instead of electromagnetic pulses. The sound pulses are usually on frequencies between 100 and 500 kHz. A higher frequency gives better resolution but less range.

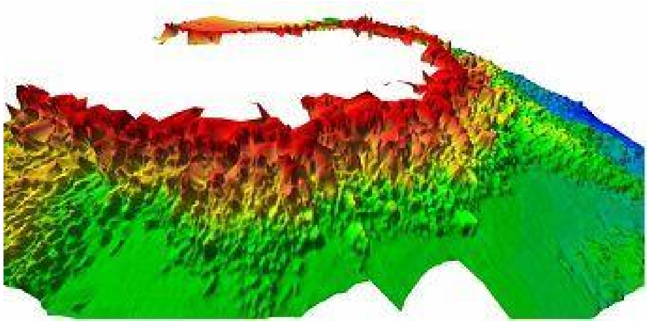


Figure 2-27 Left; Profile of Northern breakwater IJmuiden by side scan sonar Right; Side scan sonar

- **Photographic survey**
  This process is now recurring on smaller scales, with image mosaics, recorded from helicopters, aircraft, and balloons using GPS to log viewing positions (see Figure 2-28). These images are then projected onto digital terrain models of the area of interest. By repeating flybys at well-chosen intervals, changes, which would otherwise be too slow or subtle for the human eye, are clearly resolved. In South-Africa annual harbour breakwater surveys use helicopters to survey with differential GPS for imaging. Both digital and analogue images are recorded on pre-determined flight paths. The primary goal of this breakwater-monitoring is to quantify damage to breakwaters. Broken and missing armour units, as well as individual displacement of these units are quantified on an annual basis, or frequently in the case of major storm damage. Flight paths following GPS stored way stations are flown annually with images typically recorded at 25m intervals along the breakwater from a position, which is normal to the plane, defined by the bank of armour units. This image sequence is digitised and stored as the digital reference breakwater. This image sequence defines the reference frame against which future breakwater images are compared.

Figure 2-28 Digital imaging by helicopter; Crane and ball survey, photograph CSIR

2. Measurement of environmental conditions or loadings
Measurement of wind climate, wave climate and water level is necessary to predict the loading of the breakwater.

3 Breakwaters IJmuiden

3.1 Introduction

In the previous chapter a description of all aspects of breakwaters in general has been given. This chapter concentrates on all aspects of the breakwaters of IJmuiden, such as the design, construction and maintenance of the present breakwaters. The constraints of the design as well the construction are treated in this chapter. This detailed information is used in the next chapter to develop preliminary designs and a construction programme. Section 3.2 describes the old design of the breakwaters of IJmuiden and Section 3.3 contains information about the execution phase and accompanying problems. Section 3.4 treats the maintenance of the breakwater from completion of the breakwater until now and also the long-term solutions for the problems of the breakwaters are described. Section 3.5 holds the constraints of design and construction.

3.2 Design of Present Breakwaters IJmuiden

From 1868 until 1876 the old breakwaters of IJmuiden were built as a part of the fairway connection between the North Sea and Amsterdam. These breakwaters were high vertical walls, which were build of large concrete blocks. The main function of those breakwaters was wave reduction for shipping and visual guidance. In 1956 it was decided that the breakwaters had to be extended to fulfil their functions in the future, because dimensions of ships became larger and ships had difficulties to navigate into the harbour basis. At that time there was also a large amount of sediment and shipwrecks of World War II in the harbour mouth. The new breakwater had to satisfy constraints, divided in different categories:

- Social;
- Environmental;
- Financial;
- Functional;
- Technical.

Only functional and technical constraints are treated in this chapter. The functional constraints were translated into the following functions of the breakwater:

- Wave reduction for shipping;
- Guidance of the currents for shipping;
- Protection of the entrance channel against sedimentation;
- Visual guidance for shipping.

Out of these functions the following requirements with respect to the design were listed.

1. Dimensions

The new harbour mouth had to be accessible for ships of 70,000 – 80,000 DWT and the design of the breakwaters took also the development of larger ships into account. This resulted in the following dimensions:

- Depth of outer harbour must be -15.5m N.A.P.
- Width of the mouth of the harbour had to be at least 400m to have enough space to manoeuvre.
- Incoming ships which are sailing with 8 knots/hour required an extension length of 2,000m.

2. Nautical criteria

Wave and current influence have to be reduced in the harbour mouth to manage the ships safely into the harbour. The new design has to fulfil the following criteria:

- Prevent the amount of dredging in the future;
- A small current gradient across the entrance;
- Damping of wave energy in the harbour basin.

In 1956 Delft Hydraulics started the research on different lay-outs of the breakwaters and the cross sections of the breakwaters. The results of that research are summarised here:

- An asymmetric outer harbour, where the Southern breakwater head is positioned 3,154 metres into the sea and the Northern head 2,154 metres and the width of the harbour entrance is 750m (Figure 3-1). The construction level of the breakwaters varies between -10m to -15m N.A.P.
- Open "high" dams with rubble mound core, a crest with heavy concrete elements and at both sides armour layer of 2 layers of Tetrapodes.

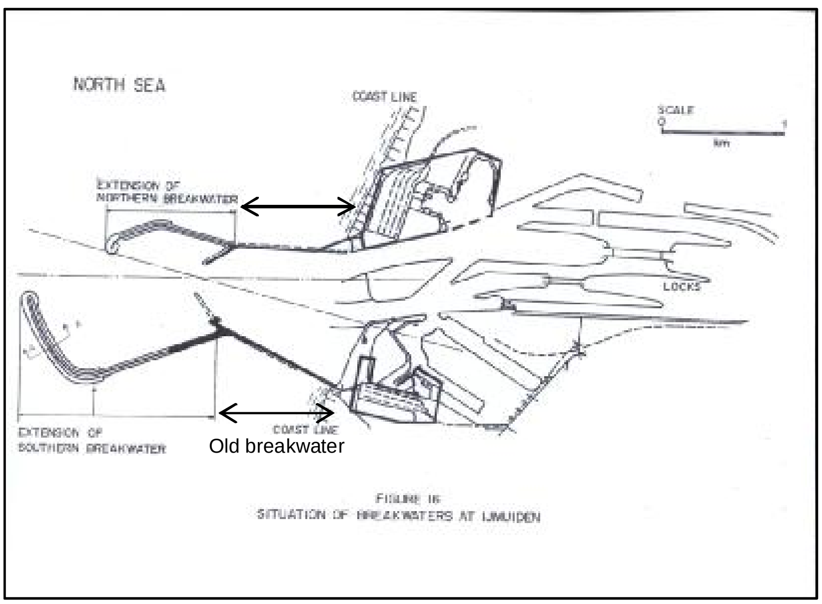


Figure 3-1 Top view of IJmuiden breakwaters

Model studies showed that the Tetrapodes at the inner slope of the breakwater were not stable during overtopping of waves. A new armour layer of stone asphalt was proposed, because of the failing of the armour layer at the inner slope. Eventually the armour layer of the total breakwater was constructed with stone asphalt because of economic and execution reasons. Besides the adjustments of the armour layer, the design of the crest element had been changed and the amounts of rock material for the core were optimised (see Appendix II for drawings of cross sections).

3.3 Construction of Breakwaters IJmuiden

The present breakwaters were built between 1964 and 1967, but the preparation activities started in 1962. In 1961 the main requirement was that the construction time of the breakwater had to be short to minimize the changes in the sandy coastal system. The construction can be divided into three parts:

1. **Core of the breakwater**
   Sinking down the matresses, subsequently the dumping of gravel layers and core layers. This part was mainly done by dumping vessels.

2. **Crest element**
   Positioning of the crest element and filling it with concrete. This operation was carried by land cranes.

3. **Armour layer**
   Stone asphalt layer was applied on the breakwater and toe was constructed with quarry material. The stone asphalt was placed on the breakwater by two jack-up platforms (Figure 3-2).

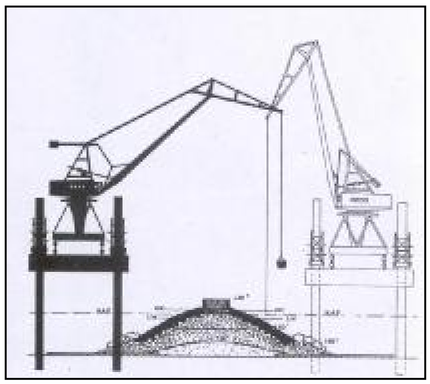


Figure 3-2 Distribution of work of the cranes

The first two phases worked out properly, but during the placement of the armour layer difficulties (settlements, cracks in the armour layer) with the stone asphalt started. The breakwater of IJmuiden was the first breakwater with an armour layer of stone asphalt, so there was no experience at all. Stone asphalt is a special material, which consists of a mixture of asphalt and small quarry stone. The armour layer of stone asphalt has to fulfil the following requirements:

	Requirements of stone asphalt layer
1	The size and the grading of the asphalt have to match with the core material and a good attachment of the asphalt on the core material is required
2	Settlements of the layer have to be minimal
3	The compaction of the stone asphalt has to be achieved without equipment but only by own weight
4	The surface of the slopes has to be dense and smooth to prevent wash out by wave attacks
5	Resistance against chemical and biological attack
6	Maintenance has to be technical feasible and has to be economical

Table 3-1 Requirements of stone asphalt

After an investigation different asphalt mixtures were adapted for the slope above the waterline and the slope under the waterline. Also the slope was altered from 1:1.5 to 1:2.0 or 1:2.5 and 1:3 on the head. Despite all investigations, problems occurred with the stone asphalt. Settlements of the layers took place at both sides of the dam with the consequence that the rigid part above the waterline could not follow the settlements of the part under the waterline, so cracks in the layers were visible. Referring to these problems three solutions were proposed:

1. Construction of a toe at the bottom of the armour layer;
2. Increase the thickness of the asphalt layer to 3.5 - 4.0 meters of the lowest part of the armour layer;
3. Creation of holes in the armour layer beneath NAP -2.0 meter, where the holes have to be 2.5 – 3.5 % of the total surface of the asphalt layer.

The first option has been chosen: at the bottom of the armour layer a toe construction with concrete blocks of 17 tonne to the level of NAP -4.0m. These measures have only been applied to the southern breakwater at the deeper parts of the trunk section at the seaside and at the head of the breakwater.

Materials

Different sorts of materials have been used for the breakwaters. The gravel, 630,000 tonnes, originated from the rivers, Meuse, Main and Moesel was transported by inland waterway ships. The quarry material, 2.5 million tonne, was transported from Belgium by ship (33%) and by train (67%). The capacity of the trains was 1,000 tonne/train, and total amount a week was 15,000-20,000 tonne/week. The stone asphalt was produced by three plants, which were positioned on the construction area and the produced 600 tonne/day. During the execution phase also 17 tonnes concrete blocks were produced to reinforce the toe construction. The weekly progress of the construction was 25 to 30m. Different causes were responsible for delays of the construction, see Table 3-2 and Table 3-3:

Part of unworkable time	Percentage of total time [%]
Storm, snow, tides	17.5
Repair of equipment	1.5
Waiting times	13
Moving of jack up platforms, etc	10
Total	**42**

Table 3-2 Unworkable time of total execution

Execution phase	Percentage of time [%]
Process dry quarry stone 300-1,000 kg	12.5
Depth soundings	6.5
Profiling of the core	18
Dumping of stone asphalt	18
Placement of crest elements	1.5
Filling of crest elements	1.5
Total	**58**

Table 3-3 Classification of workable time

Equipment

Due to the short construction time of the project a lot of equipment was required and new equipment was developed. At that time in other countries cranes were used on the dry part of the breakwater and they moved seawards with same pace as the progression of the dam. This method was for IJmuiden too vulnerable due to the rough sea conditions and the settlements of the breakwater. Wave attack and currents would delay the execution and a large amount of material would disappear into the sea. Jack-up platforms were less vulnerable for rough hydraulic conditions, so they were used for the construction in combination with dump trucks. The part under water was constructed by dumping vessels.

- **Bottom door barges**
  The lower part of the breakwater was built by five bottom door barges which were specially built for this project, because they were not available on the market. The construction of the lower part lasted three years. The vessels were self-propelling and they used the inaccurate positioning system Decca for the placement of quarry material.

- **Jack-up platforms**
  During World War II the jack-up platform was developed, a floating pontoon with movable steel piles. Description of the operation of jack-up platform: Pontoon is positioned on his position by tugboats; piles are placed on the sea bottom; than the pontoon is jacked up along the piles till a safe level above the level of storm surge. This whole procedure lasted 13 hours. On the pontoons were mobile cranes placed, which could profile 80 metres of the breakwater. Originally 4 jack-up platforms were planned for the construction, but due to changes in the design two platforms were sufficient.

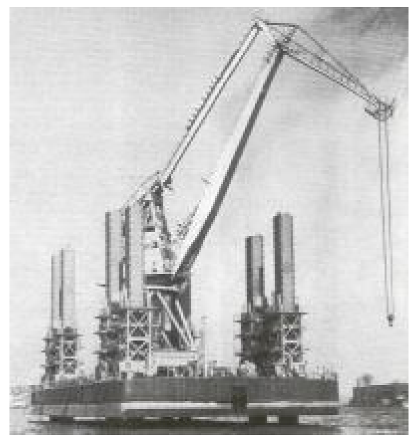


Figure 3-3 Jack-up platform Lepelaar

Jack-up platform "Lepelaar" (see Figure 3-3) had eight piles and the crane on the platform had a boom of 56m with then a lifting capacity of 25 tonnes. The Dutch government bought the jack-up platforms, because the contractors did not want to take high risks.

Construction yard

At both sides of the two breakwaters new areas were used for the production of stone asphalt and storage of construction material. The costs of the construction and organisation of the areas were € 4,500,000 and annually € 90,000 was paid for rent and licences.

On the northern bank a dune area of 36 hectares was used as working area. This area was opened up by roads and rails and also a construction harbour was positioned in this area. A part of this harbour was used for wet storage of gravel. On the southern bank an area of 20 hectares was hired from the "Staats Vissers Haven Bedrijf" and this area had also train connections. To prevent delays of the construction caused by the delays in the quarrying and the transport large storage areas were constructed. The total weekly supply was 15,000 – 20,000 tonne and 10,000 – 15,000 tonne was needed for the construction, so the weekly storage was 5,000 tonne. During the whole construction process no problems occurred with delays, but there were some problems with the unloading capacity, because only one crane was available in IJmuiden.

3.4 Maintenance of Breakwaters IJmuiden

After the problems with the construction of the asphalt layer, the problems continued in the following years. The asphalt layer showed settlements and did not have enough strength to withstand the wave forces and water pressure underneath the armour layer pushed this layer upwards. In 1971 the covering of the stone asphalt layer with concrete blocks started, although a research of Delft Hydraulics was not positive about the use of concrete blocks. 22 and 30 tonnes blocks were placed at the less attacked parts of the breakwaters and 45 tonne blocks were placed at the heads of the breakwaters. The main reason of the covering of the asphalt layer was the reduction of maintenance. The execution time of the placement of the blocks on the two breakwaters was 15 years (1972-1987).

The placement of the blocks did not solve all the problems, because many of the placed concrete blocks started to fail, like the occurrence of cracks in the surface of the blocks, corner pieces of the concrete blocks dislodged and disintegration of the concrete (see Figure 3-4 and Figure 3-5). The use of blast furnace slacks and poor concrete quality were probably responsibly for these failures. Besides these problems the concrete blocks nearby the crest were not stable, which confirmed the outcome of the research in 1971.

Figure 3-4 Poor concrete quality of the cubes

From 1970 until 1975 maintenance of the breakwaters costed € 5.5 million. The main part of the maintenance was the placement of concrete blocks in accordance with the block plan. From 1975 until 1995 the mean annual maintenance costs were € 0.75 million. Until 1995 new blocks were made, asphalt was removed and new concrete blocks and asphalt were added.

From 1995 on a proper maintenance strategy was lacking due to disagreements between several involved parties. In expectation of the optimum maintenance strategy no maintenance has been executed from 1995 onwards, only the repair of road surface on the crest of the breakwaters and the placement of 45 tonne blocks on the heads in 1999.

Figure 3-5 Overview of present stage of the breakwaters IJmuiden

Solutions for the current problems

At the moment short term emergency maintenance is necessary and this maintenance will be managed by a local department of Rijkswaterstaat, "Dienstkring Noordzeekanaal". The Civil Engineering Division has developed 15 options for the long term, which can be divided in three main types:

1. Reduce external loads
Local measures, like a wider berm or submerged dams can decrease the loading of the waves on the construction.

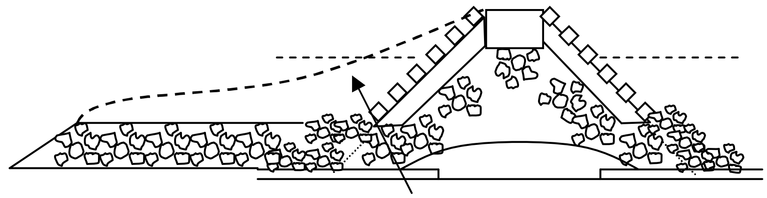


Figure 3-6 Profile with a wider berm

2. Increase external strength of the construction
Increase the strength of the construction is only realistic for the armour layer. Other parts of the breakwater are also replaceable in theory, but than a whole new construction has to be built.

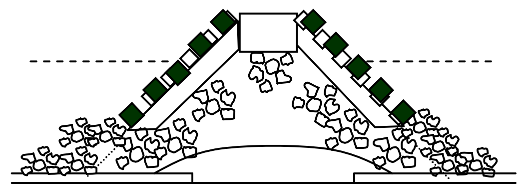


Figure 3-7 Profile with a new armour layer

3. Reduce internal water pressure in the construction
Two different ways can accomplish the reduction of the internal water pressure:

- A totally impermeable dam in which external loading can not initiate internal stresses;
- A permeable dam, in which no water pressure can be build up.

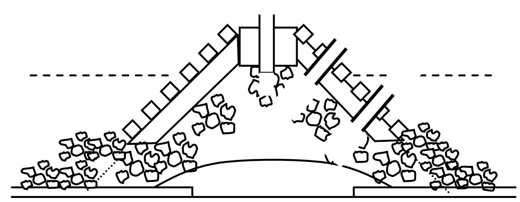


Figure 3-8 Profile with holes in the construction

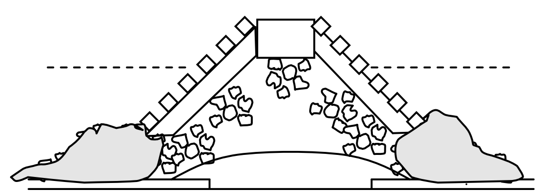


Figure 3-9 Profile with an impermeable toe construction

At the moment the Civil Engineering Division is working out alternatives of these three main types
The objective of this thesis research is to find the most economical concrete armour unit. One of the main solutions for the problems of the breakwaters of IJmuiden, is the increase of external strength of the construction. The rehabilitation of the breakwaters of IJmuiden can serve as case study for this thesis research, but the objective of this thesis is not the solution of the problems of breakwaters IJmuiden.

3.5 Constraints of Breakwaters IJmuiden

3.5.1 Introduction

The formulation of boundary conditions, objectives and assumptions gives a clear view on the situation. A distinction is made between the design constraints and construction constraints. Constraints for both design and construction form a framework, from which good design and construction methods follow. First the boundary conditions are described and subsequently the objectives and assumptions are described.

3.5.2 Boundary conditions of design and construction

Design boundary conditions

- **Meteorological data**

The meteorological data are obtained from measuring station Valkenburg, which are comparable with the data from IJmuiden. These data are given in Appendix I.

- **Wave climate**

Wave data are obtained from the Measuring station YM6, ammunition dump, 35km offshore on 21m deep water. Normative extreme wave direction W-WNW.

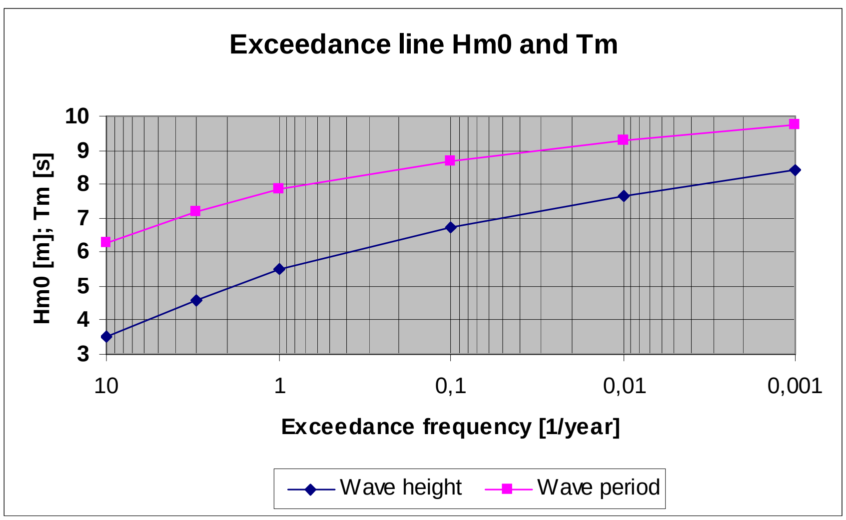


Figure 3-10 Exceedance line Hm0 and Tm, Wave boundary conditions along the Dutch coast on deep water, RIKZ (1995)

- **Local boundary conditions**

The bathymetry near the breakwaters will influence the wave boundary conditions (Figure 3-11). Since construction of the breakwaters in 1968 the entrance channel has become deeper and wider. In 1968 the depth of the entrance channel was NAP -15.5m and the width was 750m. In 1985 the entrance channel has been dredged up to a level of NAP -20.0m and has been widened to 1,600m. The highest wave penetration came from the direction W-WNW, but at this moment the range has expanded to W-NW. The effect of the expansion and the deepening of the channel are modest on the significant wave height, but the highest loads will influence a larger part of the breakwaters. In the new situation it is possible that individual waves with a height of 10m will occur, which means an increase of 30 percent in comparison with the old situation. These individual waves could be dangerous for the current asphalt layer, but will not influence the design process of a new armour layer. The main design parameter for the new armour layer of armour units is H_s = 7.5m, because this parameter determines mainly the dimensions of the armour units. These wave height boundary conditions are calculated by the half of the water depth. Further research with wave modelling studies has to be executed to get accurate data. The direction of the waves is indicated in Table 3-4.

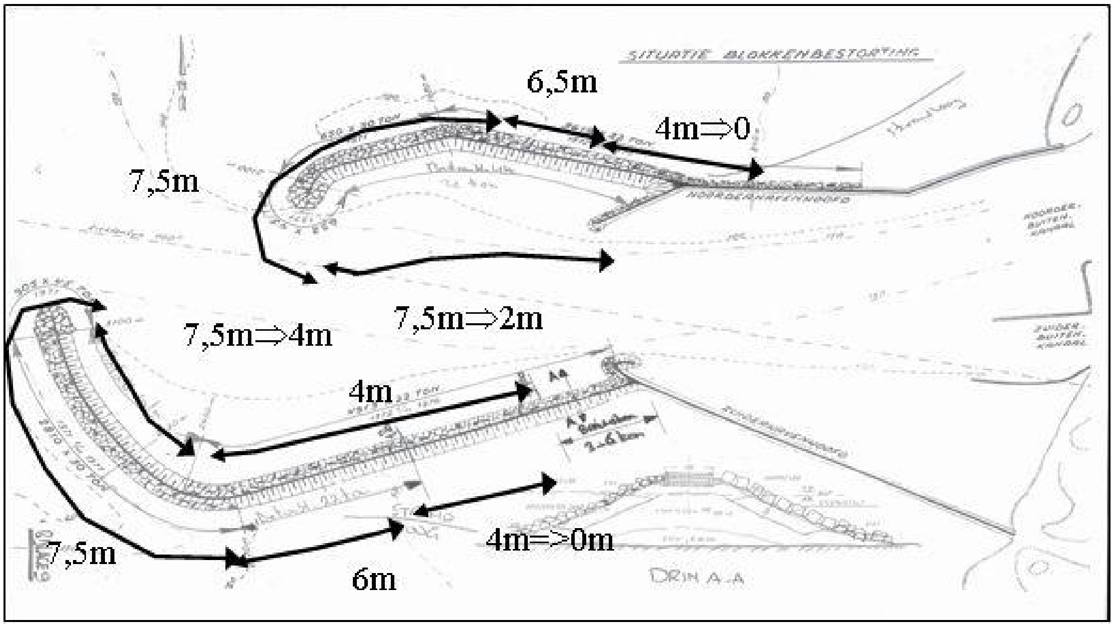


Figure 3-11 Local wave heights (one in 100 years)

Degrees	1990-1996 [%]	1997 [%]	Winter [%]	Summer [%]
0	17.8	15.6	7.6	23.8
30	6.3	6.0	4.2	7.2
60	2.8	2.1	3.1	1.3
90	1.4	1.5	2.9	0.6
120	1.4	1.6	3.8	0.5
150	1.7	2.0	4.3	0.9
180	1.5	1.2	1.8	0.7
210	10.0	14.6	24.0	11.8
240	18.3	16.1	19.8	14.0
270	9.2	7.6	8.0	9.3
300	9.8	11.1	10.0	11.0
330	19.8	20.7	10.6	18.5

Table 3-4 Percentage of wave direction; Jaarboek Monitoring Rijkswateren [1996/7]

- **Tides and currents**

The tidal range in the North Sea varies between 4.5m in Vlissingen till 1.0m in Den Helder.
Tidal level in cm to NAP (Normal Amsterdam Level).

Period	Date	High water level [cm]		Date	Low water level [cm]	
		Highest	*Mean*		*Highest*	*Mean*
1997	02/10	191	95.1	18/11	-168	-73.9
1951-1995	1/2/1953	385	-	15/3/1964	-240	-
Normal	-	250	97	-	-171	-73

Table 3-5 Tides, Location IJmuiden buitenhaven; Jaarboek Monitoring Rijkswateren [1996/7]

Time before of HW	Velocity [m/s]	Direction	Time after HW	Velocity [m/s]	Direction
6 hours	0.72	S	1 hour	0.73	N
5 hours	0.52	S	2 hours	0.47	N
4 hours	0.31	S	3 hours	0.15	N
3 hours	0.15	Var	4 hours	0.26	S
2 hours	0.57	N	5 hours	0.57	S
1 hour	1.14	N	6 hours	0.62	S
0 hours	1.04	N			

Table 3-6 Currents, location IJmuiden Southern breakwater; Tidal current atlas HP 16 [1992]

- **Sediment transport**

Direction of the sediment transport along the Dutch coast is northwards and the southern breakwater of IJmuiden blocks a large part of the sediment transport. Changes of the cross sections of the breakwater will not influence the morphology.

- **Geotechnical data**

The subsoil underneath the current breakwater consists of granular loose sand with D_{50} = 150 μm. From 1968 until 1996 settling of the breakwater has been measured.

Alignement	**Mean value of settlement (cm)**
Section 1200 - 1700	9
Section 1700 - 2300	6
Section 2300 – 3000	7.5
Section 3000 – 3200	10
Head of breakwater	14

Table 3-7 Settlement southern breakwater

- **Bathymetry**

In 1999 the bathymetry near the breakwaters IJmuiden has been mapped out. A pit in the channel in front of the breakwater has developed. The average depth along the breakwaters is approximately NAP -12.0m and decreases in the direction of the coast.

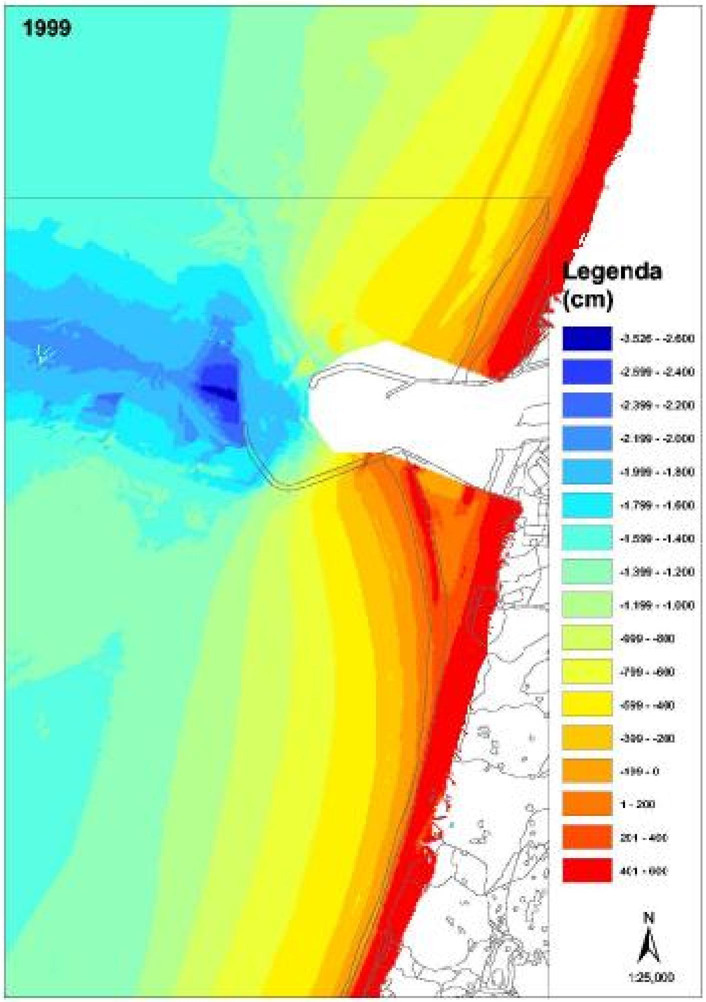


Figure 3-12 Depth contours near the breakwaters

- **Current state of the existing breakwater**

The breakwaters of IJmuiden have a lot of problems with the armour layer of stone asphalt, but the other parts of the breakwater are also important, when a new design will be made.

