DURBAN HARBOUR ENTRANCE

WIDENING AND DEEPENING PROJECT




MASTER PROJECT CT4061, GROUP CF44

FINAL REPORT, SEPTEMBER 2005

PRDW

Faculty of Civil Engineering and Geosciences

Delft University of Technology

PREFACE

This final report was prepared under a collaborative project carried out by PRDW (Consulting Coastal, Ocean and Environmental Engineers: Prestedge Retief Dresner and Wijnberg) and students of the faculty of Civil Engineering and Technical Geosciences of the Delft University of Technology in May and June 2005. The joint project was supported by PRDW and DUT and sponsored by Royal Boskalis Westminster nv and Van Oord Dredging and Marine Contractors bv. Plaxis software was kindly made available by Plaxis bv.

The final report was produced under the guidance of the DUT and PRDW:

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The group consisted of four members, students of the Faculty of Civil Engineering and Technical Geosciences of the Delft University of Technology. Two of the group members are coastal engineering students and two are geotechnical engineering students. The combination of these two Master subjects was able to cope with the geotechnical and coastal engineering problems this project has to offer. The group consisted of the following students:

Sebastiaan Frankenmolen	geotechnical engineering;
Arjan Frens	coastal engineering;
Carl Gilding	coastal engineering;
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PLAXIS

Van Oord

ABSTRACT

The Durban Harbour, run by the National Ports Authority of South Africa (NPA), is the largest harbour in South Africa. To stay competing in the 21st century the harbour entrance must be widened and deepened for the entry of Post Panamax ships. An important part of the Durban Harbour entrance Widening and Deepening project is the demolishion of the existing North Groyne at the harbour entrance and the construction of a New North Groyne approximaly 100 metres to the north. Preliminary research was already conducted by PRDW (PTY) LTD. A suitable construction method for the removal of the existing groyne and construction of the New North Groyne has been executed. Based on this construction method a design of the New North Groyne has been made. The design is tested on wave-structure interaction failure mechanisms and slope stability, including liquefaction potential, and is considered safe. A detailed design of the falling apron, utilized on the channel side slopes, is made.

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1. INTRODUCTION

In this chapter the general project description will be given. First the aims of the master project will be explained. After that a description of the location of the project will be given followed by the description of the company that is making the design. Finally the structure of the report will be discussed.

The aim of this chapter is to give a general description of what can be expected in the rest of the report.

1.1 Master project

The Master project is based on research in, or a design based on, one of the sub sectors of civil engineering. A realistic actual civil engineering problem has to be solved in a multidisciplinary interrelationship. Monographs and designs have to be specified into a complex work. The study is based on knowledge, understanding and skills gained in the preceding study years. Attention must be paid to quality control and the evaluation of the design process.

The aims of the MSc-project are:

- Learning to design in a sub-sector of civil engineering in multidisciplinary link;
- Integrated appliance of knowledge and skills from previous years;
- Application of design knowledge and skills from the first, second and third year;
- Learning to work by means of an interdisciplinary approach;
- Learning to report, present and defend the final product;
- Learning to apply elementary quality guarantee principles during the design process;
- Learning to evaluate the interdisciplinary work process.

1.2 Durban Harbour, South Africa

With a total area of 1,221,037 km² (33 times the size of The Netherlands) and a population of 33 million people, South Africa is one of the largest countries in Africa. The city of Durban is located on the South African East Coast, see figure 1-1.

figure 1-1: Durban, South Africa;

South Africa has a long tradition of trade through shipping. In the 15th century seafarers used South Africa as a resting place for journeys to the Far East. During the old days a couple of ports were established like Cape-Town, Port Elizabeth, Richards Bay and Durban. The bay around which the city of Durban lies was discovered by the Portuguese traveller Vasco da Gama in his caravel San Gabriel. He sighted land on Christmas day 1497 and therefore named it Terra Natalia. The bluff at the harbour is the only landmark of note for hundreds of miles along this section of the coast and in its lee lays the enticing lagoon, see figure 1-2.

figure 1-2: Durban harbour;

Later on Natal was named after the Governor of the Cape, Sir Benjamin D`Urban. During the 19th century Durban grew out to be a large harbour supplying a large region of the hinterland. Nowadays the harbour is run by the National Port Authority of South Africa. To stay competing in the 21st century the harbour entrance must be widened and deepened for the entry of Post Panamax ships. This widening and deepening project will be executed in close relation with the development of new tourist industry along the beaches close to the harbour entrance. The tourist industry has become of vital importance to the Durban economy.

1.3 PRDW (PTY) LTD.

Mr Verhagen introduced us to Dr Wijnberg of PRDW. This company is working on the Durban Harbour Entrance Widening and Deepening Project. Prestedge Retief Dresner Wijnberg (PTY) LTD. is an international firm of consulting coastal, ocean and environmental engineers. PRDW offers a full range of consulting engineering services. From full feasibility studies to detailed design and contract administration, including research, planning and environmental studies where needed. Their main office of 30 employees is located in Cape Town at the Waterfront. PRDW was contracted by the National Port Authority of South Africa (NPA) for the widening and deepening project of the Durban harbour.

1.4 Contents of the report

In the previous paragraphs general information about the Master project, the Durban Harbour project and PRDW has been given. This in order to give the reader some background information on the situation in which the project is being carried out.

Chapter 2 gives the reader an overview of the entire design process for coastal engineering projects.

Chapter 3 contains a summary of the preliminary design by PRDW for the Durban Harbour widening and deepening project. The different components of the project will be briefly discussed and a choice will be made as to which component will be worked out in the final report. Finally, a problem definition and objection will be formulated.

In chapter 4 the design process will be specified for the chosen element of the Durban Harbour widening and deepening project. The structure of this report and a time schedule of the project will be given.

Chapters 5 and 6 provide the physical site conditions (bathymetry, hydraulic and geotechnical). From those conditions the assumptions and requirements follow.

In chapter 7 the different construction alternatives will be made based on the requirements of the previous chapter. A multi criteria analysis will be executed to determine the best three alternatives for design in the next chapter.

Chapter 8 contains the final design of the three selected alternatives including the choice of the optimal alternative based on both its construction method and its design.

In chapter 9 and 10 the chosen alternative will be tested on, respectively, wave-structure interaction and slope stability.

Chapter 11 discusses the design and construction of a falling apron on the channel slopes.

The conclusions and recommendations will be given in chapter 12.

2. THE DESIGN PROCESS

2.1 Introduction

This chapter presents an overview of the entire design process for coastal engineering projects. In chapter 4 this design process will be specified for the chosen element of the Durban Harbour Widening and Deepening project. The choice of element will be made in chapter 3.

The aim of this chapter is to familiarize with the design process as used in coastal engineering projects.

2.2 General

In the design process both the functional as well as the structural design has to be looked at. This implies that one has to design a construction which fulfils the functional requirements, but also that the construction will not fail, collapse or be damaged seriously with a predefined probability. The objective of the design process is to find a concept that meets the requirements and that can be realized not only in terms of technical feasibility, but also in terms of cost–benefit ratio and social and legal acceptance. This implies that the solution of the design process must combine the following elements:

- Functionality;
- Technology (what is feasible);
- Environment (what is allowed or accepted);
- Cost and benefit;
- Paper work (drawing board);
- Matter (actual construction).

2.3 Abstraction level

In any design process various levels of abstraction can be discerned. In most cases it is sufficient to distinguish three levels:

- **Macro level:** the system;
- **Meso level:** a component of the system;

- **Micro level:** an element of one of the components.

2.4 Phases

During the design process certain phases can be recognised. A logical set of phases is:

- **Initiative:** formulation of the ultimate goals of the design object as part of the system;
- **Feasibility:** review of the system with respect to technical, economical, social and environmental consequences and feasibility. Requirements are formulated on the component level;
- **Preliminary design:** giving shape to the system on broad lines, including determinations of the exact functionality of the components and definition of requirements at the element level;
- **Final design:** composition of a set of drawings and specifications for the system in which the final shape of the components is fixed and the functionality of the elements is determined;
- **Detailed design:** composition of a set of drawings and specifications in which the final shape of the elements is fixed.

It must be recognized that the design process is a complex iterative process and may be described in more than one way. Another overall formulation in flowchart form is given in Appendix B.

2.5 Cyclic design

Each activity in the design process, which is represented by a cell in table 2-1, is a cyclic process in its own right, consisting of a number of steps:

- **Analysis:** assembling of available data and arrange for the provision of missing data; Drawing up a set of criteria that the design must fulfil (list of requirements), cross-checking all with respect to cost and functionality;
- **Synthesis:** generation of conceptual ideas and alternatives that broadly meet the requirements;
- **Simulation:** detailing of concepts and alternatives (by calculation, simulation, or modelling) up to a level that makes them mutually comparable. Again a crosscheck with respect to cost and functionality is required;

- **Evaluation:** assessment of the concepts and alternatives, comparison on the basis of cost and benefit;
- **Decision:** selection of the best option. If more than one option is acceptable, repeat the process in further detail, until a final decision can be made. This may involve some toggling between the abstraction levels in a particular phase of the design process.

Phases	Abstraction Level		
	System	Component	Element
Initial	Purpose		
Feasibility	Functionality	Purpose	
Preliminary design	Shape	Functionality	Purpose
Final design	Specifications	Shape	Functionality
Detailed design		Specification	Shape

table 2-1: cyclic processes;

3. PROBLEM DESCRIPTION

3.1 Introduction

This chapter sets out to give a summary of the preliminary design for the Durban Harbour Widening and Deepening project already executed by PRDW. The different components of the project will be briefly discussed and a choice will be made as to which component will be worked out. Finally, a problem definition and objection will be formulated.

The aim of this chapter is analyze the Durban Harbour project and choose a component for further study in this report.

3.2 Summary of Preliminary Design report

3.2.1 Introduction

The National Port Authority (NPA) wants to improve access to the Durban Harbour for larger vessels. The existing channel alignment is illustrated in appendix C, with the proposed new arrangement given in appendix D. The revised channel arrangement will require dredging of the approach, entrance and inner harbour channel. The existing North Groyne will be removed and replaced by a New North Groyne, aligned to provide protection to both the entrance channel and harbour basin. During the entire project the harbour has to remain open. This means half of the new entrance channel must be operational throughout the project.

3.2.2 Components

The preliminary design contains the following components, illustrated in figure 3-1:

- Breakwaters and revetments;
  - New North Groyne (A);
  - North Bank Revetment (B);
  - Southern Breakwater (C);
- Dredging work (D);
- Sub-Aqueous Tunnel (E);
- Sand Bypass (F).

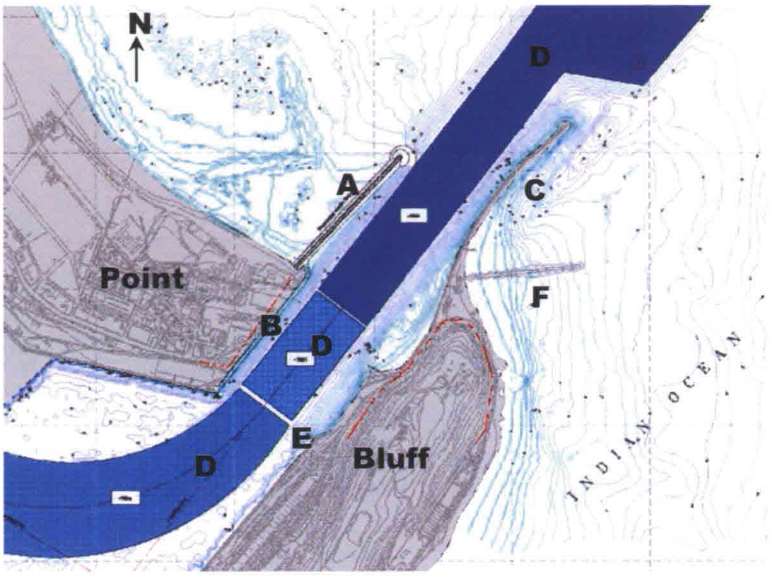


figure 3-1: project components;

3.2.3 North Groyne

The North Groyne is located at the end of the Point peninsula, see figure 3-1:

- **Function:** the main function of the North Groyne is to prevent the uncontrolled ingress of sand into the channel and the erosion of the Durban beaches. Other functions are to delineate the entrance for safe navigation, to limit the penetration of wave energy and to protect the channel banks and services in the area. The North Groyne is subject to waves of limited height and direction of approach. Waves attack the head of the groyne and then run parallel to both sides of the structure. The groyne head absorbs the bulk of the wave energy. The trunk of the groyne is largely subject to wave action along its length;
- **Changes:** due to the widening of the channel the existing North Groyne has to be removed and a New North Groyne has to be constructed approximately a hundred meters to the north. The New North Groyne is constructed with a head and a trunk of different design, optimized to their different functions as discussed above. A detailed drawing of the proposed New North Groyne can be found in appendix E. To control the ingress of sediment, the head of the groyne needs to be positioned on at least -5 m CD (Chart Datum, approximately 1 m below MSL). This will be elaborated in paragraph 5.8.1. The wider channel will result in an increase in the level of wave energy penetration. This is to be minimized by maintaining an overlap similar to the existing situation. The New North Groyne has still to be protected by the Southern Breakwater from direct wave attack. Maintaining the overlap and meeting the -5 m CD contour results in a fixed distance of less than 60 m from the head of the groyne to the proposed new channel bottom, as

illustrated in figure 3-2. A number of different concepts were considered in the preliminary design. The preferred option was found to be a rubble mound structure. Typical preliminary design sections are shown in appendix F;

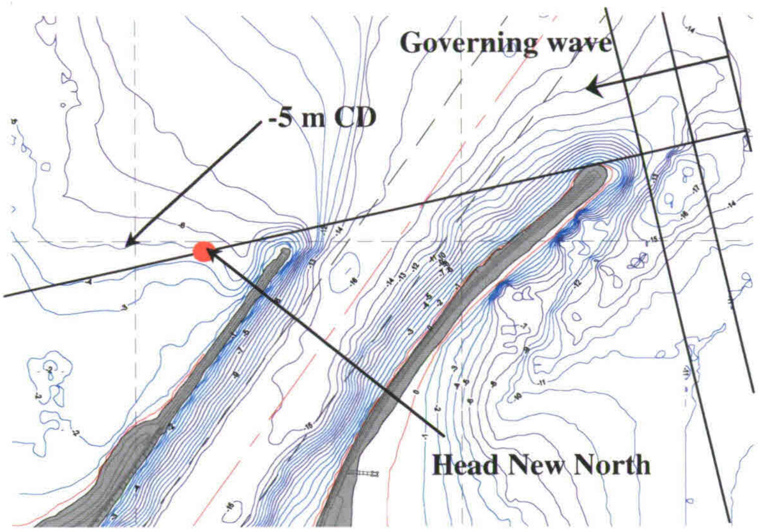


figure 3-2: location head New North Groyne;

- **Problems:** the main problem lies with the combination of the construction of the New North Groyne and the removal of the existing one. Because the harbour has to remain open at all times, the entrance channel must always be accessible. The required slope from the dredged channel to the toe of the head of the groyne is steeper than the natural sand slopes in the existing channel of approximately 1:6. A possible solution will be to pre-dredge the area on which the groyne is to be constructed. However, such dredging will tend to silt up relatively quickly in unprotected water, especially in shallow areas. Research has to be done into the design of the long term erosion and scour protection of the inner slope. Undermining of the groyne after dredging to the desired depth can occur in the event of rapid erosion and re-alignment of the lower slope. This failure mechanism can occur before permanent protection is placed; temporary stabilization of the toe is needed in that case.

3.2.4 North Bank Revetment

The North Bank Revetment is located at the Point peninsula, see figure 3-1 (B):

- **Function:** the North Bank Revetment is to a large extent an extension of the New North Groyne in order to protect the 530 m channel edge. The revetment will be subject to parallel running gravity waves and bow wave from shipping;

- **Changes:** the new channel alignment requires the removal of a large bulk of material currently accommodating various services, buildings and edge structures;
- **Problems:** the underwater slope from -1 CD to -19 m or -18 m CD will be constructed using a barge mounted backhoe. As the 1:3 slope is not expected to be stable under final design conditions, this slope has to be revetted immediately after excavation as an extension of the upper section. The final design and construction method of this revetment is therefore an important item.

3.2.5 Southern Breakwater

The Southern Breakwater is located at the end of the Bluff, see figure 3-1 (C):

- **Function:** the main function of the Southern Breakwater is to protect the harbour entrance from the prevailing southerly swells. Other functions of the Southern Breakwater are to delineate the entrance channel for safe navigation, to protect the channel banks and services in the area and to prevent the uncontrolled ingress of sand in the channel;
- **Changes:** the deepening and widening of the entrance channel can result in destabilization of the existing slope leading to the potential undermining of the breakwater. This results in the need for protection of the inner slope of the breakwater. Several sections of the seaward trunk and roundhead appear weak. A provision has been made for armouring the lee of the breakwater over a length of approximately 600 m and reinforcing the toe on the roundhead;
- **Problems:** the unknown foundation level of the breakwater causes problems with the design of the adjustments. This results in critical sections in the breakwater where a steep slope can occur during construction. Undermining of the breakwater after dredging to the desired depth can occur in the event of rapid erosion and re-alignment of the lower slope. This failure mechanism can occur before permanent protection is placed; temporary stabilization of the toe is needed. The effect of the proposed sand bypass could have unwanted effects on the breakwater. It is unknown in what way the breakwater is dependant on the sediment transport. The head of the breakwater is founded on a sand bed. This sand bed could be dynamically stable and blocking the sand transport could cause instability of the breakwater.

3.2.6 Dredging work

The channel area to be dredged is divided in three sections:

- Approach channel;
- Entrance channel;
- Inner channel.

Further dredging has to be done to accommodate the New North Groyne, to remove the old North Groyne and to remove the bulk on the North Bank to create the new channel alignment. The sections are illustrated in figure 3-1 (D):

- **Function:** deepening and widening of the channel and pre-dredging the construction area of the North Groyne;
- **Changes:** the bulk material will be removed to the required channel depth as shown in appendix G. The material will be dumped in two offshore disposal sites which have recently been used for sand borrowing. Contaminated soil will be handled in accordance to the nature and severity of its constituents. Part of the existing North Groyne material might be recycled for use in the construction of the New North Groyne or revetments;
- **Problems:** as mentioned before, during and after dredging slope stability problems might occur due to rapid erosion or scouring. Measures must be taken to prevent these effects from undermining existing or new structures. The harbour must at all times be open so the dredging work may not interfere with the incoming and outgoing vessels and a channel should always be open. The correct phasing of the dredging work is therefore very important.

3.2.7 Sub-Aqueous Tunnel

The Sub-Aqueous Tunnel is located between the Point peninsula and the Bluff; this is illustrated in figure 3-1 (E):

- **Function:** an existing group of infrastructural services runs under the channel and north embankment. These services include sewer reticulation, water, electrical cables and telecommunication cables;
- **Changes:** a new sub-aqueous tunnel will be constructed and all services will be rerouted through this tunnel. The new tunnel will have an outer diameter of around 5 m. The old tunnel will have to be decommissioned. Elements of the old tunnel can possibly be reused as a diving reef;
- **Problems:** at this stage there is not sufficient information about the design specifications of this tunnel to make a problem analysis. Both bored and immersed tunnels have their advantages and disadvantages.

3.2.8 Sand Bypass

The sand bypass will be located at the north point of the Bluff peninsula, this is illustrated figure 3-1 (F):

- **Function:** the function of the sand bypass is to prevent unwanted sediment transport into the channel. To accomplish this, a water-sand mixture is taken in at the south side of the harbour entrance, transported around the entrance and deposited;
- **Changes:** in the existing situation a sand trap is located south of the Southern Breakwater. A dredging ship pumps the sand aboard and unloads it at a jetty at the Point quay, which is connected to a distribution system. A complete new jetty will be constructed at the sand extraction area. A pumping station at the landside of the jetty pumps the slurry through a newly constructed pipeline under the channel to the distribution system on the other side;
- **Problems:** as mentioned above it is not entirely clear what the effect is of the new bypass system on the stability of the Southern Breakwater. There is no detailed design of the jetty and the pipeline under the channel bottom.

3.3 Choice of Sub-Project

3.3.1 Introduction

The different components of the project are discussed and in this paragraph a choice will be made as to which component will be worked out in this report.

3.3.2 Choice of subject

The different components are:

- **New North Groyne:** this subject has many different aspects both in geotechnical- and hydraulic engineering sense. Especially the phasing of the construction poses many interesting design questions;
- **North Bank Revetment:** the North Bank Revetment is effectively an extension of the trunk of the New North Groyne. Therefore both the designs are closely linked;
- **Southern Breakwater:** this element has many different subjects which can be discussed. Important subjects are undermining of the breakwater after dredging and effects of the sand bypass to the breakwater. The unknown foundation level of the breakwater complicates the study on these subjects;

- **Dredging work:** almost every element of the total project is dependent on dredging. Although the dredging plays a large role in this project, the actual dredging is rather straightforward. During and after dredging, slope stability problems, which affect the new and existing structures, can occur. It is much more interesting to discuss these effects than the actual dredging;
- **Sub-Aqueous Tunnel:** at this stage there is not sufficient information about the construction of this tunnel to make a problem analysis;
- **Sand Bypass:** it is not entirely clear what the effect is of the new bypass system on the stability of the Southern Breakwater. This is an interesting study subject. Little is known about the specific effects of the bypass system on the sediment transport.

3.3.3 Choice

The choice of the study subject is based on the following criteria:

- Relevance to study background;
- Sufficient level of engineering;
- Relevance to PRDW;
- Suitable for a two month study period.

Based on these criteria the choice has fallen on the New North Groyne. This subject incorporates many geotechnical- and coastal engineering problems. Furthermore the execution of especially this element calls for an optimized phasing.

3.4 Problem definition

The absence of a definitive design and optimal phasing of the construction of the New North Groyne in the Durban Harbour Entrance Widening and Deepening Project.

3.5 Objective

To create a definitive design and optimal phasing of the construction of the New North Groyne in the Durban Harbour Entrance Widening and Deepening Project.

4. THE NORTH GROYNE DESIGN PROCESS

4.1 Introduction

This chapter presents a view of the specific design process for the chosen element of the Durban Harbour Widening and Deepening project. The general design process as laid out in chapter 2 will be specified for the design of the New North Groyne.

The aim of this chapter is to create a specific design process for the design of the New North Groyne.

4.2 General

In chapter 2 it was implied that the solution of the design process must combine the following elements:

- Functionality;
- Technology (what is feasible);
- Environment (what is allowed or accepted);
- Cost and benefit;
- Paper work (drawing board);
- Matter (actual construction).

These elements will therefore be discussed in this report. The specific subject of this design, the New North Groyne, calls, due to its complex execution problems, for an active combination of the "matter" element and the "technology" and "functionality" elements.

4.3 Abstraction level

In any design process various levels of abstraction can be discerned. In most cases it is sufficient to distinguish three levels:

- **Macro level:** the system;
- **Meso level:** a component of the system;
- **Micro level:** an element of one of the components.

Three examples of the levels of abstraction are given in table 4-1.

General terms	Macro Level	Meso level	Micro level
	System	Component	Element
Example 1	New North Groyne	Dredging	Slope stability
Example 2	New North Groyne	Groyne (Head)	Armour blocks
Example 3	New North Groyne	Construction	Phasing

table 4-1: abstraction level examples;

4.4 Phases

4.4.1 Approach

The logical set of phases is described in chapter 2. The design process of the New North Groyne is already under way and the initiative, feasibility and preliminary design phase have already been completed. This report will therefore consider the two last phases of the project:

- **Final design:** Overview of the total design of the New North Groyne in combination with the optimal construction phasing;
- **Detailed design:** Composition of a set of failure analyses in which the final shape of the elements is fixed.

4.4.2 Final design

Due to the complex relation between execution and design of the New North Groyne different methods for the final design process are considered:

- **Method 1:** The different alternatives in design and execution are discussed separately and a choice of an optimal combination is made. This method is illustrated in figure 4-1;

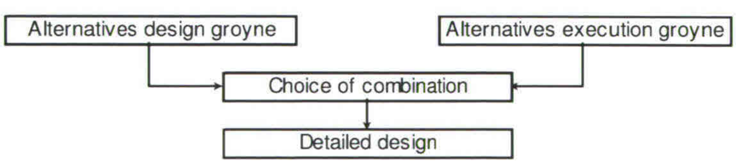


figure 4-1: final design method 1;

- **Method 2:** In this method there is a constant interaction between the design and execution. This method is illustrated in figure 4-2;

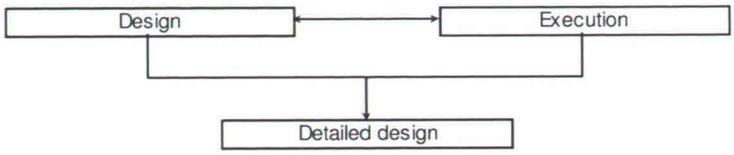


figure 4-2: final design method 2;

- **Method 3:** A number of different design alternatives are considered followed by the optimal construction method that belongs to it. This method is illustrated in figure 4-3;

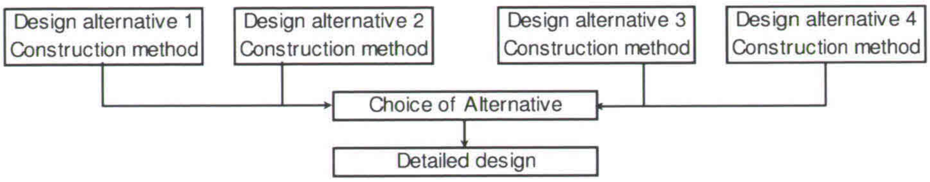


figure 4-3: final design method 3;

- **Method 4:** Essentially the same as method 3 but in this method the construction method is considered first and the design is based on that method. This method is illustrated in figure 4-4;

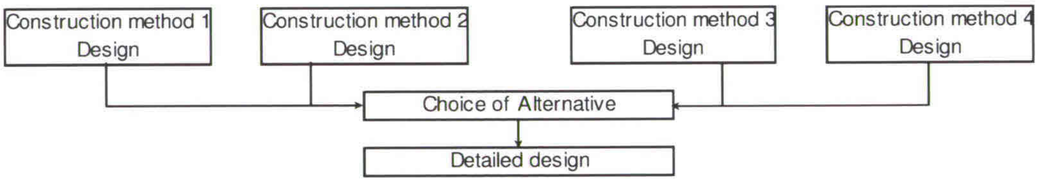


figure 4-4: final design method 4;

The final design of the New North Groyne is to such a degree dependant on the execution that it governs the entire design process. This is due to the limited possibilities for execution caused by the need to demolish the existing North Groyne and the demands on maintaining an operational entrance channel. It is therefore logical to choose method 4 where the focus lies on the construction method. An overview of the chosen design process is illustrated in figure 4-5.