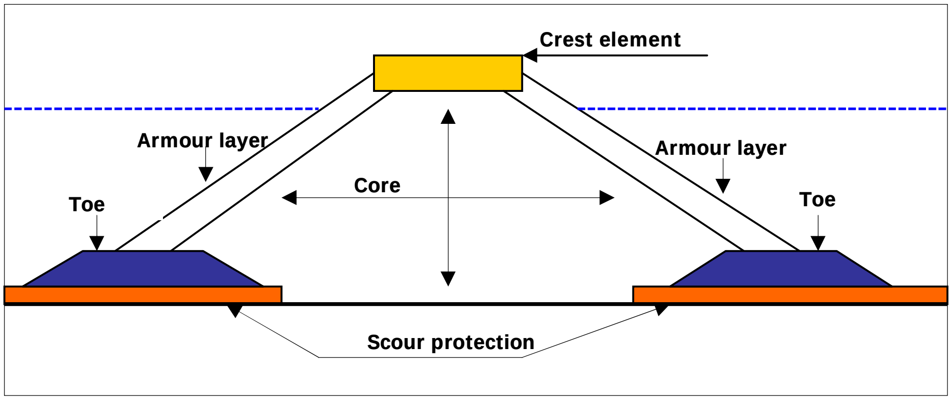


Figure 3-13 Parts of the breakwater

1. **Scour protection**

The current soil protection consists of willow mattresses with a rubble layer of 200-800 kg near the southern breakwater head and 80-200 kg on the other mattresses. Large parts of these willow mattresses may have been deteriorated or vanished.

2. **Toe**

The strength of the toe is determined by the state of the different boundary surfaces of the consecutive filter layers. Migration of material through the layers will cause settling of the construction. Migration of material takes place under storm conditions (large gradients). During design conditions a gradient with a maximum of 8 % is expected within the toe construction at N.A.P. -13.0m. The boundary surface of base material (sand) with gravel 30/all will be critical under those circumstances.

3. **Core of breakwater**

The core consists of quarry run material with grading 300-1,000 kg. The layer of gravel 30/all and the mattresses lie under the core and the asphalt layer with the concrete blocks and the crest element lie above the core. The boundary surface of this layer will not be critical under storm conditions.

4. **Armour layer**

The top layer consists of a stone asphalt layer with a thickness of 2.25m and a slope of 1:1.75 under water. Above water the thickness of the layer is 1.8m and the slope of the layer is 1:2 (1:3 at head of breakwater). On top of the asphalt layer concrete cubes of 22, 30 and 45 tonne are placed from the toe till the crest element. On many locations the cubes are glued on the slope with bitumen. The toe construction is covered with a layer of rock 1-6 tonne and 17 tonne cubes to support the concrete cubes. The current concrete cubes do not have enough hydraulic and structural stability.

5. **Crest element**

The crest element is a pre-cast element filled with concrete and it has a width of 7m, a length of 2.5m and a height of 2.0m. The elements are connected with steel rods, but according the contractor the strength of the rods is minimal. The total weight of a crest element including concrete fill is approximately 80 tonne and it is founded on a layer of core material of rock 300-1,000 kg. The crest element is covered with an asphalt layer, which made heavy transport possible during the execution phase. The crest level of the elements is NAP +4.0m (head of southern breakwater), which made a "dry" execution possible. At this moment it is uncertain if the element is totally supported by the under layer.

Construction boundary conditions

The boundary conditions of the construction are subdivided into the different phases of the execution process.

1. General

- Legal standard during the whole execution:
  1. Satisfy the legal standard on nuisance of noise, smell and vibrations;
  2. Working vessels, trucks, cranes and other equipment have to satisfy safety standards;
  3. The following rules from the "Bouwstoffenbesluit" (1999):
     The construction material may not be mixed with the surrounding soil material and must be recyclable. When the construction has lost its function, the owner is liable to remove the construction material.

- From the head of the breakwater till section 2400 of the southern breakwater is the part which is the most exposed to storm conditions. A cross section of the head and one cross section from the trunk will be studied. The most severe damages occurred at the outer slope, so the inner slope will be left out of the scope of this research.

2. Production

- Concrete mixture requirements are based on NEN 5950 (VBT 1995), published by the Dutch "Normalisatie instituut":
  1. Strength class B25 for cubes and B35 for interlocking units;
  2. Environment Class 4 (sea water);
  3. Density ρ = 3,800 kg/m^3 for heavy concrete cubes and ρ = 2,350 kg/m^3 for the other 3 elements;
  4. The use of water reducers is sustained, but the use of decelerating and accelerating agents and air entrainer is not sustained;
  5. Blast furnace slag cement, class B;
  6. The grading and quality of the aggregate material have to satisfy the NEN standard 2650;
  7. The sand has to satisfy the NEN standard 3542;
- Quality aspects of quarry stone:
  The European standard EN 13383-1 en -2, Armour stone Specification and Test Methods is since July 2004 the new standard.
- Subsoil of storage area is sand. Without geotechnical consult the maximum design value for the foundation pressure 100 kN/m^2. With a geotechnical consult the maximum foundation pressure is 250 kN/ m^2 .The area is consolidated by previous storage of construction material.

3. Placement

- Old storage area is accessible by land and water; a quay for loading and unloading is available;
- Placing requirements of filter layers

	Above water [m]	Below after [m]
Core	+/- 0.20	+ 0.5 / - 0.3
Under layer (300-1,000kg)	+ 0.4 / - 0.2	+ 0.8 / - 0.3
Armour (4-6 tonne)	+ 0.4 / - 0.3	+ 0.7 / - 0.5

Table 3-8 Tolerances follow CUR report 154 / CIRIA report 83

- Bearing capacity of the crest element is decreased, which could be caused by the settlement of core material;
- Depth in the harbour of IJmuiden is 16.5m and of the sluice 13.72m;
- Working conditions see Table 3-9.

Equipment	Weather condition	Restriction
Crawler cranes	Wind	< wind force 6
Vessels	Waves	Hs < 1.5 m
Floating barges	Waves	Hs < 0.5 m
Mobilisation of jack-up platform	Waves	Hs < 1.0 m

Table 3-9 Working conditions

The dominating factor during construction of core and under layer with equipment working on top of the core is the significant wave height possible to occur. Placement of core and under-layer (3 - 6- tonne) can proceed till Hs reach a value of 2m.

- Diving activities according NEN/ISO 9002 and VCA**.A team of 4 divers is required. A dive team costs 2,400 euro per day exclusive a working vessel. A working vessel costs € 100 – 200/ hour. Total costs of a dive team are 4,000 euro per day.

  Restrictions during dive activities:
  - Wave height < 0.5 m
  - Current < 1.5 m/s
  - The maximum diving depth near the breakwater is 15m, so a decompression tank is not required and nitrox 40/60 can be used for the diving activities, so the duration of the dive is not restricted;
  - The sight during the placement has to be at least 0.50m

3.5.3 Objectives of design and construction

Design objectives

- Widening of breakwater may not hinder the shipping traffic;
- Design wave 1/100 years; standard design wave;
- Wave overtopping is allowed, because the breakwater is designed for it;
- Lifetime of the reconstructed breakwater with new armour layer is 100 years;
- In the future the crest element has to be accessible for monitoring and maintenance;
- The current lay-out and the functions of the breakwaters have to be maintained;
- Crest height has to be maintained; so current crest height is the normative for the design;
- Stability of the toe construction must improve;
- The lifetime of the current mattresses will not be long, so a new bed protection has to be constructed;
- Prevent damage to the asphalt layer, because the removal of the asphalt could initiate wash out of core material;
- Prevention of scour holes in front of the construction.

Construction objectives

1. General

- A Life Cycle Analysis has to be carried out to investigate the durability of an alternative. All phases, from the extracting, processing, transport, construction until the end of the material is investigated.

2. Production

- Availability of quantities of material (quarry stone and concrete elements) in stock on the storage area, so delays with the production/transport will not influence the execution;
- The calculation of quantities does not take into account the losses caused by settlements of the subsoil, storm damage and depot losses;
- Efficient division of the storage area;
- Protection of armour units against severe weather conditions during the hardening process;
- Two years before the execution phase the following aspects have to be examined:

1. Research into the availability, productions, transport, and storage area of quarry stone and the preparation of the production of concrete elements;
2. Delivery contracts;
3. Research into the right equipment, which is required for the handling of quarry stone. Equipment can also be used for other large projects, like Maasvlakte II.

3. Placement

- The execution may not hinder the shipping traffic;
- Construction harbour for the vessels;
- Execution period from April till September, rest of the year the risk of damage during execution is too big;
- Recycling of demolished material;
- Prevention of storm damage, temporary precautions for execution sections.

3.5.4 Assumptions

Design assumptions

- Impermeability of the asphalt layer will not have influence on the hydraulic stability of the new armour units;
- Some of the existing concrete cubes can be a part of the new filter layer, so they do not have to be removed;
- Height of crest element is the upper limit of the new layer of armour units, because blocks, which are higher than the crest element, could move across the crest;
- The design is only preliminary; model tests should be executed afterwards to test the hydraulic stability of the designs (especially on the influence of the impermeable asphalt layer);
- Removal of the toe construction and the armour layer of the current breakwater are feasible.

Construction assumptions

- Permits for the extraction of gravel on the North Sea will be granted;
- During the HISWA, a nautical event in the Marina IJmuiden the execution will continue;
- All quarry material and concrete are available during the reconstruction of the IJmuiden breakwater;
- The equipment used for the project is available during the construction period;
- The comparison deals with a part of the reconstruction of the breakwater, but the equipment will be used for the whole project and the costs will be written off on the whole project. A clear comparison can not be made.

4 Alternative Designs of the Armour Layer

4.1 Introduction

A breakwater, like any other coastal structure, is planned as a practical measure to solve a specific problem encountered in the coastal zone. In this case the breakwater exists already, but the armour layer requires maintenance. Several stages can be distinguished in the design process for this

1. Definition of functions
2. Determination of constraints
3. Creating alternatives
4. Geometrical design
5. Choice of execution method
6. Final choice of a functional solution

The functions of the existing breakwaters of IJmuiden will not change in the future, so the design process can start with stage 2. The constraints for the breakwaters are described in Chapter 3. Stage 3, the creation of alternatives, is determined by the objective of this thesis research, a cost comparison of three different armour units, cubes, Accropodes and Xblocs. Other armour units beyond the scope of this research, only the density of the concrete of the armour units can be varied. The next step of the design process is a preliminary design of a trunk section and a head section with these armour units. Section 4.2 holds a description of this design process. Only a preliminary design is sufficient, because the comparison is the head subject of this thesis and not a final design for the reconstruction of the breakwaters IJmuiden. Step 5, a description of the construction area and the construction methods is given in Section 4.3. Stage 6 is not applicable for this thesis study, because no solution for the IJmuiden problems has to be chosen. Section 4.4 treats the placement techniques of cubes, Accropodes and Xblocs. The last section, Section 4.5 gives an overview of maintenance strategies. The next chapter, Chapter 5 compares the production and placement costs of the three armour units.

4.2 Preliminary Design of New Armour Layer

4.2.1 Introduction

One of the long-term solutions for the problems of the breakwaters IJmuiden, mentioned in Chapter 3, was the increase of external strength of the construction. The Civil Engineering Division (Bouwdienst Rijkwaterstaat) has proposed two options to reinforce the current profile. This thesis research focuses on costs of three different kinds of armour units and uses the long-term solution for the IJmuiden problems for a realistic comparison between the three armour units.
For this thesis research a cross section design of the breakwater has to be made. Designs are made for the trunk section and for the head of the breakwater. The choice for an option has roughly been made, because the research deals with a comparison between different armour units and not with the local problems of the breakwaters of IJmuiden. The two options of the Civil Engineering Divison:

1. Widening of the current profile

One option to repair the unstable toe and armour layer is covering of the existing section with sea gravel, filter layers and an armour layer of heavy rock (10-15 tonne) or armour units. By covering the current profile, the new profile will be very wide and will require a lot of material. Because the new cross section is wider en less steep, wave overtopping will be less, so the slope of the harbour side will be stable enough without adjustments.

2. Reconstruction of the toe and armour layer

This option consists of the removal of a part of the toe construction and the removal of 17-tonne concrete cubes. After this operation a new toe will be constructed and a new filter layer and armour layer with armour units will be placed on the current armour layer. Wave overtopping is also reduced with this option, so no adjustments are necessary for the inner slope.

Although the execution risks during the demolition and construction phases are larger for the reconstruction alternative, this alternative will be more economical than the widening alternative. The material costs of the widening alternative are much higher than the other alternative, so the total costs of this alternative will be also higher than the other and this alternative will not worked out.
The new armour layer can be applied with all three armour units, cubes, Accropodes and Xblocs. A further subdivision can be made with the density of concrete; three kinds are available, 2,350, 2,800 and 3,800 kg/m ^{3}. The objective of this thesis is a comparison between cubes, Accropodes and Xblocs, so at least three alternatives have to be worked out. Concrete with density of 2,350 kg/m ^{3} is called "normal" concrete and concrete with density 3,800 kg/m ^{3} is called "heavy" concrete. Concrete with a density of 2,800 kg/m ^{3} is also known as heavy concrete. The application of "heavy concrete" for interlocking elements is not usual, because the Accropode and the Xbloc find most of their stability by interlocking and not by weight. Two extremes have been chosen for the cube alternative, "normal" concrete (2,350 kg/m^3) and "heavy" concrete (3,800 kg/m^3). Density 2,800 kg/3 has not been chosen, but this density can be used later on in the sensibility analysis.

The interlocking units, Accropode and Xbloc are designed for a single layer and in the problem analysis (Section 1.2)) is chosen for a double layer of cubes. A double layer is chosen, because this concept has been used worldwide and it has worked properly. A single layer of cubes saves concrete, but has to be placed more accurately (low porosity). This alternative is beyond the scope of this research, because there is not much experience with this concept. All the choices, which have been made, are summarised in the table below.

	Density [kg/m^3]			Type of layer	
	2,350	2,800	3,800	Single	Double
Accropode	**x**			**x**	
Xbloc	**x**			**x**	
Cube	**x**		**x**		**x**

Table 4-1 Overview of all options

Eventually four designs for the trunk section and head section are worked out:

1. Concrete cubes with density 2,350 kg/m^3
2. Concrete cubes with density 3,800 kg/m^3
3. Accropodes with density 2,350 kg/m^3
4. Xblocs with density 2,350 kg/m^3

4.2.2 Design of armour layer

The present breakwaters of IJmuiden consist of core material, stone asphalt and cubes as armour layer. A part of this armour layer will be removed and a filter layer will be placed on the stone asphalt and the cubes. The activities below are valid for all four alternatives:

1. The demolition of the current toe and the mattresses;
2. Construction of an extra filter layer, quarry run 100-300kg in front of the construction to prevent the wash out of core material;
3. Construction of soil protection;
4. Construction of filter layer and thickness of filter layer based on 1/15 – 1/7 of the weight of one armour unit.

This section will deal with both the design of the armour layer and the toe construction. The quantities, dimensions and drawings are listed in Appendix II and III.

Armour layer

A widely used formula for the design of concrete armour units is the Hudson formula, although it does not include the influence of the wave period and the surf similarity parameter, ξ. Since 1942, systematic investigations into the stability of rubble slopes have been performed at the Waterways Experiment Station in Vicksburg, USA. On basis of these experiments, Hudson proposed the following formula:

The Hudson formula

$$H_s / \Delta D_n = [K_d * \cot a]^{1/3}$$

H_s	incident wave height near the toe	[m]
Δ	relative density $[(\rho_a - \rho_w)/\rho_w]$	[-]
D_n	nominal diameter of unit = $[W/\rho_a]^{1/3}$	[m]
K_d	stability factor	[-]
a	slope angle	[degrees]

	Trunk section of breakwater		Head section of breakwater	
	Breaking	Non-breaking	Breaking	Non-breaking
2 layers of cubes	6.5	7.5	4.33	5
Accropode	12	15	8.4	10.5
Xbloc	16	16	13	13

Table 4-2 K_d values; Source SPM 1984 and Xbloc K_d value by DMC

Recommended values for K_d have frequently published and updated by the Corps of Engineers in the Shore Protection Manual (see Table 4-2). The coefficient K_d is a sort of waste bin factor including all kind of unknown variables and unaccounted irregularities in the model investigations. At first, it is a function of the damage level defined as loss of stability. It also includes the effect of the shape of the blocks and the internal friction. It covers also all other influences:

- Layer thickness of the armour layer;
- Manner of placing the units;
- Roughness and interlocking of the units;
- Type of wave attack;
- Head or trunk section of the breakwater;
- Angle of incidence of wave attack;
- Size and porosity of the under laying material;

- Crest level (overtopping);
- Crest type;
- Wave period;
- Shape of foreshore;
- Accuracy of wave height measurement (reflection);
- Scale effect.

In view of this, one cannot expect a good consistency of reported values of K_d. When using the formula, one must realize what influence uncertainties have on the final result. The difference of K_d between the trunk and the head has been caused by the difference in wave attack. H_s has been measured by RIKZ (see Figure 4-1) and the local wave height near the construction is 7.5m. For interlocking armour units the K_d value is higher, because of the reduced interlocking on the head (units lie in a curve). With Hs of 7.5m and a depth of 14.5m (high water) the waves will break, so for the IJmuiden case a K_d for breaking waves will be used.

Toe construction

The toe of the breakwater has to be properly designed, because it is important for stability of the breakwater. With the formula an optimum can be found between D_{n50} and the height of the toe. A toe construction will be designed under the same boundary conditions, so one design will be appropriate for all four options.

$$H_s \ / \Delta D_{n50} * N_{od}^{-0.15} = 2 + 6.2(h_t / h)^{2.7}$$

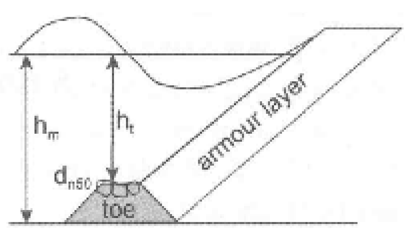


Figure 4-1 Illustration of toe

Critical values for N_{od} are:
N_{od} Character of damage
0.5 Start of damage
1.0 Acceptable Damage
4.0 Failure

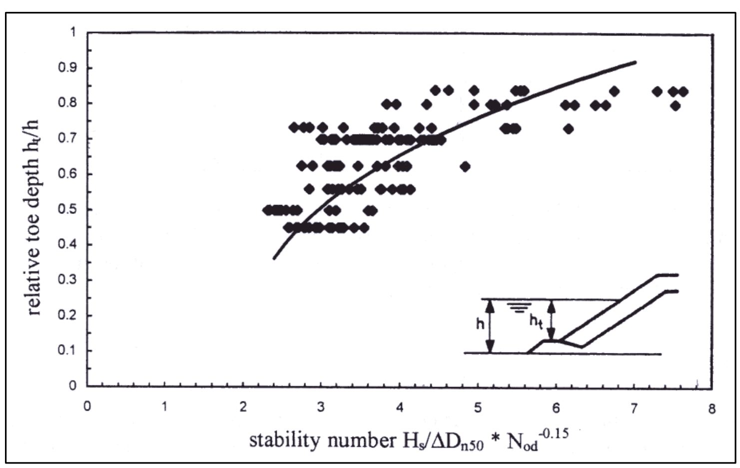


Figure 4-2 Toe stability graph; Van der Meer

These values are valid for a standard toe, with a height of 2 to 3 D and a width of 3-5 D.
The validity range is:

$0.4 < h_t / h < 0.9$
$3 < h_t / D_{n50} < 25$

After calculation the optimal toe design values are found:

ρ = 2,750 kg/m^3 (granite)
N_{od} = 0.5 (start of damage)
H_s = 7.5m with water level N.A.P. + 2.0m (high water)
h = 14.5m
h_t = 10.0m
D_{50} = 1.04 – 1.31 (3-6 tonne)

With the calculated values for the toe construction and the armour layer, drawings can be made for all four alternatives. This resulted in designs of the head section and the trunk section for all four alternatives.

Design 1 Concrete cubes with density 2,350 kg/m^3

The height of the toe is relative high, which requires heavy rock in the toe construction. Different slope steepness has been used for the calculations with the Hudson formula.

- 1:1.33 Maximum slope steepness for Accropodes and Xblocs
- 1:1.50 Maximum slope steepness for cubes
- 1:1.75 Current slope of the armour layer of the breakwaters IJmuiden.

A slope with a steepness of 1:1.5 is the most economical for an armour layer of concrete cubes, because the quantities of filter material and armour units are minimized. At the horizontal part of the armour layer the first layer of cubes is not totally extended to the crest element to minimize the amount of demolition of the current breakwater. This prevention of demolition has not only economical reasons, but also technical reasons; because demolition increases the risk of wash out of core material, so it will also increase the instability of the construction. With the Hudson formula the D_{50} can be calculated (Table 4-3). The number of units has been calculated with the assumption, that the double-layer of cubes has a porosity of 30 %. In Appendix II the drawing of the design is given.

Dimensions	Trunk section	Head section
Density [kg / m^3]	2,350	2,350
$D_{Element}$ [m]	2.75	3.10
D_{layer} [m]	5.50	6.20
D_{filter} [m]	2.40 (3-6 tonne)	2.8 (6-10 tonne)
Volume [m^3]	20.8	29.8
Weight [tonne]	48.9	70.0
Slope [-]	1:1.5	1:1.5

Table 4-3 Properties of Design 1

Design 2 Concrete cubes with density 3,800 kg/m^3

The application of high density concrete decreases the dimensions of the cubes, but the amount of units will increase. A design with high density concrete will not differ from a design with normal concrete, so the same slope steepness has been applied (see Table 4-4).
The number of units has been calculated with the assumption, that the double-layer of cubes has a porosity of 30 %. In appendix II the drawing of the design is given.

Dimensions	Trunk section	Head section
Density [kg / m^3]	3,800	3,800
$D_{Element}$ [m]	1.30	1.50
D_{layer} [m]	2.60	3.00
D_{filter} [m]	1.80 (1-3 tonne)	1.80 (1-3 tonne)
Volume [m^3]	2.2	3.375
Weight [tonne]	8.4	12.8
Slope [-]	1:1.5	1:1.5

Table 4-4 Properties of Design 2

Design 3 Accropodes with density 2350 kg/m^3

A slope with a steepness of 1:1.33 is the most economical for an armour layer of Accropodes because the quantities of filter material and armour units are minimized. This steepness is recommended by Sogreah, the engineering company which has developed the Accropode. Efficient interlocking between Accropodes with the underlying filter layer allows a homogenous and yet flexible protective layer. The Accropode has been subjected to finite element calculations to calculate the internal stresses of the unit. When wave data which have been used for the design is correct, no maintenance of the construction is required in the future. The whole armour layer consists of Accropodes, also the Accropodes on the horizontal part are hydraulic stable, because a large part of the wave energy passes the armour layer and does not break on the elements. In Appendix II the drawing of the design is given. Although the Accropodes are part of a single layer, on the horizontal part 3 cubes of the current breakwater have to be removed and also a part of the stone asphalt. Number of units has been calculated with the porosity, which is recommended by Sogreah: 11.71 12m^3 Accropode units and 8.39 18 m^3 Accropode units can be placed on 100 m^2.

Dimensions	Trunk section	Head section
Density [kg / m^3]	2,350	2,350
$D_{Element}$ [m]	3.28	3.75
D_{layer} [m]	2.95	3.38
D_{filter} [m]	2.5 (2-4 tonne)	2.7 (2.6-5.2 tonne)
Volume [m^3]	12.0	18.0
Weight [tonne]	28.2	42.3
Slope [-]	1:1.33	1:1.5

Table 4-5 Properties of Design 3

Design 4 Xblocs with density 2,350 kg/m^3

The whole armour layer consist out of Xblocs, also the Xblocs on the horizontal part are hydraulic stable. In Appendix II the drawing of the design is given. The amount of demolition of the current cubes is comparable with the Accropode design.

The Xblocs find a stable position on the slope and the highly porous armour layer minimises wave overtopping. Studies indicate that the unit sturdiness of the Xbloc is excellent and the unit is able to cope with loads that can be expected during its lifetime. When the wave data which has been used for the design is correct, no maintenance of the construction is required in the future. Number of units has been calculated with the porosity, which is recommended by DMC: 12.75 9m^3 Xbloc units and 10.53 12m^3 can be placed on 100m^2.

Dimensions	Trunk section	Head section
Density [kg / m^3]	2,350	2,350
$D_{Element}$ [m]	3.0	3.30
D_{layer} [m]	2.9	3.20
D_{filter} [m]	1.8 (1-3 tonne)	1.8 (6-10 tonne)
Volume [m^3]	9.0	12.0
Weight [tonne]	21.2	28.2
Slope [-]	1:1.33	1:1.33

Table 4-6 Properties of Design 4

4.3 Reconstruction of Breakwaters IJmuiden

4.3.1 Introduction

The main problem of breakwater construction is to build out core and cover layers consecutively in such a manner that every part which is not yet stabilised by its cover is not damaged by the environmental conditions during the construction of the breakwater. All damage, which occurs during construction, has to be repaired according to the prescribed layer profile, as the functioning of the breakwater depends on the filter design rules. Therefore, it is necessary to consider tolerances and to maintain a very strict position control during the construction of the breakwater. The risks which are threatening the breakwater during the construction can be reduced by the following methods:

- Select a specific construction period with calm weather;
- reduce the exposed length of the vulnerable part of the structure;

Several methods are available to build a breakwater. Many variations in equipment, order of construction, location (waterborne or land-based) are possible. Working out all those options would become too extensive for this thesis investigation, so a choice has to be made between the different options.

The execution rate determines the transport, storage and production of elements. The order of construction and the equipment are important for the right choice for the execution method. The following options for the order of construction are possible:

1. Execution starts at the head of the breakwater
2. Execution starts at the landside
3. Working from head to landside with one team and another from landside to head
4. First demolition of armour layer, than the reconstruction of the whole breakwater

The first two options are in principle the same, but differ in construction direction, which especially determines the way of transport of materials. Option 3 will decrease the construction time substantially, because two construction teams are working at the construction at the same time. Only at the start of the project two teams have to learn to work at the breakwater instead of one, so extra time is lost. Option 4 makes the unprotected construction very vulnerable for environmental conditions, because after the demolition of the toe construction the core material of the breakwater can be washed out by wave attacks. For this research option 1 is chosen, because it is the most efficient option.

In principle there are three methods to place the construction material in the construction:

1. Land based equipment
2. Floating equipment
3. Jack-up platforms

For this research the first option, land-based equipment, is chosen, because the costs of execution over land are much lower and waterborne equipment has a lot of restrictions with the workability. To reduce risks during construction the exposed sections are 100 meter. Activities during the reconstruction of the IJmuiden breakwaters take place on:

1. Construction yard
2. Breakwater

Normally a construction programme is based on the reconstruction of a whole structure, so total costs (e.g. mobilisation of equipment, preparation costs, etc) can be divided over the whole structure. For this thesis a part of the breakwater has been chosen to perform a cost comparison. A part is sufficient, because the goal of this thesis study is to get an insight into the differences between the three armour units.

1. Construction yard

In the Netherlands the construction of large infrastructural works has different types of requirements. This concerns not only technical problems e.g. quality certificates, but especially problems with local and national authorities on building permits. A construction yard including a concrete factory causes nuisance in the field of water- and air quality, ecology and noise. The procedures to obtain a permit could take years, because it is possible for individual citizens or organisations to lodge an appeal with court against decisions of the government. A construction yard without a concrete factory, where concrete is delivered by local concrete factories is an option. A permit will be granted faster, when there is no concrete factory on site. The concrete can be delivered by one or more factories. With more concrete factories the delivery risks of concrete are lower and the delivery of concrete for the breakwater is not their only project, so the factories are not totally dependent. When one concrete factory is used, the contractor has to deal with one party and can demand more from that factory.

Due to all procedures for permits, production on an existing casting yard could be another option. A requirement for this yard is quick access to deep water (connection to the North Sea), to minimize the transport cost. Road transport is not an option, because the weight of the armour units is high and special trucks are required to transport them. Transport on barges is the safest and cheapest way of transport. When the barge is positioned near the breakwater, a land crane on the breakwater picks the units from the barge and places them on the breakwater. To be a competitive option the transport costs have to be low and the workability of the barge has to high.

In this particular case of IJmuiden a construction yard without a concrete factory is chosen. The production of armour units will take place on the construction yard, so lifting equipment is required. The transport of the armour units from the construction yard to the breakwater will be done by flatbed lorries.

Intermezzo quality certificate

A system of quality certificates guarantees the quality of the concrete. A concrete factory with quality certificates which delivers concrete to contractors is responsible for its products. A contractor who orders concrete with a certificate is not liable for the concrete quality and thereby shifts its responsibility to the concrete factory.

In the Netherlands the foundation BMC is responsible for the extension of quality certificates.
Three forms of quality certificates:

1. Product and process certificate;
2. Certificate of design;
3. Care system (Quality, Operational Health and Safety Assessment, Environment).

The certificates are valid between one and three years and BMC checks with unexpected inspections if a factory fulfil the requirements of the certificate.

The last years most of the large coastal structure building projects had an own concrete factory on site due the large required quantities of concrete. A concrete factory on site is not obvious for the Dutch situation, because the required quality certificates and building permits.

A factory on site requires a long preparation to obtain the permits. The contractor himself has to execute quality checks and other measures to fulfil the requirements of the quality certificate. He is fully responsible for the quality of the concrete and he is liable when failure occurs.

The capacity of a casting yard for armour units depends on the available storage area and the daily number of placed units. In IJmuiden free space for storage is limited in the surroundings of the breakwater. The daily casting of armour units is approximately equal to the number of daily placed units. The costs of the moulds depend on the weight of the steel framework.

The production of armour units can be executed during all seasons, so the production period is continuous. The casting process does not differ much between the three alternatives (see Table 4-7). One crew is responsible for the casting and the stripping of the formworks, one crew for the pouring of concrete and another one for the transport of units from the casting yard to the storage yard. After one day the unit can be stripped and after three days the unit can be transported. These facts result in a typical lay-out of casting yard, see figure below.