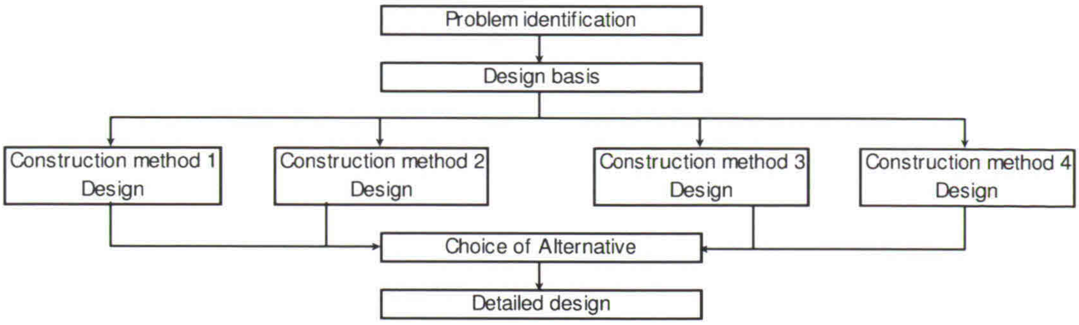


figure 4-5: the design process;

A specific formulation in flowchart form is given in appendix H.

4.5 Structural design

As discussed in chapter 2 the details of the structural design will be filled in during the phase of the *detailed design* and sometimes already the phase of the *final design*. Basically this means that each structural part should not fail or collapse with a probability, as followed from the functional design.

A structure fails when the load is larger than the strength, in other words if:

$$Z = strength - load < 0$$
$$Z = R - S < 0$$

In this equation R is the strength and S is the load. Usually R consists of a number of parameters (e.g. material properties) and S consists of a number of load values.

4.6 Probabilistic design

A project of this magnitude and in this environment demands an extensive probabilistic design. Due to the lack of sufficient data and chosen emphasis on the hydraulic design and the geotechnical analysis, it is decided to execute only a limited qualitative risk analysis. This analysis will be based on expectations of the occurring risks, derived from the available data.

4.7 Time schedule

For the planning of the report a time schedule has been used and is shown in table 4-2. The project encloses a period of eight weeks. After this period a definitive design and optimal phasing of the construction of the New North Groyne in the Durban Harbour Entrance Widening and Deepening project has been completed. The final report has been finalized in the Netherlands. During the project, the Durban harbour project site and the CSIR, the hydraulic laboratory in Stellenbosch, is visited.

Activity \ Week	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Introduction	■							
Problem Description	■							
Summary of Preliminary Design Report	■							
Choice of sub-project	■							
Problem analysis	■							
Problem definition	■							
Design Basis		■						
Marine and Geophysical aspects		■						
Requirements and assumptions		■						
New North Groyne detailed design		■	■	■	■			
Construction alternatives		■	■	■				
MCA construction method				■				
Detailed design Head			■	■	■			
Detailed design Trunk			■	■	■			
Failure systems						■	■	
Wave-structure interaction						■	■	
Slope stability						■	■	
Concluding remarks / Recommendations								■

table 4-2: time schedule;

5. SITE CONDITIONS

5.1 Introduction

This chapter provides the physical site conditions (bathymetry, hydraulic and geotechnical). From those conditions the assumptions and requirements follow. Those will be formulated in the next chapter.

The aim of this chapter is to give a description of the conditions at the site of the New North Groyne.

5.2 Bathymetric Data

The Bathymetric data for the harbour entrance is compiled using digitised SA Navy charts along with survey data obtained from a 1997 survey of the Vetch's Pier and Durban Bight area and the data obtained from the MGS 2003 survey of the existing entrance channel. The bathymetry in the project area varies from 0 m to -20.8 m below CD (Port), see box 1, and is strongly controlled by the presence of the existing North Groyne and Southern Breakwater, the harbour channels, "Flat Reef" and "Vetch's Pier". The bathymetry of the area immediately north of the groyne is characterised by shallow water depths and gently bathymetric gradients. "Flat Reef" is evident as a gently sloping ridge with 0.5 m to 1.0 m of relief, which starts near the base of the North Groyne and strikes at an acute angle to the beach to the limits of the survey block, see figure 5-1.

box 1: Level Datum and Coordinate System

The level datum throughout is Chart Datum (Port). The following conversion is required to convert this to Mean Sea Level (MSL):

MSL = CD (Port) + 0.90 m.

Chart Datum is located 13 mm below CD (Port).

The co-ordinate system is the Clarke 1880, LO31.

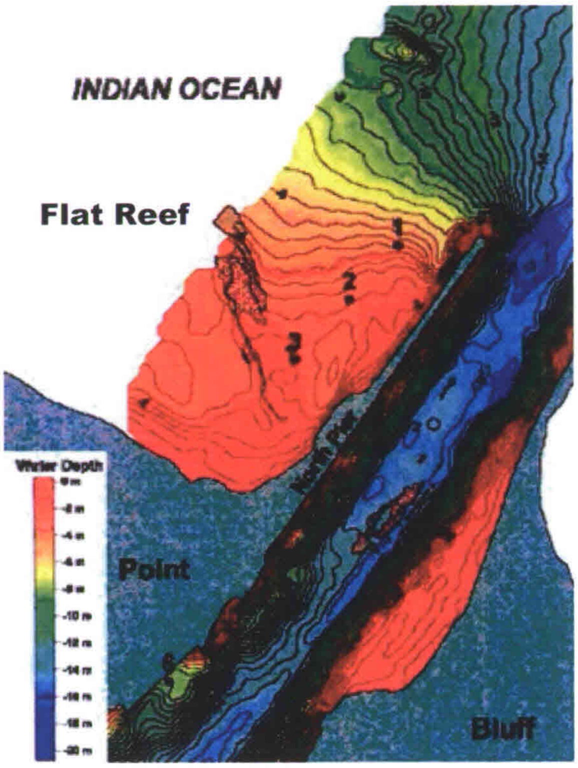


figure 5-1: bathymetry and vibrocore locations;

The NPA maintains approximate channel side slopes of 1:6 between the Bluff and Point and along the seaward extent of the channel down to 1:12 seawards of the groyne and breakwater. It should be noted that these slopes are based on a single survey that does not reflect the potential temporal variation as a result of sediment movement. Calculations have been made to determine the equilibrium slope; these can be found in appendices U and V.

5.2.1 Surficial seafloor geology

The seafloor geology is presented as a coloured interpretation in map and is illustrated in figure 5-2. The acoustic facies in the survey area include Holocene fine- to medium grade marine sand, late Pleistocene reef outcrops, scattered late Pleistocene reef outcrops, scattered or isolated exposures of rock/concrete debris and finally sub-aqueous extensions of the toe and slope of the North Groyne and Southern breakwater.

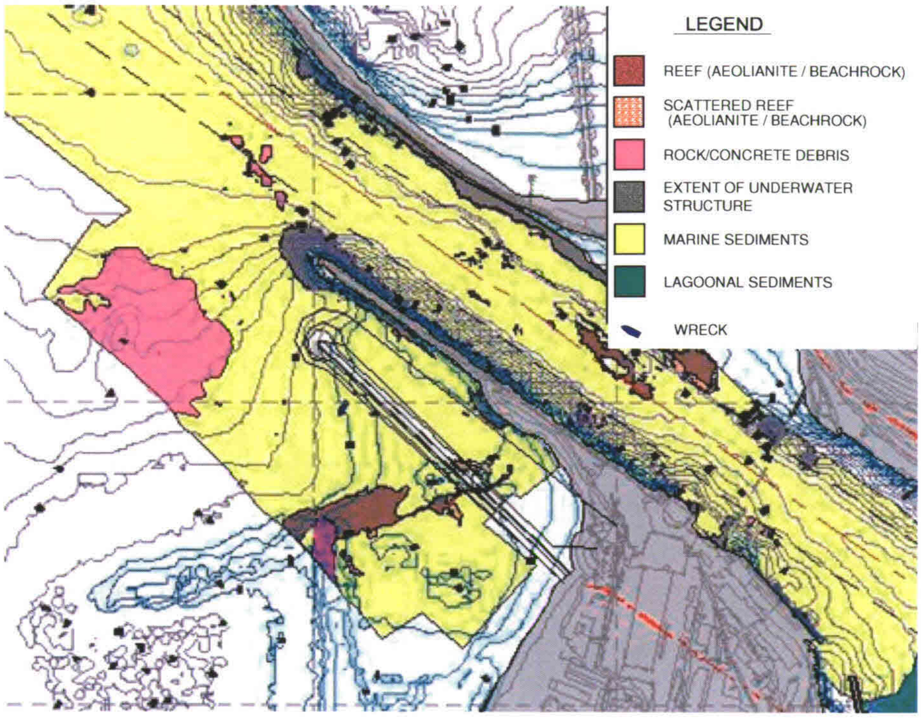


figure 5-2: seafloor geology;

The reef outcrops generally consist of late Pleistocene aeolianites (carbonate cemented dune sand) or beach rocks (carbonated cemented beach sand) that formed as coastal dunes or beach profiles during periods of low sea-level between 125,000 and 10,000 years ago.

Isolated examples of rock and concrete debris are set apart from the reef and scattered reef exposures as these objects "float" in unconsolidated marine sand and do not form part of a more laterally persistent rock outcrop or continue at depth to form part of a deeper-seated rock formation. Rock and concrete debris takes the form of concrete blocks, rubble, blocks of sandstone and Dyad tillage that were used in the construction of the original breakwaters that have moved, or has been dumped away from existing breakwater structures. The extent of the sub aqueous slopes of the existing North Groyne and Southern Breakwater have been assigned to a different acoustic facies as they are composite structures consisting of large blocks of concrete and smaller sandstone and boulders.

The unconsolidated sediment that constitutes about 90% of the survey area consists of Holocene fine- to medium grained sediment with variable calcium carbonate contents and small- to medium-scale sedimentary structures. The calcium carbonate content of the sediment is derived from skeletal detritus originating from the mechanical and biological breakdown of carbonate-secreting organisms such as shells, coral, barnacles and see-urchins.

The origin of the sedimentary structures is related to a combination of wave oscillation (small sedimentary structures) and tidal currents (medium scale sedimentary structures).

5.2.2 Seismic Survey

A maximum of 30 – 46 m of acoustic penetration in the substrate was achieved with the "Boomer" sub-bottom profiling equipment in the survey area. A number of unconformities were identified within this range and have been used to subdivide the sedimentary sequence into discrete seismic stratigraphic units. Unconformities are acoustically reflective horizons that represent erosional surfaces, or surfaces of nod-deposition that separate sedimentary units of different age and sedimentary character. The stratigraphic units represented below the Durban Harbour entrance channel include; Cretaceous marine sediments (1 million to 125 thousand years old), late Pleistocene lagoonal sediments (125 thousand to 18 thousand years old), late Pleistocene aeolianite and beachrock lithologies (125 to 18 thousand years old) and unconsolidated Holocene marine sediment (younger than 10 thousand years old). Two sections in the Durban Harbour Entrance illustrate the different sediments in combination with the depth, see figure 5-3.

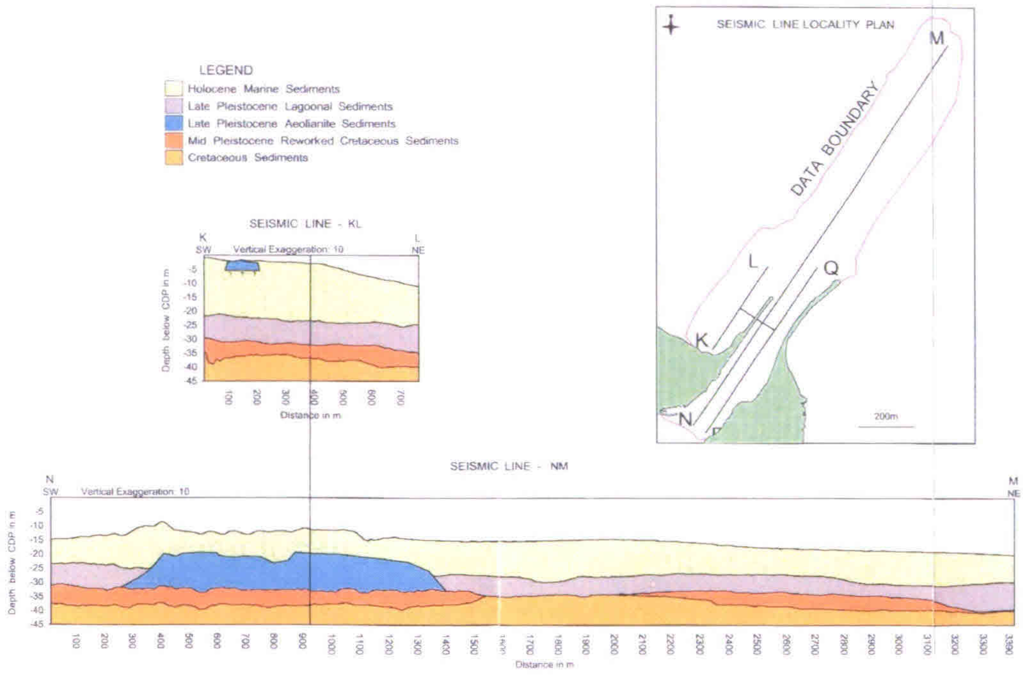


figure 5-3: sediment profiles;

The age of the stratigraphic units is based on local stratigraphic knowledge and published literature.

5.3 Sediment cores

A total of five sediment cores were collected in the Durban Harbour survey area. Three of the cores (cores 1, 2 and 3, see figure 5-1) were collected along the line of the proposed New North Groyne.

The sediments contained in all the cores are believed to be "undisturbed" and representative of the geology in the Durban Harbour area. The sediments in the three cores have been subdivided into two different sedimentary facies based on colour, grain-size statistics, calcium carbonate content, organic, carbon contents, dry density values and artificially introduced debris content. The cores and assumed cross-section of the North Groyne area is shown in figure 5-4.

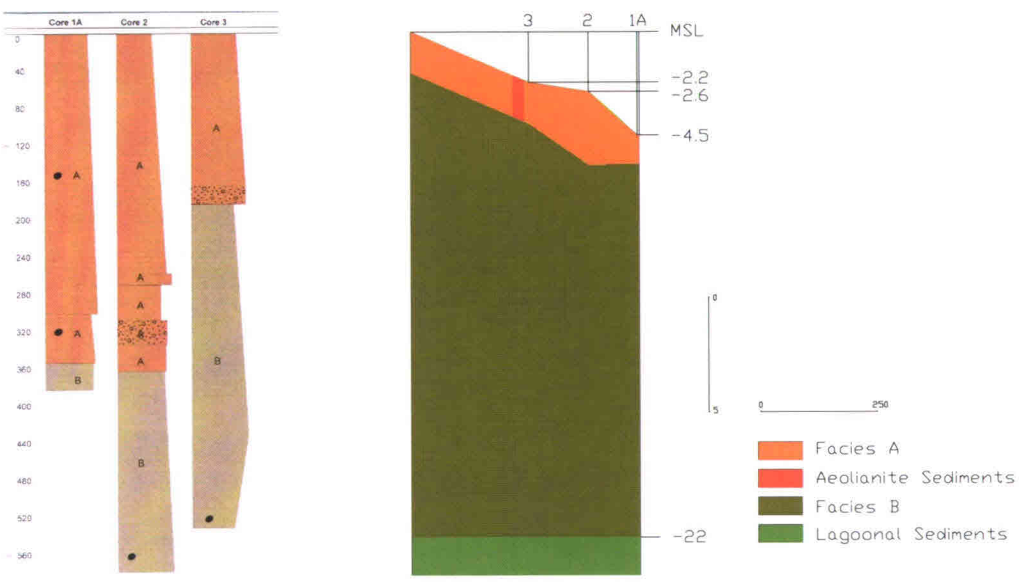


figure 5-4: vibrocores with labelled facies and ground profile assumption;

5.3.1 Facies A

Facies A consists of light reddish brown, fine to coarse grained, well sorted to moderately sorted, coarse skewed, free flowing sand with minor light grey mottling, few visible shell fragments, minor amounts of heavy minerals and low interstitial mud (<63 µm) content. The sediments of Facies A are characterised by high calcium carbonate contents and low- to moderate organic carbon contents. The sediments of this facies are generally normally grade (upward fining) to massive (structureless) sedimentary units vary in thickness from 1.85 m to 3.0 m and contain a scattered distribution of "out of context" rounded to well rounded beachrock/aeolianite pebbles (5-60 mm in diameter). A well developed, matrix supported pebble lag comprising rounded to well rounded pebbles of 5 – 65 mm in diameter is sometimes present at the base of Facies A sediments. The dry densities of Facies A sediments range from 1.34 to 1.85 tonne/m^3 and average 1.53 tonne/m^3. The sedimentary characteristics of Facies A sediments are summarise in table 5-1 below.

n=15	Mean mm	Sorting mm	Skewness φ	Gravel %	Mud %	Dry Density tonne/m^3	Org. C %	$CaCO_3$ %
Min.	0.204	0.47	-0.26	0.00	0.00	1.34	3.9	18.2
Max.	0.794	1.02	0.17	61.0	1.39	1.85	16.4	50.2
Ave.	0.43	0.76	-0.08	8.89	0.10	1.53	6.7	28.09
Colour	Light Reddish Brown							
Sedimentary Structures	Normally graded or massive sedimentary units; large out of context pebbles; basal enrichment in beachrock/aeolianite pebbles							
Sediment Maturity	Subangular to well rounded							

table 5-1: summary of the sedimentary characteristics of Facies A;

5.3.2 Facies B

Sediments of Facies B are characterised by light olive to dark olive, subangular to well rounded, near symmetrical to fine skewed, moderately sorted to well sorted, medium- to coarse grained, clean, free flowing sand with low gravel and interstitial mud contents, moderate calcium carbonate values and low organic carbon contents. The sediments of this facies occur as normally graded or massive sedimentary units that vary in thickness from greater than 0.43 m to less than 3.48 m, because of lack of further geotechnical research it is assumed that Facies B continues to a depth of – 22 m CD. The sediments have moderate to high concentrations of heavy minerals and moderate concentrations of visible shell fragments. Larger "out of context" rounded pebbles of beachrock and aeolianite (5-30 mm in diameter) are scattered throughout Facies B sediments but are less prolific than the "out of context" pebbles of Facies A sediments. The dry densities of Facies B sediments range from 1.51 to 1.73 tonne/m^3 and average 1.61 tonne/m^3. The sedimentary characteristics of Facies B sediments are summarised in table 5-2 on the next page.

n=13	Mean mm	Sorting mm	Skewness φ	Gravel %	Mud %	Dry Density tonne/m^3	Org. C %	$CaCO_3$ %
Min.	0.256	0.38	-0.16	0.00	0.00	1.51	3.7	13.2
Max.	0.836	0.92	0.12	4.0	1.71	1.73	9.1	20.5
Ave.	0.426	0.56	0.004	0.68	0.26	1.61	5.7	16.6
Colour	Light Olive							
Sedimentary Structures	Normally graded or massive sedimentary units; large out of context pebbles; basal enrichment in beachrock/aeolianite pebbles.							
Sediment Maturity	Subangular to well rounded							

table 5-2: summary of sedimentary characteristics of Facies B;

5.3.3 Differences

In general the sediments of Facies A and B are medium to coarse grained clean sands with low mud contents and occasional "out of context" pebbles scattered throughout the sedimentary succession. The differences between sediment samples of Facies A and B are mainly colour and a slightly elevated calcium carbonate content of Facies A. The most significant difference between Facies A and B sediments is however the compaction of these sediments below the seafloor. This is not evident from the core samples which are "disturbed", but rather from the amount of effort required to penetrate these sediments during the vibrocoring operation. Sediments of Facies A were significantly easier to penetrate in comparison to sediments of Facies B in that extended periods of vibrating of the vibrocore hammer were required to penetrate the sites where Facies B sediments were dominant and the fact that several core barrels could not be retrieved from these sediments once they had achieved full penetration. This variation in in situ compaction is related to the age of the sediments whereby the older sediments have had more time to achieve a better compaction, a process which has been aided through burial by the younger sediments. Facies A sediments are probably mid-Holocene to late Holocene (5,000 years and younger) in age while Facies B sediments are probably late Pleistocene to mid Holocene (125,000 to 5,000 years) in age. The measure of the in situ compaction of the sediments cannot be determined by laboratory analysis but can be evaluated by geotechnical equipment such as a cone penetrometer. The sediments of Facies A and B are predominantly free flowing sands with almost a complete lack of cohesive material or evidence of cementation and are not perceived to present any problems in terms of dredging activity. Difficult substrates such as the pebble lag deposits (core 2 and 3) are probably common in Facies A and B sediments and are not believed to be of appreciable thickness as to present a problem to dredging activities. The fact that the cores along the proposed route of the New North Groyne did not refuse on reef at shallow depths is indicative that "Flat Reef" does not extend offshore beneath unconsolidated sediments for any significant distance beyond the surface expression of this feature. The reef itself is probably composed of beachrock of late Pleistocene age (125,000 to 10,000 years old) and is not believed to extend to appreciable depths below the seafloor. Beachrock, by definition, is normally characterised by thin (< 2 m), laterally persistent (in a coast parallel direction) outcrops that "float" in unconsolidated sediments and do not have "deep roots" as is the case with aeolianite lithologies. Beachrock/aeolianite is very hard with UCS values that can exceed 40 MPa and will not be broken down easily. The fractured nature of beachrock which is normally broken up into discrete slabs by a series of coast normal and coast parallel fractures will however assist in breaking up the reef outcrops where this is required.

5.4 Determination of Soil Parameters

The known soil parameters as obtained from the vibrocoring are shown in table 5-1 and table 5-2. These parameters are sufficient to execute all planned calculations. Estimations of the other required parameters are obtained in this paragraph and are summarized in table 5-3.

Material Properties		Facies A	Facies B
Saturated density	γ_{sat} [kN/m^3]	18	19
Compression index	C [-]	200	300
Young's modulus	E [MPa]	10	30
Friction angle	φ [°]	31	35
Cohesion	c [kPa]	0	2.5
Dilatancy angle	ψ [°]	1	5
Porosity	n [-]	0.42	0.38
Permeability	k [m/s]	10^{-3}	10^{-4}
Poisson's ratio	ν [-]	0.3	0.3
Constrained modulus	E_{oed} [MPa]	14	40

table 5-3: material properties;

The saturated density is estimated by the comparing the dry density to representative dry density in table 1 of the Dutch NEN 6740 (Ref. I6). The Koppejan consolidation parameter or compression index C is defined as follows,

$$C = \frac{1}{\left(\frac{1}{C'_p} + \frac{1}{C'_s}\right)},$$

where,

C'_p is the primary compression modulus [-],

C'_s is the secular compression modulus [-].

This parameter is approximated using the NEN table. E is the Young's modulus, the elasticity modulus; approximated using the NEN table. With the known dry density of the two facies, the porosity and angle of internal friction are determined. The friction angle φ is depending on the mass density, grain angularity, and grading of the sand. For a first estimation of φ table 5-4 can be used.

Shape and grading	Loose	Dense
Rounded, uniform	φ=30°	φ=37°
Rounded, well graded	φ=34°	φ=40°

table 5-4: angle of internal friction, sand;

Finally with the help of figure 5-5 and table 5-4 an estimation of the angle of internal friction can be made.