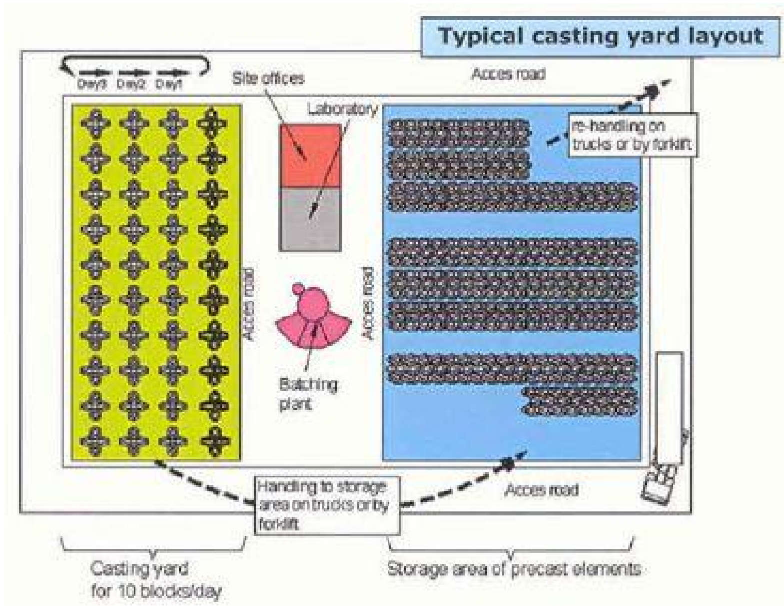


Figure 4-3 Typical lay-out of a production area [Sogreah]

Small gantry cranes are positioned on the casting yard to lift the formwork and to support the pouring process. A big gantry crane is responsible for the transport of the armour units from the casting yard to the storage area.

Figure 4-4 Concrete pump

The concrete is delivered by concrete mixers of 9 m^3 or 12 m^3 and they pour the concrete into a concrete pump (Figure 4-4), which pumps the concrete into the frameworks. An assumption is made that the daily amount of concrete can be delivered by a concrete factory in the surroundings of IJmuiden. Local problems, which could hamper the logistic process, are beyond the scope of this research.

Activities	Description
1. Preparation for stripping	Loosen the bolts and the bars of the framework
2. Stripping of framework	Carefully lifting of the framework by a gantry crane, because the play between the fresh concrete and the framework is minimal and damage of the amour unit have to be prevented
3. Cleaning of the framework	Cleaning of all sides of the framework and all borders. After the cleaning operation the framework can be positioned on a new position.
4. Applying of casting oil	Casting oil is applied on the framework to prevent attachment of the concrete to the steel framework.
6. Preparation for casting	The bolts and bars of the framework have to be tightened before the concrete is poured into the framework.
7. Applying of curing compound	Curing compound is a liquid, which is applied on the concrete to prevent shrinkage cracking.

Table 4-7 Detailed description of the stripping and casting operation

2. **Breakwater**

The most critical activity of the execution process is placement of armour units. The crawler crane (Figure 4-5), which places these units, has to work whole day (12 hours) and stagnation has to be prevented. The armour units are supplied by flatbed lorries. In case of IJmuiden a crawler crane, Manitowoc 4100W-S2 is chosen for the placement, because the crane is mobile and it has max lifting capacity of 209 tons (appendix VI). The crane just fits on the crest element, but does not have enough boom length to place the armour units above the toe construction. A temporal construction road made of quarry stone is required, on which the crane can be positioned, so it can reach the toe.

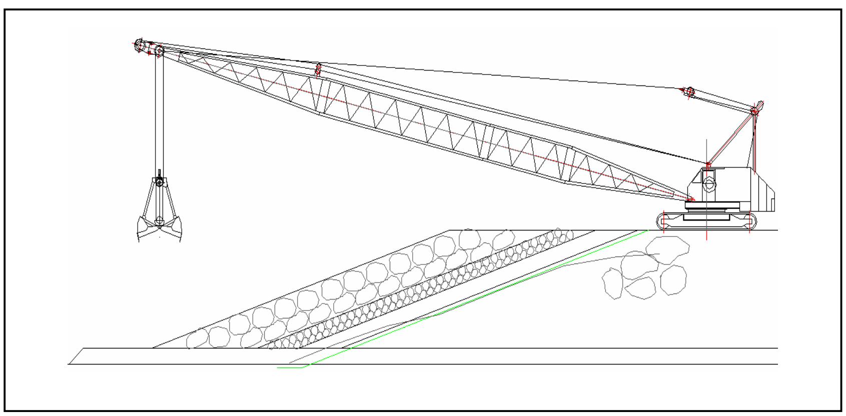


Figure 4-5 Position of the crane on the breakwater; Manitowoc 4100W-S2

After the bottom protection, filter layers and the toe construction have been constructed, the crane starts with placing of units near the toe. In order to maintain stability of the armour layer, the armour layer will be placed triangular, so first a wide base on the toe and then smaller to the crest.

4.3.2 Construction process of the reconstruction of breakwaters IJmuiden

Reconstruction of the breakwater is divided in different phases. Before a new armour layer can be constructed, a part of the old armour layer has to be removed. These phases are executed consecutively, so when a part of the armour layer of one section of 100 meter has been removed, the construction of the toe of that part can start. When the armour layer of a section has been removed, the removal of the armour layer of the next section can start. The main phases of the reconstruction are described in Figure 4-6:

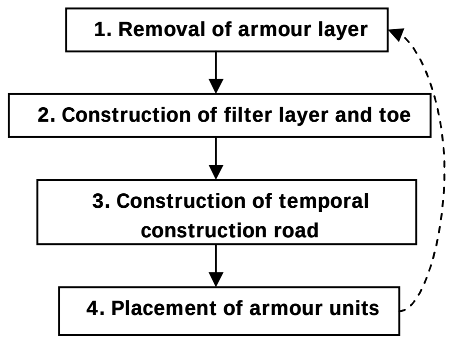


Figure 4-6 Construction phases

1. **Removal of armour layer**
   First the old toe construction will be demolished and the concrete cubes of 17 tonnes and 30 tonnes will be removed.

Figure 4-7 Dipper dredger

A dipper dredger (Figure 4-7) will remove the toe construction which is made of quarry material (1-6 tonne). The 17-tonnes cubes will be removed by the dipper dredger and the large cubes (30 and 45 tonne) will be removed by a crawler crane on a barge. The last step of the removal operation will be removal of the stone asphalt. The removal operation by the dipper dredger will last approximately 10 weeks and the special crane on the barge will work 30 weeks. Less material has to be removed for the heavy concrete cube alternative, so the removal period of the cube alternative will be shorter than the other two alternatives. Some parts of the asphalt layer of the Accropode and Xbloc alternative have to be removed by a back hoe with demolition hammer, but no removal of asphalt is required for the cube alternative.

Figure 4-8 Demolition hammer

2. Construction of filter layers and toe

After the demolition phase the bottom protection will be dumped and the filter layers will be placed on the slope of the breakwater by a side stone dumping vessel. A work week of 5 days is used for the calculations with quarry stone. The figures in the tables below are based on recent projects (Van Oord, 2004), but in practice differences occur due to local circumstances.
The differences between the quantities of quarry material of the three alternatives are small and therefore also differences in costs and construction time remain small. The total construction time, vary between 28 weeks for the Xbloc, 33 weeks for the Accropode and 29 weeks for the cube.

Characteristics	Side stone dumper	Crane
Frequency	2 trips / day	10 hours / day
Gross productivity	1,250 tonne / trip	150 tonne / hour
Efficiency	80 %	75 %
Net productivity	10,000 tonne / week	5,625 tonne / week

Table 4-8 Production rates of filter layer placement [Van Oord, 2004]

	Accropode	Xbloc	Cube 3800
Side stone dumper	198,792 tonne	171,506 tonne	178,453 tonne
Crane	72,760 tonne	57,646 tonne	59,725 tonne

Table 4-9 Quantities of quarry material

3. Construction of temporal construction road

A temporal construction road is needed to provide more space for the crawler crane, because the length of the boom of the crane is not sufficient to reach the toe construction. This construction road consists of quarry material and will be dumped by dump trucks. To reach the toe construction the width of the construction road has to be at least 14 meter. The total amount of quarry material which is dumped is approximately 33,600 m^3 (67,200 tonne). When 12 m^3 dumpers are used, 2,800 rides will be required. When the armour units have been placed, the temporal construction road has to be removed. The construction road can be constructed before and during the placement operation and it can be removed simultaneously with the placing of the armour units. When the crawler crane has placed the armour units, a back hoe removes the quarry stone and the crawler crane can place the last armour units

4. Placement of armour units

The most critical activity of the whole execution process is the placement of armour units. Placement methods are treated in Section 4.4.

4.3.3 Multi-criteria analysis

In the previous section preliminary designs of four alternatives have been made.
Before the four alternatives will be compared on cost, it is relevant to analyse globally which alternatives can be constructed in practice. The alternatives are compared on criteria, which are relevant in the construction of a breakwater. A distinction has been made between different construction stages:

- Production;
- Transport;
- Placement.

- **Production**
  Dimensions, shape and weight of the armour units determine the ease of handling during the production of the units. The Accropode and Xbloc units have approximately the same dimensions and weight and no problems occur during the pouring, hardening and transport of the units. The dimensions of the heavy concrete alternative are relatively small and will not cause any problems. Only adaptations for equipment have to be made due to the high density of the concrete
  The cubes with normal concrete have large dimensions, so the moulds of the armour units will be large and the pouring process will require a lot of time. The capacity of one concrete mixer is not sufficient to pour an armour unit (20.8 and 29.0 m^3), so two mixers have to fill one armour unit. Due to the dimensions of the units large temperature gradients can occur during the hardening process, which can cause structural instability of the armour units. Another disadvantage of this alternative is the large total amount of concrete, which is caused by the double layer of cubes.

- **Transport**
  Weight, dimensions and the number of units determine the number of trucks and the capacity of the trucks. The dimensions and weight of the concrete cube of normal concrete stand out negatively by comparison with the other three alternatives. Special cranes and flatbed lorries have to be used to transport these units.

- **Placement**
  The dimensions of the cross section of the breakwater and the lifting capacity of the crane are important parameters of the placing process. The Accropode and Xbloc units have to be placed accurately on the slope of the breakwater and cubes can be dumped when the weight of the units allows it. Accurate placement of the cubes of normal concrete is hampered by the weight of the concrete cube and another disadvantage is very wide temporal construction.

It can be concluded that the cube made of normal concrete has difficulties in every construction phase and this alternative will not be a part of the comparison. In the next chapter the Accropode, Xbloc and the heavy concrete cube will be compared on costs.

4.4 Placement Methods of Armour Units

4.4.1 Introduction

After the removal of a part of the old armour layer and the reconstruction of bottom protection and filter layers, the armour units will be placed by a crawler crane on their position. The length of the boom of the crawler crane is not sufficient to place the units near the toe, so a temporal construction road is necessary. The width of this temporal way is 14 meter. When the crawler crane is positioned on this way, the crane can reach the toe. Before the crane starts with placing on the breakwater head, this temporal construction road has to be build. This temporal construction road is built out of quarry material which is transported by dump trucks (Figure 4-9).

Figure 4-9 Dump truck

Before the placement operation starts in April, the temporal construction road of the head section (100m) is built in one week. The temporal construction road of the other sections is built simultaneously with the placement of the other armour units. When all armour units of a section are placed, a backhoe removes the temporal construction road and the back hoe follows the crawler crane. This removal operation is executed simultaneously with the placing of armour units. Logistics problems could occur on the breakwater, because different lorries, backhoes and dump trucks are present there. These problems are not taken into account. The order of construction is described in Figure 4-10.

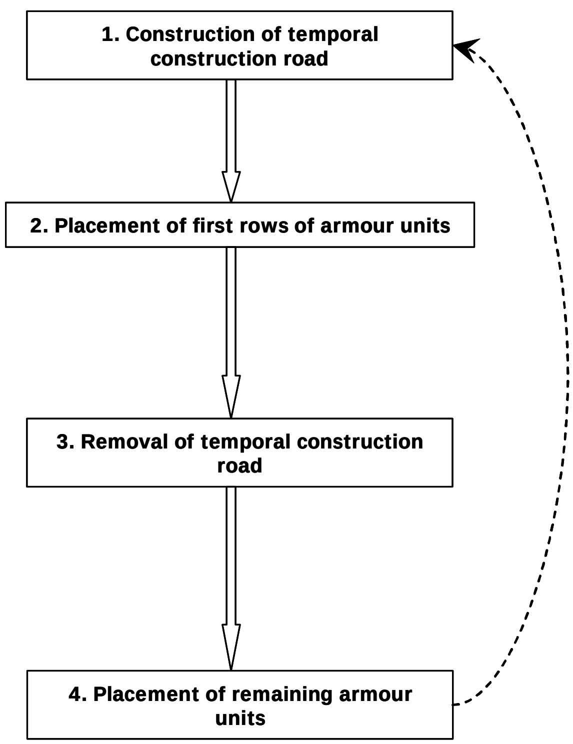


Figure 4-10 Construction scheme of placement of amour units

In 2004 an experiment has been executed to measure the placement rates of Accropodes and Xblocs. The purpose of this experiment is to make a transition of the information found in literature onto the Xbloc. In this experiment a comparison between the new Xbloc and the Accropode has been made. The main objective of this experiment is to determine the placing rates of the Xbloc and the Accropode in a scale model. With these experimental results the transition to the prototype scale placing rate has been made. This experiment is executed with dry conditions and no armour units are placed under water. The placing of armour units in water is beyond the scope of this experiment. Below a short summary of the experiment is given.

Experiment

1. Description of armour units
2. Placement methods
3. Results
4. Conclusion

1. Description of armour units

In this experiment a concrete scale Xbloc model of 4.4cm has been used. This small sized Xbloc has a theoretical placing density of $120/D^2$ [Xbloc/100 cm^2], for this Xbloc it mains 6.20 units per 100 square centimetres. The 4.8cm scale model Accropode is made of hard plastic. This small sized Accropode has a theoretical placing density of $134.4/D^2$ [Accropode/ $100cm^2$], for this Accropode it means 5.83 units per 100 square centimetres.

Placement requirements

Xbloc [DMC]

1. The packing density on the slope shall be between 98% and 105% of the theoretical value;
2. Each Xbloc shall be secured by two other Xbloc units and shall be in contact with the under layer.

Accropode [Sogreah]

1. The packing density on the slope shall be between 98% and 105% of the theoretical value;
2. Each Accropode shall be keyed in between two blocks on the row below and shall be in contact with the under layer;
3. Less than one third of the blocks shall have the anvil parallel to the slope. Units with this orientation must be distributed throughout the facing and shall not be found in groups;
4. Blocks shall be placed in deliberately varied orientations, with neighbouring units having different orientations;
5. No two adjacent blocks shall have their anvils touching

2. Placement methods

The placement of armour units by crane is simulated by the use of a string which is hand-held. There are basically two different methods of attaching (Table 4-10) the Xbloc to the string and one for the Accropode. One method of attaching the Xbloc unit to the string is with a hydraulic clamp, and the other method of attaching is by sling. The hydraulic clamp is modelled by a clip binder (see Figure 4-11). In this experiment the Accropode is only attached with a sling; the Xbloc can be attached in the same way as the Accropode. The way the sling is attached predetermines the orientation of the block on the breakwater.

The configuration of the first row of Xbloc and Accropode units is important as is has great affection the packing density, because the placing distance will be copied in the rows above, therefore the first row has an additional importance. The orientation of units has also an impact on the ease of placement of the other units on the slope. The Xbloc units in the first row shall rest on three points.

Figure 4-11 Left; Xbloc placed by hydraulic clamp Right; Accropode placed by sling

Mechanism	Advantage	Disadvantage
Sling	1. No special equipment required 2. Low cost solution	1. Difficult positioning of unit 2. Slow attachment and release
Tong	1. Easy positioning	1. Special equipment

Table 4-10 Overview of placement methods

3. Results

The theoretical density of the amour units has been described in the requirements. The density of the Xbloc on the slope was always correct; if the distance between the Xbloc units in the first along the breakwater was correct, than the density will also be correct. The distances upon the slope gave no trouble and have been automatically correct. The placement of Accropode units is more difficult because the orientation of the Accropode units determines the actual position of the centre of gravity of each unit. The distance for each row upon the slope has to be measured. This is clear from the placing densities given in Table 4-11; the Accropode placement has a wider spectrum than the placement of the Xbloc.

Experiment	Number (units)	Time (min)	Units (per hour)	Density (%)	Final result
Xbloc placed by clamp	55	22	150	100.2	Good
Xbloc placed by sling	55	25	132	100.4	Good
Xbloc placed by clamp second attempt	55	17	194	102.1	Good
Xbloc placed by sling second attempt	45	16	169	102.4	Good
Accropode place by sling	55	45	73.3	106.4	Failure
Accropode place by hand	36	25	86.4	100.7	Moderate
Accropode place by hand second attempt	28	20	84	101.5	Moderate
Accropode place by sling second attempt	45	30	90	102.0	Moderate
Accropode place by sling third attempt	45	27	100	103.8	Good

Table 4-11 Results of experiment

The scatter in the placing rate has two reasons:

1. **Kind of armour unit**
   From this experiment it can be concluded that the placement of an Accropode is more difficult than the placement of an Xbloc. Except from the stricter requirement for the Accropode than for the Xbloc another aspect determines the ease of placement; the shape of the armour unit.

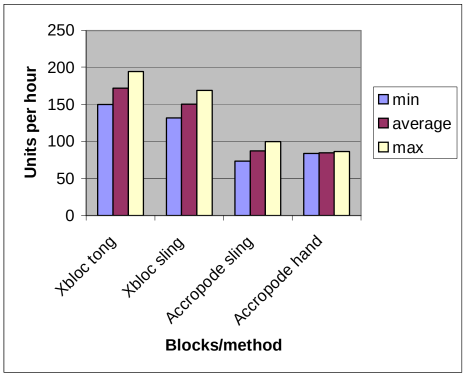
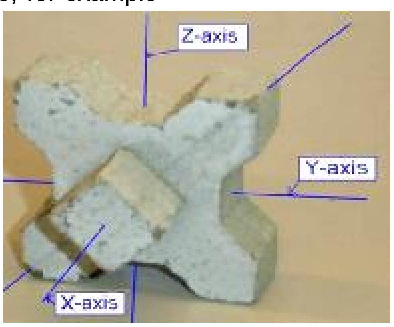


Figure 4-12 Units per hour versus blocks/method

The Accropode design is such that they are symmetric in the y axis and z-axis and both have an anvil, see Figure 4-13 The Xbloc is symmetric in both z-axis and y-axis and yz-axis (blue line), see Figure 4-13, this means that less different orientations are possible on the slope, for example



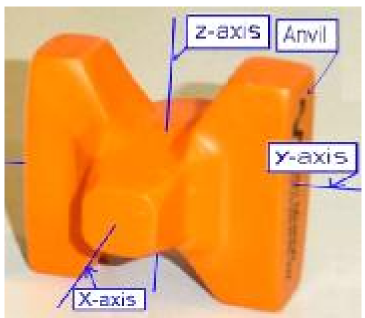


Figure 4-13 Left; Xbloc. Right; Accropode

if an Accropode unit is turned 90 degrees around the x-axis the Accropode has a new orientation but the orientation of the Xbloc is the same again, in this way the Accropode has twice as much possible orientations as the Xbloc and in that prospective more difficult to place. Secondly the placement of an Xbloc is also easier than an Accropode because it has no anvil but a notch, with this difference the Xbloc can easy be placed between two other blocks without getting jammed on the anvil.

2. Placing methods

The placement by a hydraulic clamp has two great advantages in respect to the sling: It is faster in attaching and releasing an Xbloc and it is more precise getting the right position and orientation. The sling has an advantage when a small mount of units have to be placed; as a sling can be handled by every contractor with a simple crane.

4. Conclusion

For the Accropode it is crucial to place the first row at the right distances, after that is done the requirements of Sogreah are a lot easier to achieve. The Xbloc also has to be placed at the right distances, but if the individual distances are not entirely correct the rows above will correct these distances, this will only happen if the average distance is correct. This can be seen in the first experiment with Xbloc. With this knowledge it means that the surveying the first row of Accropode units is more important than by the Xbloc units. On a prototype it is favourable to check every Accropode on the correct distances.

For the Xbloc it is sufficient to check the average distance every 4 to 5 units. In the experiment this is done by ruler, but in reality the first row of units is the most difficult row to control, for example by diver. The time differences between the Xbloc and the Accropode in the experiment are mainly caused by the time required to get the right position and orientation during placement of the units. This would also be the case on a prototype. In the scale model the time difference in placement between Accropode and Xbloc is in the order of 1.8 to 2, see placing rates Table 4-11. This difference can not automatically be translated to placing rates in practice. Due to the scale differences and the environmental conditions the placement rates of the Xbloc will be smaller than the values in the experiment. The scale factor of the placing rates between the Xbloc and the Accropode lies between the 1.25 and 2. The value of 1.25 is based on personal communication with a technician of Delft Hydraulics.

Workability

A work week consists of 5 days and a work day of 12 hours is assumed for this project. In the period April-September the breakwater is rehabilitated, because during the other months of the year the environmental conditions are too harsh.

Workable time in period 1992 – 2002 (Measuring station IJmuiden, deep water)

April	79.1 %	*	22	=	17.4 work days
May	83.3 %	*	21	=	17.5 work days
June	80.5 %	*	22	=	17.7 work days
July	83.8 %	*	22	=	18.4 work days
August	82.2 %	*	22	=	18.1 work days
September	67.1 %	*	22	=	14.8 work days
Total annual work days					**103.9 work days**

To simplify this calculation a work week consists of four workable days and one unworkable day, this means 26 labour weeks per year. This day is unworkable, because the environmental conditions (H_s=1.5m) restrict the crawler crane. To simplify the calculation the number of units used in the calculation differs from the number of units of the design (small deviation 0.1 – 0.9 %).

4.4.2 Accropode

The information on placing rates for the Accropode given by Sogreah, the patent holder, is 50 - 120 units a day (without further classification). Sogreah has indicated that the rate of placing decreases, when the dimensions of the unit increase. The placement of Accropods has to be checked by a consultant of Sogreah. These numbers have been verified with information from conference paper, experience of DMC and other sources. All sources indicated a lower placing rate, in the order of 30 – 60 units a day. Table 4-12 gives an overview of the placement requirements.

Accropode placement requirements [Sogreah]	
A	The packing density on the slope shall be between 98% and 105% of the theoretical value
B	Each Accropode shall be keyed in between two blocks on the row below and shall be in contact with the under layer
C	Less than one third of the blocks shall have the anvil parallel to the slope. Blocks with this orientation must be distributed throughout the facing and shall not be found in groups
D	Blocks shall be placed in deliberately varied orientations, with neighbouring blocks having different orientations
E	No two adjacent blocks shall have their anvils touching

Table 4-12 Placement requirements for Accropodes

The Accropodes are placed with a sling around the armour unit. In order to fit an Accropode, the armour unit needs the right orientation. In order to get the right orientation each Accropode has its own method of attaching the sling. Calculations of the placement are based on production figures of recent projects. Production rates can not be based on one project, but also on other projects with other circumstances, see table below.

Project	**year**	**type**	**number**	**weight**	**m3**	**place**	**Equipment**	**placing rate**	
Reconstruction Fregate Island	2000	Accropode	875	4,7	2	submerged	crawler crane	1.5-3	per hour
				4,9	2,085	crest	crawler crane	5	per hour
Scarborough, UK	2003	Accropode	4000	2,7-3,8	6,3-9			5-6	per hour
Hartlerpool marina,UK	1991	Accropode		6-9,6	2,6-4			8	per hour
Map Ta Phut, Thailand	1996	Accropode	44630	2,4-6	1-2,5		back hoe	12-18	per hour

Table 4-13 Placing rate of recent projects

The placing rate can be influenced by several parameters:

- Equipment and size of armour units;
- Skill of crew;
- Way of placement;
- Environmental conditions;
- Position of armour unit on the slope
- Smoothness of under layer.

- **Equipment**
  It is likely that hydraulic lifting equipment can handle armour units faster than a conventional crane; but the size of the unit is not the only factor that influences the placing rate.

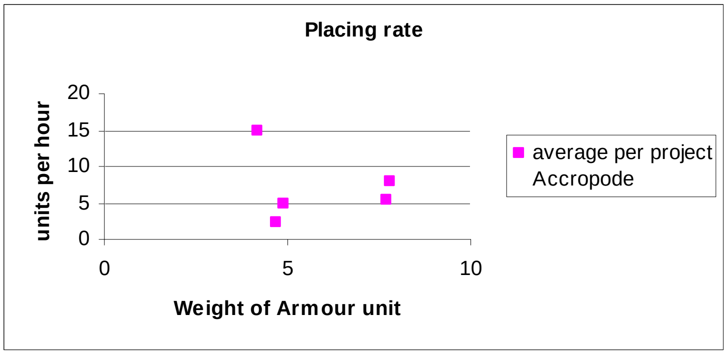


Figure 4-14 Placing rate versus armour unit weight

- **Skill of the crew**
  A well trained crew can achieve much higher placing rates and it is important that the contractor has the right equipment. Larger projects have a higher placing rates than small or repair projects; the cause of this increased placing rate is mainly the consequence of a learning process. After a while the crane driver mastered the placement of units, as a consequence the placing rate increases. This benefit of learning process pays of in larger projects, but when you only have to place a few hundred units the project is finished before the crane driver masters the placement. As a consequence repair projects have a very low placing rate.

- **Way of placement**
  Placing an Accropode from waterborne equipment is more difficult than from a stable breakwater.

- **Environmental conditions**
  In rough conditions it is more difficult to place the unit on the correct location checking the position of a block in rough conditions is very difficult or even impossible.

- **Position of the unit on the slope**
  Checking the submerged placement of the unit takes more time than placement above the waterline.

- **Smoothness of under layer**
  When the under layer is not smooth enough, difficulties with interlocking will arose.

The placement rates will be used for the following calculations, which are based on recent experiences of the construction Fregate breakwater, which have been built by Interbeton. Fregate Island is one of the islands of the Seychelles. At Fregate Island a total of 995 Accropodes have been transported to the island and have been placed on the breakwater. The placement of 2 m^3 Accropodes at the toe structure was a very time consuming operation and hampered due to strong breaking waves near the breakwater. In order to guarantee interlocking it was important that divers visually inspect the Accropodes at the toe. The Accropodes were lifted with a hook that could release the sling with the secondary hoist rope, see Figure 4-15.

Figure 4-15 Hook with secondary hoist rope

Positioning Accropodes under water was done by mains of predefined coordinates and the use of total station survey equipment. Progress of placing was very much depending on the sea and weather conditions; during rough sea conditions it was impossible to install any Accropode unit at all. The trunk of the breakwater was heavily exposed to breaking waves, this considerable slowed down the placing operation an average of 1.5 per hour was derived at the trunk.

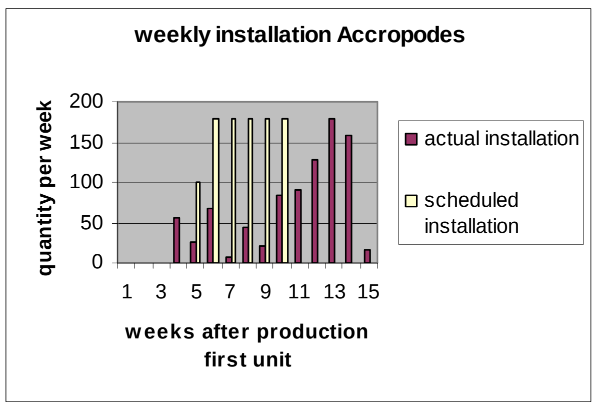


Figure 4-16 Installation of Accropodes

Around the head of the breakwater less hinder of breaking waves was experienced, (deep water), this increased the installation of Accropode units to an average of 3 per hour. Above low water level at the head of the breakwater the placement of the 4.9 tonne [$2.1m^3$] Accropode units could further accelerate by manual assistance from workers on shore, an average of 5 per hour was achieved. The placement of all Accropode units has been done by a 100 tonne crawler crane. The actual installation did not meet the scheduled rate, see Figure 4-16.
Not only the experience of Fregate Island will be used for the calculations, but also the rates of Scarborough (Table 4-13), where the situation is comparable with IJmuiden (North Sea conditions).

4.4.3 Xbloc

In 2001 the Xbloc has been developed by DMC and until now only a small project with Xblocs has been executed. So there are no prototype placement rates available for these armour units. It is difficult to predict the placement rates of Xblocs in actual practice, because there are only results available from an experiment of DMC with small scale Xblocs and Accropodes, which is described in Section 4.4.1

4.4.4 Cube

In The Netherlands and Spain concrete cubes have been used as armour units on breakwaters. The breakwaters of Scheveningen and Hook of Holland have a double layer of cubes as armour layer. In Scheveningen the cubes were placed by two mobile land cranes and in Hook of Holland the cubes were placed by two ships. These ships were specially designed for this project and comprehensively tested in laboratories. The ships were very stable and a gantry crane was fixed to the stern of the ship. This gantry had two trolleys, which had both hydraulic clamps to pick up the concrete cubes, see Figure 4-17. The placement rates of these gantry crane were high, approximately 20 cubes per hour.

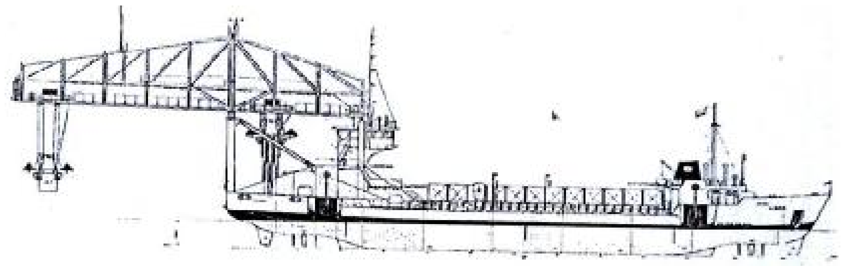


Figure 4-17 Concrete blocks dumping vessel for Hook of Holland breakwater

Figure 4-18 Block clamp for the breakwater of Scheveningen

In Spain the situation is different from IJmuiden, because very large cubes have been applied with purpose made cranes, so these figures can not be used for this thesis study (Table 4-14). This project in Spain is a reinforcement of an existing breakwater after failure. For the placement of cubes, a special crane had been built onto the breakwater. With this crane placing rates as listed in Table 4-14 are achieved.

Size	Number	Place	Distance	Speed
8 tonne	95,000	Filter layer	180m/month	
25 tonne	40,000	Toe berm at -14m		35 per day
100 tonne	17,500	On surface	40m/month	55 per day
		Submerged		35 per day

Table 4-14 Rates from a breakwater project in Spain, paper 16 of Coastlines, structures and breakwaters Conference, March 1998

Contrary with interlocking units, cubes can be dumped on the breakwater. Concrete cubes can also be placed by crane when high layer density is required, e.g. single layer of cubes (Figure 4-19). Dumping of cubes will increase the production rate, because the positioning time of dumping is shorter than the positioning time of placing.

Figure 4-19 Concrete cubes for a cruise terminal at St. Vincent, Bahamas

Dumping of cubes for the armour layer of a breakwater is generally done by a gantry crane on a ship or special crane on the breakwater. This crane is equipped with caterpillar tracks, so the crane is mobile. The crane has to be rigid to absorb the dynamic forces and a long boom is required, otherwise it can not dump cubes near the toe of the breakwater. This crane has to be build especially the reconstruction of the breakwater of IJmuiden. Two types of cranes are appropriate for this project:

1. **A specially built gantry crane**
   This crane has to satisfy the requirements of the reconstruction of the breakwaters of IJmuiden. A disadvantage of this solution that this crane only can be used for this project and all costs of this crane have to be written down on this project. The probability that this crane can be used for another project is small. The purely estimated (based on dimension and lifting capacity) price of such a crane is € 3,000,000. This information is provided by Figee B.V., a Dutch crane building company.

2. **Crawler crane based on mobile gantry crane**
   This option consists of a mobile gantry crane as base part of the construction, on which a large crawler crane is positioned. This crawler crane (DEMAG 2400) has sufficient reach and lifting capacity to dump the cubes. A remark can be make on this option, the dynamic behaviour of this crane can reduce the placement rates. The price of the base part is € 250,000 for the adaptations of the crane and the rent of a period of 2 years. The costs per week (5 labour days) of the DEMAG 2400 (450 tonne) are € 35,000. This information is provided by BAM Civiel, but these prices are estimated values based on recent project, so no rights can be derived from this figures.

The second option will be used for the calculations, because the prices of this option are the most realistic. The crane picks the cubes up from the flatbed lorry with the help of a hydraulic clamp and brings the cube to the right location. When the cube is released it is necessary that it has the right location and orientation. An advantage of dropping in water is minimizing the swing on the cube and decreasing the loading on the crane. Another method of dumping is dumping from a side stone dumper, but this method will be not taken into account because this is not appropriate for the reconstruction of the breakwaters of IJmuiden.

Dumping process

The dumping of cubes can be divided in four different phases:

1. Positioning of the cube
2. Dropping process above water line
3. Dropping process from water line to bottom
4. Landing of cubes

1. **Positioning of the cube**
   The first part of the placing process is positioning of the crane on the right position by the crane with the help of positioning equipment. When the cube is on the right location, the crane driver drops the cube above the water line or he lowers the cube to the waterline and drops the cube when the cube is in the water.

2. **Dropping above water line**
   The velocity of the cube, when it hits the water, depends only on the height of the drop. When the cube hits the water, it loses some of its kinetic energy and it slows down.

3. **Dropping from the water line to bottom**
   This process depends on the previous phase. When the cube is released a certain height above the water it has a higher velocity than the cubes which are released in the water. The point of dropping determines the degree of turbulence behind the cube.

Figure 4-20 Enclosure of air during the fall of cube

4. **Landing of cubes**
   When the falling cube hits the bottom, the first contact surface of the cube with the bottom is small. The slope of the breakwater and the quarry stone of the filter layer cause this event. Furthermore the falling cube will turn over, so the cube does not fall with a face downwards, but a corner point of the cube will be the lowest point of the cube. A consequence of that small contact surface is the turning of the cube.

Parameters

During the dumping of cubes for the armour layer of a breakwater the following parameters play a role:

1. Distance between the cubes within the dumping pattern
2. Method of dumping
3. Order of dumping
4. Accuracy of dumping
5. Orientation of cubes within the horizontal plane
6. Orientation of cubes within the vertical plane
7. Order of placement

8. Height of dumping above the waterline
9. Maximum falling depth in the water
10. Number of layers
11. Influence of waves
12. Influence of currents
13. State of filter layer
14. Density of the cubes

The slope of the breakwater is not a variable, because a gentler slope, in case of cubes, will cost more material and labour. A number of these parameters are described below.

1. Distance between the cubes with the dumping pattern

Before the placement starts, a dumping plan is listed. This dumping plan holds the coordinates of each cube, the corners of the cubes, and the distance between the coordinates can be varied in x-direction and in y-direction.

2. Method of dumping

Besides dumping of cubes by crane, it is possible to dump them by a side stone dumper.

3. Order of dumping

The most common way of dumping is working from the under part till the upper part of the breakwater diagonally. Another way of dumping is dumping horizontally (see Figure 4-21). An advantage of this method is than the chance on bouncing and rolling of the cubes into the wrong row is smaller

4. Pattern of dumping

Four possibilities for pattern of dumping:

1. The units are dumped equally to the axes (x-axis and y-axis).
2. The units are dumped under an angle of 45 degrees with the axis.
3. The units are dumped equally to the axis, but the horizontal rows jumps a distance of a half cube length plus the half distance between them.
4. The units are dumped under an angle of 45 degrees with the axis. But the units jump a distance of a half cube length plus half the distance between them.

In practice it is proven that the first option does not work, because vertical trenches will exist between the units, by which the under layer can erode (Valproeven met een betonkubus, M856 [1969]). The third and fourth option are sticked by six surrounding cubes instead of four cubes of option 2.

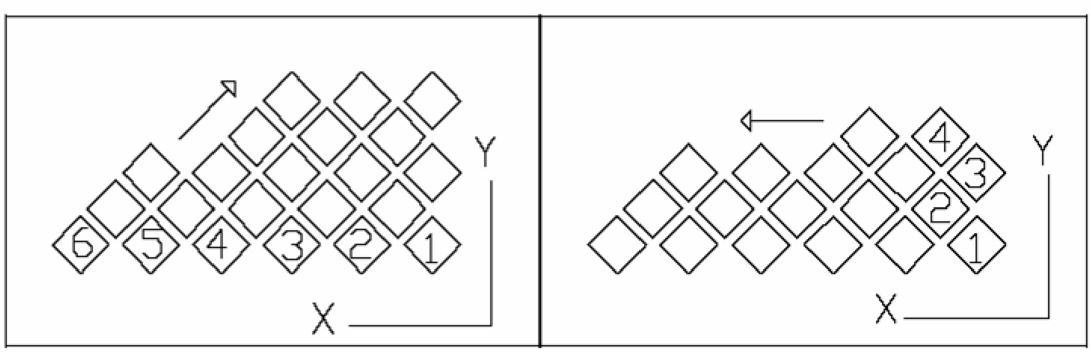


Figure 4-21 Left: Diagonal rows. Right: Horizontal rows

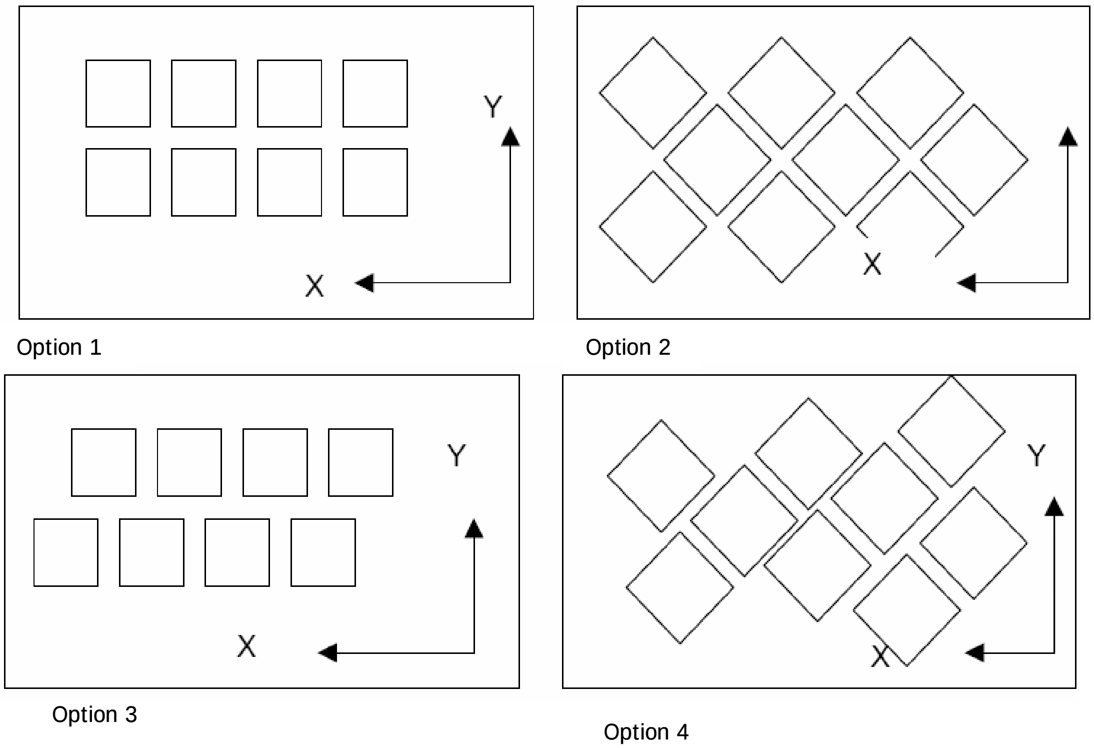


Figure 4-22 Overview of patterns of dumping

5. **Dumping height above waterline**
   The dumping height above the waterline can vary from -0.5*D_n (cube under water) till an infinite distance above the waterline. In practice the maximum dumping height will be several times the diameter of the cube. The gravity point of the cube is relevant for the determination of the height of the cube above the water line.

6. **Maximum water depth**
   The maximum depth in the water is the distance between the water line till the bottom (the transition from the toe to the slope). In practice this distance varies between 2 or 3 times D_n till 10 or 15 times D_n.

7. **Number of layers**
   Up till now all breakwaters with an armour layer of cubes have two layers of cubes.Recently research has been done with a single layer of cubes. It is possible that the porosity volume and the placement density of the second layer is influenced by the first layer.

8. **Density of the cube**
   The density of the concrete determines the weight of the cube and also the equilibrium velocity. A higher equilibrium velocity causes a shorter dumping time of the cube. It could be possible that a heavier cube, which has a shorter dumping time, has a smaller deviation from the vertical.

In 2001 Van der Vliet has done this thesis research on the behaviour of cubes during the dumping process, (Ref [17]). Due the limited time he has chosen four of the 14 parameters above, which he took into account for his research:

1. Distance between the cubes within the dumping pattern;
2. Orientation of cubes within the horizontal plane;
3. Height of dumping above the waterline;
4. Maximum falling depth in the water.

Conclusions of his research:

- Small cubes have a larger deviation from the vertical than larger cubes. This can be explained because smaller cubes do lose less energy when they hit the bottom, so they have more energy for rolling and bouncing.
- When cubes are dumped in horizontal rows, the first row has to be dumped on the slope and the second row on the toe and all following rows on the slope.
- Cubes, which are released in the water (water depth = $10*D_n$), show a larger deviation with the vertical than cubes which are released above the water line.
- Cubes, which are released above the water line, enclose an amount of air under the cube, which comes free during the fall in the water.

4.5 Maintenance Strategy Breakwaters IJmuiden

4.5.1 Introduction

Section 2.5 describes different methods of maintenance and monitoring. The lifetime of the breakwaters after the reconstruction will be 100 years. Normally a maintenance plan is designed for twenty or thirty years and maximal fifty years. This limited time horizon is chosen, because the uncertainty increases with a longer period and costs prediction is not realistic anymore. Uncertainty is determined by changes of working methods, equipment, material, prices and changes of demand. It is also difficult to give the breakwater a residue value over 100 years.

Experience on deterioration of breakwaters is lacking, because breakwaters with double-layer of cubes exist approximately 50 years and every breakwater is unique (different circumstances).The first breakwaters with single layer of interlocking units have been built since the eighties and no maintenance has been executed on them since then. Deterioration of individual units is also modest, because non reinforced concrete has a long lifetime in sea water. Only rocking of units can diminish the lifetime of an individual unit.

Maintenance philosophies

Due to all uncertainties concerning the behaviour of the breakwater during its lifetime different maintenance philosophies are composed. These philosophies are related with damage development of an armour layer in time. During the design phase a choice can be made between high construction costs and low maintenance costs or less high construction costs and higher maintenance costs.
A subdivision can be made between the different armour layers:

- **Rubble mound breakwater**
  The traditional multi-layered breakwater and the berm breakwater, which both totally consist of rock, allow a damage of 5% (see Section 4.5.2). This maintenance philosophy allows some damage during the lifetime of the breakwater to lower the initial construction costs.

- **Double layer of cubes**
  The double layer of cubes is designed with the Hudson formula, in which a K_d factor is used that allows a certain damage (0 - 5%). A choice has to be made between two options when damage has occurred:

  1. **Do nothing**
     When damage occurs to the outer layer, the inner layer can still deliver protection for the total structure. Replacement of displaced cubes and placement is not necessary. Monitoring is recommended after heavy loading in storm periods.

2. **Repair of armour layer**
   When some cubes are displaced out of the armour layer during a storm, they have to be replaced after the storm. Also new block can be placed on the armour layer.
   If more damage is tolerated, smaller cubes can be used for the armour layer. A certain damage value can be put in the Van der Meer formula. The higher the tolerated damage value, the dimensions of the cubes will be smaller. The next section will outline this formula.

- **Single layer of interlocking units**
  Damage to the armour layer of interlocking units can be fatal, because a few dislocated units can induce the failure of the total layer. Start of damage and start of failure lie close to each other. During the design phase a safety coefficient is used, because damage of the armour layer during its lifetime is not tolerated. The armour layer is designed on no damage, so no maintenance is required during the lifetime of the breakwater.

However the maintenance philosophies differ, for every type of armour layer monitoring is required to check the state of the breakwater.

4.5.2 Maintenance cubes

The alternative with the double-layer of cubes is designed with a 1/100 year wave height. In Section 4.5 the probability of exceedance of this wave height is calculated over a lifetime of 100 years, 63 %. When the design wave is exceeded during storm periods, cubes will be displaced from their original position and the armour layer will be more vulnerable for new storms. After these storms these relocated blocks can be brought back to their original position and new blocks can be placed.

In Section 5.5 the formula of Van der Meer, which calculates the number of dislocated cubes, is described. The dimensions of the cubes, which are used in this research, are calculated with the help of the Hudson formula. This formula uses a K_d coefficient, in which damage is taken into account, but damage is one of several parameters, which are taken into account, see Section 4.6. The design with two layers of cubes is checked with the formula of Van der Meer, with the following wave heights, see Section 3.2.3. The number of waves is during a storm is approximately 1000
(see Table 4-15).

Frequency [i/year]	H_s [m]	H_s offshore [m]	Tm [s]	N [-]
10	3,2	3,5	6,3	1000
1	5,3	5,5	7,9	1000
0,1	6,5	6,7	8,7	1000
0,01	7,5	7,6	9,3	1000
0,001	8,2	8,4	9,7	1000

Table 4-15 Wave heights

The number of dislocated cubes is expressed as damage number. When on a regular scale maintenance is planned, the maximum value of the damage number may not exceed 1.5
(see Table 4-4). The damage numbers of the cubes of the trunk and head section are plotted in Figure 4-23. The value is not realistic, because Van der Meer has only executed 2-D experiments on trunk sections. The Van der Meer formula is not valid for armour units on head sections.

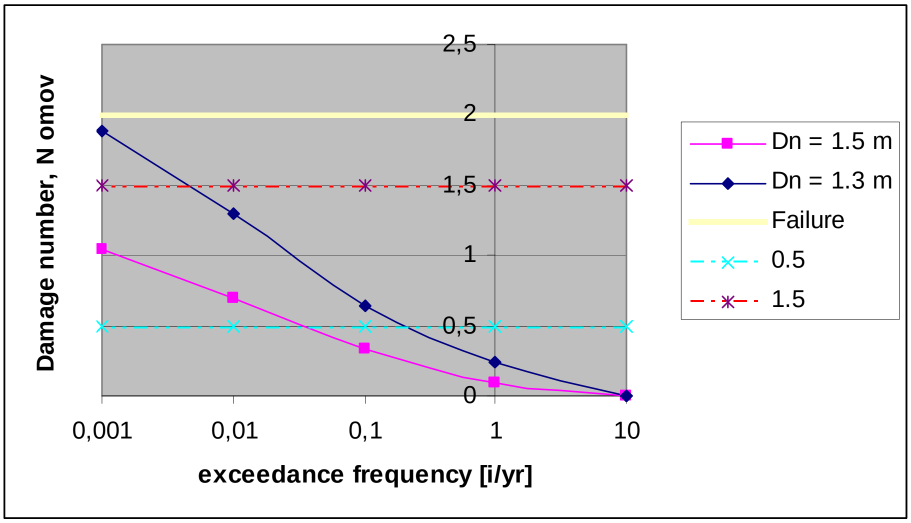


Figure 4-23 Damage number N_{omov} of concrete cubes

During the once in 100 years wave height the damage number is 1.3 for the cubes on the trunk section. This means damage, but no failure of the armour layer. Monitoring and when necessary maintenance will be carried out after heavy loadings during storm periods.

4.5.3 Maintenance interlocking units

In general breakwaters with single layer of interlocking units do not require maintenance, because they are designed with a safety margin. When the breakwater with the interlocking units has been constructed, time-based monitoring will take place. The first five years the position, orientation and condition of the interlocking units will be checked every year. When little or no change is recorded in that period, the interval period of monitoring can be increased to 5 year.

Accropode

In practice maintenance for a breakwater with interlocking units is reduced to monitoring and maintenance will be minimal. To calculate the start of damage of a breakwater with Accropodes, the formulae below can be used (recommended by Sogreah). D_n = 0.7 D if D is the height of the unit, see Table 4-16.

Start of damage $$\frac{H_s}{\Delta D_n} = 3.25$$

Start of failure $$\frac{H_s}{\Delta D_n} = 3.8$$

	D_n [m]	Δ [-]	H_s Start of damage [m]	H_s Start of failure [m]
Trunk	2.3	1.32	9.9	11.55
Head	2.63	1.32	11.3	13.2

Table 4-16 Damage wave heights of Accropode armour layer

Xbloc

In practice maintenance for a breakwater with interlocking units is reduced to monitoring and maintenance will be minimal. To calculate the start of damage of a breakwater with Xblocs, the formulae below can be used (recommended by DMC b.v.).

Start of damage
$$\frac{H_s}{\Delta D_n} = 3.5$$

Start of failure
$$\frac{H_s}{\Delta D_n} = 3.9$$

	D_n [m]	Δ [-]	H_s Start of damage [m]	H_s Start of failure [m]
Trunk	2.1	1.32	9.7	10.81
Head	2.31	1.32	12.85	11.89

Table 4-17 Damage wave heights of Xbloc armour layer

The wave height of 1/1000 years wave is 8.2m (see Figure 3-10). The chance on damage of the armour layer during the lifetime is very small for the Accropode and the Xbloc. Only damage could occur due to failures of the design and failures during the construction of the breakwater. The interlocking units are over designed, when the hydraulic stability is considered, but the dimensions are necessary to ensure the structural integrity of the units in case of rocking.

4.5.4 Analysis of maintenance strategies

The development of a maintenance strategy for breakwaters is difficult due the lack of knowledge of maintenance of breakwaters and some types of breakwaters do not require maintenance. It is difficult to express those differences in costs, but it is obvious that the maintenance costs of a double layer of cubes (tolerated damage 0-5 %) are higher than the interlocking units (no damage).

The armour layer of a double layer of cubes allows damage; a small number of cubes can be dislocated, because the second layer has reserve capacity to withstand the wave forces. After the storm periods the cubes can be replaced on the armour layer. A regular maintenance scheme can not be mapped out, because it is difficult to predict when an armour layer requires maintenance. Regular monitoring is recommended.

The armour layer with interlocking units does not require maintenance, because they are designed on no damage, which implies that a maintenance strategy is redundant. In theory this is valid, but in practice failures could be made during design, production and placement of the units, so monitoring is recommended.

The armour layer is a part of a system, the breakwater, which consists of more parts, such as the core, filter layers, toe construction and the crest element. Failures of one these elements can induce failure of the system, so monitoring and maintenance have not only to be adapted to the armour layer but also to the other parts of the breakwater.

5 Cost Comparison of Alternative Designs

5.1 Introduction

This chapter deals with the cost parameters of the construction process of a breakwater. The construction process can roughly be divided into two different construction stages:

- Production
- Placement

Transport is not a separate phase, because in every phase of the construction transport of materials and equipment takes place. Total costs of a coastal structure, such as a breakwater, are divided in initial costs and costs during its lifetime (see Figure 5-1).The initial costs consist of preparation costs, design costs and construction costs. This comparison takes only the construction costs into account, because the preparation and design costs are approximately equal the same for all three alternatives. Operation costs of a breakwater are negligible, because it does not have an active function.
The only costs during its lifetime are maintenance costs. Maintenance can be employed on all parts of the breakwater, but the emphasis lays on the maintenance of the armour layer.
This part of the breakwater is important, because it protects the total structure. The maintenance of the toe is also relevant, but for all three armour units the same, so it is out of the scope of this research. Downtime in the harbour basin caused by wave transmission through and over the breakwater plays a minor role.

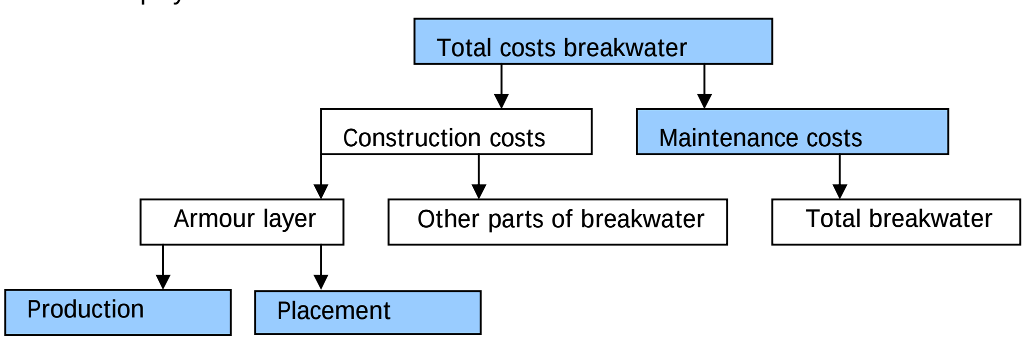


Figure 5-1 Overview of total costs

In the previous chapter designs with the different armour units have been made for the breakwaters IJmuiden. An appropriate construction method has also been chosen for the breakwaters IJmuiden and maintenance strategies have been discussed in Chapter 4. This chapter calculates and compares the production and placement costs of the three armour units. The next chapter reviews these calculations in a sensitivity analysis. Section 5.2 gives an overview of the cost factors of the comparison study. In Section 5.3 a calculation is made of the number of placement days, which determine a big part of the placement costs. Section 5.4 holds the calculation of the costs of the production process of the concrete armour units. Finally in Section 5.5 the total costs of placement and production of all three armour units are compared.

5.2 Structure of Cost Comparison Study

The construction costs depend on the breakwater geometry. A more conservative design will implicate higher construction costs, but will provide a more robust breakwater. The total costs consist of labour costs, equipment cost and material cost. This cost comparison will compare the alternatives globally and will not calculate all labour, equipment and material costs in detail.
This comparison study compares the costs of all three alternatives during the production and placement phase. This section gives an overview of all cost factors during these construction phases.

Production

- **Quarry material**
  The grading of the quarry material, which will be used for the IJmuiden breakwater, is very large. The price of the quarry material is determined by production and transport. The quarry stone is placed by side stone dumpers or by crane. The small grading classes are placed by a side stone dumper and the large stones are placed individually by crane. A thorough market analysis of quarry material is beyond this thesis, so a standard value of a quarry material price is used in this study. This price is an index number which is based on recent projects. The real costs of quarry material are time-dependent, but for this calculation index numbers are used.

Activities	Price dumper [euro]	Prices crane [euro]
Purchase stones, stockpiling and transport to crane	25 euro/tonne	30 euro/tonne
Placing of quarry stone	12.5 euro/tonne	12.5 euro/tonne
Mobilisation	750,000 euro	750,000 euro
Demobilisation	150,000 euro	250,000 euro

Table 5-1 Cost of quarry material

- **Concrete prices**
  In the Netherlands the concrete price is determined by the topography, market situation and access to infrastructure. In Limburg (southern region of the Netherlands) the cost of concrete is lower than the rest of Holland, because of the availability of aggregates in the region. The prices in the region of Amsterdam and Rotterdam are interesting for this project, because when the casting takes place on old casting yard the price of concrete from Amsterdam is relevant. One of the possibilities for pre-casting is a casting yard in Schiedam, so the price of concrete from Rotterdam is relevant.

Strength	Environment class	Consistency	Amsterdam [euro/m^3]	Rotterdam [euro/m^3]
B35	4	3	80	78
B35	4	2	79	77
B25	4	3	-	76.8
B25	4	2	-	75.8

Table 5-2 Concrete price region Amsterdam and Rotterdam ,August 2004

The required quantities of this research also influence these prices of concrete. When more concrete is required, the prices could change positively. Also the density of concrete determines the price of concrete. When a density of more than 3,200 kg/m^3 is required, all aggregates (sand and gravel) have to be removed out the mixture and finer Magnadense (kind of magnetite) is added to the mixture. Below the density of 3,200 kg /m^3 coarser Magnadense has to be added to the mixture. These differences cause also a difference in price, see Table 5-3.

Density of concrete [kg/m^3]	Cost of Magnadense [euro]/m^3 concrete	Total cost concrete/m^3 [euro]
3,100	70-80	120-180
3,800	150-200	200-300

Table 5-3 Price of material/m^3

Intermezzo heavy concrete

Heavy concrete is concrete with iron ore (magnetite) as aggregate instead of sand and gravel. The density of this concrete can reach a value of 4,000 kg/m^3. Armour element can reduce in weight and size because of this high density. Magnetite is a natural iron ore. Iron ore's physical and chemical properties do vary from mine to mine.

The raw iron ore consist of granular material with a maximum grains size of 25 mm. Magnetite is found in two mines at Kiruna, in the North of Sweden. The Swedish iron ore consist of approximately 85 % of iron oxides, the other elements are manganese, calcium, phosphorus, silicon, magnesium oxide and a large number of trace elements. The magnetite has an angular shape and a grey colour. It is chemically very stable and after long-term exposure to air and water no sign of erosion, corrosion or weathering. The angular shape of the magnetite causes a good attachment of the magnetite to the cement. Magnetite, when used as an aggregate in mass pours has the effect of lowering the maximum temperature by acting as a heat sink. The heat is retained longer and released more helping to reduce the risk of thermal cracking. Some characteristics of Magnetite:

- Thermal conductivity Magnetite (Fe304) 9.7 W/mK (Watt per meter Kelvin)
- Heat capacity Magnetite 734 J/kgK (Joule per kilogram Kelvin)

Magnetite has been frequently used in civil engineering, for example underwater flooring, ballasting of bridges (StoreBelt) and other applications. The process of production of heavy concrete does not differ with normal concrete. The concrete factory has to be adapted to the weight of heavy concrete and also the other equipment as the mixers, moulds, etc. This adaptation of equipment and the availability of magnetite are reflected into the price of heavy concrete. The price of heavy concrete depends on the location of the construction area (access to deep water) and required amount of concrete.
See Appendix IV for properties and characteristics of Magnadense.

- **Moulds**
  The price of the moulds is determined by the dimensions and the shape of the armour unit. Material and labour costs are included in the price of 2 euro/ kg steel. Manufacturing special shapes does not require extra cost.

- **Storage area**
  The rent of m^2 per week in the Region IJmuiden depends on the demanded dimensions of the area and the location of the area. The costs of storage are not taken into account in this study.

Placement

- **Transport**
  The cost of the transport during the execution depends on the number of trucks and the number of labour days. The price of a flat bed lorry is 1,000 euro/ week and it has a maximum capacity of 50 tonne.

- **Placement**
  The costs of placement of quarry material and armour units depend on the number of labour days, when the side stone dumpers and the cranes are deployed during the execution period. The equipment and labour used during the placement are time-dependent parameters:
  Dive team : 20,000 euro/week
  Manitowoc 4100W-S2 : 6,850 euro/ week (exclusive mobilisation/demobilisation)

5.3 Placement Costs of Armour Units

5.3.1 Introduction

In this section a calculation of the number of placement days is made of all three armour units. In a previous section the placing methods are already described. With this information and the percentage of unworkable conditions, which are described in the previous chapter, the number of construction weeks can be calculated. The number of construction weeks determines the labour and equipment costs, so also the total placement costs. At the end of this section a comparison is made of the placement costs of the three armour units. The calculation of placement days is divided in two parts, first the trunk section will be calculated and subsequently the head section. This calculation does not include differences between placing rates between head and trunk section, but due more severe environmental conditions on the head, a factor of 1.1 is taken into account for the production figures.

5.3.2 Accropode

The calculation is divided in two parts, placement of units on the trunk section and on the head section. The units are placed by a crawler crane, which is positioned on the crest element. To ensure a realistic comparison the unworkable days are taken into account. First an estimation of the placement time is made of a section of 10 meter. The crawler crane starts with placement of units near the toe construction and subsequently it works to the crest element. When the crane has finished a row, it has to relocate to the next row. This relocation of the crane lasts approximately 30 minutes. Due to the temporal construction road, the crane can not place the last units and it will place those units, when all armour units of a section of 100 meter have been placed and the temporal construction road has been removed. All those units are located above the waterline. For this calculation the placing rates of Fregate Island and Scaborough are used, however the environmental conditions are not totally comparable with IJmuiden. The placement rate depends on the position of the armour unit on the cross section. The cross section is divided in three parts: under waterline, breakers zone and above waterline, see Figure 5-2 and Figure 5-3.