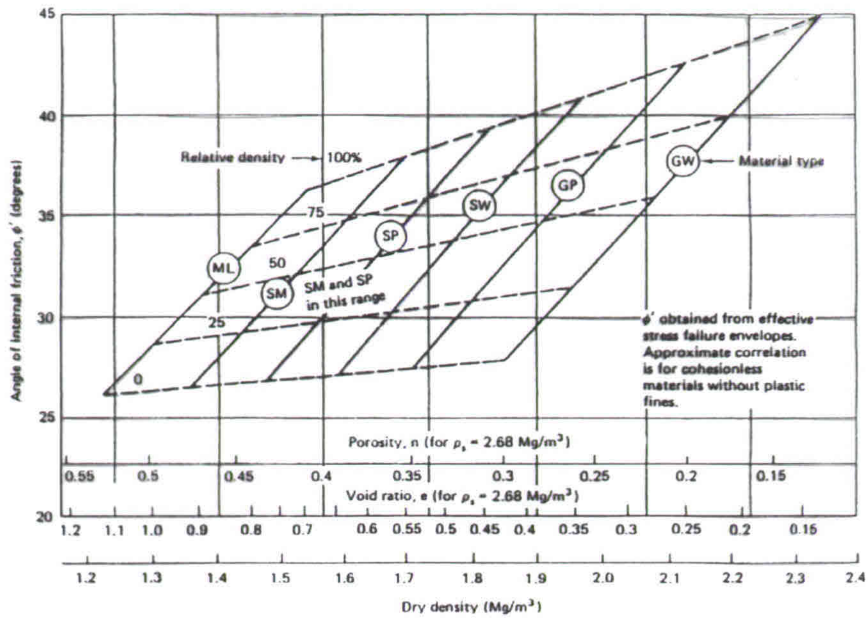


figure 5-5: parameter selection: strength parameters;

Both facies are well rounded and are uniformly graded. During vibrocoring it was observed that the packing of Facies A was much less dense than Facies B. The angle of internal friction of Facies A is estimated at 31 degrees, the friction angle of Facies B is estimated at 35 degrees. The cohesion c is zero for sand. In Facies B some cohesion is assumed caused by cementation due compression and $CaCO_3$ content. The dilatancy angle ψ is approximated using,

$$\psi = \varphi - 30^{\circ}.$$

The permeability coefficient k of sand is between 10^{-2} m/s and 10^{-6} m/s. Because Facies B is densely packed the value of 10^{-4} m/s is chosen. Facies A is loosely packed, a value of 10^{-3} m/s is chosen. The Poisson's ratio ν is usually 0.3 for sand. The constrained modulus E_{oed} [MPa] is defined as:

$$E_{oed} = \frac{(1-\nu)}{(1+\nu)(1-2\nu)} E.$$

5.5 Coastal Processes Data

5.5.1 Tide level

Tide levels are provided in the South African Navy tide tables (SAN 2004). The primary characteristics for the Port of Durban were extracted from this document and are presented in table 5-5.

Tide Parameter	Water Level (m. CD)
Highest water level (HAT)	2.3
Mean high water springs (MHWS)	2.01
Mean high water neap (MHWN)	1.36
Mean level (ML)	1.11
Mean low water neaps (MLWN)	0.87
Mean low water springs (MLWS)	0.21
Lowest water level (LAT)	0.00

CD= -0.913 MSL (where MSL is the official land levelling datum).

table 5-5: tide parameters for Durban Harbour;

For the maximum design sea levels (max design sea level = mean high water spring + calculated residual) see table 5-6 (Ref. P1).

Return Period (year)	Sea Level (m CD)
1	+2.42
5	+2.53
10	+2.57
50	+2.66
100	+2.70

table 5-6: maximum design sea levels;

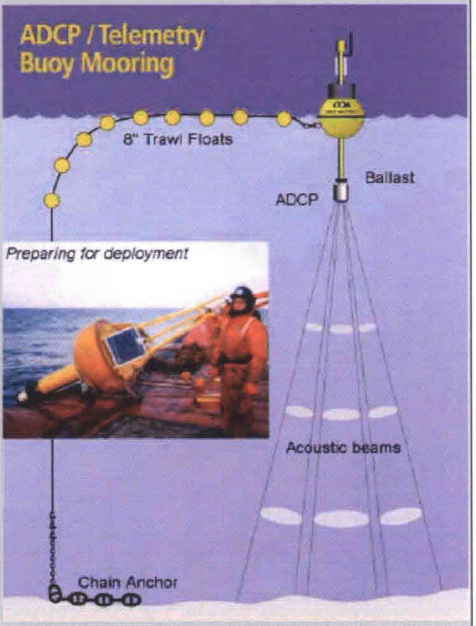
The long term sea level rise is estimated to be about 300 mm during the design horizon of 50 years.

5.5.2 Design Waves

Waves have been measured with an ADCP, see box 2, approximately 300 m off the tip of the Southern Breakwater as indicated in figure 5-6, from 25 January 2002 to 31 December 2003. Data coverage over this period was 87 % providing 1.7 years in total.

box 2: ADCP

The ADCP (Acoustic Doppler Current Profiler) uses four acoustic beams, pointing in different directions to sense different velocity components of the water column above the instrument (RDI, 1996).
For example, if the ADCP points one beam at an easterly angle and another at a northerly angle, it will measure east and north current components. One acoustic beam is required for each current component. Therefore, to measure three velocity components (i.e. east, north, and the vertical) there must be at least three acoustic beams. The built-in compass and known deviation between magnetic north and true north at the site, allow the current components to be related to true north (TN).



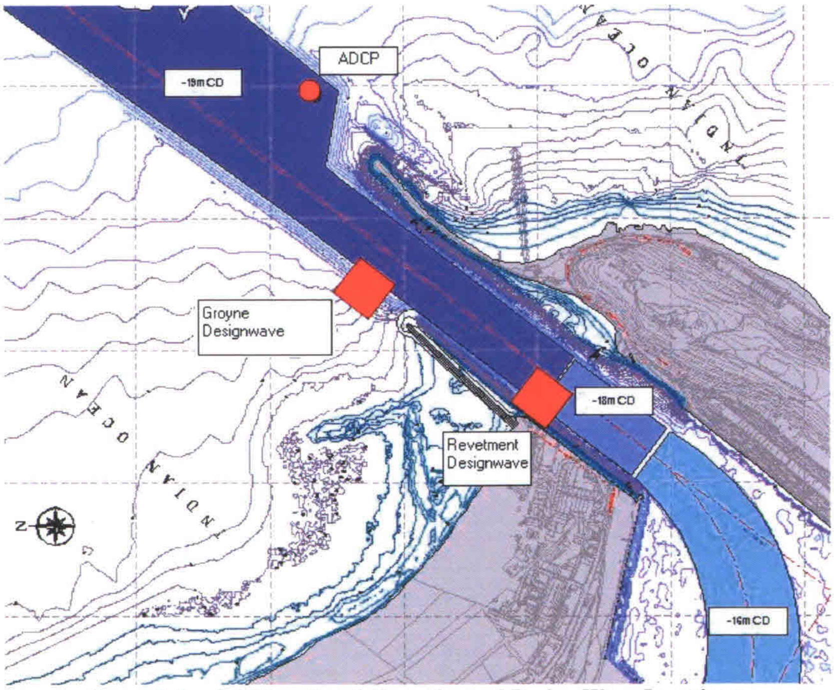


figure 5-6: ADCP location & Transformed Design Wave Locations;

Design waves were established by fitting Weibull distributions to the data measured with the ADCP. The directional design wave heights for this location are summarized in table 5-7.

Return Period [years]	Significant Wave Height [m]							
	Direction Sector [degrees]							
	22.5	45	67.5	90	112.5	135	157.5	180
1	2.4	2.9	3.2	3.8	3.8	4.0	3.4	2.2
5	2.7	3.2	3.5	4.7	4.8	5.4	4.7	2.4
10	2.8	3.2	3.6	5.1	5.3	6.0	5.2	2.5
20	2.9	3.3	3.7	5.6	5.7	6.7	5.8	2.6
50	3.1	3.4	3.8	6.1	6.4	7.6	6.7	2.8
100	3.2	3.5	3.9	6.6	6.8	8.3	7.3	2.8

table 5-7: directional design wave heights;

The peak wave period ranged between 5 s and 15 s, but was predominantly between 8 s and 10 s. The east appears to be the dominant direction, particularly during summer and autumn. The significant height of the long waves varied between 0.03 m and 0.15 m. A clear relationship was noted between the swell and long waves. A plot of the long wave spectrum showed a number of distinct concentrations of long wave energy, focusing around 40 s, 55 s, 80 s, and 100 s.

Using a numerical wave propagation model, waves at the ADCP location were transformed to a location approximately 150 m off the North Groyne head, see figure 5-6. For the estimation of the design waves the three-parameter Weibull distribution was utilized. The distribution is provided below.

$$H_s = \alpha + \beta \frac{1}{-\ln(1-p)^{-\frac{1}{k}}},$$

where:

α = location parameter [m],

β = scale parameter [m],

k = shape parameter [-],

p = probability non-exceedance [-].

In order to calculate the design waves for the New North Groyne and the North Bank Revetment the recorded data had to be transformed to the areas of interest. Numerical model

(MIKE21 BW) transformation coefficients were used to transform the data to the selected areas. For each data set independent storm events above a specific threshold limit were selected for the design wave estimate. The estimated design wave heights and the parameters defining the fitted distribution are provided in table 5-8 and table 5-9. Wave direction is not an important design parameter at this location and therefore a single omni-directional design wave was determined.

Return Interval	Southern Breakwater [m]	North Groyne [m]	North Bank Revetment [m]
1	4.1	2.5	1.07
5	4.8	2.9	1.15
10	5.2	3.0	1.19
20	5.5	3.2	1.22
50	5.9	3.3	1.26
100	6.3	3.5	1.29

table 5-8: single omni directional design waves;

Parameters	Southern Breakwater	North Groyne	North Bank Revetment
Shape Factor (k)	1.0	1.1	1.4
Alpha [m]	2.1	1.6	0.8
Beta [m]	0.5	0.3	0.1
Correlation (r^2)	0.99	0.99	0.99
Storms (N)	97	76	32
Years	1.7	1.7	1.7
Storm/year (N)	58	45	19
Threshold [m]	2	1.6	0.85

table 5-9: single omni directional design waves, parameters;

This table shows a 100 years return period wave height for the North Groyne of only 3.5 m. Wave heights are further reduced as waves penetrate the harbour. Design waves were therefore also determined in the entrance channel approximately in line with the root of the New North Groyne see figure 5-6. These wave heights are relevant for the design of a revetment along the remainder of the North Bank Channel. A 100 year return significant wave height of 1.3 m was determined for this location.

5.5.3 Currents

Currents were measured with the ADCP (Ref. P1). This was carried out with the same instrument used for wave measurements. Appendix I shows the current roses for various depths at the ADCP location.

The currents were found to remain relatively uniform through the water column, having a median speed of 0.2 m/s. The 1 % exceedance values range from approximately 0.60 m/s at a depth of 10 m to 0.73 m/s near the surface. Hydrodynamic current modelling was not conducted as part of the preliminary design process but should be considered for evaluating dredge slope stability in the final design.

The directional behaviour of the currents remains similar throughout the water column. The dominant directional sectors are NNE to ENE and SSE to S. This refers to the direction into which the current is flowing. Currents closer to the surface show more directional variability, with a distribution similar to the wind directional distribution. Near surface currents are significantly affected by winds. Sub-surface currents are less affected by winds, probably being driven by tidal and larger scale forcing mechanisms. A more detailed investigation in the tidal design might provide sufficient confidence to optimize and decrease the extent of proposed scour protection.

5.6 Meteorological Data

Wind data is available from CSIR and information on visibility, rainfall, temperatures and humidity is available from the South Africa Weather Bureau (Ref. P1).

5.6.1 Winds

The CSIR has collected wind data at the Bluff since 1995. It comprises 20 minute averaged values from October 1995 to May 2002. Wind speeds ranged from calm conditions to a maximum velocity of 32 m/s. The median speed (exceeded 50 % of the time) was 6.2 m/s, with a wind speed of 17.2 m/s being exceeded 1 % of the time.

The dominant wind directional sectors were N to NE and SSW to WSW. This corresponds approximately to the coastline orientation. Wind direction refers to the direction from which the wind is blowing.

5.6.2 Visibility/Fog

On average, fog is experienced one day every month in the period from March to December.

5.6.3 Rainfall

The average annual rainfall for Durban is 1,009 mm. The highest rainfall occurs during the summer months, between October and March. The average monthly rainfall is 67 mm, with the highest 24-hour rainfall on record being 197 mm. On average, 130 days a year have at least 1 mm of rain.

5.6.4 Temperatures

Durban has a sub-tropical climate. The hottest months are between December to March, when average temperatures range between 23° C and 25° C. The average high temperature during this period varies between 27° C and 28° C. The highest temperature on record is 40° C. On average, there are four days a year on which a temperature higher than 32° C is recorded.

5.6.5 Relative Humidity

The average morning relative humidity is 83 %. This is highest between December and March, when it averages between 86 % and 87 %.

5.7 Channel Design

5.7.1 Design vessel

The design vessel for the harbour entrance is the 6600 TEU Post Panamax container vessel. The significant characteristics of this ship are provided in table 5-10.

Characteristic	Value
TEU's	6600
DWT	80 000 t
Displacement	100 000 t
Length Overall (Loa)	347 m
Length between perpendiculars (Lbp)	333.5 m
Beam (B)	42.8 m
Loaded (Design) Draft (D)	14.0 m

table 5-10: significant characteristics of the 6600 TEU post Panamax container vessel;

5.7.2 Entrance Channel width

The entrance channel should have a minimum bottom width of 220 meters, see table 5-13. This is considered more conservative than the PIANC (1997) guideline and has been validated by the vessel simulation study (Ref. I8).

5.7.3 Channel Depth

Acceptable conditions in the harbour basin are determined by safety requirements for ship handling. Tugs will not be able to control the design vessel in winds exceeding 20 m/s and wave heights exceeding 1.5 m (Ref. P1). Safety criteria for the design ship will limit the access for loaded vessels to wave, wind and tidal conditions where a nominal under keel clearance of 0.6 m and maximum swept lane width of less than 70 % of the available channel is maintained at all times, see table 5-11. The approach channel depth is determined by accounting for the factors given in table 5-12.

Parameter	Depth [m]	
	PIANC	PORT
Loaded Draft of the Vessel	14	14
Allowance for Trim	0.3	0.3
Allowance for Sounding error	0.1	0.2
Allowance for Siltation	0.2	0.3
Nominal Underkeel Clearance	0.6	0.5
Allowance for Tide (CD)	0	0
Allowance for Vessel Motion	0.8	0.7
Total Depth Requirement	16	16

table 5-11: depth of basin channel;

Parameter	Inner depth [m] CD	Outer depth [m] CD
Loaded Draft of the Vessel	14	14
Allowance for Sounding Errors	0.2	0.2
Allowance for Siltation	0.3	0.5
Nominal Underkeel Clearance	0.6	0.6
Allowance for Tide (MLWS)	0	0
Allowance for Vessel Motion including Squat	2.9*	3.7*
Total Depth Requirement	18.0	19.0
*PRDW(2001)		

table 5-12: approach channel depth;

5.7.4 Summarised channel alignment

The existing channel alignment is shown in appendix C. The proposed preliminary alignment is illustrated in appendix D (it was selected as being the most favourable alignment in Ref. P1). The channel characteristics have been based on the PIANC guidelines and BS 6349 Part 5, they are summarised below in table 5-13.

Channel bottom width:		
Straight section	220 m	PIANC: width ≥ 5 x ship beam
Bend	270 m	PIANC: width ≥ 6 x ship beam
Channel depth:		
Protected from waves	-16 m CD	
Exposed to waves	-18 m CD	
Outer channel	-19 m CD	
Channel alignment:		
Bend radius	1500 m	PIANC: minimum radius = 1500 m
Bed slope stability:		
Protected areas	1:3	BS 6349 Part 5: 1:3 ≤ Slope ≤ 1:4
Exposed areas	1:12	BS 6349 Part 5: 1:6 ≤ Slope ≤ 1:12.
Excavated in rock	Nearly vertical	BS 6349 Part 5: Nearly vertical
Revetment slopes	1:1.5	

table 5-13: channel characteristics;

The side slopes selected for the channel conform to those currently used in the port. These have been found to be stable.

5.8 Sediment transport

The sediment transport along the Durban coast consists of an average 600,000 m^3/year flow of sand from the south to the north. To prevent the sediment from clogging the entrance channel a sand trap is located at the Bluff, south of the Southern Breakwater. In the existing situation a dredging ship, the Piper, pumps the sand aboard and unloads it at a jetty at the Point quay, which is connected to a distribution system, figure 5-7 shows the dredging ship and the existing distribution system.

figure 5-7: dredging ship the Piper and distribution centre;

In the proposed situation a fixed sand bypass system will be constructed. The resulting discharge of 500,000 m^3/year onto the Durban beaches south of the harbour entrance, relative to the present discharge of 260,000 m^3/year (average for the period 1996-2002), will result in accretion of the beach. An analysis is conducted by the CSIR to assess seasonal variability in transport conditions. This analysis is conducted with measured ADCP data (1 July 2002 to 30 June 2003). The percentage variation in the seasonal net transport rate, relative to the annual rate is, indicated in table 5-14.

Season	Net transport rate relative to annual [%]
Winter	+6.6
Spring	-11.8
Summer	-7.3
Autumn	+12.6

table 5-14: seasonal variation in sediment transport;

It is known that the direction of sediment transport can occasionally change from the north direction to the south direction. At the moment it is unknown when this occurs. Assumed is, based on table 5-14, that sediment transport from north to south occurs mainly in the spring and summer.

5.8.1 Entrance channel

The primary concern is whether, under the changed wave conditions resulting from the harbour widening, the New North Groyne will be sufficiently long to prevent transport of sand from the beach/surf zone into the harbour entrance channel. It is anticipated that most significant transport would be driven by wave action occurring under a range of conditions, particularly storm events. PRDW commissioned the CSIR to conduct mathematical modelling. The study was completed in November 2003 (Ref. I9). The study area is defined by the nearshore from which sand could conceivably be transported toward the harbour channel, i.e. the Vetch's Bight region.

Employing the SBEACH cross-shore model in tandem with closure depth formulations allowed an assessment of whether or not a significant quantity of sediment will be transported seaward of the end of the North Groyne, from where it could easily be transported into the harbour channel.

An analytical method for the calculation of the annual depth of closure is presented by Hallermeier (1978, 1981). This relation also holds true for longer time periods (Stive *et al*, 1992). Hallermeier's method has been found to be robust, with good correlation to the closure depth measured for erosive wave conditions (Nicholls *et al*, 1998), although it is generally regarded as giving a conservative (i.e. deeper) estimate of the depth of closure compared to that shown by measurements. Hallermeier's relation is given by the following formula:

$$d_1 = 2.28H_s - 68.5\left(\frac{H_s^2}{gT_s^2}\right),$$

where,

d_1	Closure depth for the time period considered, referenced to mean low water	[m],
H_s	Significant wave height exceeded for 12 hours during the time considered	[m],
T_s	Associated wave period	[s],
g	Acceleration due to gravity	[m/s^2].

The non-breaking significant nearshore wave height is required as input to the formula. In this study, the ten-year hindcast offshore dataset was used, as it covers a long enough period to include variability in conditions. The ten-year hindcast offshore dataset was transformed to the nearshore ADCP position. In order to make allowance for the sheltering effects experienced at the area of interest, these data were further transformed to the end of the North Groyne using the refraction coefficients determined from the refraction study.

Using the annual wave heights transformed to 18 m water depth, the depths of closure for the end of the North Groyne were calculated, using the Hallermeier's relation. The annual depth of closure varies between 3.6 m and 4.4 m with an average of 4.2 m. As the wave data set covers a period of ten years, the ten-year depth of closure will be 5.2 m. The seabed depth at the end of the New North Groyne (-5 m CD) is comparable with the predicted annual maximum transport depths (-4.2 m CD) adjacent to the groyne using Hallermeier depth of closure approach. The depth of the seabed at the end of the New North Groyne (-5 m CD) is fractionally less than the predicted maximum depth of transport (-5.2 m CD). However conditions conducive to transport at this depth are estimated to be rare (less than 0.1 % of the time). Therefore no meaningful deposition of sediment into the future harbour channel is anticipated.

After construction of the New North Groyne and dredging of the entrance channel, readjustment of the nearshore profiles adjacent to the groyne can be expected, resulting in some slumping of material into the entrance channel. Once the profiles have adjusted to a new equilibrium no further significant movement of material into the entrance channel is to be expected. This slumped material will need to be dredged initially; thereafter minimal maintenance dredging of the entrance channel as a result of material ingress from the north should be required.

5.8.2 Durban beaches

The configuration of the New North Groyne as well as the deepening of the entrance channel will have possible consequences on nearshore wave conditions, on sediment transport and thus on the Durban beach stability. The distribution of sand is vital for the maintenance of the Durban recreational beaches north of harbour entrance, see figure 5-8. Without constant sediment supply the beaches will erode which will have an unwanted impact on the Durban tourist industry. The beach plan shape can be controlled and manipulated to that desired through varying the sand bypass operating strategy. The CSIR research concludes that the New North Groyne position and length does not affect the ability of the beach plan shape to be manipulated by varying the operating strategy.


figure 5-8: Durban beaches;

5.9 Safety factors

5.9.1 Partial material factors

Due to high uncertainty in the determination of the material parameters a partial safety coefficient for each parameter has to be taken into account. The coefficients are defined in table 5-15.

<table>
<tr><th rowspan="4">Type of geotechnical construction</th><th rowspan="4" colspan="2">Partial material factor (γ_m)</th><th colspan="5">Limit states</th></tr>
<tr><th colspan="4">Ultimate</th><th>Serviceability</th></tr>
<tr><th>1A</th><th>1B</th><th>1A</th><th>1B</th><th></th></tr>
<tr><th colspan="2">positive</th><th colspan="2">negative</th><th></th></tr>
<tr><td>All geotechincal constructions</td><td>$\gamma_{m,g}$</td><td>volumetric weight of the soil</td><td>1.1</td><td>1.1</td><td>1</td><td>1</td><td>1</td></tr>
<tr><td colspan="8">Foundations of constructions</td></tr>
<tr><td>Raft/strip foundation</td><td>$\gamma_{m,\phi'}$</td><td>tangent of internal friction angle</td><td>1.15</td><td>1.15</td><td>1</td><td>1</td><td>1</td></tr>
<tr><td></td><td>$\gamma_{m,fundr}$</td><td>undrained shear strength</td><td>1.35</td><td>1.35</td><td>1</td><td>1</td><td>1</td></tr>
</table>

table 5-15: material safety coefficients;

5.9.2 Slope stability factors

In this design the following slope stability safety factors are used:

- During construction 1.2;
- During operation 1.5.

6. THE DESIGN BASIS

6.1 Introduction

In this chapter the boundary conditions, assumptions and requirements are given based on the previous chapter.

6.2 Boundary conditions

6.2.1 Technical boundary conditions

TB 1: The design vessel for the harbour entrance is the 6600 TEU post Panamax container vessel;

TB 2: The New North Groyne will have to fit in the design of the new channel alignment;

TB 3: The bathymetry of the area will have to be taken into account during the design;

TB 4: The geology of the seafloor will have to be taken into account during the design;

TB 5: The stratigraphy below the New North Groyne will have to be taken into account during the design;

TB 6: The material properties of the subsoil will have to be taken into account during the design;

TB 7: The tide levels provided in the South African Navy tide tables (Ref. I7) will have to be taken into account during the design;

TB 8: The 100 year return significant wave height will have to be taken into account during the design;

TB 9: Tidal currents will have to be taken into account for the design of the New North Groyne;

TB 10: The meteorological data will have to be taken into account during the design;

TB 11: The sediment transport along the Durban coast will have to be taken into account during the design;

TB 12: Partial material safety factors will be taken into account during the design;

TB 13: Safety factors will have to be taken into account for the stability analyses of slopes;

TB 14: The lifetime of the construction will have to be taken into account;

TB 15: A design storm will have to be taken into account;

TB 16: The accessibility of the New North Groyne will have to be taken into account during the design;

TB 17: An economical slope of the armour layer will be used during the design;

TB 18 The NPA prefers the use of Dolos units as armour material;

TB 19: The cap blocks in the middle of the New North Groyne will have to be of a sufficient thickness;

TB 20: For good interlocking and stable positioning of the dolosse the roundhead will have to have a sufficiently large radius.

6.2.2 Functional boundary conditions

FB 1: The function of the Southern Breakwater and North Groyne is to protect the harbour entrance against wave attack and to prevent siltation of the channel;

FB 2: The level of operational availability for safe navigation will have to be taken into account during the design;

FB 3: The effects of the harbour entrance construction process on the current shipping operations in the port will have to be taken into account during the design

FB 4: The road traffic impact will have to be taken into account during the design;

FB 5: The interface with Durban Point Development Company will have to be taken into account;

FB 6: The width and the height of the cap of the New North Groyne will have to be sufficient to accommodate safely a maintenance truck;

6.2.3 Social, cultural and historical boundary conditions

SB 1: The Durban beach close to the harbour entrance will have to be taken into account.

6.2.4 Environmental boundary conditions

EB 1: The reuse of the existing groyne materials and the use of dredged material will have to be taken into account during the design;

EB 2: Disposal of surplus material will have to be in accordance with the Dumping at Sea Act (1980);

EB 3: Contaminated sediments will have to be taken into account during the design;

6.3 Assumptions

A 1: Dutch legislation will suffice if there is a lack of information on the South African legislation;

A 2: The unknown soil parameters are assumed and can be found in table 5-3;

A 3: Facies B continues to a depth of –22 m CD;

A 4: The layer composition of the subsoil is given in figure 5-4;

A 5: Sediment transport from north to south, based on table 5-14, is assumed to occur mainly in the spring and summer;

A 6: It is assumed that if the existing North Groyne is maintained on a level of +1 m CD in the surf zone, the groyne can keep its sediment retaining function;

A 7: The end part of the existing North Groyne that is not located in the surf zone can be removed to a level of -4 m CD;

A 8: The material that will be used to construct the New North Groyne core, can be removed (land based) by using backhoes and bulldozers;

A 9: The Point area offers enough space to sort and temporarily store the removed material;

A 10: The waterborne removal of the core construction of the existing North Groyne can be done by a hydraulic backhoe dredger until the foundation level is reached;

A 11: The dredging of the existing North Groyne area, from foundation level to the desired channel depth of – 19 m CD, can be done by using a trailer suction dredger;

A 12: The total length of the North Groyne is approximately 550 meter. The groyne can be divided into two parts:

- The first part between 0 and 230 meter has an approximated width of 20 meter;
- The second part of the groyne, between 230 and 550 meter, has a width of 8 meter, see appendix N.