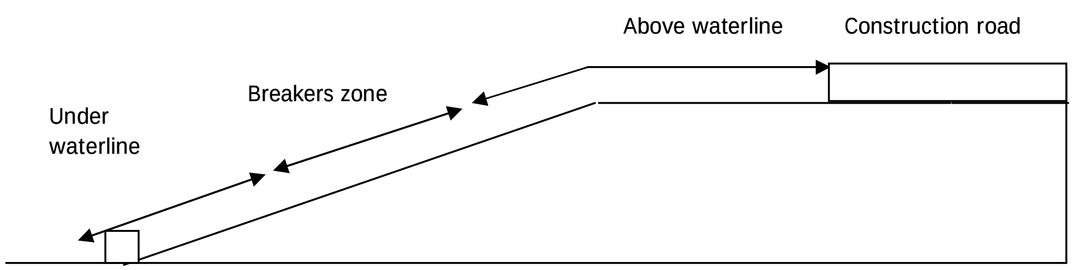


Figure 5-2 Definition of placement zones

Under waterline	3	units/hour
Breaker zone	1.5	units/hour
Above waterline	5	units/hour

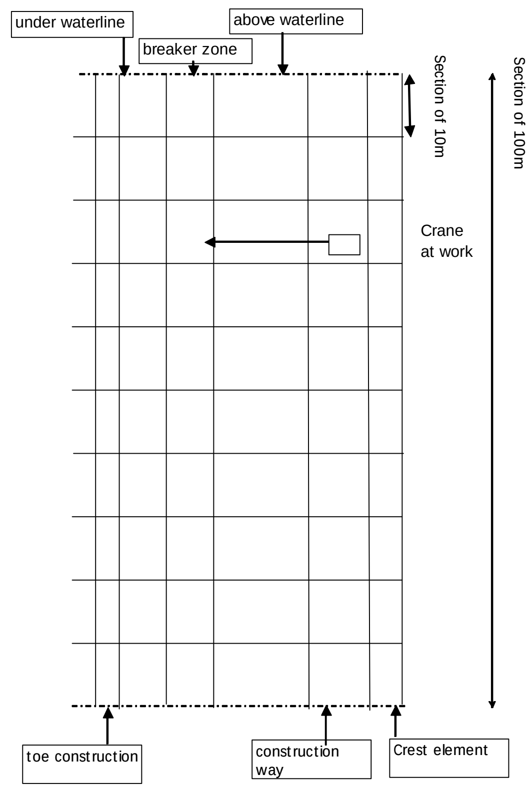


Figure 5-3 Top view of breakwater

Trunk section

3,360 units have to be placed on a section of 700m. On a section of 100m 30 rows can be placed, 16 units a row and four elements have to be placed later due the construction road.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	3	3	1
Breakers zone	2	1.5	1.33
Above waterline	7	5	1.4
Dive inspection			1.0
Movement crane			0.5
Total			5.25

Table 5-4 Procedure of one row

After the placement of one row, a diver inspects the position of the units. This operation lasts one hour. The total production of one row is 5.25 hour, so 2 rows per day can be placed. The total effective labour time is 10.5 hour. This is realistic, when a labour day of 12 hours minus the breaks is considered (productivity of 88%). On a section of 100m 30 rows of Accropodes have to be placed, this means 15 days for the placement of Accropodes. After a section of 100m is placed, the construction road will be removed and the remaining units will be placed. 8 rows of remaining units can be placed per day (10.4h) and for a section of 100m 3.75 days are required, (see Table 5-5). Table 5-6 gives an overview of all activities on a section of 100m.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Above waterline	4	5	0.8
Movement crane			0.5
Total			1.3

Table 5-5 Procedure of the placement of the remaining units of a row

Activities	Duration [days]
Placement of units	15
Placement of latest unit	3.75
Total	18.75

Table 5-6 Overview of activities of a section of 100m

Head section

390 units have to be placed on the head of the breakwater, but due to the construction road 120 units can not be placed. The section is divided into 30 rows of units, so for one row 9 units are placed by a crane on the breakwater and later on, when the temporal construction has been removed the other 4 units will be placed.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	3	3	1
Breakers zone	2	1.5	1.33
Above waterline	4	5	0.8
Dive inspection			1.0
Movement crane			0.5
Total			4.65

Table 5-7 Placement of one row

The production of one row is 4.65 hour and the daily production is 2 rows *1.1 is 10.2 hours. On a section of 100m 30 rows of Accropodes have to be placed, this means 15 days for the placement of Accropodes. After a section of 100m is placed, the construction road will be removed and the remaining units will be placed. 8 rows of remaining units can be placed per day (10.4h) and for a section of 100m 3.75 days are require, (see Table 5-8). Table 5-9 gives an overview of all activities on a section of 100m.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Above waterline	4	5	0.8
Movement crane			0.5
Total			1.3

Table 5-8 Procedure of the placement of the remaining units of a row

Activities	Duration [days]
Placement of units	15
Placement of rest of units	3.75
Total time for section 100 m	18.75

Table 5-9 Placement activities of a section of 100m

5.3.3 Xbloc

The placement of the Xblocs follows the same procedures as the placement of Accropodes. Untill now a breakwater with an Xbloc armour layer has never been constructed, so production figures are unknown. The placement rates which have been used in the calculation are based on the rates of the Accropode calculation and a difference factor of 1.5 (realistic value between 1.25- 2.0) is assumed, which implies the production rates:

Under waterline	4.5	units/hour
Breakers zone	2.25	units/hour
Above waterline	7.5	units/hour

Trunk section

3,850 units have to be placed on the whole trunk section and 550 units have to be placed on 100 m section, but due the construction road 130 units (20*4 + 10*5) can not be placed. On the 100m section 30 rows have to be placed.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	3	4.5	0.67
Breakers zone	2	2.25	0.89
Above waterline	7	7.5	0.93
Diver inspection			1.0
Movement crane			0.5
Total			4.0

Table 5-10 Placement of one row

5 rows can be placed in two days, so a section of 100m is placed in 12 days.
After a section of 100m is placed, the construction road will be removed and the remaining units will be placed. On a section of 10m the remaining blocks are divided in two rows of 4 units and one of 5 units. 10 rows of remaining units can be placed per day (10.7 h) and for a section of 100m 3 days are required, see Table 5-11. Table 5-12 gives an overview of all activities on a section of 100m.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Above waterline	4	7.5	0.53
	5	7.5	0.66
Movement crane			0.5
Total section 10m			3.22

Table 5-11 Procedure of the placement of the remaining units of a row

Activities	Duration [days]
Placement of units	12
Placement of latest unit	5
Total time	17

Table 5-12 Overview of activities of section of 100m

Head section

450 units have to be placed on the head of the breakwater, but due to the construction road 120 units can not be placed. The section is divided into 30 rows of units, so for one row 11 units are placed by a crane on the breakwater and later on, when the temporal construction has been removed the other 4 units will be placed.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	3	4.5	0.67
Breakers zone	3	2.25	1.33
Above waterline	5	7.5	0.67
Diver inspection			1.0
Movement crane			0.5
Total			4.17

Table 5-13 Placement of one row

The production of one row is 4.17 hour and the daily production is 2.2 rows *1.1 is 10.1 hours. On a section of 100m 30 rows of Xblocs have to be placed, this means 14 days for the placement of Xblocs. 10 rows of remaining units can be placed per day (10.5h) and for a section of 100m 3.0 days are required, see Table 5-14. Table 5-15 gives an overview of all activities on a section of 100m.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Above waterline	4	7.5	0.53
Movement crane			0.5
Total			1.05

Table 5-14 Procedure of the placement of the remaining units of a row

Activities	Duration [days]
Placement of units	14
Placement of rest of units	3
Total	17

Table 5-15 Overview of activities of a section of 100m

5.3.4 Concrete cube

The dumping of the cubes differs from the placing methods of interlocking units. The cubes are dumped by crane and not placed. This procedure will increase the production rates of the cubes. The production rates which will be used in the calculation are based on production rates of the breakwaters of Hook of Holland and Scheveningen. A distinction is made between the cubes dumped under the waterline and the cubes dumped above the waterline.

Under waterline	10	units/hour
Above waterline	15	units/hour

Trunk section

20,558 elements have to be placed on the trunk section and a section of 10 meter is divided in 7 rows with a double layer of cubes, 294 cubes in total. A temporal construction road is not required, when a special gantry crane dumps the cubes. The lack of this temporal construction road decreases the total number of construction days. A total row can be dumped, without restrictions.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	2*11	10	2.2
Above waterline	2*10	15	1.33
Movement crane			0.5
Total			4.0

Table 5-16 Placement of one row

2.5 rows of double-layer cubes are placed on a labour day. Diver assistance is not necessary, because the cubes find their own ideal position on the breakwater. On a section of 100m 70 rows with a double layer of cubes have to be dumped, this means 28 days for the dumping of cubes.

Head section

2328 units have to be placed on the head section and a section of 10 meter is divided in 6 rows with double layer of cubes, 234 cubes in total. The construction road prevent the direct placement of 70 units, those will be placed later. At the start the first layer of 16 units is placed and subsequently the second layer is placed on this layer.

Position	Number of units [-]	Placement rate [unit/hour]	Duration [hours]
Under waterline	2*11	10	2.2
Above waterline	17	15	1.13
Movement crane			0.5
Total			3.8

Table 5-17 Placement of one row

To make a distinction between placement on the trunk and on the head the production time of one row is multiplied by factor 1.1. 2.5 rows of double-layer cubes can be placed on labour day. Diver assistance is not necessary, because the cubes find their own ideal position on the breakwater. On a section of 100m 60 rows of cubes have to be dumped, this means 24 days for the dumping of cubes.

5.3.5 Analysis of placement of armour units

The main procedure of the placing operation is for every armour unit the same. First the unit is attached to the crane; subsequently the unit is lifted and moved to its position. When the armour unit is positioned, it can be released from the crane. The main difference of the placing methods of the three armour units is the difference between placing and dumping. Placing of armour units requires accuracy, which decrease the production rates. The placement rates of dumping of cubes are very high, but a disadvantage of this alternative is the use of a special crane.

Section		March				April				May				June				July			
	Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	Removal of armour layer	▒	▒	▒	▒																
	Construction of filter layer and toe				▒	▒	▒	▒													
	Construction of construction road				▒																
	Placement of armour units					■	■	■	■												
2	Removal of armour layer					▒	▒	▒	▒												
	Construction of filter layer and toe								▒	▒	▒	▒									
	Construction of construction road								▒												
	Placement of armour units									■	■	■	■								
3	Removal of armour layer									▒	▒	▒	▒								
	Construction of filter layer and toe												▒	▒	▒	▒					
	Construction of construction road												▒								
	Placement of armour units													■	■	■	■				
4	Removal of armour layer													▒	▒	▒	▒				
	Construction of filter layer and toe																▒	▒	▒	▒	
	Construction of construction road																▒				
	Placement of armour units																	■	■	■	■

Table 5-18 Part of the construction scheme of the reconstruction of breakwaters IJmuiden

0 gives the number of working weeks including unworkable days. In April starts the placement operation at the head of the breakwater and after 26 working weeks all equipment is demobilised. Before the placement of armour units starts, a part of the current armour layer is removed and subsequently a new filter layer and toe is placed. Simultaneously with the placement of filter layers the temporal construction road is constructed. The last step for this section is the placement of the armour units. All these steps are described in a construction scheme and a part of the scheme is given Table 5-18. The number of placement weeks in this scheme is an average of the three armour units. The order of construction is for every section the same and this is also valid for the second year of construction. In Table 5-19 the number of placement days of the armour units is described. At page 65 it is assumed that a workweek of 5 days has one unworkable day. With this assumption a translation can be made between the number of placing days to the number of placing weeks. A large difference exists between the interlocking units and the cubes. The large number of cubes (double layer) explains this difference; however the placing rate of cubes is higher than the interlocking units.

Unit	Trunk (100m)	Head (100m)	Total section	
	[days]	[days]	[days]	[weeks]
Accropode	18.75	18.75	150	40
Xbloc	15	17	122	32.5
Cube3800	28	24	220	49

Table 5-19 Number of construction days for all three armour units

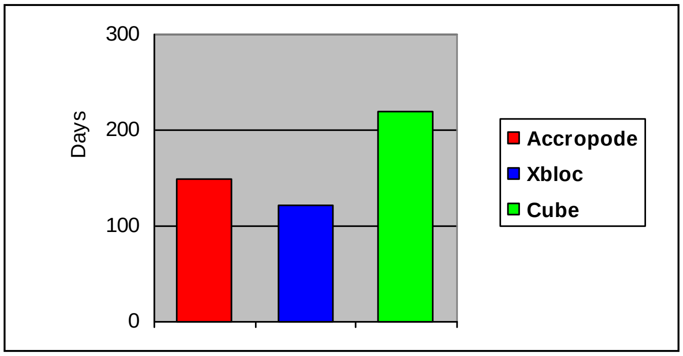


Figure 5-4 Distribution of the placement days

Equipment

Now the duration of the placement of armour units is known, the cost of this operation can be calculated. The time-dependent parameters, such as equipment and labour, determine the largest parts of the differences between the three alternatives. The crawler crane is used for the placing of the armour units. This crane is rented for the period of April – September (26 weeks) and is mobilised in March and demobilised in October. The mobilisation and demobilisation operation last both a week. The costs of mobilisation are equal to one week rent of the crawler crane. The costs are based on a work week of 60 hours.

For diving inspections a dive team of four divers and working vessel is used.
The costs of the temporal construction road are determined by the total quantities of quarry material, which is required to build this temporal construction road. Among others quarry material out of the demolished toe construction can be used for this construction road. This reduces the costs/tonne of the quarry material; a price of 20 euro/ tonne is estimated. This price is lower than the costs for the quarry material used for the filter layers (section 6.4.4), but relatively expensive, because the quarry material can only be transported in small quantities by dump trucks instead of side stone dumpers. Total costs of the temporal construction road is 67,200 tonne * 20 euro = € 134,400.

Transport is needed the whole construction process, from the preparation of the construction area till demobilisation of all equipment. Different types of vessels are used for the supply of aggregates and quarry material. Concrete mixers are used during the production of the armour units and flatbed lorries are used for the transport of armour units from the construction area to the breakwater (Table 5-20). The number of flatbed lorries depends on the daily required units for the placement of units on the breakwater. In this comparison only the costs of flatbed lorries are taken into account

Activities	Duration [min]
Loading of truck	5
Driving from construction yard to breakwater	20
Unloading	5
Driving from breakwater to construction yard	20

Table 5-20 Cyclus of truck

The number of flatbed lorries differs for the three alternatives, because the daily required units on the breakwater and the volume of the units differs. The price of a flat bed lorry is 1,000 euro/ week and it has a maximum capacity of 50 tonne. In the two tables below an overview of the number of trucks and the total costs of the trucks is given.

Type	Volume [m^3]	Weight [tonnes]	Units/ day	Max. units/hour	Units/truck	Number of trucks [-]
Accropode	18	42	32	5	1	5
	12	28			1	5
Xbloc	12	28	40	7.5	1	6
	9	21			2	4
Cube	3.375	13	100	15	3	5
	2.2	8.5			5	3

Table 5-21 Number of trucks

Type	Volume [m^3]	Production [weeks]	Number of trucks	Cost of trucks [euro]	Total costs [euro]
Accropode	18	5	5	25,000	200,000
	12	35	5	175,000	
Xbloc	12	5	6	30,000	142,000
	9	28	4	112,000	
Cube	3.375	6	5	30,0000	159,000
	2.2	43	3	129,000	

Table 5-22 Cost of trucks

However the cube alternative has more units to transport, the number of trucks is less than the interlocking alternatives. This can be explained by the light weight of the units, by which more units can be transported by one truck. Not all trucks will be used during the placement, because the number of trucks is calculated for the maximum placement rate, so the efficiency of the trucks will not be 100%. Now the construction programme and the cost factors are known, an overview of the costs during the placement is given in the table below.

Type	Equipment	Numbe r [-]	Production [weeks]/[years]	Cost/ week [euro]	Mob/demo b [euro]	Total cost [euro]
Accropode	Construction road					134,400
	Crawler crane	1	40 / 2	6,850	27,400	301,400
	Diving		40 / 2	20,000		800,000
	Total					**1,235,800**
Xbloc	Construction road					134,400
	Crawler crane	1	33 / 2	6,850	27,400	253,450
	Diving		33 / 2	20,000		660,000
	Total					**1,047,850**
Cube	Base structure					250,000
	Crawler crane	1	49 / 2	35,000	140,000	1,855,000
	Total					**2,105,000**

Table 5-23 Overview of the placement costs

With the data above it can be concluded that the costs of a dive team are relatively high in comparison with the costs of a crawler crane, it is possible to replace the dive team by camera and spotlight which is mounted on the crane. At the moment the application of a crane with camera equipment is an experimental phase. The costs of this adaptation are approximately 250,000 euro and apparently it will be more due repairs. All cranes are also equipped with positioning equipment (GPS RTK); these costs are not taken into account.

Type	Equipment	Number [-]	Production [weeks]/[years]	Cost/ week [euro]	Mob/demob [euro]	Total cost [euro]
Accropode	Construction road					134,400
	Crawler crane	1	40 / 2	6,850	27,400	301,400
	Camera	1				250,000
	Trucks					200,000
	Total					**885,800**
Xbloc	Construction road					134,400
	Crawler crane	1	33 / 2	6,850	27,400	253,450
	Camera					250,000
	Trucks					142,000
	Total					**779,850**
Cube	Base structure					250,000
	Crawler crane	1	49 / 2	35,000	140,000	1,855,000
	Trucks					159,000
	Total					**2,264,000**

Table 5-24 Overview of the placement costs without divers

Remarks

The calculations are based on many parameters, which show variation and a number of them are based on assumptions. Remarks can be made on these parameters and the way they are used in the calculations.

- Labour day of 12 hours, but representative value is 10.5 hours, which mean an efficiency of 88 %. 2 crews are required to work 12 hours a day, because the maximum duration of a labour day is 8 hours. In the previous calculations only one crew has been taken into account. Also weekend shifts have not been taken into account.
- Labour week of 5 days and it is stated that one day is unworkable. This is based on the mean value of wave data in the period 1992-2002. It is stated that a crane can not work, when the wave height exceeds 1.5m. No distinctions have been made between the crawler crane and the special crane.
- Comparison between the three alternatives is not fair, because a special crane is used for the cubes and the other two alternatives do not have special equipment. A half of an hour is assumed for the crane movement from row to row, this assumption increases the placement time of alternatives with many rows.
- A dive team is hired for the whole week, but the team works only a few hours per day. A crane equipped with cameras and spotlights is cheaper, but this technique is in an experimental phase. And camera equipment has also to deal with bad visibility in the water near the breakwater, so it does not function continuously. It also assumed that the application of inspection is not necessary for the dumping of cubes.

- To indicate the difference of placing rates between the trunk and the head, a factor of 1.1 is taken into account. The curve of the head and the weight of the head units cause this placing rate difference.
- It is assumed that the weight of the units does not influence the placing rates, because heavier units will be placed with cranes with more lifting capacity.
- The number of units is adjusted to easy the calculation; the deviations are relatively small (0.1 – 0.9 %).
- Depreciation, interest, maintenance, repair, insurances and fuel are not taken into account. The comparison is not complete because only some activities of the total process have been taken into account.
- Introduction problems with the special crane for the cubes are not taken into account.

5.4 Production Costs of Armour Units

5.4.1 Introduction

This section has listed all cost factors, which play a role during the production process of armour units. The cost factors, which are described in Section 5.2 are calculated for all three armour unit and subsequently compared in a cost analysis.

5.4.2 Accropode

1. Production of Accropode

The frameworks are divided in 4 pieces for transport purposes and a base plate is not needed; only a plastic foil or a wooden plate is enough. This holds in theory, but in practice a concrete base plate is needed to guarantee the quality of the heavy concrete units. If sufficient space is available, it may be possible to store the Accropodes in the position in which they


Figure 5-5 Casting of Accropodes

were cast, thus avoid double handling. This is not an option for the IJmuiden case, so the Accropodes are stored on a storage area. The standard procedure is also applied to the casting of Accropodes. A concrete pump is used for the pouring of the units because pouring from the mixer is difficult due the height of the units. The pouring of the 18 m^3 units requires a large capacity of the concrete pump. The daily placements of Accropodes and the shortage of storage area determine the daily production of armour units. In this case approximately 40 moulds are required. As the breakwater rehabilitation starts with placing 18 m^3 on the head, 18 m^3 units could be made in advance to divide the casting and minimize the number of moulds; 20 moulds will be sufficient.

Units [m^3]	Units [-]	Moulds [-]	Casting days	Daily volume of concrete [m^3]
12	3,332	40	83.3	480
18	390	20	19.5	360
Total	3,722	60	103	

Table 5-25 Number of casting days

The daily required amount of concrete is 480 m^3 (Table 5-25). This quantity can be delivered by a local concrete factory, which has a daily production of 800 m^3 and efficiency of 70 %, so in real terms 560 m^3 concrete can be delivered every day on site. The logistic problems are beyond the scope of the study, because these are equal for all three alternatives. A casting crew consists of 8 men and the workday consists of 8 hours. The standard hourly wage is 33 euros. The labour costs are listed in Table 5-26.

Crew [persons]	Workday [hours]	Casting [days]	Hourly wage [euro]	Total costs [euro]
8	8	103	33	217,536

Table 5-26 Labour costs

2. Material

The design of Accropode is protected by Sogreah. Sogreah signs a licensing contract with the user, covering both the right to use the trademark and technical assistance throughout the period of site work. The user has to pay for every used m^3 concrete, which is used for the armour units. The price is € 15, - / m^3 concrete. The price of concrete in the Amsterdam region for B35 with consistency range 3 is € 80, - and licence cost for this project are 705,000 euro (see Table 5-27 and Table 5-28)

Volume of unit [m]	Number of units	Total volume concrete [m3]	Cost of concrete [euro]
12	3,332	39,984	3,198,720
18	390	7,020	561,600
Total	3,722	47,004	3,760,320

Table 5-27 Material costs

Type of Cost	Cost [euro]
Material cost	3,760,320
Licence cost	705,060
Total cost	**4,465,380**

Table 5-28 Total concrete costs

The cost of quarry material depends on the production cost in the quarry, transport, storage and placement. The cost of transport, storage and placement are time-dependent, but this comparison uses euro/tonne material. These prices are based on index-numbers, see Table 5-29.

Type of equipment	Material cost [euro]	Placing cost [euro]	Total cost [euro]
Side dumping vessel	4,970,000	2,484,900	7,454,700
Crane	2,182,800	909,500	3,092,300
Total	7,152,800	3,394,400	10,547,000

Table 5-29 Quarry material cost

3. Moulds and equipment

Two different sizes of Accropodes are used on the breakwater, 12 m^3 and 18 m^3, so two different frameworks are required. The costs of frameworks are expressed in euro/kg steel, this inclusive production and material costs (Table 5-30). The present price of kilogram steel is € 2,-.

Unit [m3]	Number of moulds	Weight of mould (kg)	Surface of mould (m^2)	Steel plate thickness (mm)	Costs [€]	Total costs [€]
12	40	2,160	31.46	6	4,320	172,800
18	20	3,00	40.37	6	6,400	128,000
Total	60					300,800

Table 5-30 Moulds of Accropode

The moulds of the trunk section will be used approximately 84 times, so they are not written down yet and they can be used in the future. Also the moulds of the head section can be used for another project, there only used 20 times. Two small gantry cranes are required for the casting, stripping and pouring procedures. The maximum lifting capacity of these cranes is determined by the weight of the frame work. A big gantry crane is required for the transport of the Accropodes from the casting yard to the storage yard. The lifting capacity is determined by the weight of the biggest Accropode, 45 tonne (Table 5-31).

Equipment	Number [-]	Production [weeks]	Cost/ week [euro]	Mob/demob [euro]	Total cost [euro]
Gantry crane (10 tonne)	2	21	1,500	3,000	66,000
Gantry crane (30 tonne)	1	17	5,000	10,000	95,000
Gantry crane (50 tonne)	1	4	9,000	18,000	54,000
Total	4	21			215,000

Table 5-31 Cost of equipment

4. Storage

Three days after the pouring, the Accropodes are moved to the storage area. They can be stacked on one level or on two levels, if the ground conditions permit it. Twelve $18m^3$ Accropodes can be stacked on 100 m^2 and fifteen 12 m^3 on 100 m^2. The calculation in Table 5-32 is made for the storage of one layer of Accropodes. Double-layer storage of Accropodes is also an option, than 1.3 ha is sufficient. These dimensions are based on the storage of all the units, which is not realistic, but it gives a good insight for the comparison between the units (Table 5-32).

Figure 5-6 Storage area

Units [m^3]	Number [-]	Storage area [ha]
12	3,332	2.22
18	390	0.33
Total		2.55

Table 5-32 Storage area Accropode

5.4.3 XBloc

1. Production of Xbloc

The standard procedure is also applied to the casting of Xblocs, because with the height of the units (3.0 – 3.3m) a construction road on a higher level is not an option. A concrete pump is used for the pouring of the units and the frameworks are divided in 2 pieces. In practice a concrete base plate is needed to guarantee the quality of the heavy concrete units. In Figure 5-7 concrete is poured into a 3m^3 framework.

Figure 5-7 Casting of Xblocs

The daily placements of Xblocs and the shortage of storage area determine the daily production of armour units. In this case approximately 40 moulds are required. As the breakwater rehabilitation starts with placing 12 m^3 on the head, 12 m^3 units could be made in advance to divide the casting and minimize the number of moulds; 20 moulds will be sufficient

Units [m^3]	Units [-]	Moulds [-]	Casting days	Daily volume of concrete [m^3]
9	3,841	40	96.025	360
12	447	20	22.35	240
Total	4,288	60	119	

Table 5-33 Number of casting days

The daily required amount of concrete is 360 m^3 (Table 5-33). This quantity can be delivered by a local concrete factory, which has a daily production of 800 m^3 and efficiency of 70 %, so in real terms 560 m^3 concrete can be delivered every day on site. The logistic problems are beyond the scope of the study, because they are equal for all three alternatives. A casting crew consists of 8 men and the workday consists of 8 hours. The standard hourly wage is 33 euros. The labour costs are listed in Table 5-34.

Crew [persons]	Workday [hours]	Casting [days]	Hourly wage [euro]	Total costs [euro]
8	8	119	33	251,328

Table 5-34 Labour costs

2. Material

The design of XBloc is protected by Delta Marine Consultants. DMC signs a licensing contract with the user, covering both the right to use the trademark and technical assistance throughout the period of site work. The user has to pay for every used m^3 concrete, which is used for the armour units. The price is € 15, - / m^3 concrete. The price of concrete in the Amsterdam region for B35 with consistency range 3 is € 80,- (Table 5-35 and Table 5-36)

Volume of unit [m^3]	Number of units	Total volume concrete [m^3]	Cost of concrete [euro]
9	3,841	34,569	2,765,520
12	447	5,364	429,120
Total	4,288	39,933	3,194,640

Table 5-35 Concrete cost

Type of Cost	Cost [euro]
Material cost	3,194,640
Licence cost	598,995
Total cost	**3,793,635**

Table 5-36 Total concrete cost

The cost of quarry material depends on the production cost in the quarry, transport, storage and placement. The cost of transport, storage and placement are time-dependent, but this comparison uses cost/tonne material. These prices are based on experiences with projects in the past (Table 5-37).

Equipment	Material cost [euro]	Placing cost [euro]	Total cost [euro]
Side dumping vessel	4,287,650	2,143,825	6,431,475
Crane	1,729,380	720,575	2,449,955
Total	6,017,030	2,864,400	8,881,430

Table 5-37 Quarry material cost

3. Moulds and equipment

Unit [m^3]	Number of moulds	Weight of mould (kg)	Steel plate thickness (mm)	Costs [euro]	Total costs [euro]
9	40	2,230	5	4,460	178,400
12	20	2,600	5	5,200	104,000
Total	60				282,400

Table 5-38 Moulds of Xbloc

These figures are based on the weight of the framework of a 3 m^3 unit. The weight of the framework of 3 m^3 Xbloc unit is 20% higher than the framework of the 3 m^3 Accropode unit. This same percentage is assumed for the 9 and 12 m^3 units (Table 5-38). Two small gantry cranes are required for the casting, stripping and pouring procedures. The maximum lifting capacity of these cranes is determined by the weight of the frame work. A large gantry crane is required for the transport of the Xblocs from the casting yard to the storage yard. The lifting capacity is determined by the weight of the biggest Xbloc, 28 tonne, see Table 5-39.

Equipment	Number [-]	Production [weeks]	Cost/ week [euro]	Mob/demob [euro]	Total cost [euro]
Gantry crane (10 tonne)	2	24	1,500	6,000	78,000
Gantry crane (30 tonne)	1	5	5,000	10,000	35,000
Gantry crane (20 tonne)	1	19	3,000	6,000	63,000
Total	4	24			176,000

Table 5-39 Cost of equipment

4. Storage

Three days after the pouring, the Xblocs are moved to the storage area. They can be stacked on one level or on top of another on two levels, if the ground conditions permit it. 12.75 12m^3 Xblocs can be stacked on 100 m^2 and 15.4 9m^3 on 100 m^2 (Table 5-40). Double-layer storage of Xblocs is also an option, than 1.42 ha is sufficient. These dimensions are based on the storage of all the units, which is not realistic, but it gives a good insight for the comparison between the units.

Figure 5-8 Storage of Xblocs

Units [m^3]	Number [-]	Storage area [ha]
9	3,841	2.49
12	447	0.35
Total		2.84

Table 5-40 Storage area of single layer Xblocs

5.4.4 Concrete cube

1. Production of cubes

Gantry cranes are also applied to the casting of the cubes, but it also possible to pour the concrete directly from the concrete mixer into the framework, because the height of the units (1.3-1.5 m) makes this possible. The pouring of the 2.2 – 3.375 m^3 units does not require a concrete pump, so a bucket will used for the pouring of the cubes or directly pouring from the mixer. However the mixer has to be adapted to the high density concrete (3800 kg/m^3) and also parts of the concrete factory have to be reinforced. These adjustments increase the costs of high density concrete. The moulds of the cubes are built out of different pieces to one entity. The moulds are placed on a bottom plate and than are filled with concrete and after one day the framework can be lifted (see Figure 5-9).

Figure 5-9 Lifting of concrete units

The daily placements rates of cubes and the shortage of storage area determine the daily production of armour units. In this case approximately 90 moulds are required (Table 5-41).

Units [m^3]	Units [-]	Moulds [-]	Casting days	Daily volume of concrete [m^3]
2.2	20,558	90	228.4	200
3.375	2,328	90	25.9	305
Total	22,886	180	254.3	

Table 5-41 Number of moulds of cubes

The daily required amount of concrete is 305 m^3. This quantity can be delivered by a local concrete factory, which has a daily production of 800 m^3 and efficiency of 70 %, so in real terms 560 m^3 concrete can be delivered every day on site. A double casting crew consists of 16 men and the workday consists of 8 hours. The standard hourly wage is 33 euros. Labour costs are listed in Table 5-42.