A top level of the existing North Groyne of + 4.0 m CD is assumed. The slope of the foundation is assumed to be 1:2.

A 13: Material of the existing North Groyne can be used as core material for the New North Groyne;

A 14: The stable dredged slope in unprotected waters is 1:10;

A 15: For the design of the New North Groyne a general package density of ϕ = 0.90 for the breakwater head is assumed to be safe;

A 16: In the New North Groyne design it is assumed that a 10 m extension of the sea gravel and core layer filter is sufficient to protect the head from scouring;

A 17: For the trunk a H_s of 2.5 m is assumed;

A 18: It is assumed that a ship navigates minimal 60 metres from the bank of the New North Groyne;

A 19: In the slope stability analyses it is assumed that the groyne itself is a stable structure and will not experience large deformations;

A 20: The Durban area is not a seismic active region;

A 21: The dredged sand from the Southern Breakwater area has approximately the same properties as the sand in the New North Groyne area.

6.4 List of requirements

6.4.1 Technical requirements

TR 1: The design vessel for the harbour entrance is the 6600 TEU Post Panamax container vessel. The significant characteristics of this ship are provided in table 5-10;

TR 2: The New North Groyne has to fit the design of the new channel alignment. The channel characteristics have been based on the PIANC guidelines and BS 6349 Part 5. They are summarised in table 5-13. The proposed channel layout can be found appendix D;

TR 3: The bathymetry of the area immediately north of the groyne is characterised by shallow water depths and gently bathymetric gradients, see figure 5-1. The NPA maintains approximate channel side slopes of 1:6 between the North Bank Revetment and the channel and between the New North Groyne and the channel and 1:12 seawards of the New North Groyne;

TR 4: The seafloor geology is presented as a coloured interpretation in map and is illustrated in figure 5-2;

TR 5: The stratigraphic units represented below the Durban Harbour entrance channel include: Cretaceous marine sediments, late Pleistocene lagoonal sediments, late Pleistocene aeolianite and beachrock lithologies and unconsolidated Holocene marine sediment. Two sections in the Durban Harbour Entrance illustrate the different sediments in combination with the depth, see figure 5-3;

TR 6: The compositions of the three relevant vibrocores are shown in figure 5-4. The properties of the two relevant Facies A and B are shown in table 5-1 and table 5-2;

TR 7: The primary characteristics for the port of Durban were extracted from the South African Navy tide tables (SAN 2004) and are presented in table 5-5;

TR 8: The estimated design wave height and the parameters defining the fitted Weibull distribution are provided in table 5-8 and table 5-9;

TR 9: The currents remain relatively uniform through the water column, having a median speed of 0.2 m/s. The 1 % exceedance values range from approximately 0.60 m/s at a depth of 10 m to 0.73 m/s near the surface, see appendix I: Current roses;

TR 10: The meteorological data includes wind, visibility/fog, rainfall, temperatures and relative humidity data and can be found in paragraph 5.6;

TR 11: The sediment transport along the Durban coast consists of an average 600,000 m^3/year flow of sand from south to north. The ingress of sediment into the entrance channel from the north must be minimized;

TR 12: The partial safety coefficients for geotechnical constructions are defined in table 5-15;

TR 13: The safety factors for the stability analyses of the slopes can be found in paragraph 5.9.2;

TR 14: The lifetime of the New North Groyne has to be at least 50 years;

TR 15: The New North Groyne and Revetment will have to withstand a design storm with a recurrence period of 1:100 years;

TR 16: The accessibility of the New North Groyne has to be the same as that of the existing North Groyne;

TR 17: An economical slope for the armour layer is 1:1.5;

TR 18: Dolos units must be used for the construction of the New North Groyne;

TR 19: The cap blocks in the middle of the New North Groyne must be 1.0 meter thick;

TR 20: The radius of the 11 t dolosse for the roundhead must be 10 meters.

6.4.2 Functional requirements

FR 1: The New North Groyne must keep its function of protecting the harbour entrance against wave attack and preventing siltation of the channel;

FR 2: The new harbour entrance must provide a high level of operational availability for safe navigation;

FR 3: The harbour entrance construction process does not adversely affect the current shipping operations in the port;

FR 4: The road traffic impact in the Durban city centre must be minimized;

FR 5: The design of the landside infrastructure takes into account the Masterplan development, proposed transportation corridors and storage areas. All essential infrastructure and buildings will have to be protected during the construction phase;

FR 6: The width of the cap of the New North Groyne must be 8 m and the top of the core has to be at +3.0 m CD.

6.4.3 Social, cultural and historical requirements

SR 1: The Durban tourist beaches, including those outside the Point Area, are highly important to the Durban tourist industry and may not be damaged.

6.4.4 Environmental requirements

ER 1: Optimal economical recycling of existing groyne materials and dredged materials must be executed;

ER 2: The unusable dredged material will have to be dumped offshore in two designated areas, which are located a few miles outside the coast;

ER 3: Contaminated sediments must be treated to prevent damage to the environment.

7. CONSTRUCTION METHOD

7.1 Introduction

In this chapter different construction alternatives will be discussed. The most efficient construction approach is likely to be one where the dredging, removal of the existing North Groyne and construction of the New North Groyne are closely linked. First the existing situation will be discussed. Then five alternative construction methods will be described. The alternatives will be evaluated using a Multi Criteria Analysis and 3 alternatives will be chosen for detailed design in the next chapter.

The aim of this chapter will be to generate different alternatives for the economic construction of the New North Groyne with minimal hindrance to shipping, minimal beach erosion and maximal recycling of materials.

7.2 Existing situation

7.2.1 History

This paragraph contains a short history of the Durban Harbour Entrance. The focus will be on the different constructions in the area of the existing North Groyne and the New North Groyne. In the turbulent history of Durban a lot of attempts have been made to construct a safe entrance to the Durban lagoon. During removal of the existing North Groyne and construction of the New North Groyne, old constructions or contaminated soils can cause problems. Knowledge of the location and specifics of these possible problems can reduce the risk of unexpected extra work or damage to equipment.

Port of Durban

The lagoon that currently holds the port of Durban was discovered in 1497 by Vasco da Gama. The Bluff is the only landmark in hundreds of miles in any direction. Combined with the protected lagoon this seemed an excellent location for a harbour and a colony. In the 1840's the emigration to Durban really took off, various ships with emigrants arrived in the following years and the colony grew.

Sandbar

The larger ships now arriving exposed the problem of the sandbar in the lagoon entrance. This sandbar was in a state of relative stability, refuelled by the flood tide deposition of sediment and eroded by the rivers flowing into the lagoon and the ebb tide. The transport of sediment along the Durban coast is generally in northern direction but may change to southern under certain conditions. The average depth of the bar at low tide was approximately 6 feet (2 m) and at high tide about 12 feet (4 m). Even the high water level was not enough to give enough clearance over the bar. The treacherous waters around the harbour entrance caused many ships to run aground and while most of the time passengers could be saved, the loss of property caused problems to the starting colony. For save unloading, passengers and goods from arriving ships had to be loaded in smaller boats to be carried over the bar. Due to lack of docking facilities the passengers had then to wade through the waters of the shallow lagoon to reach the shore (Ref. I1).

Milne

This was the situation when Durban's first of many harbour engineers arrived in 1849. John Milne first made a lengthy analysis of the properties of the lagoon and the harbour entrance. He decided that the only way to remove the bar was employ the power of tidal scour. He reasoned that the ebb tide, enlarged by the inflow of rivers into the lagoon, could be squeezed between two piers and the subsequent scour would remove the sandbar. Although this method of removing sediment had worked in other harbours, Milne did not realize that the inflow caused by the rivers was not large enough to really solve the problem. Milne got to work and made good progress with the north pier using rocks quarried in the Bluff and transported to the lagoon using the first South-African railroad. Differences of opinion between Milne and harbour board members caused his resignation in 1858 after which another Harbour Engineer was appointed.

Vetch

After much deliberation in Durban the problem of the harbour entrance was delegated to London where Captain James Vetch, a Royal Engineer attached to the Harbour department of the Admiralty, was appointed to make a design. Vetch did not bother to visit Durban and proposed to build a great basin in front of the bar. Not having visited the site he had come to the erroneous conclusion that the seas where fairly gentle and a timber and rubble structure would absorb any wave forces likely to be encountered. Vetch's grand scheme, see figure 7-2, was accepted and construction tendered in London. Under Engineer in Chief James Abernethy, contractor Thomas Jackson started to build in 1861. At first the work progressed at a phenomenal rate and the citizens of Durban were very impressed. During the Durban stormy summers though became painfully clear that the structural design was not robust enough and the heavy waves would soon demolish the fragile breakwaters. This problem combined with much higher material costs than expected caused the contractor to bankrupt and Durban was left with slowly deteriorating breakwaters.

Coode

After the debacle of Vetch and Abernethy's schemes the eminent London engineer Sir John Coode was appointed to make a proposal. Busy in London himself he sent one of his assistants to Durban to take a look at the harbour entrance. The assistant spoke to locals and the current harbour engineer Peter Paterson. Based on their theories he got the faulty assumption that the harbour entrance was under laid by a solid rock reef at a depth of 24 feet. This report reached Coode in 1870 and his first proposal was essentially the same as Milne's old plan. In 1877 Coode visited Durban and by then surveys had placed the non-existent reef at a depth of only 12 feet. Coode revised his plan and his final proposal was to strengthen Vetch's North Pier and build a new South Pier at a staggering cost of £548,000 including blasting and dredging of the reef, see figure 7-2. The lawyer Harry Escombe viciously attacked this plan. After 3 years of discussion in commissions Escombe was appointed in 1881 as chairman of the Natal Harbour Board.

Innes

Harry Escombe appointed Edward Innes, a pupil of Sir John Coode, as Harbour Engineer. Innes started to work devoutly on the construction of a north and south pier. Although he still believed in the tidal scour he also ordered dredging ships that made good results. In 5 years Innes made more progress than any other engineer before him. In 1887 he was too sick to continue working and his friend Charles Crofts assumed responsibility for the harbour works.

Methven

Crofts was overlooked as Harbour Engineer and the job went to Cathart Methven. New dredgers arrived and dredging at open sea now became possible. The situation in 1890 is shown in figure 7-1. While Methven also believed in tidal scour Harry Escombe decided in 1893 that permanent dredging would be the only solution. Methven wanted to extend the north pier while Escombe would not sanction any more funds. He believed in Innes' theory of an overlapping south pier. Experts where called in and they agreed with Methven which meant the end of Escombe's political career. The controversy also meant the dismissal of Metvhen and the appointment of Charles James Crofts as Chief Harbour Engineer. An overview of the works proposed and executed up to Methven's time is shown in figure 7-2.

Crofts

While extending the north pier Crofts did a lot of work inside the harbour, building new wharves and docks. Crofts was, in his period as Harbour Engineer, advised by Sir Charles Hartley and Sir John Wolfe Barry. He continued the dredging works and by his early retirement in 1907, caused by political bickering, Durban finally had an International, safe, deep water harbour. Crofts achievements are best illustrated by his statistics, shown in table 7-1.

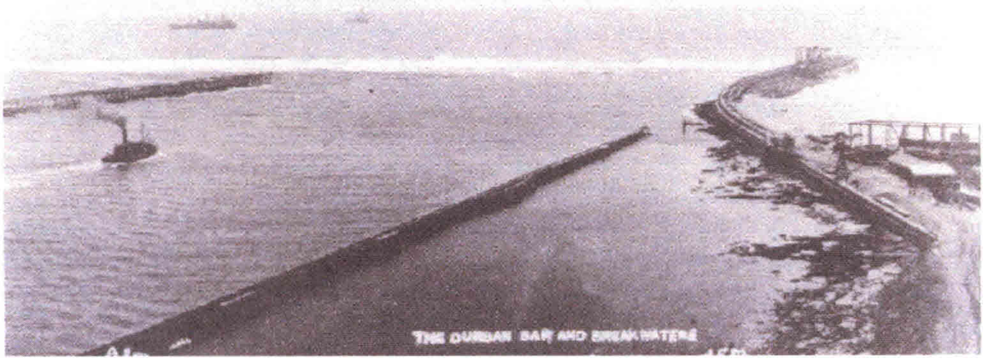


figure 7-1: white waters over the bar in 1890;

Available depths of water entrance at low water	
Average of 1895	12 feet 1 inch
Average of 1907	32 feet 8 inches
Tonnage entering	
1895	319,878 tonnes
1907	2,542,130 tonnes
Draught of deepest vessel	
1895	19 feet 0 inches
1907	30 feet 7 inches

table 7-1: general statistics relating to Durban Harbour (Crofts 1913);

figure 7-2: overview of proposed and executed works;

Steamers

The giant steamers of the early 20th century came in gradually greater numbers to the waters of Durban. What brought them was the cheap coal railed from the inland mines to the landing facility at the Bluff. Coal dust from the Bluff bunkering facility together with the ash jettisoned from the steamers boilers, soon built up in the lagoon. The dredgers, which were now working at the submerged sand beds, picked up this muck and deposited outside the harbour. Here the turbulent sea did its work and soon the Durban beaches had their first black soiling.

Groyne Shortening

In later years, when the theory of tidal scour removing the sandbar had already been disproved, the cant in the North Groyne constructed by Methven and Crofts posed a risk for entering ships. It was removed somewhere between 1936 and 1938. The proposed removal of the cant appears on a 1936 drawing and it is supposed that the actual removal would not have taken more than 2 years. The original drawing of the cant by Crofts and Methven (NPA archive) is shown in appendix J the removed part (NPA archive) is indicated in appendix K.

Relevant data

A summary will be given of remnants of old constructions, descriptions of existing constructions and problems that are expected to be encountered in the process of constructing the New North Groyne.

1 Milne's Pier;
2 Vetch's Pier;
3 Coal pollution;
4 Innes, Methven and Crofts' Pier;
5 Removed pier.

figure 7-3: overview relevant data

1. Milne's Pier

Remnants of Milne's old pier were found during construction in the Point area. Because of their historical value the uncovered parts where left and covered by a concrete capping. It is to be expected that during removal of the Point berths, for the widening of the entrance channel, more remnants will be uncovered. According to Milne's logs, he used rock from the Bluff quarry.

2. Vetch's Pier

Vetch's Pier has been partly removed in the past, even in Methven's time, but parts still remain. The remains can be seen from the beach. Certain design alternatives or later projects may require removal of this pier. As with Milne's pier Abernethy used Bluff rock in construction of the pier.

3. Coal Pollution

The old coal berths were located along the side of the Bluff. Spilling of coal and dumping of the ashes caused large pollutions in that area. Most of the polluted soil has since been removed by dredging but it can be expected that during removal of the Point area and dredging in the entrance channel some contaminated soil will be encountered.

4. Innes, Methven and Crofts' pier

The existing North Groyne has been constructed by, subsequently, Innes, Methven and Crofts. The rock used during the construction comes from the quarry at Umgeni. Only one drawing showing cross sections and foundation levels is available. Appendix N shows the available information.

5. Removed pier

The cant in the North Groyne has been removed in 1936. It is to be expected that some rocks were not removed. During dredging in the area of the former cant careful attention will have to be paid not to damage the dredging equipment on remnants of the existing groyne. Little is known about the removal of the cant but the as built specifics are shown in appendix K.

7.2.2 Entrance channel layout

The existing channel alignment is shown in appendix C The proposed new alignment is illustrated in appendix D.

7.2.3 Infrastructure

The existing North Groyne is at the moment accessible by road and boats of limited draught. The main road to the Point area runs right through the city of Durban. The eThekwini Municipality wants to minimize the transport of rocks by truck through its city centre (FB 10). An alternative for road traffic is the use of a decommissioned railroad running through the Point area, see figure 7-4.

figure 7-4: existing railroad with extension options;

7.2.4 Point Area

The Point has in recent years become a high crime area. Drugs trade and other criminal activities have made most of the streets unsafe to walk, especially at night. The eThekwini Municipality is trying to restore the Point's former glory by encouraging foreign investors to build apartments and hotels. The Point Development is now underway and old buildings and warehouses are being demolished to create room for new luxury apartments. The New North Groyne starts close the boundary between the NPA and the Point Development Company areas. During construction close attention will have to be paid to the interaction with the PDC (FR 8 and FB 9). The buildings on the NPA site of the Point Area are all owned by the NPA and are ready to be demolished. A special case is the sandhopper station located at the start of the New North Groyne, see figure 7-5.

figure 7-5: sandhopper station;

This station coordinates the distribution of sediment, dredged from the sand trap, along the Durban beaches. It will be decommissioned when the permanent sand bypass is completed. A period of time may exist when the station is already decommissioned while the new sand bypass system is not yet operational. In that case a solution will have to be found to prevent large erosion of the Durban beaches. As mentioned before the Durban tourist beaches, including those outside the Point Area, are highly important to the Durban tourist industry and should not be damaged (SR 1). Another point of attention is the seawater intake of the uShaka sea aquarium. The intake is located approximately 200 meter north of Vetch's Pier. Obstruction of the aquarium intake caused by large sediment deposits, perhaps buffers, on the beaches will result in claims from the aquarium.

7.2.5 Environmental

Environmental circumstances during construction have such a large influence on the productivity and the final quality of the structure that it is necessary to have statistical data on the conditions at the construction site. This meteorological data can be found in paragraph 5.6. During construction of the New North Groyne there might be phases where the wave and/or current protection is not fully completed. It is important to make an assessment of the risk of damage to the New North Groyne or other structures in the area. The best execution period has to be determined or assumed with wave and weather conditions taken into account. Two important aspects of the environmental data will be discussed briefly: the wave data and the sediment transport data.

7.2.6 Waves

An overview of the occurring waves and their effect on workability and expected damage to the New North Groyne during construction is shown in appendix L.

7.2.7 Sediment transport

It is known that the direction of sediment transport can occasionally change from the north direction to the south direction. At the moment it is unknown when this occurs. Sediment transport from north to south occurs mainly in the spring and summer (A 5).

7.2.8 Moored ships (berths)

There are two berths located close to the harbour entrance: the Point berths and the Bluff berths. Because the harbour entrance construction process may not adversely affect shipping operations in the port (FR 3) damaging or creating operational difficulties to moored or mooring ships is not allowed. Damage can occur due to wave penetration into the harbour. While in the final design the wave penetration will be minimized, during construction phases may occur in which there is less protection. A study (Ref. P4) on wave penetration has concluded that the existing North Groyne does not provide much protection against wave penetration. Removing it is not expected to lead to significant increase in wave penetration. In figure 7-6 the Durban Harbour Entrance model at the CSIR and the existing situation are illustrated.

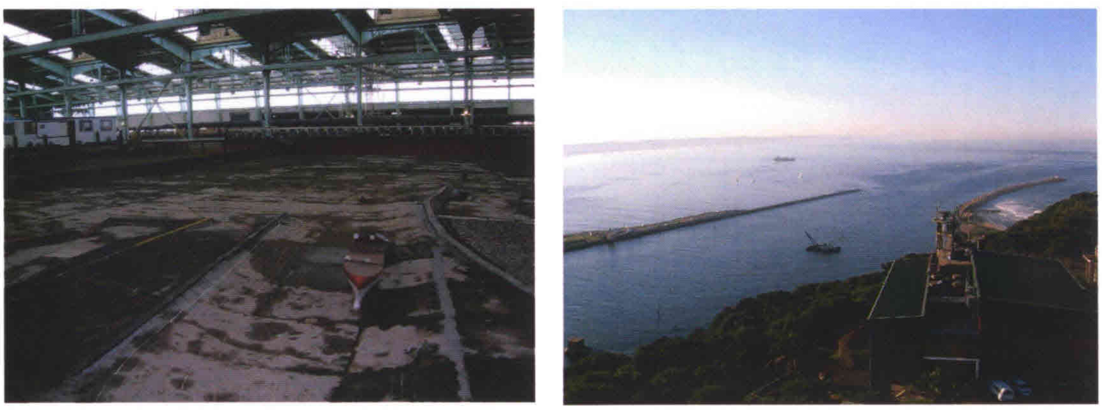


figure 7-6: Durban harbour entrance, the CSIR model and the existing situation;

7.3 Planned work New North Groyne

The history and the current situation of the North Groyne area have been discussed. The following actions can be executed in the North Groyne area for the Durban Harbour Entrance Widening and Deepening Project:

- **Removal of the existing North Groyne (NG):** The removal process of the existing North Groyne can be divided in two parts:

  - Removal of the North Groyne to a level upon which the groyne can still execute its main function: prevention of the ingress of sand into the entrance channel;
    - It is assumed that if the North Groyne is maintained at a level of +1 m CD in the surf zone, the remaining part can execute its function (A 6). The length of the surf zone is determined in appendix M;
    - The end part of the existing North Groyne, which is not located in the surf zone can be removed to a level of -4 m CD (A 7).
  - Removing of the remaining part of the existing North Groyne to the desired channel depth of -19 m CD.
- **Pre-dredging of the groyne head area:** pre-dredging of the head area may be required to construct it in sufficiently deep water. To ensure that the dredge hole stays open for a sufficient period of time a large area for construction of the head needs to be dredged. This because a slope of 1:10 is considered stable for this dredge hole during the construction period (A 14). The head of the New North Groyne will be constructed on this slope. If pre-dredging of the head area will not be chosen, other solutions have to be found;
- **New North Groyne construction:** the New North Groyne construction consists of 2 parts:
  - Construction of the trunk including revetment. The trunk functions primarily as a barrier for sediment transport;
  - Construction of the head that functions as a breakwater, minimizing wave penetration and protecting the trunk.
- **Falling or Launching apron:** A falling or launching apron is required if the head and/or trunk of the New North Groyne are not constructed in a pre-dredged area. This to ensure the channel side slope remains stable under scouring in the channel. A falling or launching apron is considered a viable solution for this problem.
- **Channel widening and deepening:** The channel will be dredged to a depth of -19 m CD next to the New North Groyne and to a level of -18 m CD next to the North Bank Revetment (TR 2). The initial slope between the New North Groyne and the channel will be approximately 1:6 and the initial slope between the North Bank Revetment and the channel will be approximately 1:3 (TR 3). After erosion it is expected a gentler slope will develop;
- **Removal and construction of the North Bank Revetment (NBR):** The New North Bank Revetment is a continuation of the New North Groyne revetment.

The order and extent in which these required actions are executed results in different alternatives. With the known requirements the construction sequence alternatives in table 7-2 are generated.

	Partial removal NG	Complete removal NG	Pre-dredge Head	Pre-dredge Trunk	Pre-dredge NBR	Construction Head	Construction Trunk	Construction NBR	Apron Head	Apron Trunk	Channel dredging
Alternative 1	1	1	2	2	2	3	3	3	-	-	4
Alternative 2	1	2	2	-	-	3	1	1	-	1	4
Alternative 3	2	2	2	-	-	3	1	1	-	1	4
Alternative 4	3	3	-	-	-	2	1	1	2	1	4
Alternative 5	1	2	-	-	-	2	1	1	2	1	3

table 7-2: generation of alternatives;

1. The complete removal of the existing North Groyne (phase 1) will be followed by the complete construction of the New North Groyne (phase 3) after pre-dredging the head area (phase 2);
2. The partly removal of the existing North Groyne simultaneous with the construction of the trunk of the New North Groyne (phase 1). After pre-dredging the head area (phase 2), the construction of the head of the New North Groyne will be done (phase 3);
3. The construction of the trunk of the New North Groyne (phase 1) will be followed by the removal of the existing North Groyne (phase 2). After pre-dredging the head area, the head of the New North Groyne will be constructed (phase 3);
4. The complete construction of the New North Groyne (phase 1 and 2) without pre-dredging the head area will be followed by the complete removal of the existing North Groyne (phase 3);
5. The partly removal of the existing North Groyne simultaneous with the construction of the trunk of the New North Groyne (phase 1) will be followed by the construction of the head of New North Groyne (phase 2), without pre-dredging the head area.

7.4 Description of construction sequences

In the following paragraphs these construction sequences will be specified. The implications and effects of each alternative are given in the next paragraph.

7.4.1 Alternative 1

The complete removal of the existing North Groyne will be followed by the complete construction of the New North Groyne after pre-dredging the head area. The sequence of this construction alternative is illustrated in figure 7-7.