Crew [persons]	Workday [hours]	Casting [days]	Hourly wage [euro]	Total costs [euro]
16	8	255	33	1,077,120

Table 5-42 Labour costs

2. Material

The price of high density concrete in the Amsterdam region is approximately 250 euro/m^3.
The cube shape is not a protected trademark, so no fees have to be paid for the use of the concrete armour units (Table 5-43). Table 5-44 gives the total costs of the quarry material.

Volume of unit [m]	Number of units[-]	Total volume concrete [m3]	Cost of concrete [euro]
2.2	20,558	45,228	11,307,000
3.375	2,328	7,857	1,964,250
Total	22,886	53,085	13,271,250

Table 5-43 Concrete cost

Figure 5-10 Filling of bucket

	Material cost	Placing cost	Total cost
Side dumping vessel	4,461,325	2,230,663	6,691,988
Crane	1,791,750	746,563	2,538,313
Total	6,253,075	2,977,226	9,230,301

Table 5-44 Quarry material cost

3. Moulds and equipment

Unit [m3]	Number of moulds	Weight of mould (kg)	Surface of mould (m^2)	Steel plate thickness (mm)	Costs [€]	Total costs [€]
2.2	90	634	8.45	6	1,268	114,120
3.375	90	844	11.25	6	1,688	151,920
Total	180					266,040

Table 5-45 Moulds of cube

Two small gantry cranes are required for the casting, stripping and pouring procedures. The maximum lifting capacity of these cranes is determined by the weight of the frame work. A big gantry crane is required for the transport of the cube from the casting yard to the storage yard. The lifting capacity is determined by the weight of the biggest cube, 13 tonne. Table 5-45 and Table 5-46 give the total costs of the moulds and the gantry cranes.

Equipment	Number [-]	Production [weeks]	Cost/ week [euro]	Mob/demob [euro]	Total cost [euro]
Gantry crane (10 tonne)	3	51	1,500	12,000	241,500
Gantry crane (15 tonne)	1	51	2,300	4,600	121,900
Total	4				363,400

Table 5-46 Overview of gantry cranes

Figure 5-11 Left; storage area Right; Bottom plate for frameworks

4. Storage

Three days after the pouring, the cubes are moved to the storage area. They can be stacked 3 or 4 high, if the ground conditions permit it. 18.9 2.2m^3 cubes can be stacked on 100m^2 and 16 3.375m^3 on 100 m^2 (Table 5-47). On every side of the cube 0.5m free space is available for the block clamp. Double-layer or three-double storage of cubes is also an option, than 6.2 and 4.1 ha is sufficient. These dimensions are based on the storage of all the units, which is not realistic, but it gives a good insight for the comparison between the units.

Units [m^3]	Number [-]	Storage area [ha]
2,2	20,558	10.9
3,375	2,328	1.5
Total		12.4

Table 5-47 Storage area of cubes

5.4.5 Analysis of production of armour units

The direct costs are divided in time-dependent and time-independent factors. Material and moulds are time-independent and the costs of labour and equipment depends on the number of production days. In the table below the Accropode alternative is the standard (100%) and the other two alternatives are compared with this standard.

	Accropode	[%]	**Xbloc**	[%]	**Cube 3800**	[%]
Casting days	103	100	119	115	255	248
Labour [euro]	217,536	100	251,328	115	1,077,120	495
Concrete [m^3]	47,000	100	40,000	85	53,000	113
Concrete [euro]	4,465,000	100	3,794,000	85	13,271,000	297
Quarry material [m^3]	10,600,000	100	8,881,000	83	9,230,000	87
Moulds [-]	300,000	100	282,000	94	266,000	89
Equipment [-]	215,000	100	176,000	82	266,040	123
Storage [ha]	2.55	100	2,84	111	12.4	486

Table 5-48 Overview of costs of the production of armour units

This research does not require a detailed calculation and description of all activities during the production phase, but it gives a simplified model of reality. The most important cost parameters have been described and calculated in the previous sections.

- **Casting days**
  The differences in number of casting days are determined by the number of units and the number of moulds. It is assumed that the pouring of heavy concrete does not cause any problems. Weight and dimensions does not slow down the casting process, because heavier equipment can be deployed. The number of moulds, which is used for this calculation, is determined by the maximum daily placement of units on the breakwater. The total number of units is restricted by the maximum daily capacity of the concrete factory. The difference between the cube and the interlocking units is great, a factor three. This difference is mainly caused by the large number of cubes, which have to be produced.

- **Concrete**
  The Xbloc alternative stands out positively on total quantity of concrete and total costs. The cost of concrete is an important cost factor of the total costs and saving on concrete costs means substantial savings on total costs. Double layer of cubes and the price of heavy concrete cause the high total costs of the cube alternative. The Xbloc alternative has more units, but smaller units, which explains the lower total concrete quantity than the Accropode.

- **Quarry material**
  The costs of quarry material used in this calculation depend on index number (euro/tonne) of other projects. This method is not correct, because the dumping of quarry material is also a time-dependent parameter. The costs of quarry material is not a part of the total costs of armour units, so these costs are not take into account in the final comparison.

- **Moulds**
  The costs of the moulds of all alternatives do not differ much, however the number and dimensions differ. The costs of moulds are relatively small and when the project has been finished they can be used for another project. The state of the frameworks and the number of times which they have been used determine the (residual) value of the moulds. The (residual) value of the moulds of the three alternatives will not differ a lot after the casting operation. The number of

times, which the formwork of the cube has been used, is bigger than the Accropode and the Xbloc. This will not influence the state of the framework, because they are made of steel.

- **Equipment**
  Only the lifting equipment has been viewed and all other equipment has been excluded of the comparison. Type of equipment depends on dimensions and mostly on the weight of the units Differences of costs between the alternatives can be explained by the number of casting days, type of equipment and number of gantry cranes. The cube alternative stands out negatively in comparison with the other two. These high costs are mainly caused by the number of casting days

- **Storage**
  The number of calculated hectares of storage area outlines only the maximum dimensions of the storage area for armour units. In practice not all armour units stand on a storage area, but the production and placement operations go along. A minimum storage of armour units is required in case of delays and delivery of raw materials.

5.5 Total Costs of Armour Units

5.5.1 Introduction

In the previous sections the construction of the breakwater and the maintenance strategy for the breakwater has been described. This section compares the three alternatives. Alternatives can be compared with several methods, very detailed on costs or globally on criteria. In a detailed cost comparison the cost of all construction phases are listed and at the end the total costs of each alternative will be compared with the other alternatives. On the contrary a multi-criteria analysis compares the alternatives with different criteria and weights. A multi-criteria analysis is beyond the scope of this research, because the three alternatives do not differ on several criteria.

5.5.2 Total cost comparison

The total costs are divided in the costs of each phase; production and placement. Transport is not an individual phase, because transport is used during every construction phase. Also maintenance is not considered in the cost comparison, because it is difficult to predict the costs of maintenance.
In every phase different cost factors determine the largest part of the total costs; material costs determine the largest part of the production costs and equipment costs determine the large part of the placement costs. The duration of the placement of the units is an important parameter, because interest, depreciation, insurances and the number of repairs are time-dependent. Also fuel consumption and maintenance of equipment depend on the number of working hours.
The figures below show the costs of material and equipment, which determine a large part of the total costs.

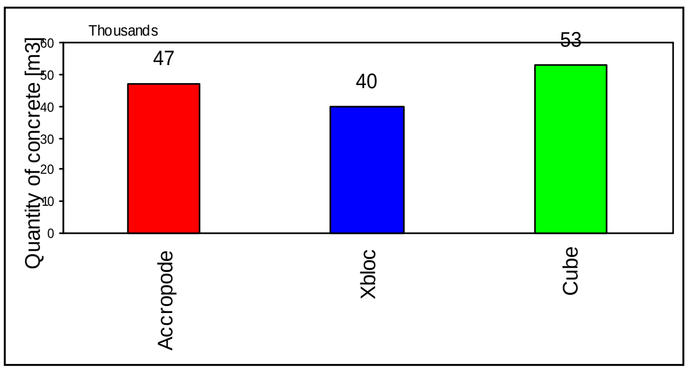


Figure 5-12 Quantities of concrete

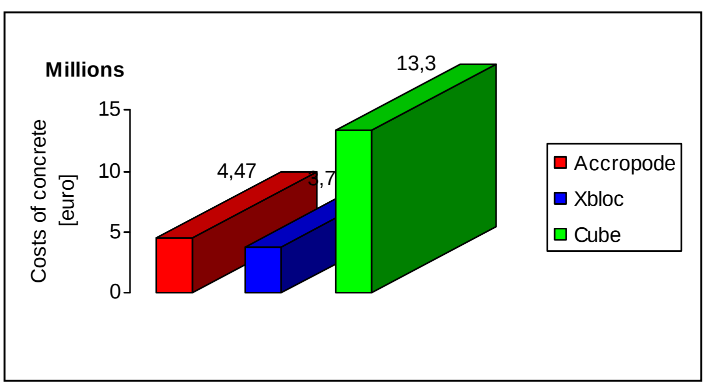


Figure 5-13 Costs of concrete

The double-layer cube alternative has the highest concrete cost. This is not only caused by the high price of high density concrete, but also by the large quantities of concrete. The design has also the most units. However the Xbloc alternative has more units, but smaller units than the Accropode, the total concrete quantity is lower and the total concrete costs are lower.

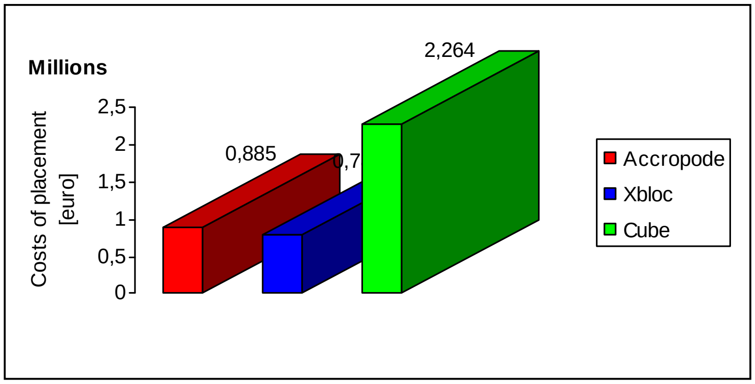


Figure 5-14 Cost of equipment

The differences between the interlocking units and the cubes can be explained by difference in construction time and the costs of the special crane, which is used during the dumping cubes.
Table 5-48 gives an overview of the most important parameters, which contribute to the total costs of a breakwater.

Phase	Cost factors		Accropode	Xbloc	Cube3800	Accropode	Xbloc	Cube3800
Production	Concrete		€ 4,465,380	€ 3,793,635	€ 13,271,250	100%	85%	290%
	Moulds		€ 300,800	€ 282,400	€ 266,040	100%	94%	88%
	Production time		103 days	119 days	255 days	100%	116%	248%
		Labour	€ 217,536	€ 251,328	€1,077,120	100%	115%	495%
		Equipment	€ 215,000	€ 176,000	€266,040	100%	69%	123%
	Max. storage with one layer high		2,55 ha	2,84 ha	12,3 ha	100%	111%	482%
	Total		**€ 5,198,716**	**€ 4,503,363**	**€ 14,880,450**	**100%**	**85%**	**286%**
Placement	Placement time		40 weeks	33 weeks	49 weeks	100%	83%	123%
		Trucks	€ 200,000	€ 142,000	€ 159,000	100%	71%	80%
		Equipment	€ 685,800	€ 637,850	€ 2,105,000	100%	93%	307%
	Total		**€ 885,800**	**€ 779,850**	**€ 2,264,000**	**100%**	**88%**	**240%**
Total			**€ 6,084,516**	**€ 5,283,213**	**€ 17,144,450**	**100%**	**87%**	**282%**

Table 5-49 Overview of total costs

The costs of quarry material are left out of the comparison, because these costs are not part of the armour layer, but part of the total breakwater. However the dimensions of the filter layers (so also the costs) depend on the dimensions of the armour layer. The differences between the alternatives are small.

Quarry material		Accropode	Xbloc	Cube3800	Accropode	Xbloc	Cube3800
	Production	€ 7,152,800	€ 6,017,030	€ 6,253,075	100%	84%	88%
	Placement	€ 3,394,400	€ 2,864,400	€ 2,977,226	100%	84%	88%
	Total	€ 10,547,200	€ 8,881,430	€ 9,230,301	100 %	84%	88%

Table 5-50 Cost of quarry material

It is obvious that the cube alternative has the highest costs in both the production and placement phase. This can be explained by the high costs of the high density concrete and also the total quantity of concrete is very large (double layer of armour units). However the cubes are small, the total placement time is high caused by the large number of units. The differences between the Accropode and Xbloc alternative in both phases are relatively small, but the Xbloc alternative stands out positively. The most cost factors are approximately equal for the Accropode and the Xbloc, but the alternatives differ on concrete quantities and the costs of placement. Other cost factors such as moulds do not determine the choice for an alternative. Also storage area is not an issue, because the difference between the Accropode and the Xbloc is small. The storage of cubes will not be a problem, because they can be stacked can higher than other units.

6 Sensitivity Analysis

6.1 Introduction

In the previous chapter the production and placement costs of the three armour units have been calculated. A sensitivity analysis is used to investigate the effect of different conditions and assumptions on the outcome of this costs comparison. The most straightforward example is testing a design under more severe conditions than expected. The key issues in carrying out sensitivity testing are:

- What range of values to test? This depends on the purpose of the sensitivity analysis and on the degree of variability. It is best to have in mind a hypothetical confidence interval so that sensitivity testing is carried out for a reasonable range of values, without finishing with unrealistic extremes.
- Which values and assumptions to tests?
- Which combination of values and assumptions to test?

With these combinations of values and assumptions several scenarios can be constructed. Section 6.2 deals with the scenarios and accompanying parameters. Section 6.3 views the uncertainties of the design and holds an optimisation of the designs. Section 6.4 deals with risks which could occur during the construction of a breakwater. The next chapter holds the conclusions and recommendations.

6.2 Scenario Variables

6.2.1 Introduction

The calculations which have been made in Chapter 5 are based on different parameters, which all have a certain uncertainty. All these parameters form more or less a certain scenario. Different scenarios are possible: optimistic, realistic and pessimistic. The calculations in Chapter 5 have been based on a realistic scenario (workability is based on realistic values). The execution can be delayed by several factors and delay can cause extra costs. When the construction time exceeds contract construction time, penalties have to be paid. To diminish the delay time, more equipment and labour have to be involved. For example when the construction of filter layers shows delay, the placement of armour can not start and total construction time will increase. The duration of the project influences the direct costs of project, because a number of costs factors are time-related costs, e.g. labour, fuel and equipment. Most important parameters of the construction costs:

- Placement rates;
- Workability (environmental conditions);
- Equipment;
- Number of moulds;
- Concrete;
- Storage.

The costs of a maintenance strategy influence also the total costs of a breakwater. This influence is discussed in this section.

6.2.2 Parameters

This chapter deals with the cost parameters, which determine the total construction and maintenance costs of a breakwater. First an overview of the costs of each armour unit is given in the tables below and in Figure 6-1. A distinction is made between the trunk and head section and the costs of the whole section and the costs per m^1 breakwater.

Accropode	**Trunk – 12m^3**		**Head – 18 m^3**	
Length breakwater	700m	1m	100m	1m
Concrete [m^3]	39,984	57.12	7,020	70.2
Units [-]	3,332	4.8	390	3.9
Concrete costs [euro]	3,798,480	5,426	666,900	6,669
Production costs [euro]	499,050	713	234,286	2,343
Placement costs [euro]	777,600	1,111	108,200	1,082
Costs/ unit [euro/-]	**1,510**		**2,588**	
Total costs [euro]	5,075,130	7,250	1,009,386	10,094

Table 6-1 Accropode costs for head and trunk section

Xbloc	**Trunk – 9 m^3**		**Head – 12 m^3**	
Length breakwater	700m	1m	100m	1m
Concrete [m^3]	34,569	49.4	5,364	53.6
Units [-]	3,841	5.5	447	4.5
Concrete costs [euro]	3,284,055	4,692	509,580	5,096
Production costs [euro]	505,902	723	203,826	2,038
Placement costs [euro]	655,100	936	124,750	1,248
Costs/ unit [euro/-]	**1,154**		**1,863**	
Total costs [euro]	4,445,057	6,350	838,166	8,382

Table 6-2 Xbloc costs for head and trunk section

Cube	**Trunk – 2.2 m^3**		**Head – 3.375 m^3**	
Length breakwater	700m	1m	100m	1m
Concrete [m^3]	45,228	64.6	7,857	78.6
Units [-]	20,558	29.4	2,328	23.3
Concrete costs [euro]	11,307,000	16,153	1,964,250	19,643
Production costs [euro]	1,409,188	2,013	200,012	2,000
Placement costs [euro]	1,879,636	2,685	285,000	2,850
Costs/ unit [euro/-]	**709**		**1,051**	
Total costs [euro]	14,595,824	20,851	2,449,262	24,492

Table 6-3 Cube costs for head and trunk section

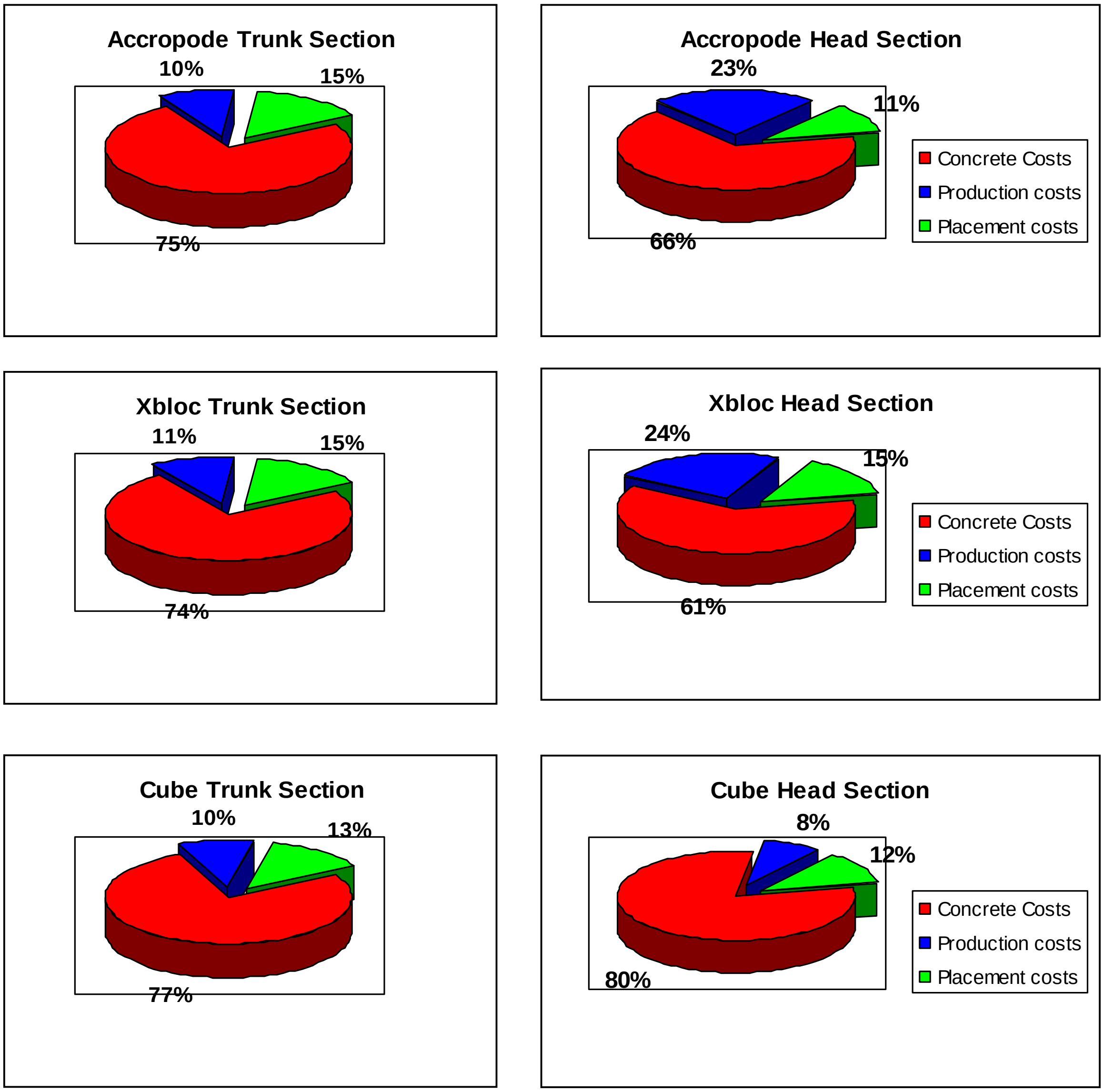


Figure 6-1 Division of costs per m^1 breakwater

	Accropode	Xbloc	Cube
Total cost trunk [euro]	7,250	6,350	20,851
Total cost head [euro]	10,094	8,382	24,492

Table 6-4 Overview of the total cost per m^1 breakwater

The differences between the three armour units and the differences between the trunk and head section have a large range. The production costs are higher in percentage terms due to the higher costs of the lifting equipment for the head section of the interlocking units. The cost parameters, which cause these differences, are described in this section. A description of the maintenance parameters is also given in this section. The costs given in Table 6-4 are only the costs of the top layer, the armour layer. The total costs per m^1 breakwater for the reconstruction of IJmuiden are higher than these costs, because other costs, as removal and filter layer costs, increase the total costs per m^1 breakwater.

Construction parameters

- **Placement rates**

Placement rates of armour units are based on experiences of recent projects, but it is difficult to transfer these rates into the IJmuiden situation. Several parameters determine these placement rates, for instance the environmental conditions, visibility in the water, dimensions of the units, dimensions of the breakwater and the execution method. Also the position of a unit on the slope is important. The placement rate is one of the most important parameters for the total construction time and also for the total construction costs. The placement rates of all three alternatives are based on values, which show a certain variation. In the previous chapter, the calculation of the placement time of row has used placement rates of recent projects, transposition of the crane and dive inspection. It could be that this calculation has calculated things twice, because the placement rate includes already transposition of the crane and dive inspections. It does not have influence on the comparison, because this has been done for all three armour units. This costs comparison has only taken a part of the southern breakwater, but in practice the whole southern and northern breakwater will be reconstructed. When both breakwaters are reconstructed, investment costs of cranes and special equipment can be written off on the project, so these investment costs are relatively smaller.

Accropode

The Accropode placement rates, which have been used for the calculation, are based on real-life construction projects. The placement rates show a large variation, varying from minus 2 units to 50 units a day. A placement of minus 2 can be explained by the fact that units were placed in the wrong way so they had to be replaced.

Xbloc

Placements rates of construction projects with Xblocs are not available, because up till now no project has been executed. It is difficult to predict these placement rates, but the placement rates of Xblocs can be based on Accropode experiences. An experiment has been executed to research the differences between the two units. The outcome of this experiment was a higher placement rate (80 - 100%) of the Xbloc in comparison with the Accropode. Some doubts can be cast on this outcome, because the laboratory situation is not comparable with the real-life situation on the breakwater. The placement rates have two parameters, which have a certain uncertainty, real placement rates of Accropodes and the outcome of one experiment. When in practice the placement rates of Xblocs are the same as Accropode, the placement time of Xbloc will be longer. For the IJmuiden case the placement time of both units will be the same, when the Xbloc placement rates are 1.1 times higher than the Accropode rates.

Cubes

The placement of a double-layer of cubes differs from the placement of the interlocking units, because the cubes can be dumped. Dumping of units increases the placement, because accurate placing is not required. Rates, which have been used for the calculation, are the based on the rates of the Hook of Holland breakwater. Two specially built ships with gantry cranes were responsible for the dumping of the cubes. In IJmuiden it is possible to make use of land-based crane. This crane is movable and it has long boom and a high lifting capacity. The used price/ week is an estimated value, because the crane has to be specially built for this project

- **Workability**

Environmental conditions play an important role during the construction of breakwaters. In the Netherlands construction of coastal structures takes place from April till September, because in winter the conditions are too difficult to work. In the calculations an H_s of 1.5m has been chosen as limit till crawler cranes can work. The restrictions for divers are more severe: Hs<0.5 m, current <1.5 m/s, visibility. The restrictions for divers are not relevant here, because the inspections of divers are replaced by camera-inspections. But also for these inspections the visibility in the water has to be good, otherwise position and orientation of the units can not be checked.

In the calculations it is assumed that one day of the work week is unworkable due environmental conditions. This assumption is based on measurements of offshore wave data near IJmuiden and the mean value of unworkable conditions (H_s>1.5m) is 20 %, but what is the variation of these data over the six months per year and what is the variation over the period of 10 years (1992 – 2002) per month (see Appendix V)? Is it possible to see some trend in these data? No, the variation per month and per year is substantial and no trend can be seen. It is clear that good and bad years exist, so it is advisable to take an optimistic and pessimistic scenario into account. In October the reconstruction of the breakwaters stops due to the environmental conditions. Equipment has to be demobilised and the breakwater has to be left in a perfect state to prevent damage in the winter. The last placed layers of armour units have to be protected by heavier units to prevent rolling and rocking of the armour units. This cost factor has to play a role in the calculation of the construction time.

- **Equipment**

In practice the productivity of equipment is never 100 % due to repair activities, refuelling, maintenance and breaks of personnel. For example the crawler crane, which places the armour units, can not work continuously the whole day long. In the previous chapter a net labour day of 10.5 hours has been taken into account. This is a realistic value, because the efficiency of the crane has been taken into account in the placement rates of the armour units. Refuelling can be done daily after the placement of the rows and maintenance can be executed on the unworkable days and in the weekend. The productivity of equipment will decrease, when the environmental conditions are bad and more repairs are necessary.

The crawler crane, which is positioned on the breakwater, has to be relocated onshore when the wave height exceeds a height of 3.0m. This height is approximately equal to the 10 times a year wave height. In a pessimistic perspective the crane has to go onshore once a month during the execution period of 6 months, so 6 times. For the placement of cubes a special crane is used, but the costs of this purpose made equipment is unknown. It is assumed that the costs are approximately 5 times higher than a normal crawler crane. This assumption is based on equipment costs of a project in South Africa, where comparable equipment has been used.

- **Number of moulds**

The number of moulds determines the number of casting days and the dimensions of the storage area. Three alternatives are possible:

	Number of moulds	Production time	Storage area
1	< max. daily placement	Long	Large
2	= max. daily placement	Middle	Small
3	> max. daily placement	Short	Large

Table 6-5 Overview of solutions with different kind of moulds

In practice alternative 2 is frequently chosen with an extra storage of a few weeks production to cope with delays of raw materials and other delays

- **Concrete**

Two kinds of concrete have been used for this cost comparison study, the two interlocking units used normal concrete and the cube used heavy concrete. An advantage of heavy concrete is the reduction of the dimensions of the units. Only the reduction of the third dimension (layer thickness) is interesting for this reconstruction of the breakwater, because the slope of the breakwater has to be protected by armour units. Eventually the slope is covered by smaller units, but a larger number of units.

The price of the used density of 3,800 kg/m^3 is very high in comparison with normal concrete (three times as high). This difference makes the application of high density concrete for large quantities not attractive. High density concrete could be attractive for special constructions and small quantities. Another option of high density concrete is more realistic, namely concrete with a density of 3,100 kg/m^3. The costs of this concrete are much lower than concrete of 3,800 kg/m^3 and little bit higher than normal concrete. The composition of the mixture changes when a density of more than 3,200 kg/m^3 is required and these other aggregates increase the total costs of the concrete mixture.

Normal concrete for interlocking units is more expensive than for cubes, licence costs of 15 euro/m^3 have to be paid. This difference in price/m^3 concrete is substantial, especially when large quantities are required. The concrete costs of the cube alternative can be reduced by using concrete with lower density or the double layer has to be replaced by a single layer. The use of normal concrete is not an option, because the dimensions of those cubes are too large, which cause problems in the production and placement phase (see Chapter 4). Options with density 2,800 are treated in the next section.

- **Overview of all equipment**

The list of equipment in the production stage is not complete, only the number and kind of gantry cranes have been taken into account. In practice more equipment is on site and this equipment has influence on the total costs of the three alternatives. Equipment costs are time-related and the production time of the alternatives differs, so eventually the total costs do.

- **Storage**

Availability of storage areas is never a problem for breakwater construction in less populated countries. In the western world space for storage of materials is difficult to find in port areas and the rent of available area rather high. Normally the client, the government, has certain areas in the port area, originated from former construction operations in the port area. So storage area will not be the problem for IJmuiden.

Maintenance parameters

- **Maintenance**

Nowadays life-cycle management of structures is an important issue, but in practice it is not suitable for breakwaters. During a lifetime of 100 years many parameters, like inflation, interest, maintenance methods, level of engineering, will have a certain uncertainty. It is impossible to predict the situation over a century, because many things can change in such a period. Also the residual value of a breakwater after a period of 100 years is hard to predict. The interlocking units, Accropode and Xbloc, are both designed maintenance free, so on this subject no distinction can be made between the units

- **Monitoring**

Monitoring costs depend on the time interval and the type of monitoring. No choice has been made for the three units, so no parameters can be varied.

6.2.3 Scenarios

In the previous chapter all costs of the alternatives have been calculated, but the alternatives can be optimised. By varying some parameters, the costs of an alternative can be reduced. In this case the costs of the cube alternative are relatively high in comparison with the interlocking unit alternative. In this section the type of crane, type of concrete and combination of those two are changed. The best way to minimise the costs is to switch from a double layer to a single layer, which reduces the total costs by half. This is not totally true, because the K_d value and the porosity differ. But a single layer of cubes is a different alternative and is beyond the scope of this thesis research. Three options have been viewed to reduce the costs of the cube:

1. Normal crane instead of special crane
2. Concrete with density 2,800 kg/m^3 instead of 3,800 kg/m^3
3. Normal crane and concrete with density 2,800 kg/m^3

1. Normal crane

Instead of a special crane a normal crane is used in this option and the following characteristics change:

- Costs of the crane per week are € 6,850 instead of € 35,000;
- Construction road is required;

- Placing instead of dumping, so placing rate decreases from 15 and 10 to 10 above the waterline and 7 under the waterline.