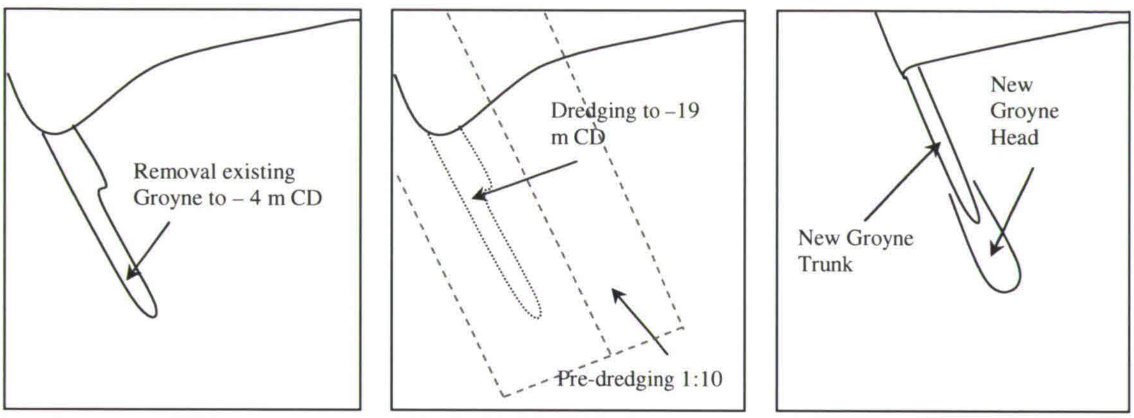


figure 7-7: construction alternative 1;

Removal existing North Groyne

This construction alternative will start with the partial removal of the existing North Groyne. As mentioned before this is only possible to a level upon which the groyne can accomplish its main function: prevention of the ingress of sand into the channel. Therefore the first 330 meter will be removed to a level of +1.0 m CD, the other part of the North Groyne, which is not located in the surf zone can removed to a level of -4 m CD or to its foundation depth, whichever is higher, see appendices M and N. Breaking up and removing of concrete sections, concrete units and rock have to be done. The material, which will be suitable for construction the core of the New North Groyne (A 13), will be removed (land based) using backhoes and bulldozers (A 8). The Point area offers enough space to sort and temporarily store this material (A 9). Appendix O discusses the expected volume of the removed material. After this initial stage, the remaining 330 meter North Groyne construction will be removed (land based) followed by the removal of the remaining core construction (waterborne or land based) until the foundation level. The waterborne removal will be done using a hydraulic backhoe dredger (A 10). The obtained material will also be sorted and stored in the Point area (A 9).

Dredging

The existing North Groyne area, which is now at foundation level, will be dredged to the desired channel depth of – 19 m CD (TR 2) with side slopes of 1:6 (TR 3) on the south side and 1:10 (A 14) on the north side where the New North Groyne will be constructed. This will be done using a trailer suction dredger (A 11). The material will be dumped offshore in two designated areas, located a few miles outside the coast (ER 2). Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment (ER 3).

Construction New North Groyne Trunk and Head

After the dredging work is finished, the construction of the New North Groyne can start. The trunk and head can be constructed using both land based and marine equipment. Because of the executed dredging works, a huge amount of material is needed to construct the elements of the New North Groyne. The area around the New North Groyne offers enough working space, due to the removal of the existing North Groyne. Materials from the existing North Groyne can easily be used for the core of New North Groyne and material from the demolished buildings.

7.4.2 Alternative 2

In this alternative the partly removal of the existing North Groyne goes simultaneous with the construction of the trunk of the New North Groyne. After pre-dredging the head area, the construction of the head of the New North Groyne will be done. The sequence of this construction alternative is illustrated in figure 7-8.

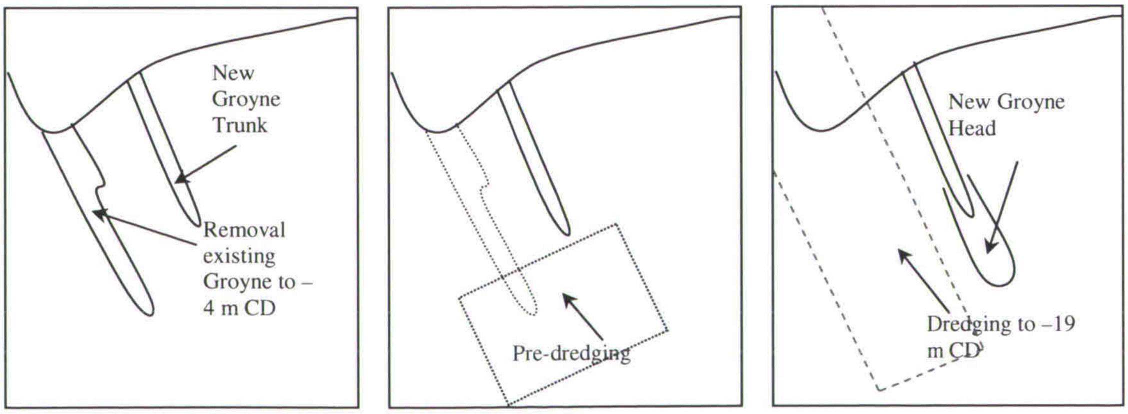


figure 7-8: construction alternative 2;

Removal existing North Groyne

The removal of the existing North Groyne will be executed in the same way as described in alternative 1 with 1 change: the removal of the remaining core section will only be executed after the New North Groyne trunk is completed.

Construction New North Groyne Trunk

The construction of the New North Groyne starts with the construction of the trunk. This will be done at the same time as the removal of the existing North Groyne. The consequence of this is that extra material must be provided to construct the core. During construction of the New North Groyne trunk, space problems for marine equipment can occur due to the closeness of the existing North Groyne.

Pre-dredging

After the construction of the trunk of the New North Groyne and the removal of the existing North Groyne to its foundation level, the pre-dredging of the New North Groyne head area can start. The head of the New North Groyne will be constructed with its base at a level of approximately -16 m CD or lower. Because of the natural slopes of 1:10 (A 14), a large area needs to be dredged in order to ensure that this dredge hole stays open for a sufficient long period of time. The minimum required pre-dredged area is shown in figure 7-9.

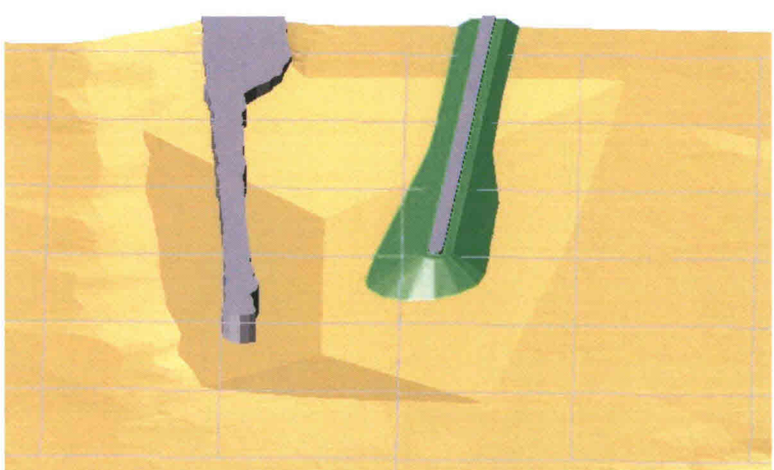


figure 7-9: minimum pre-dredged area, grey square equals 100 x 100 m;

Construction New North Groyne Head

After the pre-dredging of the head area, the construction of the head of the New North Groyne can start. The slope of the New North Groyne must be provided with a protection against erosion and scouring after dredging of the channel. This will be done using a falling or launching apron.

Channel Dredging

After the completion of the New North Groyne, the channel can be dredged to the desired channel depth of – 19 m CD (TR 2) with side slopes of 1:6 (TR 3). This will be done using a trailer suction dredger (A 11). The material will be dumped offshore in two designated areas, located a few miles outside the coast (ER 2). Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment (ER 3).

7.4.3 Alternative 3

The construction of the trunk of the New North Groyne will be followed by the removal of the North Groyne. After pre-dredging the head area, the head of the New North Groyne will be constructed. The sequence of this construction alternative is illustrated in figure 7-10.

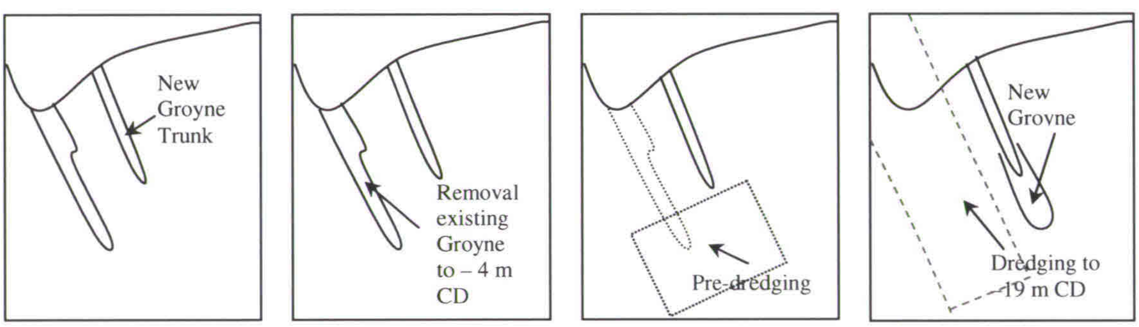


figure 7-10: construction alternative 3;

Construction New North Groyne Trunk

This construction alternative will start with the construction of the trunk of the New North Groyne. All the material must be provided to construct the core and armouring. During construction of the New North Groyne trunk, space problems for marine equipment can occur due to the closeness of the existing North Groyne.

Removal existing North Groyne

Before the removal of the existing North Groyne can start, the trunk of the New North Groyne will be completed. In this alternative the ingress of sand into the channel is prevented. After finishing the trunk of the New North Groyne, the existing North Groyne top part will be removed (land based) using backhoes and bulldozers (A 8) followed by the removal of the remaining core construction (waterborne or land based) until the foundation level. The waterborne removal will be done using a hydraulic backhoe dredger (A 10). The obtained material will be sorted and stored at the Point area. This material will be used to construct the core of the head of the New North Groyne (A 13). The Point area offers enough space to sort and temporarily store this material (A 9).

Pre-dredging

After the construction of the trunk of the New North Groyne and the removal of the existing North Groyne to its foundation level, the pre-dredging of the New North Groyne head area can commence. The head of the New North Groyne will be constructed with its base at a level of approximately -16 m CD or lower. Because of the natural slopes of 1:10 (A 14), a large area needs to be dredged in order to ensure that this dredge hole stays open for a sufficient long period of time. The minimum required pre-dredged area is shown in figure 7-9.

Construction New North Groyne head

After the pre-dredging of the New North Groyne head area, the construction of the head can start. The revetment of the New North Groyne must be provided with a protection against erosion and scouring after dredging of the channel. This will be done using a falling or launching apron.

Channel Dredging

After the completion of the New North Groyne, the channel can be dredged to the desired channel depth of – 19 m CD (TR 2) with side slopes of 1:6 (TR 3). This will be done using a trailer suction dredger (A 11). The material will be dumped offshore in two designated areas, located a few miles outside the coast (ER 2). Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment (ER 3).

7.4.4 Alternative 4

The complete construction of the New North Groyne without pre-dredging the head area will be followed by the complete removal of the existing North Groyne. This construction alternative guarantees prevention of the ingress of sand into the channel. The sequence of this construction alternative is illustrated in figure 7-11.

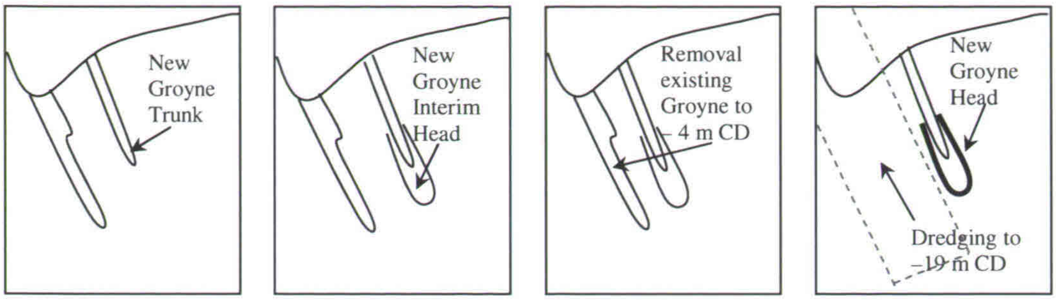


figure 7-11: construction alternative 4;

Construction New North Groyne

This construction alternative will start with the construction of the trunk of the New North Groyne. All the material must be provided to construct the core. During construction of the New North Groyne trunk, space problems for marine equipment can occur due to the closeness of the existing North Groyne. After the construction of the trunk, the construction of the New North Groyne head commences. The slope of the New North Groyne, head and trunk, must be provided with a protection against erosion and scouring after dredging of the channel. This will be done using a falling or launching apron. After finishing the dredging of the channel to the desired depth, the final protection of the New North Groyne head will be placed.

Removal existing North Groyne

After the construction of the New North Groyne trunk the partial removal of the existing North Groyne will start. As mentioned before this is only possible to a level upon which the groyne can accomplish its main function: prevention of the ingress of sand into the channel. Therefore the first 330 meter will be removed to a level of +1.0 m CD, the other part of the North Groyne, which is not located in the surf zone can removed to a level of -4 m CD or to its foundation depth, whichever is higher, see appendices M and N. Breaking up and removing of concrete sections, concrete units and rock have to be done. After finishing the New North Groyne, the remaining part of the existing North Groyne top part will be removed (land based) using backhoes and bulldozers (A 8) followed by the removal of the remaining core construction (waterborne or land based) until the foundation level. The waterborne removal will be done using a hydraulic backhoe dredger (A 10). The obtained material will be sorted and stored at the Point area. This material will be used to construct the New North Groyne core of the head part. The Point area offers enough space to sort and temporarily store this material (A 9).

Pre-dredging

No pre-dredging of the New North Groyne head area is required in this construction alternative. The head will be founded on the existing sea bottom.

Channel Dredging

After the completion the New North Groyne, the channel can be dredged to the desired channel depth of – 19 m CD (TR 2) with side slopes of 1:6 (TR 3). This will be done using a trailer suction dredger (A 11). The material will be dumped offshore in two designated areas, located a few miles outside the coast (ER 2). Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment (ER 3). After finishing the dredging of the channel to the desired depth, the final protection of the New North Groyne head is placed.

7.4.5 Alternative 5

The partly removal of the existing North Groyne goes simultaneous with the construction of the trunk of the New North Groyne and will be followed by the construction of the head of New North Groyne, without pre-dredging the head area. The sequence of this construction alternative is illustrated in figure 7-12.

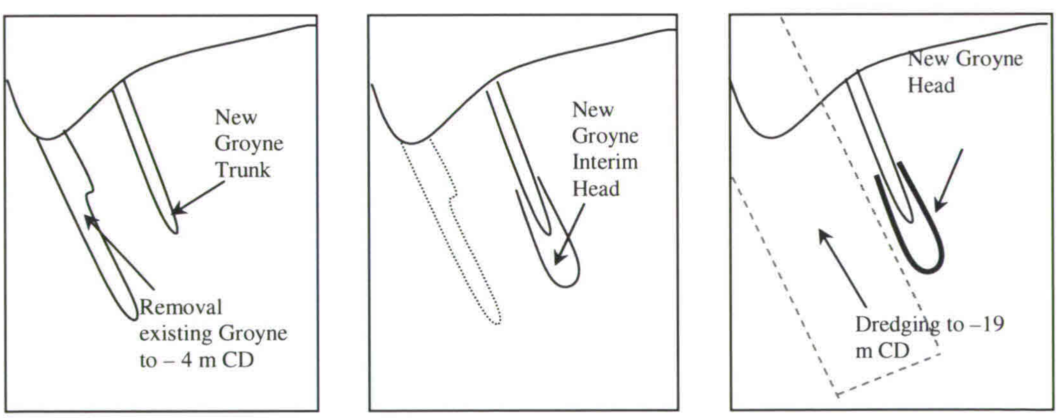


figure 7-12: construction alternative 5;

Removal existing North Groyne

The removal of the existing North Groyne will start simultaneous with the construction of the trunk of the New North Groyne. The main function of the groyne, prevention of the ingress of sand into the channel continues in this construction stage. The first 330 meter will be removed to a level of +1 m CD (A 6), the other part of the existing North Groyne that is not located in the surf zone can removed to a level of -4 m CD (A 7), see appendices M and N. Breaking up and removing of concrete sections, concrete units and rock have to be done. The material that will be suitable for construction the core of the New North Groyne (A 13) will be removed (land based) using backhoes and bulldozers (A 8). The Point area offers enough space to sort and temporarily store this material (A 9). Appendix O discusses the expected volume of the removed material. After the completion of the New North Groyne trunk, the remaining 330-meter existing North Groyne construction will be removed (land based) followed by the removal of the remaining core construction (waterborne or land based) until the foundation level. The waterborne removal will be done using a hydraulic backhoe dredger (A 10). The obtained material will also be sorted and stored in the Point area.

Construction New North Groyne

The construction of the New North Groyne starts with the construction of the trunk. This will be done at the same time of the removal of the existing North Groyne. This has the consequence that extra material must be provided to construct the core. During construction of the New North Groyne trunk, space problems for marine equipment can occur due to the closeness of the existing North Groyne. After the construction of the trunk, the construction of the New North Groyne head can commence. The slopes of the New North Groyne must be provided with a protection against erosion and scouring after dredging of the channel. This will be done using a falling or launching apron. After finishing the dredging of the channel to the desired depth, the final protection of the New North Groyne head can be placed.

Channel Dredging

After the completion of the New North Groyne, the channel can be dredged to the desired channel depth of – 19 m CD (TR 2) with side slopes of 1:6 (TR 3). This will be done using a trailer suction dredger (A 11). The material will be dumped offshore in two designated areas, located a few miles outside the coast (ER 2). Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment (ER 3).

Pre-dredging

No pre-dredging of the New North Groyne head area is required in this alternative. The head will be founded on the existing sea bottom.

7.5 Multi criteria analysis

7.5.1 Introduction

In this paragraph the impact of the considered construction alternatives will be evaluated on different criteria. Depending on their importance, criteria will have different weight factors. Points will be awarded to alternatives for each criterion and these points are multiplied by the weight factors. Eventually the construction alternatives with the most points will be elected as the final design.

7.5.2 Impact of alternative 1:

- **Material use:** A huge bulk of material will be required because the trunk of the New North Groyne will be build at large depth. A large rubble mound is therefore required but no extra rock for falling or launching aprons will be necessary, see appendix P. Compared to the other alternatives this alternative is the most recycling friendly. Because the construction of the New North Groyne only commences after the total removal of the existing North Groyne, all material removed can be recycled. Storage of the removed material in the Point Area will be required;
- **Shipping:** The complete removal of the existing North Groyne before construction of a new one will require the entering and exiting ships to visually navigate on the buoys and harbour lights. The channel will, for a period of time, not be protected from the north. Wave penetration will be higher compared to other alternatives;
- **Moored ships:** Due to higher wave penetration the risk of damage to moored ships on the Bluff berths is also high compared to other alternatives. Computer model testing has proven this effect not to be significant.
- **Damage to New North Groyne:** The trunk of the New North Groyne will not be unprotected for long because dredging work will not interrupt the construction. No significant loss of material is expected;
- **Sediment transport:** For a large period of time there will be no protection against sediment transport into the channel. A flow of sediment from the north into the channel is to be expected;
- **Dredging:** Compared to other alternatives much more pre-dredging needs to be done. The entire trunk will be build as a new structure at the same depth as the head in other alternatives. The safety of dredging will be increased because no falling or launching apron is required in this alternative. Lastly the increased sediment transport in the channel will require more maintenance dredging during construction to keep the channel at depth;

- **Construction time:** Because the removal of the existing North Groyne and the construction of the New North Groyne do not take place simultaneously and much more dredging and rock dumping will have to be done, the construction time is expected to be long, compared to other alternatives;
- **Design Aspects:** No problems with slope stability and liquefaction are expected in this alternative because the New North Groyne will be a completely new structure. Due to the great height of the rubble mount some settlement will occur in the subsoil so some extra height will have to be given to the structure.

7.5.3 Impact of alternative 2:

- **Material use:** The New North Groyne is constructed at the same time as the demolishment of the existing North Groyne. Materials can be recycled without having to store them. The trunk of the New North Groyne will be constructed in shallow water so much less material will be used compared to alternative 1. The head will be constructed in a pre-dredged area and will still therefore require a large core;
- **Shipping:** The existing North Groyne will be partly removed but still present on its foundation level, buoys will need to be used to warn ships of its presence. The channel will, for a period of time, be only partly protected from the north. Some wave penetration into the channel can occur;
- **Moored ships:** There is less wave penetration compared to alternative 1 due to the presence of the submerged part of the existing groyne. The penetration will be higher compared to alternatives 3, 4 and 5 but it is still not considered very significant.
- **Damage to New North Groyne:** The trunk of the New North Groyne will be unprotected during the pre-dredging work for the construction of the head. Some damage to the relatively unprotected groyne can be expected;
- **Sediment transport:** In the pre-dredging phase there will be less protection against sediment transport caused by the partly finished New North Groyne and the removed head of the existing North Groyne;
- **Dredging:** Pre-dredging will have to be done for construction of the head. During the pre-dredging and dredging of the channel slopes the risk of damage to the dredging equipment caused by the falling or launching apron exists. Some maintenance dredging, due to sediment transport will need to be done;
- **Construction time:** The trunk of the New North Groyne is build simultaneous with the demolishing of the existing North Groyne. The pre-dredging will take some time but removal of the existing groyne can continue and dredging can begin quickly.

Construction of the head of the New North Groyne can only be commenced when the pre-dredging is completed;

- **Design aspects:** Design of a falling or launching apron will be required for the trunk of the New North Groyne and the North Bank Revetment. Slope stability analyses will be required for the channel slopes, to determine stability after dredging and the final slope. Some settlement can occur, mainly in the head section.

7.5.4 Impact of alternative 3:

- **Material use:** The trunk of the New North Groyne will be constructed in shallow water so much less material will be used compared to alternative 1. The head will be constructed in a pre-dredged area and will still require a large core.
The trunk of the groyne will be constructed before the removal of the existing groyne so it cannot be build with recycled materials. The head of the New North Groyne will probably require enough core material to recycle all the material of the existing North Groyne. Some storage of material will be required;
- **Shipping:** The existing North Groyne will be partly removed but still present on its foundation level, buoys will need to be used to warn ships of its presence. The protection of the channel against wave penetration is slightly better, compared to alternative 2;
- **Moored ships:** There is less wave penetration compared to alternative 1 and 2 due to the presence of the existing North Groyne. The protection will be less compared to alternative 4 and 5 caused by the only partly finished New North Groyne;
- **Damage to New North Groyne:** The trunk of the New North Groyne will be unprotected during the pre-dredging work for the construction of the head. Some damage to the relatively unprotected groyne can be expected;
- **Sediment transport:** In the pre-dredging phase there will be less protection against sediment transport caused by the partly finished New North Groyne and the removed head of the existing North Groyne;
- **Dredging:** Pre-dredging will have to be done for construction of the head. During the pre-dredging and dredging of the channel slopes the risk of damage to the dredging equipment caused by the falling or launching apron exists. Some maintenance dredging will need to be done;
- **Construction time:** The trunk of the New North Groyne will be build before any demolishing or dredging can be done, this will take extra time. The pre-dredging will take some time but removal of the existing groyne can continue and dredging can

commence quickly. Construction of the head of the New North Groyne can only be started when the pre-dredging is completed;

- **Design aspects:** Design of a falling or launching apron will be required for the trunk of the New North Groyne and the North Bank Revetment. Slope stability analyses will be required for the channel slopes, to determine stability after dredging and the final slope. Some settlement can occur, mainly in the head section.

7.5.5 Impact of alternative 4:

- **Material use:** Both the trunk and the head of the New North Groyne will be build in shallow water and will therefore require less material than the other alternatives. Once the falling or launching apron is activated extra material will be required to strengthen the head. Because the existing North Groyne is only demolished after the New North Groyne is completed no recycling of material can be done;
- **Shipping:** Because the existing North Groyne will only be removed after the New North Groyne is almost finished the wave penetration will be minimized. The channel will stay visually delineated for a long period of the construction; only during the removal of the existing groyne extra buoys will be needed;
- **Moored ships:** The wave penetration is at no point larger than the final wave penetration with the New North Groyne in place. Model research has already determined no damage to moored ships will occur in the new situation;
- **Damage to New North Groyne:** The trunk of the New North Groyne will be protected during the dredging work by the temporary head. Some damage to the temporary head is acceptable;
- **Sediment transport:** During the entire period of the project full protection against sediment transport will exist;
- **Dredging:** No pre-dredging is required in this alternative; this has a positive effect on the dredging cost. The dredging of the channel will have higher risk of damage to the dredging equipment caused by the falling or launching aprons in the head and the trunk. The expected extent of the damage to a backhoe dredger is not very large. No maintenance dredging for sediment transport is required during construction;
- **Construction time:** The entire New North Groyne will be build before any removal of the existing North Groyne is started. Although no pre-dredging will have to be done the construction time will be long compared to other alternatives;
- **Design aspects:** Design of a falling or launching apron will be required for the trunk and head of the New North Groyne and the North Bank Revetment. Slope stability

analyses will be required for the channel slopes at the head and trunk, to determine stability after dredging and the final slope. No large settlements are expected to occur.

7.5.6 Impact of alternative 5:

- **Material use:** Both the trunk and the head of the New North Groyne will be build in shallow water and will therefore require less material than the other alternatives. After the falling or launching apron is activated extra material will be required to strengthen the head. The construction of the trunk "interim-head" can be done with recycled material because the removal of the existing North Groyne takes place simultaneous to the construction of the New North Groyne;
- **Shipping:** The existing North Groyne will be partly removed but still is present on its foundation level, buoys will need to be used to warn ships of its presence. The protection of the channel against wave penetration is slightly better than with alternative 2;
- **Moored ships:** There is less wave penetration compared to alternative 1 and 2 due to the presence of the existing North Groyne. The protection will slightly be better than alternative 3 because the new head will be finished earlier. The protection will be less than alternative 4;
- **Damage to New North Groyne:** The trunk of the New North Groyne will be protected during the dredging work by the temporary head. Some damage to the temporary head is acceptable;
- **Sediment transport:** During the entire period of the project almost full protection against sediment transport will exist. Only the transport over the partly removed existing North Groyne can occur which is not expected to be much;
- **Dredging:** No pre-dredging is required in this alternative. The dredging of the channel will have higher risk of damage to the dredging equipment caused by the falling or launching aprons in the head and the trunk. The expected extent of the damage to a backhoe dredger is not very large. No maintenance dredging for sediment transport is required during construction;
- **Construction time:** The trunk of the New North Groyne is build simultaneous with the demolishing of the existing North Groyne. With no pre-dredging required, the construction time is expected to be short, compared to other alternatives.
- **Design aspects:** Design of a falling or launching apron will be required for the trunk and head of the New North Groyne and the North Bank Revetment. Slope stability analyses will be required for the channel slopes at the head and trunk, to determine stability after dredging and the final slope. No large settlements are expected to occur.