The number of placement days increases, because the crane places the cubes (0)

Trunk (100m)	Head (100m)	Total section	
[days]	[days]	[days]	[weeks]
35	30	275	69

Table 6-6 Number of placement days with normal crane

Equipment	Total costs [euro]
Construction road	134,400
Trucks	223,000
Crane	513,750
Total	**871,150**

Table 6-7 Placement costs of normal crane

	Special crane	Normal Crane	Change [%]
Placement [weeks/years]	49 / 2	69 / 3	+40
Placement [euro]	2,264,000	871,150	-62
Total costs	17,144,450	15,751,600	-8

Table 6-8 Comparison between special crane and normal crane

Although the placement time increases, the placement costs decrease considerably (0 and 0). A disadvantage of the normal crane is the long placement time, three years, which means more mobilisation and demobilisation costs. A solution to reduce the placement time is to use an extra crane to place the cubes on the slope, but this option is not presented out here. The influence on the total costs is considerable, a decrease of 8 % in comparison with the alternative with the special crane. A normal crane is more attractive than a special crane, because the special crane is estimated to be five times more expensive.

2. Concrete with density 2,800 kg/m^3

In Chapter 4 two kinds of concrete have been chosen for the cube alternative, 2,350 and 3,800 kg/m^3. Two have extremes have been chosen, but in this sensitivity analysis it is interesting to consider an option with concrete with density 2,800 kg/m^3. With this density the dimensions of the cube increase and the number of units decreases.
D_n= 2,05m
W = 24 tonne
Cost of concrete 2,800 kg/m3 = 120 euro/m^3

	2,800 kg/m^3	3,800 kg/m^3
Number of units trunk [-]	8,250	20,558
Number of units head [-]	850	2,328
Total [-]	9,100	22,886

Table 6-9 Number of units of both options

	2,800 kg/m^3	3,800 kg/m^3	Change [%]
Concrete [m^3]	82,000	53,000	+55
Concrete cost [euro]	9,840,000	13,250,000	-26
Total costs	12,470,000	17,070,000	-27

Table 6-10 Comparison between the two options

The total costs are not only changed by the lower by the lower concrete costs ,but also by the lower placement costs, because with density 2,800 kg/m^3 less units have to be placed. The placement costs are lower due to the lower number of units, but they are not calculated in detail. The total costs are influenced substantially, a decrease of 27%, mainly by the decrease of the concrete costs. The price of m^3 concrete is the most important cost parameter.

3. Normal crane and concrete with density 2,800 kg/m^3

Concrete reduction is known; see option 2, but for this option also the type of crane has been changed so the placement time and the placement costs change also (0 and 0).

Trunk (100m)	Head (100m)	Total section	
[days]	[days]	[days]	[weeks]
16	13	125	31

Table 6-11 Placement time with normal crane and density 2,800 kg/m3

Equipment	Total costs [euro]
Construction road	134,400
Trucks	140,000
Crane	240,000
Total	**514,400**

Table 6-12 Placement costs

	2,800 kg/m^3	**3,800 kg/m^3**	**Change [%]**
Concrete [m^3]	82,000	53,000	+55
Concrete costs [euro]	9,840,000	13,250,000	-26
Placement costs	514,000	2,213,000	-77
Total costs	11,900,000	17,070,000	-30

Table 6-13 Comparison between the options

In Table 6-13 one can see that the total costs are reduced with 30% by the use of concrete with density 2,800 kg/m^3 and a normal crane. When all three options are viewed, it can be seen that the concrete costs are the normative costs and the concrete costs have the largest influence on the total costs. Option 3 is the most attractive of options with cubes in this study, because concrete with density 2,800 kg/m3 is used and a normal crane is used for the placement of the cubes.

Another way to reduce construction costs is to work within one working season, so extra costs such as mobilisation, demobilisation costs, etc can be prevented. If a project can be constructed depends on the environmental conditions, duration of a labour week and the number of equipment. In this option the number of equipment is not varied, because one well trained crew is responsible for the placement and more cranes will decrease placing rates (start up problems). A fourth option to reduce costs is:

4. Construction time in one year

Only the Accropode and Xbloc option are reviewed for this option. In Chapter 5 is assumed that one of the five labour days is unworkable (workability of 80%), which resulted in the following construction period:

	[days]	[weeks]
Accropode	150	40
Xbloc	122	33

Table 6-14 Overview of construction time of the interlocking units

When a very optimistic scenario is assumed, where no unworkable weather occurs during the labour week 131 work days are available. When also the weekends are included as work days 183 are available in this optimistic scenario. If a percentage of 20 % of not workable days is taken into account for this scenario with weekends 146 days are available. It can be concluded that the reconstruction with Accropode and also Xbloc units is completed within one season (April - September) with a workability of 80 %, when the construction continues during the weekends. Working during the weekends is more expensive than during the week, but these extra costs are less than the costs of the reconstruction in two years.

6.3 Optimisation of the Design of the 3 Alternatives

The largest part of the distribution of the total costs is determined in the design phase. In this phase material, quantities, execution method and equipment are chosen, so this phase has a lot of influence on the total costs. However the designs are based on assumptions and the hold the following uncertainties:

- Stability of toe construction;
- State of crest element is unknown;
- No modelling of wave field has been done and no laboratory tests have been done;
- Impermeability of the asphalt layer will not have influence on the hydraulic stability of the new layers;
- The geometry of the breakwater has been changed during the years, current state is not totally known;

Other options, such as smaller units and other designs, have to be tested to optimise a design. Reduction of construction costs can be achieved by reducing the dimensions of the armour units. The total quantity of concrete will decrease and also the thickness of the filter layers will be less. Only the number of units will increase, so the placement operation will last longer. A distinction can be made between the single layer interlocking units and the double layer of cubes.

Single layer interlocking units

In Section 2.4 the failure mechanisms of single layer interlocking units are described. The Accropode has a safety margin of approximately 20 %, difference between H_s of design value and H_s of start of damage. The safety margin of the Xbloc is 25 %. It is recommended by Sogreah and DMC to use the design values, because no damage is tolerated to the armour layer. A unit of a single layer of interlocking units derives its strength from the interaction with the other block. Weight of the units plays a minor role. Dangerous situations can occur when one or more units are displaced, because the whole layer can become instable and total failure of the layer is possible. The single layer does not have any reserve capacity, because it is a single layer. The application of high density concrete for interlocking units has not been used till now, but it is interesting to test it.

Double layer of cubes

Double layer of cubes derives its strength from the weight of the cubes and when units of the top layer are displaced, the second layer has a reserve capacity. Start of damage is permitted, because the armour layer has reserve capacity, but in the short term replacement of units is advisable. The present design with a double layer of cubes is designed with a once in 100 years (H_s=7.5m) wave height. When some damage is tolerated, a once in 10 years wave height (H_s=6.5m) can be used for the design. The Hudson formula will be used to calculate the dimensions of the new cube with K_d for the trunk section of 6.5 (breaking waves) and a K_d for the head section of 4.33 (breaking waves).

$$H_s / \Delta D_n = [K_d * \cot a]^{1/3}$$

H_s incident wave height near the toe [m]
Δ relative density $[(\rho_a - \rho_w)/\rho_w]$ [-]
D_n nominal diameter of unit = $[W/\rho_a]^{1/3}$ [m]
K_d stability factor [-]
a slope angle [degrees]

	Volume [m³]	Weight [tonne]	Number [-]	Concrete [m³]
Trunk	1.4	5.33	29190	40866
Head	2.2	8.4	3200	7040
Total			32390	47906

Table 6-15 Properties of design with smaller cubes

The number of cubes is calculated with the drawings of the old design, so a small deviation is expected. The damage development of the new design can be calculated with the Van der Meer formula:

$$H_s / \Delta D_n = \left(6.7 \frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} - 0.5$$

N = number of waves
N_{od} = number of displaced blocks
N_{or} = number of rocking blocks
N_{omov} = N_{od} + N_{or}
s_{om} = $2p / g * H_s / T_m^2$ (fictitious wave steepness)

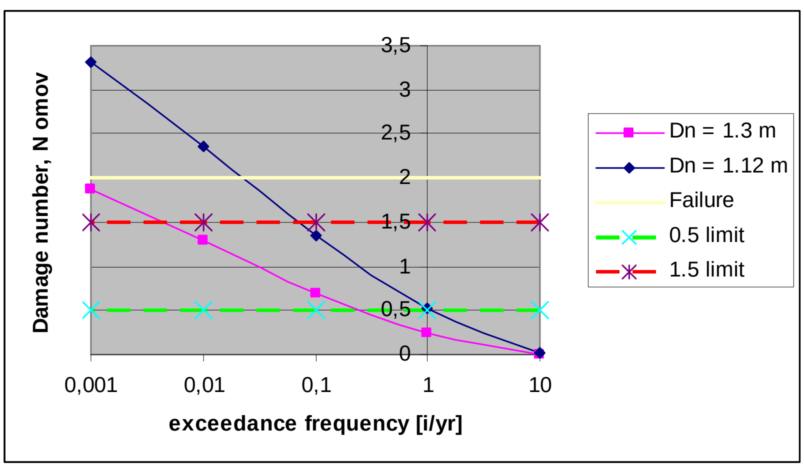


Figure 6-2 N_{od} value

Failure of the construction has to be prevented, so the damage number during design conditions has to be less than 1.5. The only parameter, which can be varied is N, the number of waves during a storm. The other parameters, such as wave height and wave period are given. The first attempt was a high value of N, 3000 waves and the result of that was a damage number of more than two during design conditions. So the design values of the Hudson formula and the check with the Van der Meer formula did not match. The second attempt with a lower value of N, 1000 waves, shows a better result (Figure 6-2) .During design conditions, once in 10 years wave height, the damage number is 1.34, between the once in 100 years wave height is viewed, the damage number is 2.35. This is too high, because failure of the breakwater has occurred. It can be concluded that a smaller cube size is not preferable.

However Figure 6-2 shows the damage development of a cube on the head section, this is not a good representation of reality, because Van der Meer did not take all effects into account. Van der Meer has executed only 2-D experiments, so his formula is not representative for the cubes on the head section. But also for the cubes on the trunk section the effects of longitudinal waves, which are created by the head of the breakwater, are not taken into account so the tests are not complete.

7 Conclusions and Recommendations

7.1 Conclusions

The objective of this thesis research was to find the most economical armour unit. To make a clear overview of the costs of three different concrete armour units the reconstruction of the breakwaters IJmuiden is used as case study. The results of this case study can be translated in to the costs of breakwaters in general, because situation of IJmuiden is not unique. The armour units are placed from the breakwater itself and the armour units are produced in the neighborhood of the breakwater. Only the environmental conditions differ for each breakwater construction project and these influence the construction time and construction costs. The construction method with accompanying equipment is adapted to circumstances of a breakwater construction project.

The production and the placement of concrete armour units are researched in this study and also the maintenance of the units is viewed. The cost comparison study focuses only on the armour layer of breakwater and the other parts of the breakwater are out of the scope of this study. The costs per m^1 breakwater are not the full costs, because other parts such as the toe and filter layer are not included. Conclusions are divided into different parts, first general conclusions and further on the placement and production conclusions are described.

General

- Concrete costs form the largest part of the total construction costs, between 73 to 77% of the total costs for the trunk section and between 61 to 81% of the total costs for the head section. It can be concluded that the concrete costs are the most important part of the total costs.

- Applications with high density concrete for cubes are interesting, but expensive. Dimensions of the armour units will decrease, but this is only interesting for the thickness of the armour layer. The entire slope of the breakwater has to be protected, so more small units are required. A balance has to be found between the density and the cost/m^3 placed concrete. The density of concrete determines both and the density has to be varied to find the most economical alternative. An armour layer with a double layer of cubes with normal concrete (density 2,350 kg/m^3) can not be executed due to the large dimensions of those cubes, which will cause problems during the production and placement of those cubes. The sensitivity analysis shows that concrete with density 2,800 kg/m^3 can reduce the total costs considerably in comparison with concrete with density 3,800 kg/m^3.

- Knowledge of maintenance and experience with maintenance strategies is limited. A maintenance strategy is not required for interlocking units, because those units are designed maintenance free. A double layer of cubes can be designed also maintenance free, but it usually allows a small percentage of damage (0-5%) and then a maintenance strategy is required. Monitoring the state of the breakwater is advisable regardless if a breakwater is designed maintenance free. Monitoring of the breakwater implies monitoring of the whole breakwater, not only the armour layer.

- The outcome of the cost comparison of breakwaters IJmuiden gives the following results, but these costs are only the costs of the armour layer per m^1 breakwater.

	Accropode	**Xbloc**	**Cube3800**
Total cost trunk [euro]	7,250	6,350	20,851
Total cost head [euro]	10,094	8,382	24,492

This overview gives an impression of the costs of the armour layer, but a choice between the armour units can not be based on this comparison only. More criteria (workability, reconstruction of other parts of the breakwater, etc.) play a role during the decision-making process

Placement

- The Xbloc has the lowest number of placing weeks and the cube the highest number of placing weeks. The placement weeks are expressed as percentages:

	Accropode [%]	**Xbloc [%]**	**Cube 3800 [%]**
Percentage placement weeks	100	87	123

The difference in placement weeks can be explained by:

1. High placement rates of Xbloc in comparison with Accropode
2. Xbloc has fewer units in comparison with the cube (double layer)

- Lowest number of placing weeks does not mean automatically lowest placement costs, because more parameters (kind of equipment, number of equipment, etc) influence the placement costs. In the IJmuiden case the Xbloc alternative has the lowest placement costs, because the placement time is short and no special equipment has been used. Only a temporal construction road has been constructed for the placement of armour units

- Inspections by dive teams have to be minimised or to be replaced by under water cameras. Safety precautions and environmental conditions determine the periods that divers can inspect the breakwater. The situation of IJmuiden restricts the diving periods, by which the total placement time increases. Inspections are necessary for interlocking units to check the position of the units. The armour layer has to fulfil the placement requirements otherwise Sogreah and DMC can not guarantee the safety of the armour layer.

- Dumping of cubes is the fastest way of placing armour units, but requires a special crane to absorb the dynamic loads, which are caused by dumping. However the high placement rate of a special crane, the costs of a special are too high in comparison with a normal crane (factor 5) and the use of a normal crane is recommended. Interlocking units are accurately placed, but the way of attaching (sling) to the units has to be improved. A new way of attaching can increase the placement rates.

Production

- The Xbloc has the lowest concrete costs and the cube the highest concrete costs. The concrete costs are expressed as percentages:

	Accropode [%]	**Xbloc [%]**	**Cube 3800 [%]**
Percentage concrete costs	100	85	290

The Xbloc has the lowest concrete costs, 15 % less than the Accropode alternative, however the Xbloc alternative has more units. The cube alternative has two layers, by which the total quantity of concrete is much larger than the interlocking alternatives. Not only the quantity of concrete, but also the kind of concrete determine the concrete cost. The two interlocking alternatives use concrete, "normal concrete" with density of 2,350 kg/m^3, but the cube alternative uses the more expensive "heavy concrete" with density 3,800 kg/m^3. It is concluded that double layer of cubes with high density concrete (3,800 kg/m^3) is expensive in comparison with the other two alternatives.

7.2 Relations with Other Studies

As mentioned earlier the Civil Engineering Division (CED) has viewed different alternatives for the reconstruction of the breakwaters of IJmuiden. The report of this study called "Invulling scenario herstel en noodmaatregelen havenhoofden IJmuiden" will be issued in December 2004. Five alternatives are presented in this report:

1. Armour layer of quarry stone
2. Single layer of Xblocs
3. Single layer of cubes
4. Double layer of cubes
5. Penetrated quarry stone

These alternatives are compared on different criteria such as environment, maintenance, construction and costs. The following conclusions are made after the research:

- The alternative with the single layer of cubes is preferable, because it has the best quality/price ratio after a multi-criteria analysis and a costs calculation. The Xbloc alternative is the second best alternative;
- The alternative with the double layer of cubes is preferable for the armour layer of the head of the breakwater;
- Model research is required for the final design of the reconstruction of the breakwater.

The outcome of both reports differs, as the cube is the less favourable alternative of this report and the most favourable of the CED report. An important difference between the reports is the lacking of a single layer of cubes alternative in this report. Also the CED report has reviewed more criteria and has taken into account all other parts of the breakwater and not only the armour layer. A remark can be made on the assumption of the CED that no filter is required for the single layer cube alternative. This influences the costs and thus the results. The porosity of 30 % of the single layer of cubes, which has been taken into account in CED report, is high, because it is the same value as for the double layer of cubes (30 % porosity has been taken into account for the alternative in this thesis report).

7.3 Recommendations

- Placement rates and placement difficulties have to be better documented during the construction of breakwater projects. This information is valuable for the design of new projects.
- More research has to be done into new innovative construction methods and equipment, which can work during extreme conditions. The research has to focus on new ways of attaching the armour units and the placing of units on the slope with special attention for the units on the head.
- Monitoring of the armour layer has to be carried more frequently, because information on the deterioration of whole armour layer and individual armour units is lacking. This is valid for cubes and interlocking units as well. A survey has to be made of the used maintenance strategies of breakwaters all over the world. This survey could give a good overview of the used methods and the costs of maintenance.
- Armour layers with Xbloc units have not been constructed up till now and placement rates are only based on one experiment. It is wise to do an evaluation, when a number of breakwaters with Xbloc units have been built. The results of this evaluation have to be compared with the results of the experiment.
- An alternative with an armour layer which consists of a single layer of cubes with density 2,800 kg/m^3 has to be analysed. This alternative could be competitive with the interlocking units. The sensitivity analysed that an alternative with density 2,800 kg/m^3 can be more economical than an alternative with 2,800 kg/m^3.

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Company brochures and oral information

Minelco
Sogreah
DMC
BAM Civiel
Interbeton

Appendix I. Meteorological data

VALKENBURG (ZH) normalen tijdvak 1971-2000

210	Temperatuur °C gemiddeld	Temperatuur °C gemiddeld minimum	Temperatuur °C gemiddeld maximum	Relatieve vochtigheid % gemiddeld	Relatieve vochtigheid % 12.00 UT	Neerslag duur in uren	Neerslag in % van de tijd	Neerslag som in mm	Verdamping som in mm	Globale Straling som in J/cm2	Zonneschijn in uren	Zonneschijn in % langst mogelijke duur	Luchtdruk in hPa	Wind snelheid in m/s	Wind-vector snelheid in m/s	Wind-vector richting in graden	Aantal dagen met windkracht >= 4 Bft	>= 5 Bft	>= 6 Bft	>= 7 Bft	>= 8 Bft
jan	3,4	0,7	5,6	86	84	61,4	8	64,9			54,0	21	1015,6	6,2	3,4	219	24	17	10	4	1
feb	3,3	0,4	6,0	85	80	42,1	6	43,4			82,7	30	1016,2	5,7	2,8	206	22	14	7	3	1
mrt	5,8	2,4	9,0	83	77	59,5	8	60,5			121,4	33	1014,8	5,8	2,6	237	25	16	8	3	0
apr	8,0	4,0	11,8	80	71	37,1	5	43,5			172,9	41	1014,1	5,1	1,9	309	24	13	4	1	0
mei	12,1	7,6	16,3	78	69	34,3	5	48,3			224,4	46	1015,3	4,7	1,6	325	24	12	3	0	0
jun	14,6	10,5	18,4	79	71	38,8	5	65,1			211,0	42	1015,9	4,7	2,3	295	23	11	2	0	.
jul	16,9	13,0	20,6	80	72	32,3	4	66,1			217,9	43	1016,3	4,7	2,3	293	24	12	3	0	.
aug	17,1	12,8	21,2	79	69	28,2	4	61,1			212,0	47	1016,1	4,3	2,0	283	22	10	3	0	.
sep	14,5	10,5	18,1	83	74	47,5	6	85,8			139,7	37	1015,6	4,5	2,0	252	21	11	4	1	0
okt	10,8	7,2	14,1	84	77	55,5	7	87,8			103,9	31	1014,8	5,0	2,2	220	22	13	6	2	0
nov	7,0	4,1	9,5	86	82	67,2	9	88,7			57,7	22	1014,3	5,7	2,8	232	23	15	8	2	0
dec	4,7	2,1	6,9	87	84	62,5	8	73,9			44,0	18	1014,5	6,0	3,3	221	24	17	9	3	0
winter	3,8	1,1	6,2	86	83	166,0	7	182,2			180,7	23	1015,4	6,0	3,2	215	70	48	26	10	2
lente	8,6	4,7	12,4	80	72	130,9	6	152,3			518,7	40	1014,7	5,2	2,0	290	73	41	15	4	0
zomer	16,2	12,1	20,1	79	71	99,3	4	192,3			640,9	44	1016,1	4,6	2,2	290	69	33	8	0	.
herfst	10,8	7,3	13,9	84	78	170,2	7	262,3			301,3	30	1014,9	5,1	2,3	235	66	39	18	5	0
jaar	9,9	6,3	13,1	83	76	566,4	6	789,1			1641,6	34	1015,3	5,2	2,4	258	278	161	67	19	2

Aantal dagen met:

210	Temperatuur maximum >= 30 °C	maximum >= 25 °C	maximum >= 20 °C	maximum >= 15 °C	maximum < 0 °C	minimum < 0 °C	minimum < -5 °C	minimum < -10 °C	10cm < 0 °C	Weersverschijnselen mist	regen	sneeuw	hagel	onweer	ijsvorming	Neerslag droog	>0 mm	>=0,1 mm	>= 1 mm	>=10 mm	Zonneschijn zonloos	<= 20 %	>= 80 %
jan	.	.	.	.	3	11	4	1	14	7	21	5	3	1	1	7	24	18	12	1	13	20	1
feb	.	.	.	0	2	12	3	1	15	6	16	5	3	1	1	9	19	13	9	1	8	14	2
mrt	.	.	0	2	0	7	1	0	12	6	20	3	3	1	0	9	22	17	12	1	6	14	3
apr	.	0	1	6	.	3	.	.	9	4	19	1	3	2	0	10	20	14	9	1	3	10	4
mei	.	2	6	17	.	0	.	.	3	4	18	0	1	3	.	12	19	13	9	1	2	9	6
jun	0	3	8	25	.	.	.	.	1	3	19	.	0	4	.	10	20	14	10	2	2	9	4
jul	1	5	14	31	.	.	.	.	0	3	18	.	0	4	.	12	19	13	9	2	2	9	4
aug	1	4	17	31	.	.	.	.	0	4	18	.	0	4	.	11	20	14	9	2	1	7	5
sep	.	0	6	27	.	.	.	.	0	7	19	.	0	4	.	8	22	16	12	3	3	10	2
okt	.	.	1	12	.	1	.	.	4	7	21	0	1	3	.	8	23	18	13	3	6	14	3
nov	.	.	.	1	0	5	0	.	8	5	23	2	4	3	1	6	24	19	14	3	10	18	2
dec	.	.	.	0	1	10	2	0	12	6	23	4	4	1	1	6	25	18	13	2	14	21	1
winter	.	.	.	0	6	33	9	2	41	19	60	14	10	3	3	22	68	49	34	4	35	55	4
lente	.	2	7	25	0	10	1	0	24	14	57	4	7	6	0	31	61	44	30	3	11	33	13
zomer	2	12	39	87	.	.	.	.	1	10	55	.	0	12	.	33	59	41	28	6	5	25	13
herfst	.	0	7	40	0	6	0	.	12	19	63	2	5	10	1	22	69	53	39	9	19	42	7
jaar	2	14	53	152	6	49	10	2	78	62	235	20	22	31	4	108	257	187	131	22	70	155	37

VALKENBURG (ZH)

EXTREMEN tijdvak 1971 t/m 2000

210	Temperatuur °C																								Neerslag mm						
	maandgemiddelde												absoluut																		
	laagste	jaar	hoogste	jaar	laagste minimum	jaar	hoogste minimum	jaar	laagste maximum	jaar	hoogste maximum	jaar	laagste minimum	jaar	dag	hoogste minimum	jaar	dag	laagste maximum	jaar	dag	hoogste maximum	jaar	dag	laagste maandsom	jaar	hoogste maandsom	jaar	hoogste dagsom	jaar	dag
jan	-2,3	85	6,9	75	-6,1	85	4,3	75	0,6	87	8,8	75	-16,4	79	1	10,0	93	16	-9,6	87	14	13,8	99	5	3,9	97	159,9	88	28,0	84	23
feb	-3,4	86	7,5	90	-7,1	86	4,8	95	-0,1	86	10,3	90	-14,0	86	24	10,9	90	20	-5,5	86	9	15,9	98	15	0,8	86	107,0	90	19,4	96	12
mrt	2,6	87	8,3	91	-1,0	87	4,9	81	6,1	87	12,1	91	-11,1	71	5	11,1	98	30	-3,8	71	6	20,8	90	17	9,0	93	111,5	81	24,8	98	6
apr	5,8	86	10,4	93	1,9	86	6,0	98	9,5	86	15,0	87	-4,4	96	1	13,7	93	29	3,6	86	11	25,9	93	30	3,3	76	109,6	98	29,1	94	4
mei	9,5	91	15,1	92	5,9	87	10,3	92	12,6	91	19,7	92	-1,5	76	1	17,0	98	13	6,2	79	4	29,7	98	11	7,5	89	140,5	79	34,9	86	20
jun	12,8	91	17,0	76	8,4	72	12,3	98	16,0	91	21,5	76	1,7	91	2	18,7	00	20	10,0	75	3	33,5	96	7	14,3	88	141,6	98	51,4	82	22
jul	15,1	78	20,6	94	11,1	78	15,5	94	17,7	74	25,5	94	5,4	80	22	19,8	94	31	13,1	78	7	33,1	94	24	5,5	83	174,8	87	45,4	87	17
aug	15,5	93	20,4	97	11,2	74	15,8	97	18,8	93	25,2	97	5,5	73	22	19,7	97	11	14,5	93	22	34,6	90	4	9,9	76	166,4	92	38,2	88	25
sep	11,9	86	17,4	99	7,2	86	13,6	99	15,8	86	21,3	99	1,2	71	16	17,8	89	16	11,0	74	25	28,7	91	3	12,7	77	173,6	74	40,0	74	5
okt	8,0	74	13,0	95	4,1	74	9,6	89	11,0	74	17,1	95	-4,4	97	29	16,0	97	9	4,0	85	31	24,5	83	4	18,9	95	188,6	74	40,5	74	4
nov	2,8	93	10,3	94	0,5	85	7,2	94	5,2	93	12,4	94	-7,1	98	23	13,7	96	3	-2,5	93	22	17,5	84	1	32,7	94	211,4	00	43,0	83	27
dec	-0,6	95	7,8	74	-3,6	95	5,7	88	1,9	95	9,5	74	-10,6	78	31	12,2	00	12	-7,6	78	31	15,4	77	23	20,2	72	155,6	99	36,8	85	26

210	Zonneschijn uren in %						Windsnelheid m/s						Temperatuur: aantal dagen met ..... per maand															
													Maximum								Minimum						10 cm	
	laagste maandsom	laagste percentage	jaar	hoogste maandsom	hoogste percentage	jaar	Hoogste uurgem.	jaar	dag	hoogste windstoot	jaar	dag	grootste >= 30 °C	jaar	grootste >= 25 °C	jaar	grootste >= 20 °C	jaar	grootste < 0 °C	jaar	grootste < 0 °C	jaar	kleinste < 0 °C	jaar	grootste < -10 °C	jaar	grootste < 0 °C	jaar
jan	23,5	9	73	93,4	38	96	25,2	76	2	38,1	83	18							13	87	24	97	1	88	11	85	27	79
feb	39,3	14	79	132,4	48	75	23,1	83	1	35,0	84	6							16	86	26	86	1	95	9	86	27	86
mrt	55,9	15	81	202,5	55	93	22,1	86	24	36,5	87	27					2	90	4	71	16	96			3	71	22	72
apr	106,5	26	75	248,1	60	96	26,2	73	2	37,6	73	2			3	93	6	87			6	96					18	84
mei	88,0	18	83	331,4	68	89	19,0	00	28	29,0	00	28			7	00	16	92			3	79					10	90
jun	130,6	26	77	309,7	62	76	14,4	87	6	23,1	94	21	3	76	10	76	16	92									4	75
jul	142,5	28	00	321,8	64	94	15,9	85	22	25,2	89	30	4	76	16	94	29	94									1	71
aug	144,5	32	77	303,9	67	76	15,9	86	27	24,7	94	18	4	75	16	97	28	00									1	73
sep	78,4	21	84	196,1	51	97	20,6	75	27	26,8	90	21			3	91	20	99									4	86
okt	51,7	16	98	151,6	46	95	24,2	86	20	37,6	86	20					5	90			6	83					15	74
nov	25,6	10	76	107,6	41	89	25,7	72	13	35,5	83	27							5	93	17	85					19	85
dec	20,7	9	82	74,8	31	94	22,1	86	19	32,4	77	24							9	96	27	95			1	96	28	95

210	Aantal dagen met ..... per maand																																			
	Weerverschijnselen												Neerslag												Zonneschijn					
	grootste met mist	jaar	grootste regen	jaar	kleinste regen	jaar	grootste sneeuw	jaar	grootste hagel	jaar	grootste onweer	jaar	kleinste droog	jaar	grootste droog	jaar	grootste >= 1mm	jaar	kleinste >= 1 mm	jaar	grootste >= 5	jaar	grootste >= 10	jaar	grootste zonloos	jaar	kleinste zonloos	jaar	grootste >= 80 %	jaar
jan	19	71	29	88	8	96	18	85	11	81	4	95	1	84	21	96	21	84	1	96	13	88	5	98	20	90	4	98	6	97
feb	13	72	26	95	1	86	12	78	11	73	5	99	1	95	19	86	18	95	1	85	9	00	3	90	17	93	1	00	6	99
mrt	13	74	29	88	13	96	9	95	11	95	7	95	1	79	18	96	24	88	3	73	10	88	5	98	13	81	2	97	12	93
apr	8	81	27	98	6	96	5	82	11	73	7	98	3	98	23	96	19	98	1	76	6	99	2	98	10	81			12	96
mei	12	84	26	83	4	89	4	79	4	83	8	00	3	73	27	89	18	83	2	89	11	79	5	79	8	83			13	92
jun	9	80	26	98	8	76			2	97	9	80	2	80	18	86	17	87	3	96	11	98	5	98	5	81			12	76
jul	13	83	25	93	8	94			2	84	8	95	2	78	23	94	18	93	2	83	10	93	7	93	5	88			10	76
aug	11	77	26	87	7	95			1	95	9	99	2	77	24	95	17	92	3	83	11	92	6	92	4	92			15	75
sep	15	76	27	84	10	97			3	74	8	99	1	80	20	97	19	98	5	86	13	74	7	94	8	84			7	81
okt	13	77	30	98	12	95	1	86	6	74	10	81	1	98	19	95	22	74	5	85	14	00	7	74	11	88	2	99	8	95
nov	11	94	28	00	17	93	12	85	12	73	8	90	1	78	13	89	24	00	7	93	15	00	7	77	18	76	4	95	7	89
dec	15	81	30	99	15	96	13	81	11	76	4	00	1	99	14	96	24	99	5	96	13	93	6	99	19	71	8	00	6	96

210 VALKENBURG (ZH)

Standaard afwijkingen	Temperatuur °C			Relatieve vochtigheid %	Neerslag		Verdamping	Globale Straling	Zonneschijn	Lucht druk	Wind
	gemiddeld	gemiddeld minimum	gemiddeld maximum	gemiddeld	duur in uren	som in mm	som in mm	som in J/cm2	in uren	in hPa	in m/s
jan	2,6	2,8	2,5	3	28,6	37,9	-	-	19,1	6,5	1,6
feb	2,6	2,7	2,4	4	22,6	28,0	-	-	25,6	5,6	1,3
mrt	1,7	1,7	1,7	3	28,2	29,1	-	-	35,9	5,2	0,9
apr	1,1	1,1	1,4	4	19,3	26,6	-	-	37,7	4,4	0,8
mei	1,5	1,2	1,9	4	19,3	32,0	-	-	55,9	3,2	0,7
jun	1,0	0,9	1,4	3	18,8	31,4	-	-	50,3	2,9	0,6
jul	1,3	1,0	1,9	4	18,5	42,1	-	-	43,6	2,2	0,6
aug	1,2	1,1	1,7	3	14,6	37,2	-	-	41,8	2,5	0,6
sep	1,1	1,4	1,3	3	27,0	47,5	-	-	26,6	4,1	0,9
okt	1,2	1,3	1,3	3	26,5	45,0	-	-	23,7	4,8	1,0
nov	1,5	1,6	1,5	3	24,8	40,1	-	-	17,8	5,2	1,0
dec	1,9	2,0	1,8	3	28,5	34,3	-	-	15,3	6,2	1,1

Appendix II. Drawings of cross sections breakwater

Appendix III. Quantities of rehabilitation breakwaters

In the tables below the quantities of the different alternatives are given. The head and trunk section are described separately. The trunk section is 700m and the head section 100m.