7.5.7 Criteria

Based on the expected impact and requirements defined in the Design Basis chapter, the following criteria are specified:

- Construction Costs:
  - Supply of material;
  - Equipment;
  - Dredging work.
- Environmental:
  - Recycling of materials;
  - Volume of dredging work.
- Shipping safety:
  - Safety of ships in channel;
  - Safety of ships on berths.
- Damage chance:
  - Chance of damage to groyne during construction;
  - Chance of damage to beaches;
  - Chance of damage to dredging equipment.
- Construction Time.

7.5.8 Determining weight factors

In table 7-3 the criterion weight factors are determined by comparing the criteria to each other. The criterion in each row is compared to the criteria in the columns. If the criterion in the row is considered more important than the one in the column it is awarded a "1". If the criterion in the column is considered more important a "0" is awarded. The criterion with the most points is awarded the highest weight factor.

	Construction costs	Environmental	Shipping safety	Damage chance	Construction time	Total	Awarded weight factor
Construction costs		1	0	0	1	2	**3**
Environmental	0		0	0	1	1	**2**
Shipping safety	1	1		1	1	4	**5**
Damage chance	1	1	0		1	3	**4**
Construction time	0	0	0	0		0	**1**

table 7-3: criterion weight factors;

The safety of ships in the channel and on the berths is considered paramount to all other criteria. The chance of damage will bring much higher costs in claims and rebuilding of structures than is expected to be saved using different equipment and materials. The social and economical impact of damage to the Durban beaches is also considered very important (SR 1). The environmental impact is not considered very important, compared to the other criteria, only the recycling of materials and the disposal of dredging materials are taken into account.

7.5.9 Analysis

Each alternative is awarded a score between 0 and 5, 5 meaning a good score and 0 a bad score, see table 7-4.

	Construction costs	Environmental	Shipping safety	Damage chance	Construction time	Total
	3	**2**	**5**	**4**	**1**	
Alternative 1	0	3	3	5	0	**41**
Alternative 2	3	5	3	3	4	**50**
Alternative 3	3	3	4	4	4	**55**
Alternative 4	4	2	5	2	3	**52**
Alternative 5	4	3	3	2	5	**46**

table 7-4: alternative scores;

7.5.10 Conclusion

Alternative 3 scores best with 55 points. Alternative 4 is the second best construction method with 52 points.

7.6 Conclusions and Recommendations

7.6.1 Conclusions

Based on the Multi Criteria Analysis (MCA) the construction alternatives that are chosen for the detailed design are alternative 3 and alternative 4. Construction alternative 1 will also be considered. Although this alternative scored low in the MCA it is an interesting design subject and should be elaborated.

7.6.2 Recommendations

The possibility of damage to the New North Groyne during construction is briefly mentioned in this chapter. The longer the unprotected trunk of the groyne is exposed to wave conditions the larger the chance of damage. It is advisable to look into the extent of the damage caused by occurring wave heights. Small damage is probably acceptable but large damages will be costly and can seriously hamper the dredging work.

Research will have to be done to ensure the stability of the Durban tourist beaches. During the construction the existing hopper station will be removed and the sediment supply to the beaches will be temporarily stopped. A temporary hopper station can be build or a buffer of sediment can be placed on the beaches. This buffer might affect the sediment transport into the channel during construction and its effects will have to be evaluated.

Construction alternatives for the North Bank Revetment are not discussed in this chapter. It is assumed that the revetment will be an extension of the New North Groyne trunk revetment. Other solutions are nevertheless also possible. A sheet pile quay wall instead of a slope can be a land saving and viable solution. The evaluation and design of these possible alternatives will not be included in this report but might be worthwhile to consider.

The initial dredge slope between the channel and the New North Groyne will be approximately 1:6, after erosion it is expected a gentler slope will develop. Research to this slope development will be done in chapter 10.

Several alternatives mentioned in this chapter require a falling or launching apron to protect the trunk revetment and the North Bank Revetment. A design and proposed construction method of this falling or launching apron will be considered in chapter 11.

8. DESIGN OF ALTERNATIVES

8.1 Introduction

In this chapter different designs will be made for all of the three considered construction alternatives. First the relevant design bases as proposed in chapter 5 will be described. Next the general design will be made by applying the design rules to the cross-sections of the head and the trunk separately. The following design elements are taken into account: armour layer, first underlayer, toe berm, core, filter and scour protection. The three design alternatives will be evaluated using a Multi Criteria Analysis. Combining this analysis with the results of the MCA in the construction method chapter, one of the alternatives will be chosen for the detailed design.

The aim of the chapter is to make designs for the New North Groyne based on the 3 alternatives chosen in chapter 7.

8.2 Design basis

8.2.1 Position

The position of the New North Groyne is determined mainly by the navigational requirements of the widened channel and to control sediment transport. Simulations with ships entering into and departing from the harbour were executed by the CSIR to ensure channel dimensions that satisfied the requirements of the National Ports Authority. The extent to which the groyne could be positioned northwards of the existing position was limited by the need to minimize wave penetration. It was not considered practical to move the southern breakwater and therefore the New North Groyne had to be positioned as far as possible to the north in order to obtain the required width for the new channel. In that position the distance between the centre of the New North Groyne and the channel will be 60 metres.

In figure 8-1 the final position that was selected for the New North Groyne is illustrated. This layout was selected based on:

- Navigational requirements(TR 1, TR 2, FR 2 and FR 3) ;
- Avoiding significant wave reflection from the New North Groyne into the entrance channel;
- Ensuring sufficient water depth at the head of the structure to avoid sediment transport into the channel (TR 11).

The New North Groyne has two main functions. The function of the head is breaking the incoming waves and the function of the trunk is reducing the sediment transport.

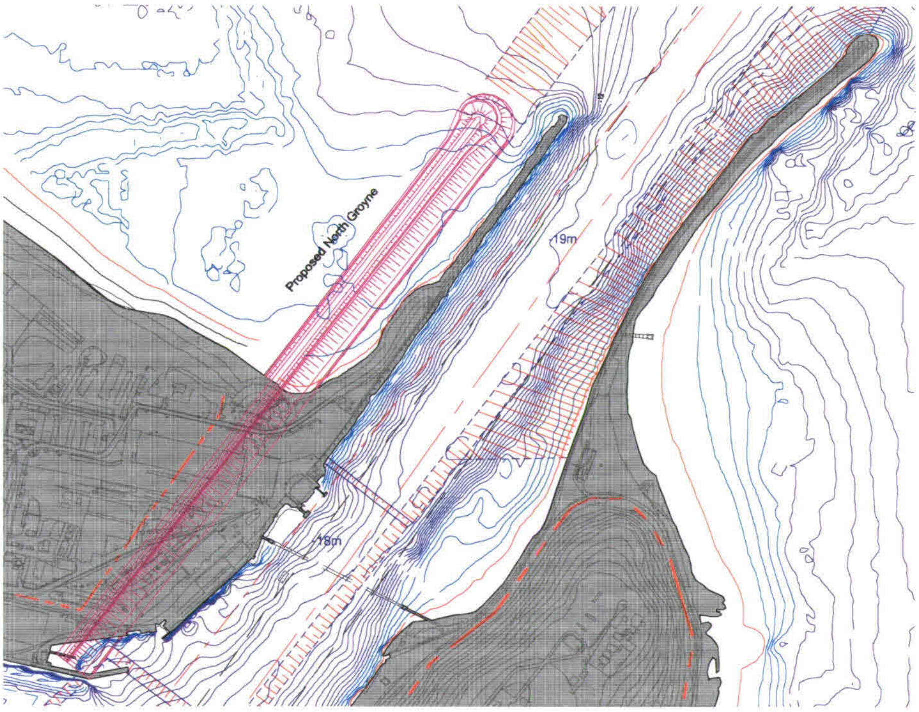


figure 8-1: final position New North Groyne;

8.2.2 Crest height

The crest level is generally determined on the basis of the functional requirements. It must be clear that the selection of a too high crest level leads to excessive use of material because the volume of the structure is proportional to the square of its height. Selection of a low crest level may also have serious consequences for the construction method (exclusive use of marine equipment) and the maintenance. The New North Groyne has to have the same accessibility as the existing North Groyne (TR 16); this is not possible with a low crest.

The crest level of the New North Groyne is designed to approximately the same level as the existing North Groyne. The existing groyne is high enough (+4.0 m CD (A 12)) to withstand the wave attack. This is why the level of the newly designed crest will be at the same height. The crest will be made with cap blocks. The cap blocks can be used as a road for construction and maintenance equipment. If the groyne is opened for public, the road provides a safe way onto the groyne. Placing the cap blocks is a very secure job, because after placing it is never possible to fill voids under the cap-block. To protect the cap-blocks, it is recommended to place the armour layer or underlayer material up to the full height of the cap-blocks. The width of the cap will be 8.0 m (FB 6) to ensure that a truck can drive on the groyne and maintenance to the groyne can be done land based.

The top of the core has to be at +3.0 m CD, this enables the dump trucks to drive safely on the groyne during construction (FR 6). With a crest height at +4.0 m CD and a core height at +3.0 m CD the cap blocks will only be 1.0 meter thick in the middle, this will be a sufficient thickness (TR 19).

8.3 Alternative 1: Complete pre-dredging

The complete removal of the existing North Groyne will be followed by the complete construction of the New North Groyne after pre-dredging the head and trunk areas. There is a lot interaction between the different steps in the design process. To maintain a logical order some used values are calculated later on in the paragraph.

8.3.1 Head

Armour layer

The 1:100 year design significant wave height is 3.5 m. (TR 8). Because the wave measurements are from a small period of time, a safer value of 4.0 m is used for the significant wave height at the head. The slope of the groyne is 1:1.5 (TR 17). Stability of dolosse and rock is evaluated with Hudson's equation (CERC, 1984). The Van der Meer equation is only used for waves on the trunk (Ref. AH).

Hudson's equation:

$$W = \frac{\rho_s H_s^3}{K_d \Delta^3 \cot \alpha},$$

where,

W	Armour weight	[tonne],
H_s	Significant wave height at toe of structure	[m],
ρ_s	Density of armour material	[tonne/m^3],

Δ	Relative density:	$\Delta = \dfrac{(\rho_s - \rho_w)}{\rho_w}$	[-],
α	Slope angle		[°],
K_d	Reduction coefficient		[-].

The formula is applicable for slopes not steeper than 1:1 and not gentler than 1:4. The coefficient K_d represents many different influences. One of the functions of this coefficient is the level of damage defined as "loss of stability". It also includes the effect of the shape of the block and the internal friction between the blocks. Results of calculations with the Hudson formula can be found in table 8-1.

H_s [m]	Armour Type	ρ_s [tonne/m^3]	K_d [-]	W [tonne]
3.5	Dolos	2.40	8.0	7.3
3.5	Rock	2.65	2.8	14.0
4	Dolos	2.40	8.0	11.0
4	Rock	2.65	2.8	20.7

table 8-1: Hudson formula results, head;

It can be concluded that rock is not a practical option for the head of the groyne. For the head of the groyne dolosse of 11 t will be needed. The armour layer of the head will be build up with dolos blocks (TR 18). Dolosse are preferred in South Africa above other types of armour units due to the present experience, the extensive research and knowledge. Dolos blocks can easily be made in large quantities.

Armour type

The head of the groyne is relatively vulnerable to damage since the armour units are less supported and/or less interlocked due to the curvature. In general, damage occurs on the inner quadrants, which is understandable looking at the 3D shape, see figure 8-2.

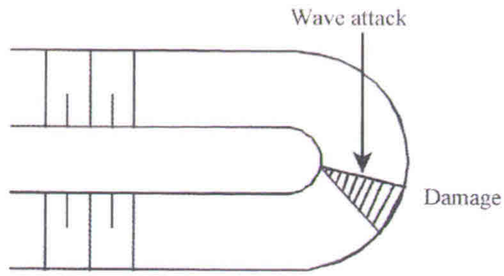


figure 8-2: curvature of groyne head;

Therefore, the head of a groyne is often reinforced, either by:

- Using larger size armour units;
- Increasing the density of the armour units;
- Reducing the slope.

Neither of the structural solutions is ideal; larger and heavier blocks pose construction problems and a reduced slope angle may cause a hazard to navigation. In this section more information is given about the dolosse and the optimal type of dolos for the head of the New North Groyne will be chosen.

figure 8-3: South African dolos;

The dolos armour unit, as shown in figure 8-3, was designed by E.M. Merrifield and was first used in the East London breakwater in 1964. The dolos shape comprises a central shank with a fluke at either end, as shown in figure 8-4. The primary dimension is the dolos height (H). The ratio between the shank width (B) and the height is termed the waist ratio (w_r), which generally has a value between 0.32 and 0.36. The width of the fluke end (A) is typically equal to 0.2*H. A straight chamfer or a curved fillet is often included at the fluke-shank interface. The size of the chamfer (D) or the radius of the fillet is expressed as a ratio of the dolos height.

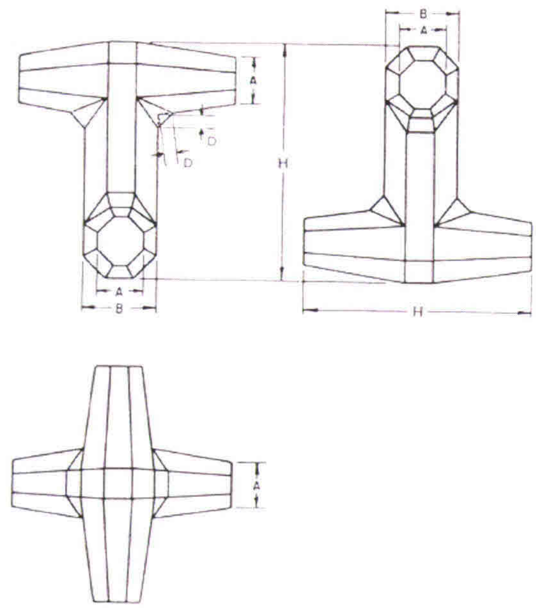


figure 8-4: dolos dimensions;

The armour layer thickness depends on the type of dolos (volume and waist ratio) and the way they are placed (packed). This is expressed in the packing density, which gives the relation with the porosity, the number of layers and the shape factor:

$t_n = \tau_n \cdot V^{1/3}$	Armour layer thickness	[m],
$\tau_n = n \cdot c_n$	'Relative' armour layer thickness	[m],
$\phi = n \cdot c_n \cdot (1 - \frac{P_f}{100})$	Packing density	[-],
$V = \frac{M}{\rho_s} = 0.675 \cdot w_r^{1.285} \cdot H^3$	Block volume (JAZ, 1980)	[m^3],
$D_n = V^{1/3} = 0.877 \cdot w_r^{0.428} \cdot H$	Equivalent cube diameter (JAZ, 1980)	[m],
$N_n = \phi_n \cdot V^{-2/3} = \phi_n / D_n^2$	Number of units, per unit area of slope	[1/m^2],
$w_r = 0.34 \cdot \left(\frac{M}{20}\right)^{1/6}$	Waist ratio	[-],
	$0.32 \leq w_r \leq 0.36$ (Zwamborn and Beute, 1972),	

where,

n	Number of units comprising the thickness of the armour layer	[-],
c_n	Shape factor (layer coefficient)	[-],
P_f	'Fictitious' porosity of the armour layer	[%],
H	Dolos height	[m],
M	Dolos mass	[tonne],
ρ_s	Density of armour material	[tonne/m^3].

To calculate the layer thickness, the shape factor and the porosity will have to be known. The U.S. Army Coastal Engineering Research Centre (Ref. I10) and Carver and Davidson (Ref. I11) have found the following values after research, see table 8-2.

Source	n	Placement	c_n [-]	P_f [%]
U.S. Army CERC (1977)	2	Random	1.00	63
Carver & Davidson (1977)	2	Random	0.94	56

table 8-2: porosity and shape factor;

The excellent hydraulic stability of the dolos is due to its shape, which permits interlocking with adjacent units and also results in a highly porous armour layer. The rather slender dolos shape is, however, susceptible to breakage. This breakage has contributed directly or indirectly to the failure of a number of rubble mound breakwaters, most notably that of Sines, Portugal, in 1978 and 1979. In other cases the gradual breakage of dolos units over time has necessitated major repairs to the breakwaters, e.g. at Mossel Bay. These breakages have led to increased research activity to assess the structural integrity of armour units in general and the dolos in particular.

In 1980 Zwamborn (Ref. I12) did specialised research on the dolos, with a 'fictitious' porosity of 51.5 percent. This 'fictitious' porosity gives a proper measure of reality. The corresponding parameters are found in table 8-3.

Packing	$\phi_{n=2}$ [-]	$\tau_{n=2}$ [-]	$c_{n=2}$ [-]	$P_{f(n=2)}$ [%]
Light	0.83	1.74	0.87	51.5
Mean	1.00	2.06	1.03	
Dense	1.15	2.36	1.18	

table 8-3: dolos properties according to Zwamborn (1980);

Maximum interlocking can be expected when the layer thickness is equal to the dolos height, viz. $t_{n=2} = H$. This is called the optimum layer thickness (Ref. I12). If a waist ratio $w_r = 0.34$ and a porosity of P_f = 51.5 % are assumed, the dolos packing density corresponding to a layer thickness H follows from the calculations given below:

$$\tau_n = n \cdot c_n = t \cdot V^{-1/3},$$

$$t_{n=2} = H \text{ ,}$$

$$\tau_n = H * (0.675)^{-1/3} * 0.34^{-1.285/3} * H^{-1} = 1.81 \text{ m,}$$

$$\phi = \tau_n * (1 - \frac{P_f}{100}) = 0.87 \text{ .}$$

In 1982 Zwamborn and Van Niekerk (Ref. I13) did additional model tests on dolos packing density. Although the results from Zwamborn (1980) indicated that there is a 'optimum' packing density for $t_{n=2} = H$, it was clear from the variation in the results found by Zwamborn that it would be very difficult to achieve this optimum in practice. Therefore they advised it would be better to use ϕ = 1.0 as a minimum average relative packing density as a preliminary design value. The 13 % extra dolosse will ensure that, in practice, the minimum packing density achieved should nowhere be less than ϕ=0.87, accepting well-controlled placing techniques. Thinner dolos armour layers (ϕ = 0.65) are not regarded safe because of the very small reserve stability. Thicker dolos armour layers (ϕ = 1.3 to 1.7) would ensure a higher stability as well as a higher reserve stability. Such layers could be used effectively in heavily attacked parts of the breakwater, such as at the breakwater heads.

For the design of the New North Groyne a general package density of ϕ = 0.90 for the breakwater head will be used. This assumption (A 15) is based on the results of extra testing and on the experience from the breakwaters of the port of Coega (Ref. AH), where ϕ = 0.90

was used. The benefit is 10 % less dollose per square meter, which makes the armour layer around 10 % cheaper and faster to realize.

This results in the following armour layer dimensions for a 20 ton dolos:

$M = 20$	Mass	[tonne],
$w_r = 0.34 \cdot \left(\frac{M}{20}\right)^{1/6} = 0.34$	Waist ratio	[-],
$V = \frac{M}{\rho_s} = \frac{20}{2.4} = 8.33$	Block volume	[m^3],
$D_n = V^{1/3} = 2.03$	Equivalent cube diameter	[m],
$P_f = 51.5$	'Fictitious' porosity	[%],
$\phi = 0.90$	Packing density	[-],
$\tau_n = n \cdot c_n = \frac{\phi}{1 - \frac{P_f}{100}} = 1.86$	'Relative' armour layer thickness	[m],
$N_n = \phi_n \cdot V^{-2/3} = 0.219$	Number of units	[dolosse/m^2].

Volume of concrete per square meter:

$$Q = N_n \cdot V = 1.82 \text{ m}^3/\text{m}^2,$$

Total layer thickness: $t_n = \tau_n \cdot V^{1/3} = 3.76$ m,

Solid layer thickness: $t_s = Q = 1.82$ m,

Voids layer thickness: $t_v = t_n - t_s = 1.94$ m,

Porosity check: $P_r = t_v / t_n = 51.5$ %.

The surface of the New North Groyne, on which the dolosse will be placed, has a value of 1010 m^2. This follows from the toe dimensions section.

Number of dolosse to be placed:

$$Y = Surface \cdot N_n = 222 \text{ dolosse}.$$

To calculate the amount of required concrete, 2 formulas can be used:

$$C = Surface \cdot Q = 1843 \text{ m}^3,$$

$$C = Y \cdot V = 1850 \text{ m}^3.$$

The second calculation will be used, because the whole number of dolosse is used. This calculation has also been done for other masses of dolosse. The results can be found in table 8-4. The values of $\tau_{n=2} = 1.856$ and $P_f = 51.5$ are constant for every dolos mass.

M	H	w_τ	V	D_n	N_n	Q	t_n	t_s	t_v	Y	C
2	1.662	0.320	0.833	0.941	1.016	0.847	1.746	0.847	0.899	1027	856
5	2.255	0.320	2.083	1.277	0.552	1.149	2.37	1.149	1.221	558	1163
7.5	2.564	0.320	3.125	1.462	0.421	1.316	2.713	1.316	1.397	426	1332
10	2.842	0.320	4.166	1.609	0.348	1.448	2.986	1.448	1.538	352	1467
11	2.932	0.320	4.583	1.661	0.326	1.495	3.082	1.495	1.587	330	1513
12	2.999	0.320	5.000	1.71	0.308	1.539	3.173	1.539	1.634	311	1555
15	3.253	0.324	6.250	1.842	0.265	1.658	3.418	1.658	1.760	268	1675
20	3.556	0.340	8.333	2.027	0.219	1.825	3.762	1.825	1.938	222	1850
30	4.070	0.360	12.50	2.321	0.167	2.089	4.307	2.089	2.218	169	2113

table 8-4: dolosse comparison;

It is interesting to see is that an increase of the mass of the dolos-units results in a decrease of the number of required dolosse (Y) and an increase of the total volume of required concrete (C), this is illustrated in figure 8-5.

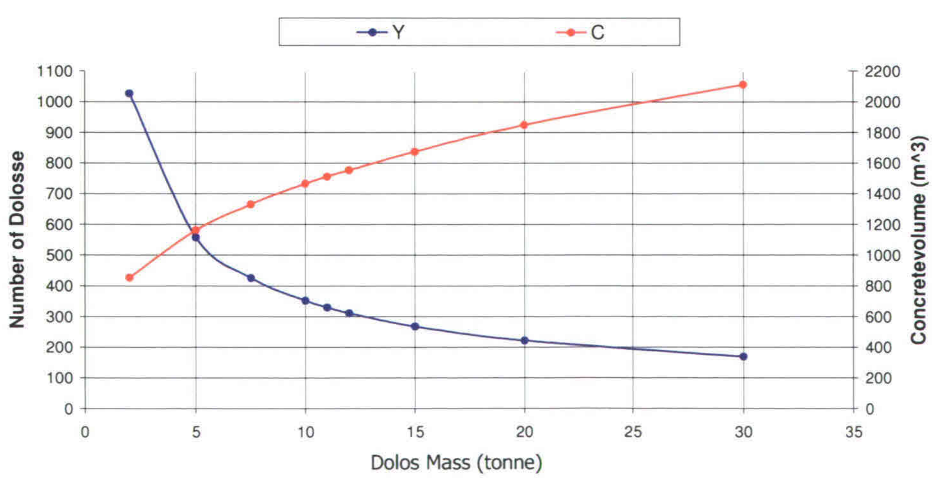


figure 8-5: dolos mass comparison;

The required armour units for the head of the New North Groyne will be 11 t dolosse. At the Southern Breakwater 20 t dolosse will be used for maintenance purposes. The existing crane on the Southern Breakwater, see figure 8-6, is suitable to do this work. This crane cannot be moved to the North Groyne. An alternative has to be found. Placing the dolosse using waterborne equipment is extremely difficult due to the present waves and will therefore not be done. A land based crane is preferred. Furthermore there are already a few moulds for the 20 t dolos present on the Southern Breakwater. Therefore it is interesting to see which type will be chosen as the optimal type of dolosse for the New North Groyne, the 11 t or the 20 t.

figure 8-6: 20 t crane on the Southern Breakwater;

11 t dolos

Using the 11 t dolos, an amount of 330 dolosse is required. Placing takes 15 minutes per dolos, so the total placing time will be approximately 83 hours. To construct the dolosse, moulds are needed. One mould costs about 50,000 rand (≈6,250 euro). Per mould one dolos can be constructed per day. Before placing, the dolosse have to cure for 28 days. For the construction of the dolosse, 1,513 m^3 of concrete is required. The cost of concrete is approximately 1,000 rand/m^3 (≈125 euro/m^3).

20 t dolos

The 20 t dolosse result in a heavier and more stable layer. This results in lower maintenance costs compared to the use of the smaller dolosse. The toe has to be thicker; this depends on the armour layer thickness. The radius of the head is larger, this ensures good interlocking. Due to the dolosse being larger than necessary, optimal interlocking should not be required. A number of 222 dolosse of 20 t is needed and 2 moulds for the 20 t dolosse are already available on the Southern Breakwater. The placing time is about the same for every type of dolos, about 15 minutes. The placing will take 56 hours. For the construction of the 20 t dolosse, 1,850 m^3 of concrete is required. Using 20 t dolosse requires 337 m^3 more concrete than using 11 t dolosse.

Comparison

Under normal circumstances the 11 t dolos is the best choice. The design wave has a large safety factor (7.3 ton dolosse required for the 3.5 m design wave), therefore no problems in maintenance will be expected. For the 20 t head, extra rubble (larger radius and toe) and an extra amount of 337 m^3 of concrete is required to construct the dolosse. The placing of the 11 t dolosse will take 27 hours longer, but it can be done with a smaller crane. More moulds are needed if is chosen to construct the 11 t dolosse in the same time as the 20 t dolosse, but if there is no time pressure this is not a problem. Using the 11 t dolosse the armour layer thickness will be 3.08 m.

Radius

For good interlocking and stabile positioning of the 11 t dolosse a radius of 10 meters for the roundhead will be sufficient (TR 20). Using the 20 t dolosse should result in a larger radius.

Underlayer

The layer directly under the armour layer is the first underlayer. It is obvious that the units forming this layer may not pass through the voids in the armour layer. The filter must be "geometrically impermeable". It is recommended that the weight ratio of subsequent layers of quarry stone be kept between 1/10 and 1/25 (D_{n50} ratio between 2 and 3). For the armour layer on the head 11 t dolosse will be used. Therefore rocks with a gradient between 300-1,000 kg will be required for the head of the groyne as an underlayer. In this way a stable connection to the trunk layer has been created, see paragraph 8.3.2, section under layer.

The thickness τ of the underlayer is calculated in the following way:

$$\tau = nk_t\left(\frac{W_{50}}{\rho_a}\right)^{1/3},$$

where,

τ	Thickness of the layer	[m],
n	Number of stones across the layer	[-],

k_t	Layer coefficient	[-],
W_{50}	Armour unit weight	[kg],
ρ_s	Density rock	[kg/m^3].

n [-]	k_t [-]	ρ_s [kg/m^3]	W_{50} [kg]	τ [m]
2	1.0	2650	650	1.25

table 8-5: layer thickness underlayer;

The underlayer on the head consists of 300 kg to 1,000 kg rock with a thickness of 1.5 m (1.25 + extra safety).

Toe

The toe berm is the lower support of the armour layer. The necessary depth beneath Mean Low Water Springs (MLWS = 0.21 m CD) of the armour layer is:

$$H_s \cdot 1.5 = 5.25 \text{ m.}$$

The result is a depth of –5.5 m CD (0.21 m CD – 5.25 "+" safety). In traditional literature, one finds a recommended weight that is equal to the first underlayer. Nowadays the weight is based on the toe's most important concern, the stability. This can be checked using the Van der Meer equation (1995):

$$d_{n50} = 0.625 \frac{H_s}{\Delta S^{0.15}} - 0.15 h_t ,$$

$$W_{50} = \rho_r d_{n50}^3 ,$$

where,

S	Damage value	[-],
h_t	Depth of the top of the toe	[m].

The results of the Van der Meer equation are shown in table 8-6.

H_s [m]	h_t [m]	S [-]	d_{n50} [m]	W_{50} [kg]
3.5	5.71	2	0.39	154

table 8-6: results Van der Meer equation, toe;

The recommended minimum weight of the rocks of the toe is 0.15 t. The toe will be constructed with the heavier 0.3 - 1.0 t rock resulting in an easier construction because this is the same rock that will be used for the underlayer. The toe starts on the underlayer with a slope of 1:1.5 opposite to the underlayer, for the layer thickness of the 11 t dolosse (3.08 m) a height of 1.86 m is needed. For extra safety the toe will start at –7.5 m CD with a slope of 1:1.5 up to –5.5 m CD. The berm will be stable for a width of $3.3 \cdot D_{n50} = 2.0$ m.

With the knowledge of the exact armour layer dimensions, the armour surface for the dolosse can be calculated. The head can be dimensioned as a cone with its top on +9⅔ m CD (z = 0). The dolosse are placed between –7.5 m CD (z = -17 1/6) and +3 m CD (z = -6). There will be no dolosse at the section where the trunk connects the head. This will be a part of 74 degrees, seen from the centre of the head. This results in a $74/360 \cdot 100 = 20.48$ % (pie piece) of the roundhead that will be the transition to the trunk (no dolos placing), this is illustrated in figure 8-7.

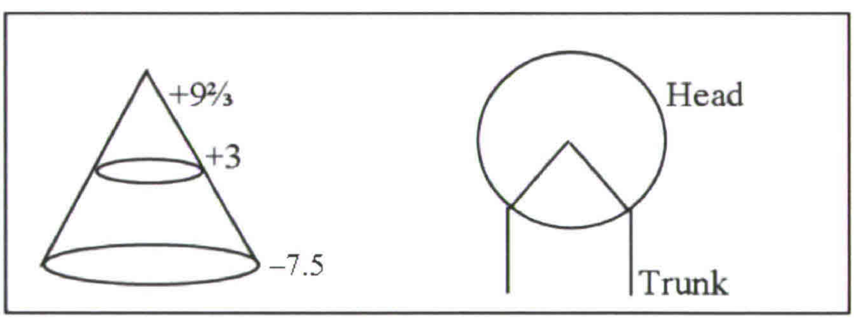


figure 8-7: armour surface calculation;

Roundhead (cone with its top on +9⅔ m CD):

$$O_r = \int_{6\frac{2}{3}}^{17\frac{1}{6}} 2\pi \cdot 1.5 \cdot z dz = 1179 \text{ m}^2.$$

Flat top:

$$O_f = \pi \cdot 10^2 - \pi \cdot 4^2 = 84 \text{ m}^2.$$

Total reduced armour surface:

$$O_t = 0.8 \cdot \left(O_r + O_f\right) = 1010 \text{ m}^2.$$

Core

In most cases, the material of the first underlayer is such that no extra filter layer is required for the transition to the core. Assuming a weight ratio between the first underlayer and the core between 1/10 and 1/25, results in a 10-60 kg core. For the core, a material called "quarry run" or "tout venant" is usually used, indicating that it is meant to represent the finer fractions of the quarry yield curve. The quarry run generally has a wide ($1.5 < D_{85} / D_{15} < 2.5$) to very wide ($D_{85} / D_{15} > 2.5$) grading. The use of large units in the core is not a problem from a stability point of view, although it does have a negative influence on wave transmission and sand tightness. For the core, the quarry run is used and rocks and old rubble removed from the existing North Groyne. Before it can be used for the core the concrete must be crushed to useful dimensions. This is an efficient and economical way of reusing the old material.

Filter

Specifically under the seaward toe large pressure gradients, that tend to wash out material from the seabed through the structure, may occur. Even extension of the core material under the toe berm may not guarantee the integrity of the structure as a whole. Loss of material in this region is an important threat to the stability of the armour layer. There are different examples in literature that show that substandard filters have caused the failure of a breakwater. It is therefore recommended that a geometrically impermeable filter should be placed under the seaward part of the groyne (weight ratio 1/10-1/25). This filter may consist of a number of layers of granular material or of a geotextile or other mattress. In this case a granular filter layer will be designed. The geotextile is often used to decrease the required filter height, which in this case is not necessary for the head. The pressure gradients under the centre of the structure are generally much lower. Here, the quarry run (10-60 kg) may act as a filter of sufficient quality. Since the layers of the granular filter are constructed at a considerable depth under water, it is necessary to construct the different layers with sufficient thickness to guarantee the presence of that particular material at any location. It will also be useful to ascertain the presence of the required material by inspection. In practice, this means that no layers thinner than 0.5 m will be designed. Under the 300-1000 kg underlayer a 0.5 m thick core layer (10-60 kg) will be placed. Under this core layer a 0.5 m thick layer of sea gravel will be placed, this is illustrated in appendix Q.

Scour protection

Just in front of the New North Groyne, the seabed may be eroded due to the concentration of currents. This can cause a loss of bed material directly in front of the toe, which may cause a geotechnical stability problem. No extensive research into the occurrence of current concentration has yet been done. Due to the extent of the expected damage, scouring should occur and due to the dredged slope on which the groyne is founded it is decided to place a 10 m long scour protection at the head of the groyne. It is assumed that a 10 m extension of the sea gravel and core layer filter is sufficient to protect the head from scouring (A 16).

8.3.2 Trunk

Armour layer

The armour layer of the trunk will be dimensioned with different data than used for the head. The 1:100 year design significant wave height of 3.5 m will not directly affect the trunk. The wave energy will dissipate on the head or the wave will run parallel to the trunk. The required armour for the trunk depends on three types of wave attacks: wave penetration, long shore transport and ship waves. These three types will be applied for a trunk slope of 1:1.5.

Wave Penetration

On the North Bank Revetment a H_s of 1.29 m was derived. For the trunk a H_s of 2.5 m is assumed (A 17), because there are no direct measurements for wave attack at this section. With this H_s, the required armour weight can be calculated using the Hudson and Van der Meer equations. The results of calculations using the Hudson equation can be found in table 8-7.

Hudson's equation:

$$W = \frac{\rho_s H_s^3}{K_d \Delta^3 \cot\alpha},$$

where,

W	Armour weight	[tonne],
H_s	Significant wave height at toe of structure	[m],
ρ_s	Density of armour material	[tonne/m^3],
Δ	Relative density	[-],
	$\Delta = \dfrac{(\rho_s - \rho_w)}{\rho_w}$,	
α	Slope angle	[°],
K_d	Reduction coefficient	[-].

H_s [m]	Armour Type	ρ_s [tonne/m3]	K_d	W [tonne]
2.5	Dolos	2.40	15.8	1.4
2.5	Rock	2.65	4.0	3.6

table 8-7: Hudson formula results, trunk;

The use of dolosse is not relevant, comparing the production costs and construction time for realisation on the whole groyne trunk of such small dolosse to the use of rock. An armour weight of 3-6 t rock is required.

Van der Meer

Van der Meer assumed the effect of the wave period to be linked with the shape and intensity of breaking waves. Therefore the Iribarren parameter is used:

$$\xi = \frac{\tan\alpha}{\sqrt{s}},$$

where,

s	wave steepness: $s = \dfrac{2\pi H}{gT^2}$	[-],
α	angle of the seaward slope of a structure	[°].

The Van der Meer equation also deals with a certain influence resulting from the permeability or porosity of the groyne structure as a whole:

$$\xi_{transition} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\left(\frac{1}{P+0.5}\right)},$$

$\xi > \xi_{transition}$ results in the use of the equation for surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\sqrt{\cot\alpha}\,\xi_m^P,$$

$\xi < \xi_{transition}$ results in the use of the the equation for plunging waves:

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

It must be noted that the value of H_s in the expression $\frac{H_s}{\Delta \cdot D}$ is measured at the location of the toe of the structure after elimination of any wave reflection. On the North Groyne, $\xi_{\text{transition}}$ = 4.42 and ξ = 6.32, it can be concluded that the waves on the trunk of the groyne are surging waves. The results of the surging waves calculations can be found in table 8-8.

H_s [m]	N [-]	S [-]	T_m [s]	P [-]	D_{n50} [m]	W [tonne]
2.5	3600	2	12	0.4	1.08	3.3

table 8-8: Van der Meer results, trunk;

Longshore transport

Longshore transport is not allowed at the trunk. The equations of Burcharth and Frigaard give a recommendation for oblique waves:

$$\frac{H_s}{\Delta D_{n50}} < 3.5.$$

For this formula, the wave that runs parallel to the trunk must be used, this is the H_s = 3.5 m.

H_s [m]	ρ_s [kg/m^3]	Longshore limit	D_{n50} [m]	W_{50} [tonne]
3.5	2650	3.5	0.72	0.7

table 8-9: Burcharth and Frigaard longshore transport, trunk;

Ship waves

The design ship entering the harbour has a width of s = 42.8 m and a draft of 14 m. The entrance is 19 m deep, the surface (b) is 340 m wide and the bottom is 220 m wide. The banks have a slope of 1:1.5. When the profile is not rectangular, but trapezoidal like this case, the width is equal to the width at the water surface, but the depth (h) to be used is $\frac{A_c}{b}$.

$$A_c = \left(\frac{220+340}{2}\right)\cdot 19 = 5320 \qquad \text{Waterway cross-section} \qquad [\text{m}^2],$$

$$h = A_c / b = 15.6 \qquad \text{Depth} \qquad [\text{m}].$$

The limit speed, V_l, is derived by iteration. This is the maximum speed for the channel. In the entrance the speed is restricted for safety reasons. We will calculate here with the maximum speed as a safety:

$$\frac{A_s}{A_c} - \frac{V_l^2}{2gh} + \frac{3}{2}\cdot\frac{V_l^{2/3}}{(gh)^{1/3}} = 1, \qquad \frac{A_s}{A_c} = 0.113.$$

where,

$$V_l = 6.7 \text{ m/s}.$$

The design speed then will be:

$$V_d = 0.9 \cdot V_l = 6.1 \text{ m/s}.$$

The water-level depression z is found by iteration:

$$\frac{V_d^2}{gh} = \frac{2\cdot z/h}{(1 - A_s/A_c - z/h)^{-2} - 1}, \qquad z = 1.17 \text{ m}.$$

It is assumed that the ship navigates minimal 60 metres from the bank (A 18). The resulting ship axis will be maximal 88 metres (y) out of the entrance channel centre. This leads to a stern wave of:

$$z_{\max} = 1.5 \cdot z_{ecc} = 1.5 \cdot (1 + \frac{2y}{b}) \cdot z = 2.67\,.$$

The return current (u_r) is also found by iteration:

$$\frac{u_r}{\sqrt{gh}} = \left[\frac{1}{1 - A_s / A_c - z / h} - 1 \right] \cdot \frac{V_d}{\sqrt{gh}}\,,$$

$$u_r = 1.58 \text{ m/s.}$$

Due to the eccentricity this becomes:

$$u_{r-ecc} = (1 + \frac{y}{b}) \cdot u_r = 1.99 \text{ m/s.}$$

The secondary wave height (H) is:

$$H = \xi \cdot h \cdot \left(\frac{s}{h} \right)^{-0.33} \cdot \frac{V_d^4}{(gh)^2} = 0.79 \text{ m,}$$

ξ is the coefficient of proportionality and represents the ship's geometry, ξ = 1.2 gives a reasonable upper limit. The Froude number is:

$$Fr = \frac{V_d}{\sqrt{gh}} = 0.49\,.$$

The wave period will be (with $c = V_d \cdot \cos\phi$ and $\phi = 35^\circ$):

$$T = \cos\phi \cdot V_d \cdot \frac{2\pi}{g} = 3.2 \text{ s.}$$

The next step is to determine the stone classes that can handle these loads. The stern wave in the canal requires:

$$d_{n50} = \frac{z_{\max}}{\Delta \cdot 1.8 \cdot \cot \alpha^{0.33}} = 0.82 \text{ m},$$

corresponding with $W_{50} = 1.46$ t.

For the return flow is required (with $\tan \phi = 1$):

$$d_{n50} = 1.2 \cdot \frac{u_r^2}{\Delta \cdot 2g \cdot \sqrt{1 - \frac{\sin^2 \alpha}{\sin^2 \phi}}} = 0.25 \text{ m}.$$

For the secondary waves the load becomes:

$$d_{n50} = \frac{H \cdot \sqrt{\xi}}{3.6 \cdot \Delta} = 0.24 \text{ m},$$

($\xi = \frac{\tan \alpha}{\sqrt{s}}$ is the Iribarren parameter, where $s = \frac{2\pi H}{gT^2}$).

It is obvious that the stern wave in this case dominates the shipload.

Choice

The different procedures give the following results, which can be found in table 8-10.

Type	Specification	Weight [tonne]
Wave penetration	Hudson	3.6
	Van der Meer	3.3
Longshore transport	Burcharth and Frigaard	0.7
Ship waves	Stern wave	1.5

table 8-10: different procedure results;

The wave penetration is governing in this situation. Rocks with a grading of 3–6 t are required for the armour layer of the trunk.

Thickness of armour layer

The thickness of the armour layer is calculated in the following way and can be found in table 8-11:

$$\tau = nk_t \left(\frac{W_{50}}{\rho_a} \right)^{1/3},$$

where,

τ	Thickness of the layer	[m],
n	Number of stones across the layer	[-],
k_t	Layer coefficient	[-],
W_{50}	Armour unit weight	[kg],
ρ_s	Density rock	[kg/m^3].

n [-]	k_t [-]	ρ_s [kg/m3]	W_{50} [kg]	τ [m]
2	1.0	2650	4500	2.4

table 8-11: armour layer thickness, trunk;

The armour layer will consist of 3-6 t rocks with a thickness of 2.5 m (2.4 + extra safety).

Underlayer

It is recommended that the weight ratio of subsequent layers of quarry stone should be kept between 1/10 and 1/25 (D_{n50} ratio between 2 and 3). For the armour layer 3-6 t rock will be used, resulting in the requirement of rocks for the underlayer with a gradient between 300-1000 kg. This connects to the underlayer of the head.

The thickness of the underlayer is calculated in the following way:

$$\tau = nk_t \left(\frac{W_{50}}{\rho_a} \right)^{1/3}.$$

n [-]	k_t [-]	ρ_s [kg/m3]	W_{50} [kg]	τ [m]
2	1.0	2650	650	1.25

table 8-12: armour thickness underlayer, trunk;

The underlayer on the trunk consists of 300 kg to 1000 kg rock with a thickness of 1.5 meters (1.25 + extra safety).

Toe

As derived for the head section the toe can be situated with its top at –5.5 m CD. The toe's most important aspect is the stability. This can be checked using the Van der Meer equation (1995) in the same way as has been done for the head section. In the head section the recommended weight for the toe was 0.15 t. The optimal method of construction is to use the same material as the underlayer for the toe. The toe will be constructed using 300-1000 kg rock. The toe starts at the underlayer with a slope of 1:1.5 opposite to the underlayer. For the layer thickness of the armour layer a toe height of 1.6 m is required. The berm will be stable for a width of $3.3 \cdot D_{n50} = 2.0$ m, this is illustrated in appendix Q.

Core

For the core, quarry run will be used combined with rocks and old rubble removed from the existing North Groyne, similar as the construction of the head. Before using it as core material the concrete must be crushed to useful dimensions. This is an efficient and economical way of reusing the old material.

Filter

The whole trunk will be founded on one general filter layer. This filter will have the same dimensions as the filter used for the head of the New North Groyne.

Scour protection

The requirement of a scour protection is only expected at the channel side slope because of currents caused by tides and passing ships. The same scour protection as applied at the head will be placed on the pre-dredged slope between the toe of the trunk and the channel.

8.4 Alternative 3: Pre-dredging of the head

The construction of the trunk of the New North Groyne will be followed by the removal of the existing North Groyne. After pre-dredging the head area, the head of the New North Groyne will be constructed.

8.4.1 Head

Because the head area will be pre-dredged, the design in this alternative will be the same as alternative 1.

8.4.2 Trunk

The difference with the trunk design in alternative 1 is that in this alternative almost no pre-dredging will be executed in the trunk area, only equalizing of the bottom is necessary. Therefore most of the trunk will be constructed in shallow water (-3 m CD to -1 m CD). Only the transition between the head and the trunk will be in deeper water. The transition starts at the head around –16 m CD and will proceed with a dredged slope of 1:10 for 130 m up to –3 m CD.

Armour layer

In the final situation the shallow trunk will be subject to the same loads as the trunk in alternative 1. Armour calculations have therefore the same results. The armour layer will consist of 3-6 t rocks and will have a thickness of 2.5 m (2.4 + extra safety).

Underlayer

It is recommended that the weight ratio of subsequent layers of quarry stone be kept between 1/10 and 1/25 (D_{n50} ratio between 2 and 3). For the armour layer 3-6 t rock will be used, resulting in the requirement of rocks with a weight between 300-1000 kg for the underlayer with a thickness of 1.5 m (1.31 + extra safety).

Channel slope protection

The trunk of the New North Groyne will be founded on the seabed at a level varying from –3 m CD to +0 m CD. The slope between the toe of the groyne and the channel bottom at –19 m CD has, in case of a foundation level of +0 m CD, a length of 45 meters. This results in a slope of approximately 1:2.4 for the trunk, this is illustrated in appendix R. This slope will not be stable under the occurring scouring conditions, see box 3. The slope must be 1:6 (TR 3). This slope will be stable under the tide and wave conditions in the channel. A revetment that basically consists of a top layer and a filter layer with protection at the toe will therefore protect the slope. Because the groyne is already constructed when the slope will be dredged and because the stability of the slope needs to be guaranteed, a regular revetment cannot be used. It is assumed that scouring will happen with such speed that damage to the construction will occur before a normal revetment is placed. Therefore an alternative revetment needs to be found.

The structure has to cover a rather large area and needs to be sand-tight but permeable to water. Moreover, it has to be stable under all prevailing flow and wave conditions. Five alternative revetments are discussed:

- Dredge and dump (reference alternative);
- Rolling protection;
- Floating Mattress;
- Launching apron;
- Falling Apron.

Each alternative will be briefly discussed and a choice will be made based on a Multi Criteria Analysis.

box 3: Scouring: the removal of soil particles by currents.

General scour: lowering of streambed elevation (also called bed degradation) over long reaches due to head cuts and changes in hydrology controls (such as dams, sediment discharge, or river geomorphology) is termed general scour. General scour often occurs during the passage of a flood, but is sometimes masked because sediments deposit to the original lines and grades on the falling stage of the hydrograph. General scour involves the removal of material from the bed and banks across all or most of the width of a channel. This type of scour may be natural or man-induced and requires geomorphic and sedimentation analyses to quantify. Analytical tools are helpful in evaluating long-term general scour.

Contraction Scour: Scour that results from the acceleration of the flow due to a contraction, such as a bridge, is called contraction scour. This type of scour also occurs in areas where revetments are placed such that they reduce the overall width of the stream segment. Contraction scour is generally limited to the length of the contraction, and perhaps a short distance up- and downstream, whereas general scour tends to occur over longer reaches.

Local scour: The scour that occurs at a pier, an abutment, an erosion control device, or other structure obstructing the flow is called local scour. These obstructions cause flow acceleration and create vortexes that remove the surrounding sediments. Generally, depths of local scour are much larger than general or contraction scour depths, often by a factor ten. Local scour can affect the stability of structures such as riprap revetments and lead to failures if measures are not taken to address the scour.

Dredge and dump (reference alternative)

The simplest version of a revetment is a geotextile construction with stones on top, see figure 8-8. A geotextile is rolled down from the bank with the assistance of a diver. A fascine roll tied to the lower end can give some stiffness to ease the process. For the considered depths (+0 m CD to –19 m CD) an alternative could be to let a pontoon pull a steel pipe which is reeled around with the geotextile down the slope. A side stone-dumping vessel then dumps stones on the geotextile, upward from the toe to prevent the whole geotextile sliding down. Another possibility is not to use a geotextile and simply dump different layers on the slope to create a closed filter. This alternative will make the revetment thicker and will require more material.

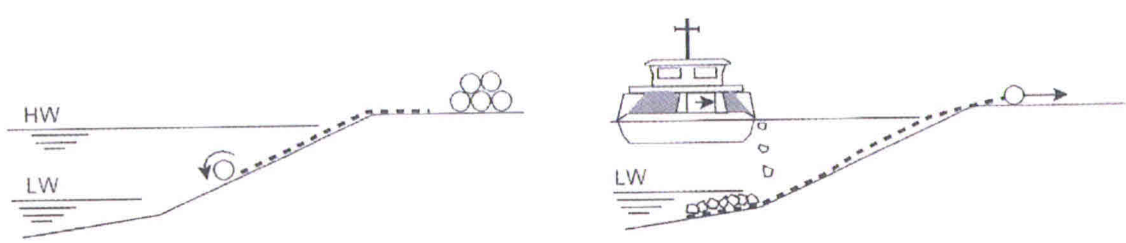


figure 8-8: dredge and dump;

Advantages:

- Simple method with lots of experience;
- Cheap method;
- Closed filter;
- Low maintenance.

Disadvantages:

- Large chance of instability of the groyne during slope dredging;
- Difficult to connect the geotextile;
- Difficult to place the geotextiles, 'suspending effect'.

Rolling protection

A mattress or block mat is constructed that contains a filter and armour layer, an example is illustrated in figure 8-9. This mattress or block mat is flexible enough to be rolled on a pipe on a vessel. It is connected to the toe of the groyne and winded off during dredging when the slope has reached the required steepness. This kind of slope/bottom protection was used during the constructing of the Eastern Scheldt Storm Surge Barrier in the Netherlands, see box 4.

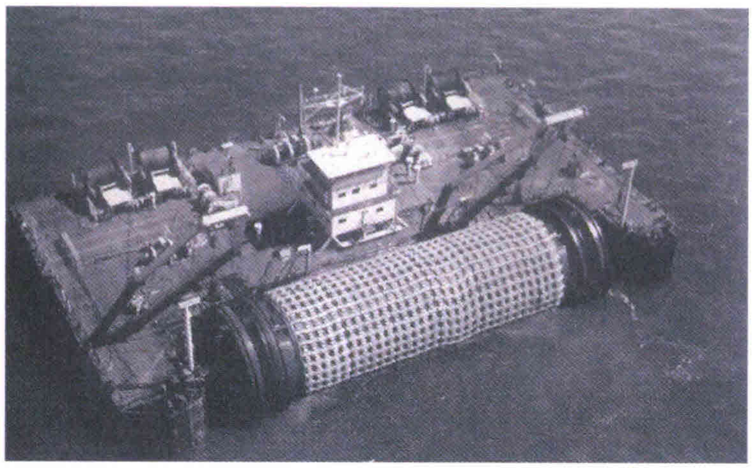


figure 8-9: sinking block mat in Eastern Scheldt;

Advantages:

- Limited filter height;
- Closed filter;
- Applicable on steep slopes;
- Experience in the Netherlands.