Accropode

Activities	Quantity [m^3]
Trunk section – 100m	
1 Removal	4400m^3 quarry stone 1-6 tonne
	200 17- tonnes concrete cubes
	96 30- tonnes concrete cubes
	160 m^3 stone asphalt
2 Construction of soil protection	2716 m^3 gravel
3 Construction of toe and filter layers	1272m^3 10/60 kg
	1864 m^3 60/300kg
	1620 m^3 0,1-300kg
	7432 m^3 2-4 ton
	1408 m^3 3-6 ton
4 Placement of elements	476 units (12 m3)
Head section- 100m	
1 Removal	5600m^3 quarry stone 1-6 tonne
	240 17- tonnes concrete cube
	112 45-tonnes concrete cube
	200 m^3 stone asphalt
2 Construction of soil protection	1400 m^3 gravel
3 Construction of toe and filter layers	550m^3 10/60 kg
	776m^3 60/300 kg
	1260m^3 0,1-300 kg
	4900m^3 1-3 ton
	1408 m^3 3-6 ton
	6076m^3 2,6-5,2 ton
4 Placement of elements	390 units (18m^3)

XBloc

Activities	Quantity [m^3]
Trunk section – 100m	
1 Removal	4400 m^3 quarry stone 1-6 tonne
	200 17- tonnes concrete cubes
	96 30-tonne concrete cubes
	160 m^3 stone asphalt
2 Construction of soil protection	2710 m^3 gravel
3 Construction of toe and filter layers	1270 m^3 10/60 kg
	1864 m^3 60/300kg
	1620 m^3 0,1-300kg
	5779 m^3 1-3 ton
	1408 m^3 3-6 ton
4 Placement of elements	384 units (9 m^3)
Head section – 100m	
1 Removal	5600m^3 quarry stone 1-6 tonne
	240 17- tonnes concrete cube
	112 45-tonne concrete cubes
	200 m^3 stone asphalt
2 Construction of soil protection	1830 m^3 gravel
3 Construction of toe and filter layers	720 m^3 10/60 kg
	872 m^3 60/300 kg
	1260 m^3 0,1-300 kg
	5800 m^3 1-3 ton
	1408 m^3 3-6 ton
4 Placement of elements	447 units (12m^3)

Cube 2350

Activities	Quantity [m^3]
Trunk section – 100m	
1 Removal	4400 m^3 quarry stone 1-6 tonne
	200 17- tonnes concrete cubes
	96 30-tonne concrete cubes
	160 m^3 stone asphalt
2 Construction of soil protection	2710 m^3 gravel
3 Construction of toe and filter layers	1270 m^3 10/60 kg
	1864 m^3 60/300kg
	1944 m^3 0,1-300kg
	4120 m^3 1-3 ton
	4560 m^3 3-6 ton
4 Placement of elements	817 units (20,8 m^3)
Head section – 100m	
1 Removal	5600m^3 quarry stone 1-6 tonne
	240 17- tonnes concrete cube
	84 45-tonne concrete cubes
	100 m^3 stone asphalt
2 Construction of soil protection	?m^3 gravel
3 Construction of toe and filter layers	? m^3 10/60 kg
	?m^3 60/300 kg
	1512 m^3 0,1-300 kg
	4000 m^3 1-3 ton
	?m^3 3-6 ton
	5700 m^3 6-10 ton
4 Placement of elements	644 elements (29,8 m^3)

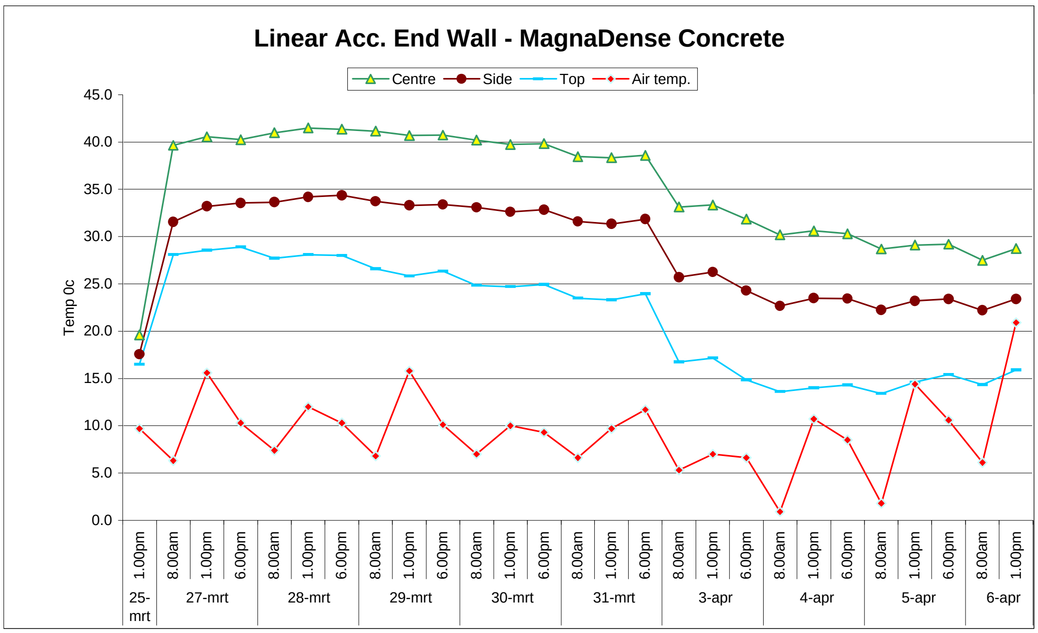
Cube 3800

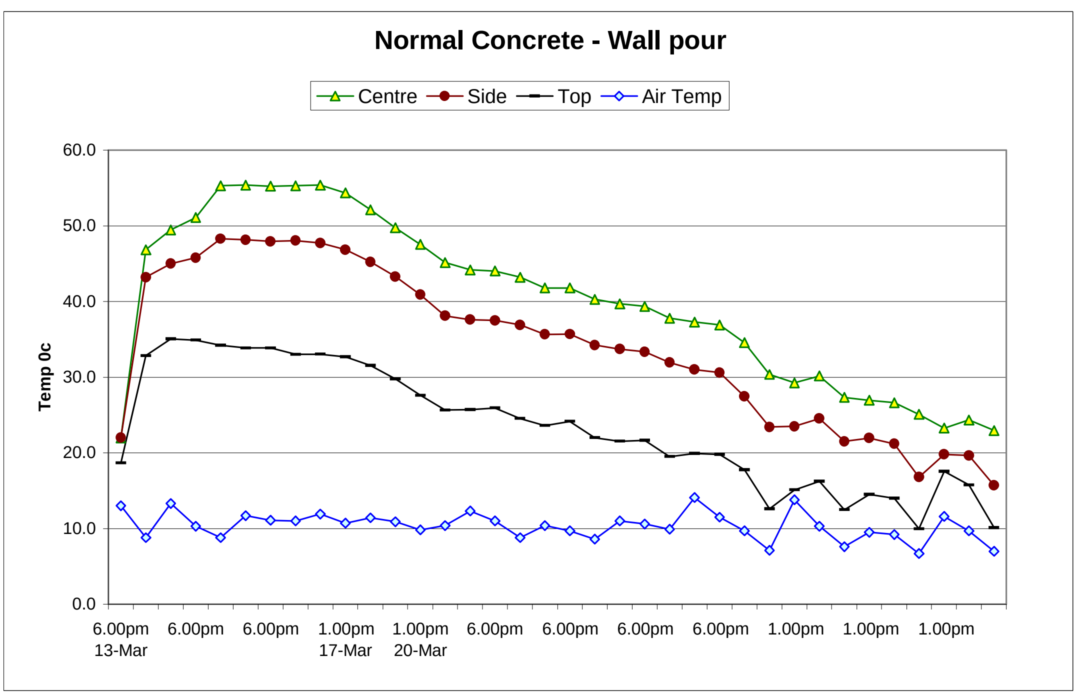
Activities	Quantity [m³]
Trunk section – 100m	
1 Removal	4400 m³ quarry stone 1-6 tonne
	200 17- tonnes concrete cubes
	64 30-tonne concrete cubes
	0 m³ stone asphalt
2 Construction of soil protection	2710 m³ gravel
3 Construction of toe and filter layers	1270 m³ 10/60 kg
	1864 m³ 60/300kg
	1944 m³ 0,1-300kg
	6181 m³ 1-3 ton
	1408 m³ 3-6 ton
4 Placement of elements	2937 units (2,2 m³)
Head section – 100m	
1 Removal	5600m³ quarry stone 1-6 tonne
	240 17- tonnes concrete cube
	70 45-tonne concrete cubes
	0 m³ stone asphalt
2 Construction of soil protection	1770 m³ gravel
3 Construction of toe and filter layers	710 m³ 10/60 kg
	856 m³ 60/300 kg
	1512 m³ 0,1-300 kg
	5194 m³ 1-3 ton
	1408 m³ 3-6 ton
4 Placement of elements	2328 units (3,375m³)

Appendix IV. Heavy concrete

Heat of hydration

MagnaDense Concrete, when used as an aggregate in mass pours has the effect of lowering the maximum temperature by acting as a heat sink. The heat is retained longer and released more slowly helping to reduce the risk of Thermal cracking.





Fine aggregates

MINELCO
minerals customised for industry

INDICATIVE PRODUCT SPECIFICATION - 2004

Product MagnaDense 2

Chemical Analysis

CaO	0.19	%
SiO_2	0.6	%
Fe	70.7	%

Screen Analysis

Opening (mm)	2.0	1.0	0.5	0.25	0.125	0.063
Passing (%)	98	83	49	23	10	4

Moisture 3.0 %
Density 5.1 g/cm^3
Bulk density 3.2 t/m^3

Product MagnaDense 4

Chemical Analysis

CaO	0.5	%
SiO_2	1.2	%
Fe	69.8	%

Screen Analysis

Opening (mm)	4.0	2.0	1.0	0.5	0.25	0.125	0.063
Passing (%)	98	72	53	43	34	24	13

Moisture 3.7 %
Density 5.1 g/cm^3
Bulk density 3.2 t/m^3

Date 2004-May
Customer DMC/TUD
Reference -
Issued by Leon Hendriks

The above chemical and physical data are expected average figures for deliveries during 2004. They are given in good faith but without guarantee. All chemical analyses are in the dry state. Oxides are not indications of the phases present, but only conventional representations of elements.

Coarse Aggregates

MINELCO
minerals customised for industry

INDICATIVE PRODUCT SPECIFICATION - 2004

Product MagnaDense 14

Chemical Analysis

CaO	1.50	%
SiO_2	4.0	%
Fe	65.5	%

Screen Analysis

Opening (mm)	< 16.0	< 10.0	< 5.0	< 2.0
Passing (%)	99	86	32	12

Moisture 2.5 %
Density 4.9 g/cm^3
Bulk density 2.8 t/m^3

Product MagnaDense 30

Chemical Analysis

CaO	3.5	% (+/- 1.0)
MgO	1.1	% (+/- 0.1)
Al_2O_3	0.6	%
SiO_2	7	% (+/- 1)
TiO_2	0.3	%
V_2O_5	0.2	%
Fe	60	% (+/- 1)

Screen Analysis

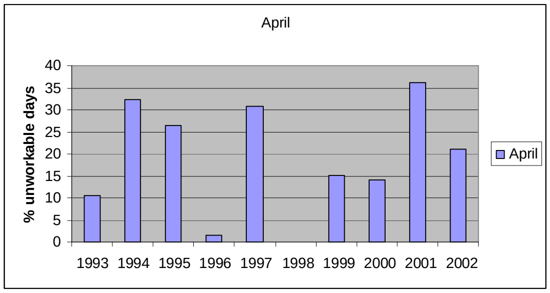
Opening (mm)	25	15	10	5
Passing (%)	85-95	50-70	30-50	5-15

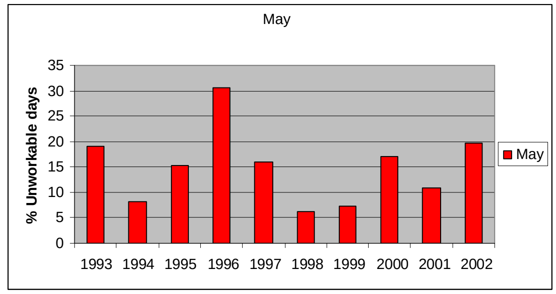
Moisture < 1 %
Density > 4.4 g/cm^3
Bulk density 2.8 t/m^3
Angle of repose 39-41 degrees

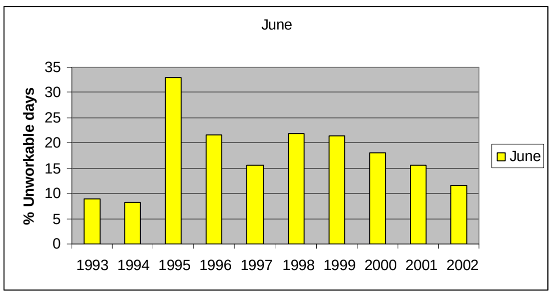
Date 2004-May
Customer DMC/TUD
Reference -
Issued by Leon Hendriks

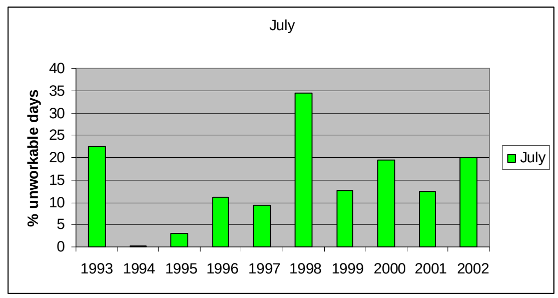
The above chemical and physical data are expected average figures for deliveries during 2004 They are given in good faith but without guarantee. All chemical analyses are in the dry state. Oxides are not indications of the phases present, but only conventional representations of elements.

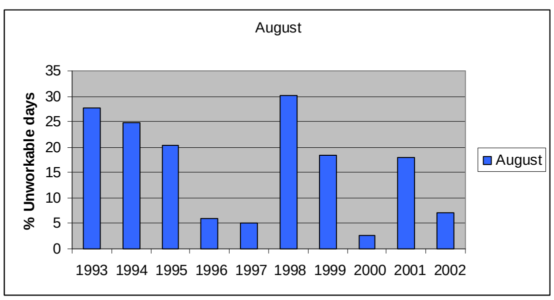
Appendix V. Workability

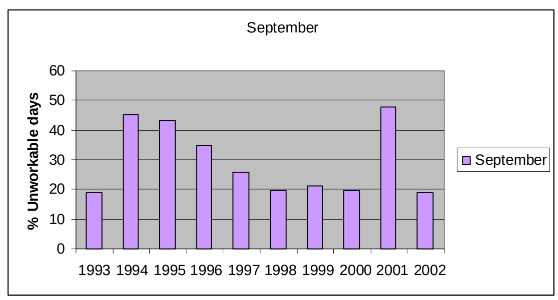


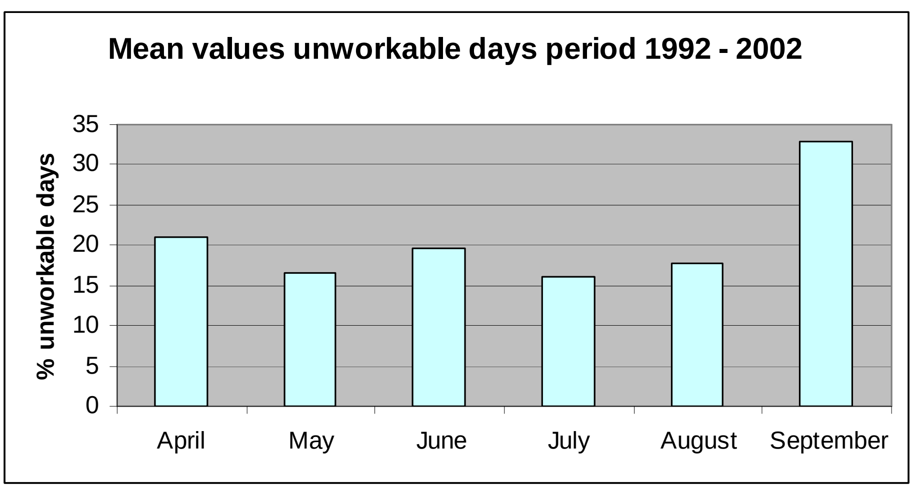


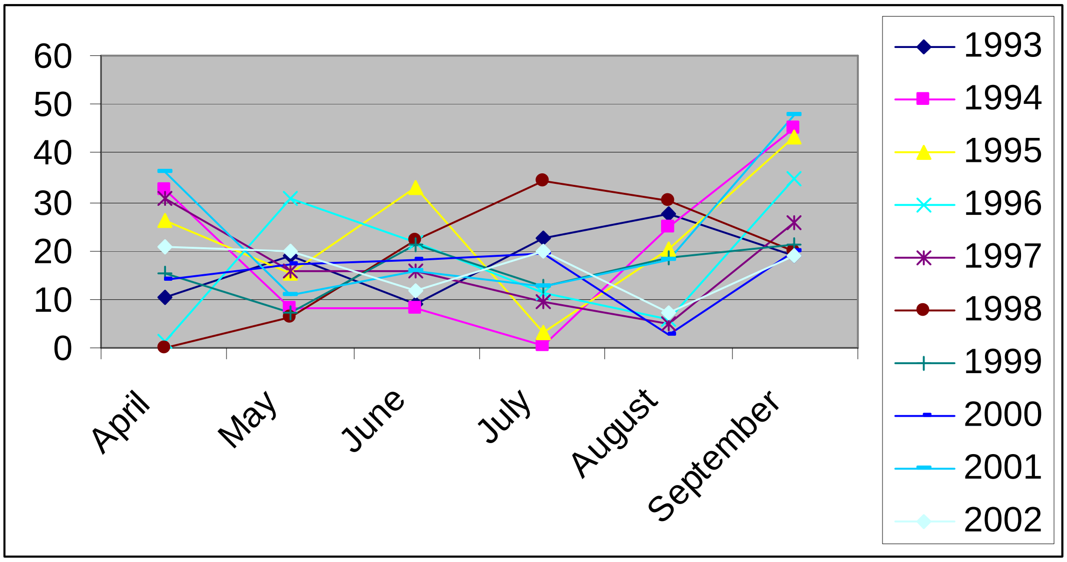




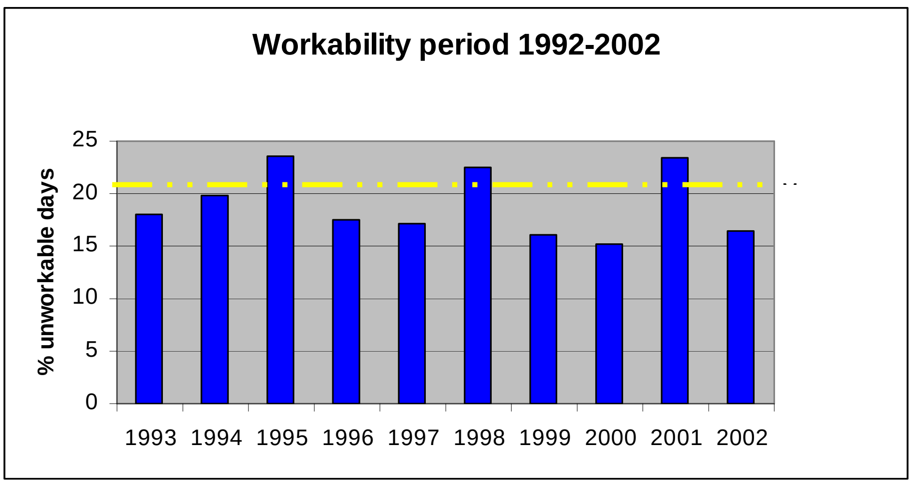








60
50
40
30
20
10
0
April
May
June
July
August
September
1993
1994
1995
1996
1997
1998
1999
2000
2001
2002



Workability period 1992-2002
% unworkable days
25
20
15
10
5
0
1993
1994
1995
1996
1997
1998
1999
2000
2001
2002

Appendix VI. Equipment

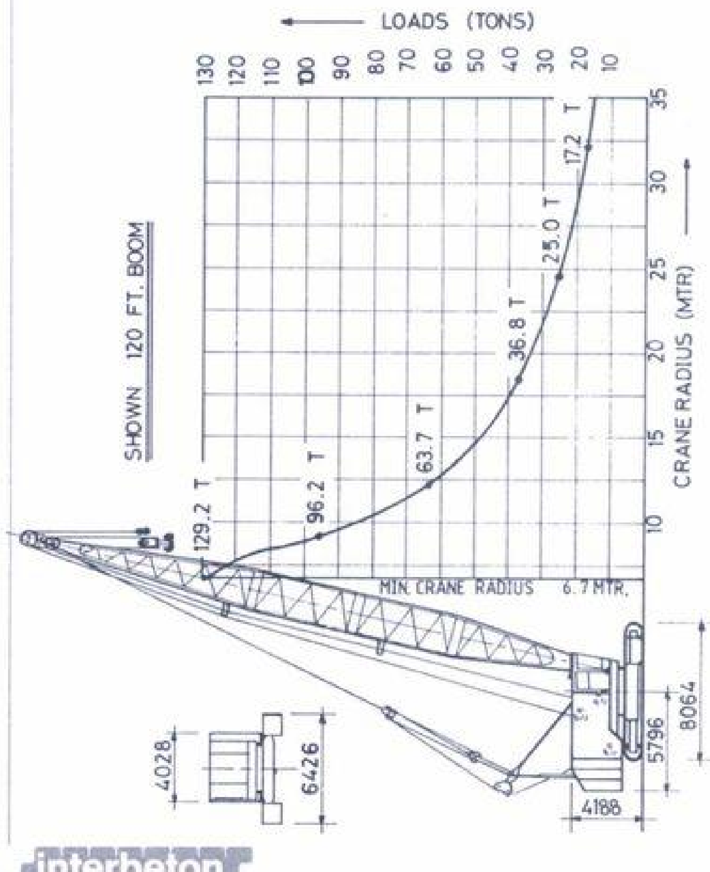
CRAWLER CRANES

interbeton bv
international division of hollandsche beton groep nv

ib nº 1558

MANITOWOC 4100W-S2

MAX. LIFTING CAPACITY	209 TONS
ENGINE	CUMM. NTA-855-C360
WEIGHT APPROX.	204.4 TONS
TRAVEL SPEED	2.2 KM/H
SWING SPEED	4 RPM
MAX. SINGLE LINE PULL	145 KN
BOOM LENGTH	21.3 - 48.8 MTR. (70 FT. - 160 FT.)
BOOM LENGTH AVAILABLE	36.6 MTR. (120 FT.)



interbeton

Appendix VII. Placement of armour units

Xbloc

Figure 8-1 Correct placement

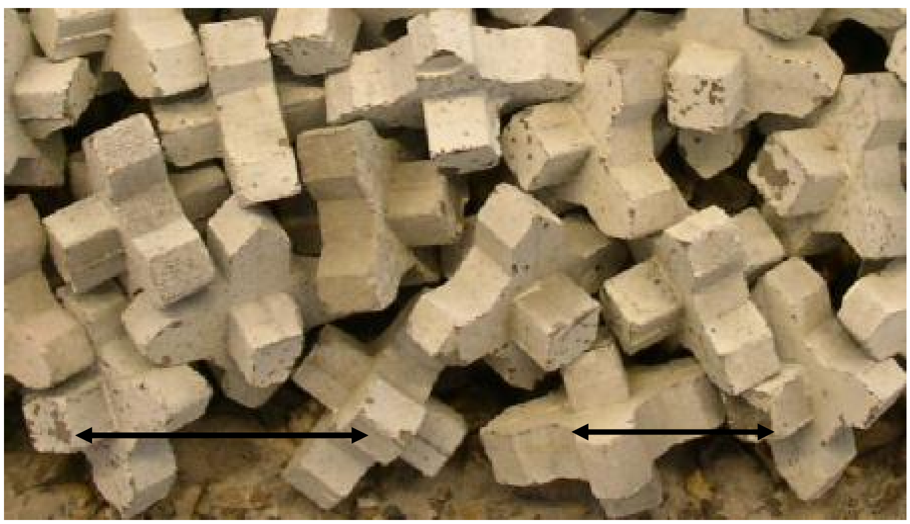


Figure 8-2 Wrong distances in the first row

Figure 8-3 Loose Xbloc

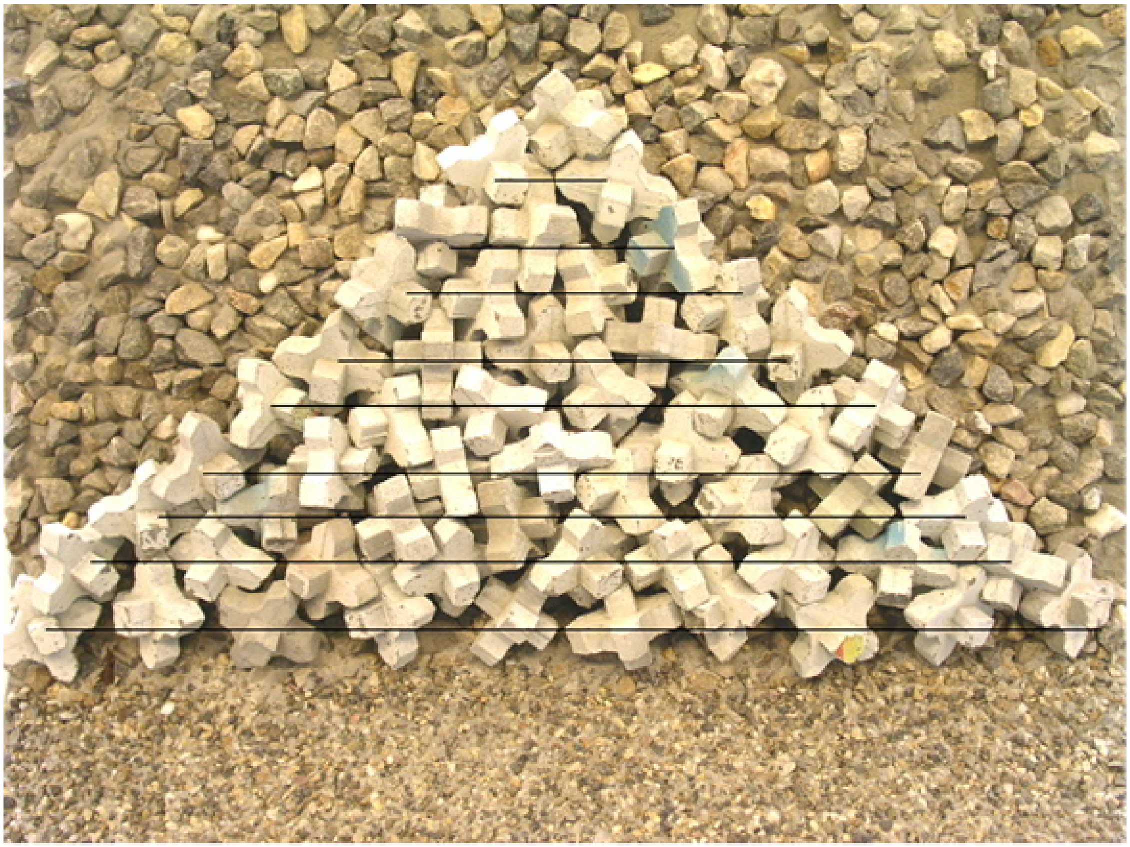


Figure 8-4 Placement of Xblocs

Accropode

Figure 8-5 Correct placement

Figure 8-6 Sagging unit

Figure 8-7 Anvils touching

Figure 8-8 Loose unit

Figure 8-9 Loose unit

Figure 8-10 Placement of Accropode for experiment DMC March 2004

Figure 8-11 Incorrect density and loose units

Core-loc

Figure 8-12 Correct placement

Figure 8-13 Gap with crest wall

Figure 8-14 Gap in berm