Disadvantages:

- Heavy loads on joints at toe;
- Vulnerable for structural damage;
- Specialized equipment required;
- Chance of instability of the groyne during slope dredging.

box 4: Rolling protection

A special type of bottom protection was designed for the Eastern Scheldt Storm Surge Barrier. The protection used in this case had to be of the granular-filter type and needed to be laid in a single operation per mattress. The three-layer filter was wrapped up in geotextile sheets that were kept in the correct shape by a steel mesh, just like gabions. Special equipment was built to handle these mats. This type of mattress was chosen for that part of the sill where loss of bottom material would cause structural failure. A geotextile alone was not considered to be safe enough in the design lifetime of 200 years.

Floating Mattress

The mattresses are floated to the site and then stretched out onto the bottom where they will be subsequently ballasted after connecting them to the toe, this is illustrated in figure 8-10. Each mattress overlaps its predecessor. In the past thick willow or bamboo mattresses were the only suitable structures. However, since the introduction of geotextiles adapted structural designs are used. The sand-tightness, formerly obtained by the thick layer of osiers can now be acquired by using a geotextile sheet. This modern version of bottom protection is based on splitting up the functions:

1. Strength is obtained by using geotextile of the desired material;
2. Sand-tightness is obtained by using geotextile of the correct mesh;
3. Floatation is obtained by adding willow or bamboo bundles (as the old way), or a floating pipe;
4. Rigidity during the sinking operation is obtained by adding one willow or bamboo grating as compared to two in the old way.

A considerable quantity of ballast is needed to fix the sheet in position. This is usually provided after the sheet has been initially ballasted during the placement operation. Initial ballasting is used to sink the sheet when the protection is floated in place. In addition, this keeps the sunken or unrolled sheet on the bottom after sinking. This initial ballast has to be followed immediately by part of the final ballast to secure the protection against the next flow attack. The final ballast should remain stable on the sheet and should not roll or slide away.

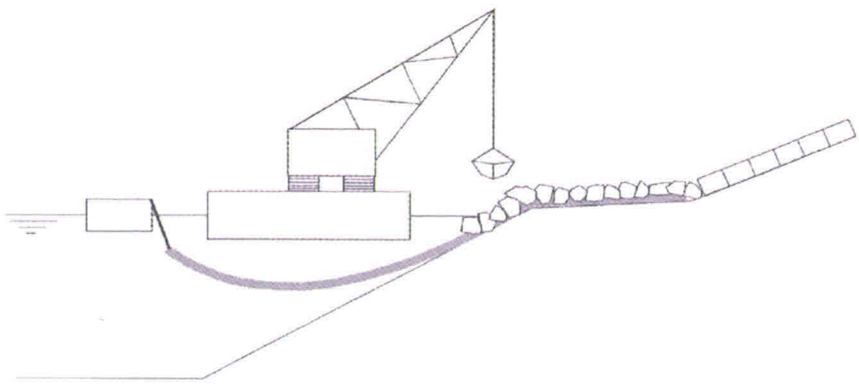


figure 8-10: floating mattress;

Advantages:

- Limited filter height;
- Closed filter;
- Experience;
- Less expensive equipment required.

Disadvantages:

- Heavy loads on joints at toe;
- Chance of instability of the groyne during slope dredging.

Launching apron

Another option is to cover the toe of the revetment using a flexible mattress, which can follow the scour slope and thus retain the bed material, this is illustrated in figure 8-11. After dredging a steep slope scouring causes the apron to "launch" itself on the slope. The mattresses are made of geotextile for sand-tightness, wire netting for strength and concrete blocks or gabions as ballast.

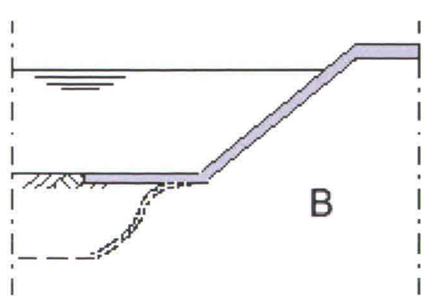


figure 8-11: launching apron;

Advantages:

- Closed filter;
- Relatively cheap method;
- Small risk of damage if correctly executed.

Disadvantages:

- Not much experience with the height differences as in the NNG case;
- Heavy loads on joints at toe;

Falling Apron

The 'falling apron' is often applied in rivers in India and Bangladesh. The falling apron is a toe protection that can adjust to scour and follow any bed erosion downward, thus continuing to protect the toe of the bank protection. The idea is to store an amount of stones at the toe of the revetment during construction. When the construction is finished an instable steep slope, starting at the channel boundary, will be dredged. When the scour starts to develop and the slope is getting less steep (eroding), the slope reaches the stones. When it runs under, the material is launched onto the developing slope. There is a risk of damage for the dredging equipment and workers when the falling starts when the dredger is still operating. The loose elements are assumed to cover the slope to a thickness large enough to retain most of the bed material. Maintenance is required because the created protection is an open filter.

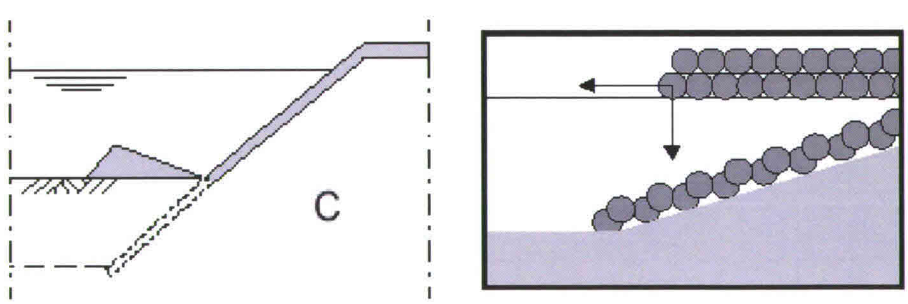


figure 8-12: falling apron;

Advantages:

- Simple method;
- Experience in India and Bangladesh;
- No joints required;
- Small risk of damage if correctly executed.

Disadvantages:

- No experience in coastal areas;
- Large amount of rock required to ensure safety;
- Maintenance required.

Multi Criteria Analysis

This method is comparable to the one used in chapter 7. A number of criteria are specified:

- Safety:
  - Damage to the groyne: Scouring and Loads on joints;
  - Damage to dredging equipment.
- Experience with method;
- Equipment and material cost:
  - Type of equipment;
  - Type of material;
  - Volume of material.
- Maintenance required.
  - Due to materials used;
  - Due to outflow of sediment.

The criteria are rated, see chapter 7, the resulting weight factors are shown table 8-13.

	Safety	Experience with method	Equipment and material costs	Maintenance required	Total	Awarded weight factor
Damage to the groyne		1	1	1	3	**4**
Experience with method	0		0	1	1	**2**
Equipment and material costs	0	1		1	2	**3**
Maintenance required	0	0	0		0	**1**

table 8-13: criterion weight factors;

The stability and safety of the groyne is considered paramount to all other criteria. The results of the MCA are shown in table 8-14.

	Safety	Experience with method	Equipment and material costs	Maintenance required	Total
	4	**2**	**3**	**1**	
Dredge and dump	0	5	4	4	**26**
Rolling protection	2	2	1	3	**18**
Floating Mattress	2	4	2	3	**25**
Launching apron	3	2	3	3	**28**
Falling apron	3	3	4	1	**31**

table 8-14: alternative scores;

The falling apron alternative is, based on the Multi Criteria Analysis, chosen as the best option for the revetment of the channel side slope of the trunk scoring 31 points. The second option is a launching apron, scoring 28 points. It should be noted that instability of the New North Groyne as expected for the dredge and dump alternative is not acceptable at all. This is not reflected in the MCA but should be considered when another alternative than the falling apron is chosen. This note holds, in a lesser extend, also true for the rolling protection and floating mattress alternatives.

Toe

The dimension of the toe for the channel side depends on the falling apron. In chapter 11 the design for the combination of the toe and the falling apron will be made more detailed. It is assumed that the armour layer will continue as a falling apron with a slope of 1:2 and a thickness of 1.5 m. The toe on the north side of the groyne will be constructed mostly in shallow water (-3.5 m CD to 0 m CD). The 50 metres transition part from the head is in deeper water (-8.5 m CD to –3.5 m CD). It is decided to construct a toe that also has the filter function. Because of the small depths, there will be no space for the standard design, a granular filter under the toe. Due to the thick filter bed, the level of the toe berm becomes too high. Therefore the under layer will be constructed as a shoulder instead of as a toe and will extend 2 metres under the armour layer. Under the 1.0 m thick underlayer and for 2.0 m under the core, a geotextile will be placed as a filter. The other part of the core will not need a filter construction. In the deeper water at the head part, the toe starts from the underlayer with a slope of 1:1.5 opposite to the underlayer. The layer thickness of the armour layer requires a toe height of 1.6 m. The berm will be stable for a width of $3.3 \cdot D_{n50}$ = 2.0 m. The toe will also be placed on a 1.0 m thick underlayer of the same material and with a geotextile. This is illustrated in appendix R.

Core

The core will use quarry run combined with rocks and old rubble from the existing North Groyne, similar as the construction of the head. Before using it as core material, the concrete must be crushed to useful dimensions. This is an efficient and economical way of reusing the old material.

Scour protection

The requirement of a scour protection is only expected on the channel side. The falling apron will function as a scour protection in this part.

8.5 Alternative 4: No pre-dredging

This alternative consists of the complete construction of the New North Groyne without pre-dredging. The head area will be followed by the complete removal of the existing North Groyne.

8.5.1 Head

The head of the New North Groyne will be constructed without pre-dredging at a level of approximately -5 m CD.

Armour layer, underlayer and core material

The conditions in the final situation are the same for all the alternatives, therefore the same armour layer of 11 t dolosse is required. This also results in a similar underlayer construction and core material as the other alternatives.

Armour layer and falling apron

In the construction method chapter, chapter 7, it was proposed to start with the construction of the apron. After collapsing of the apron, the dolosse can be placed. Because calculations have determined the required armour depth at -5.5 m CD, it is possible to place the dolos armour before the existing North Groyne is removed. Under the base of the armour layer, a 2 m high toe and a 1 m underlayer using a geotextile construction are required as foundation and filterlayer. A depth of –8.5 m CD has to be dredged in the head area. The radius of the head at –8.5 m CD will be 35.3 m (radius + slope + toe = 10 + (3+8.5)*1.5 + 8). The stable slope to dredge will be 1:10 (A 14), this has to be done to create a height difference of 4.5 metres (from –8.5 m CD up to –4 m CD). Therefore the required distance will be 35.3 + 45 = 80.3 m. The present depth is around –5 m CD and the distance from the centre of the head to the existing groyne at a depth of –4 m CD is 90 m. This distance is sufficient for the dredging work that has to be done to place the armour layer. This solution is far easier to realise and more safe to construct than placing the dolosse on a fallen apron. A falling apron is still required to protect the channel slope against scouring.

Slope protection

The head will be founded at a level of –8.5 m CD. The slope between the toe of the groyne and the channel bottom at –19 m CD has a length of 25 meters. This results in a slope of 1:2.4. Similar to the trunk this constructed slope will not be stable under scouring conditions, see box 3. The slope must be 1:12 (TR 3), this is a stable slope. Based on the similar situation and need of connection between the head and the trunk revetment, it is decided to use both for the head as for the trunk the same type of apron: the falling apron.

8.5.2 Trunk

Because the trunk area will not be pre-dredged in this alternative, the design will be almost the same as alternative 3. The only difference is that the transition area between the head and the trunk will be smaller. The transition starts at the head around –8.5 m CD and will proceed with a dredged slope of 1:10 for 50 m up to –3.5 m CD. This is illustrated in appendix S.

8.6 Multi criteria analysis

The alternatives are judged on the same criteria as proposed in chapter 7. In this paragraph the awarded scores of the construction method chapter will be reconsidered on basis of the design. Assumptions were made that now can be proved or disproved, which results in a corrected score chart and a optimal alternative.

The only influencing change made to the designs, is the required deepening of the head area to – 8.5 m CD in alternative 4. In this way the armour layer does not have to be placed on the fallen falling apron. Because of the changed armour placement, alternative 4 scores on two points:

- **Damage chance:** during construction the groyne is much safer because the head armour does not have to be placed on the fallen falling apron (from 2 to 4 points);
- **Construction time:** the dolosse can be placed direct, without waiting on the fall of the falling apron (from 3 to 4 points).

The new score of the different alternatives can be found in table 8-15.

	Construction costs	Environmental	Shipping safety	Damage chance	Construction time	Total
	3	**2**	**5**	**4**	**1**	
Alternative 1	0	3	3	5	0	**41**
Alternative 3	3	3	4	4	4	**55**
Alternative 4	4	2	5	4	4	**61**

table 8-15: alternative scores;

8.7 Conclusion

Alternative 4 has the best score with 61 points. Alternative 3 is the second best construction method with 55 points. From this, alternative 4 is chosen as the best alternative for the detailed design. AutoCAD drawings of alternative 4 can be found in appendices S and T.

8.8 Recommendations

No sufficient data is available on the occurring currents in the channel at the location of the head and trunk of the New North Groyne. For verification of the chosen dimensions it is recommended to obtain this information.

9. WAVE STRUCTURE INTERACTION

9.1 Introduction

There is a strong interaction between a wave and a wave damping structure like a breakwater. This interaction is visible in front of the structure (reflection), on the slope of the structure (run-up) and behind the structure (overtopping and transmission). Even if a breakwater structure is stable under the action of waves, there is interaction between the structure and the wave field near the structure. Various phenomena that lead to different wave patterns in the vicinity of the structure are discerned:

- Wave reflection;
- Wave run-up;
- Wave overtopping;
- Wave transmission.

Before using any of the expressions given in this chapter, it is useful to analyze which phenomenon influences the design problem in question.

The aim of this chapter is to verify the design of the New North Groyne on a selection of wave structure interaction phenomena.

9.2 Wave reflection

9.2.1 Seelig

Coastal structures reflect some portion of the incident wave energy. If reflection is significant, the interaction of incident and reflected waves can create an extremely confused sea with very steep waves that are often breaking. This is a difficult problem for many harbour entrance areas where steep waves can cause considerable manoeuvring problems for smaller vessels. Strong reflection also increases the seabed erosion potential in front of protective structures. Waves reflected from some coastal structures may contribute to erosion of adjacent beaches.

The measure in which the waves are reflected of the groyne is of importance for the pattern and the intensity of the wave motion in the surrounding area. The measure of the reflection coefficient K_r is defined as relationship between the height of the reflected wave and the height of the incoming significant wave height.

9.2.2 Wave reflection coefficients for sloping structures: head-on waves

The bulk reflection coefficient for straight non-overtopped impermeable smooth slopes and conventional rubble-mound breakwaters can be estimated from the following equation (Seelig 1983),

$$K_r = \frac{a\xi^2}{\left(b+\xi^2\right)} \quad \text{and} \quad \xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{gT_p{}^2}}}.$$

For irregular waves H is replaced by H_s and T is replaced either by $T_p\left(\xi_{op}\right)$ or $T_m\left(\xi_{om}\right)$.

The New North Groyne is in an area with irregular waves. Therefore T_p is used for the calculations. The input and the results of the calculation are given in the table 9-1.

$\tan\alpha$	H_s [m]	T_p [s]	ξ_{op} [-]	a [-]	b [-]	K_r [-]
0.67	3.5	15	6.679	0.56	10	0.4575

table 9-1: results Seelig calculation;

From table 9-1 follows,

$$K_r = H_r / H_I = 0.458,$$

$$H_r = 1.60 \text{ m}.$$

Using the Seelig (1983) equation the reflected wave height is 1.6 m. The calculations can be evaluated with figure 9-1 this shows the fitting of the model test results by Allsop and Hettiarachichi (1988). For the slope 1:1.5 with ξ_{op} it follows that K_r = 0.45. This is close to the calculated values. In the calculation overtopping is not taken into account. This means that some of the water will not be reflected because it went over the crest.

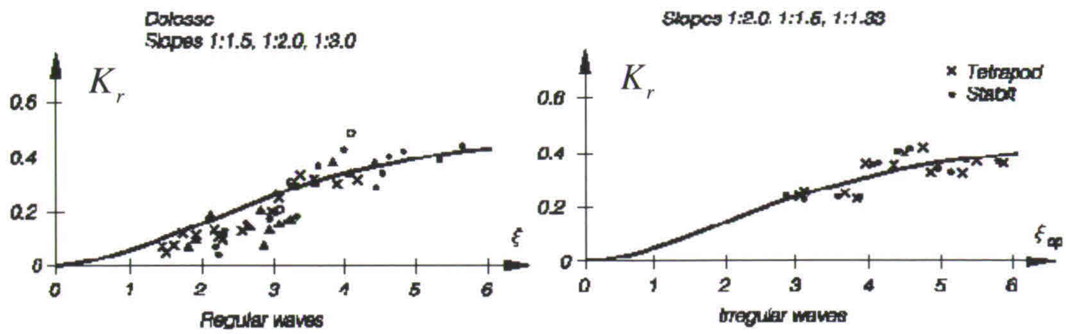


figure 9-1: reflection coefficients for slopes with concrete armour units. Head-on waves (Allsop and Hettiarachchi 1988);

9.2.3 Cress

In the Cress program calculations are made for wave reflection on slopes. The angle of the slope is α = 34 degrees. The calculation rule gives the reflection coefficient for various structures: smooth impermeable slopes, slopes with rubble or slopes with concrete elements. The used reflection rule is for the slope with concrete elements as on the head of the New North Groyne.

Parameter	Value
H_s [m]	3.5
T_p [s]	15
Slope α [°]	34

table 9-2: reflection slope, Cress input;

Cress uses the following equation to calculate the reflection coefficient, using the values of table 9-2,

$$K_r = \frac{0.5 + \xi_{0p}^2}{8 + \xi_{0p}^2},$$

where,

L_{0p} Deep water wave length based on peak period [s^2],

H_{sr} Reflected significant wave height [m].

The results are shown in table 9-3.

L_{0p} [s²]	ξ [-]	K_r (concrete elements) [-]	H_{sr} (concrete elements)[m]
351	6.18	0.41	1.45

table 9-3: results Cress calculation wave reflection;

The Cress program makes calculations using 3 parameters and varying permeability of the structure. The Iribarren number is between the values of ξ_{op} and ξ_{om}. The resulting reflected wave height is 1.45 m, see table 9-3.

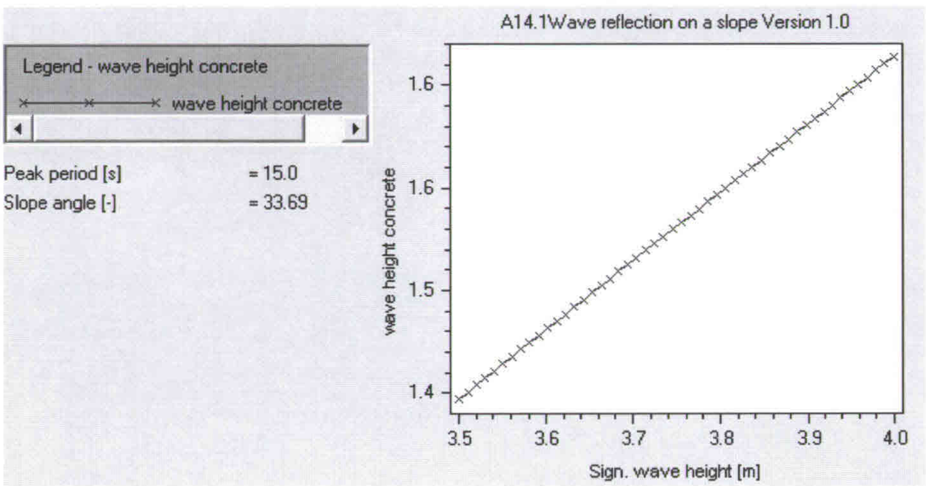


figure 9-2: varying the wave height in Cress;

In figure 9-2 the significant wave height has been varied between 3.5 m and 4.0 m. The resulting reflected wave height varies between 1.4 m and 1.7 m. For the calculation H_s = 3.5 m is used.

9.2.4 Postma [1989]

Postma has investigated the reflection from infinitely long rock slopes. He found a clear influence of the breaker parameter ξ and the "permeability" P as defined by Van der Meer. Postma (1989) gives the accurate approach in the formula:

$$K_r = 0.081 P^{-0.14} \cot \alpha^{-0.78} s_{op}^{-0.44}.$$

The formula can only be used within the validity range of the various parameters as shown in table 9-4.

Parameter	Range	Value	Range
P	0.1 <	0.5	< 0.6
cot α	1.5 <	1.5	< 6
s_{op}	0.004 <	0.0099	< 0.06
ξ_{op}	0.7 <	6.68	< 8
K_r	0.1 <	0.51	< 0.8

table 9-4: parameters for Postma (1989) equation;

The Postma (1989) equation is valid for use on the head of the New North Groyne. The calculation results are given in table 9-5. The Postma equation has as result of K_r = 0.768 giving the reflected wave a height of 2.7 m.

s_{op} [-]	cot α	P [-]	K_r [-]	H_i [m]	H_r [m]
0.0099	1.5	0.5	0.77	3.5	2.69

table 9-5: Postma (1989) calculation results;

9.2.5 Summary

Using the different ways for calculating the reflection there is a difference between the calculated reflected wave heights. Non-overtopped impermeable smooth vertical walls reflect almost all the incident wave energy, whereas permeable, mild slope, rubble-mound structures absorb a significant portion of the energy. Structures that absorb wave energy are well suited for use in harbour basins. In table 9-6 all the results of the reflection calculations are given. The results differ between 1.45 m and 2.7 m. The Seelig and Cress value were considered to be true instead of the high Postma value. The values are calculated with storm conditions. It is assumed that hindrance to ships and extra erosion is neglectable with the calculated values.

Seelig (1983)	Cress	Postma (1989)
$K_r = 0.45$	$K_r = 0.41$	$K_r = 0.768$
$H_r = 1.6$ m	$H_r = 1.45$ m	$H_r = 2.7$ m

table 9-6: wave reflection;

9.3 Wave run-up

9.3.1 SPM

Wave run-up is the phenomenon in which an incoming wave crest runs up the slope to a level that may be higher than the original wave crest. The vertical distance between still water level MSL and the highest point reached by the wave tongue is called the run-up z. From this definition, it is clear that we can only speak of run-up when the crest level of the structure is higher than the highest level of run-up. Run-up figures are mainly used to determine the probability that certain elements of the structure will be reached by the waves. In most cases, the 2 % run-up is given: the run-up level that is exceeded by 2 % of the incoming waves.

The head of the New North Groyne has a rough slope (dolosse) and a permeable core (P =0.4) the following equations for run-up are used,

$$z_{u\%}/H_s = D \qquad \text{for} \qquad (D/B)^{1/C} \le \xi_{om} < 7.5\,,$$

where,

$z_{u\%}$	Run-up level exceeded by u % of the incoming waves	[m],
H_s	Significant wave height	[m],
B, C, D	Parameters SPM 1984	[-],
ξ_{om}	Iribarren number (mean wave)	[-].

The equation is suitable for locations where the following formula is true:

$$\xi_{om} = 5.343 \; (D/B)^{1/C} \le \xi_{om} < 7.5\,,$$

$$3.2907 \le \xi_{om} < 7.5 \qquad \text{and} \qquad z_{u\%}/H_s = D\,.$$

The equation for the run-up is suitable for this situation,

$$z_{ui\%}/H_s = D \quad \text{and} \qquad z_{u\%}/H_s = 1.97\,.$$

Run-up parameters for rubble covered and permeable slopes are given in table 9-7.

H_s [m]	Run-up [%]	D [-]	$z_{u\%}$ [m]
3.5	0.1	2.58	9.03
3.5	1	2.15	7.525
3.5	2	1.97	6.895
3.5	5	1.68	5.88
3.5	10	1.45	5.075
3.5	Sign.	1.35	4.725
3.5	Mean.	0.82	2.87

table 9-7: run-up parameters for covered and permeable slopes;

Reduction coefficients,

γ_{sr}	Surface roughness	= 0.5,
γ_b	Berm	= 1.0,
γ_{sw}	Shallow water	= 1.0,
γ_{wa}	Wave angle	= 1.0.

This results in,

$$z_{uR} / H_s = \left(z_{u,2\%} / H_s \right) \gamma_{\substack{surface\\roughness}} \gamma_{berm} \gamma_{\substack{shallow\\water}} \gamma_{\substack{wave\\angle}} = 0.985 = D,$$

$$z \approx H_s = 3.5 \text{ m}.$$

The New North Groyne cap-block is placed 4.0 m above CD (2.89 m + MSL). Run-up will occur with the significant wave height. If the groyne would be constructed higher, run-up will still take place until the height is 7.0 m.

9.3.2 Cress

In the Cress program calculations are made with wave run-up on sloping structures. The input is shown in table 9-8.

Parameter	Value
H_s [m]	3.5
T_p [s]	15
n_1 slope [cot]	1.5
n_2 slope [cot]	1.5
B berm width [m]	2
D depth [m]	20.1
n_b slope berm [cot]	1.5
r roughness [-]	0.5
D_h depth berm [m]	6
P probability exceedance [%]	2-50

table 9-8: run-up input parameters Cress;

The probability of exceedance is varied between 2 and 50 %, see figure 9-3. The calculation results for 2 % exceedance are shown in table 9-9.

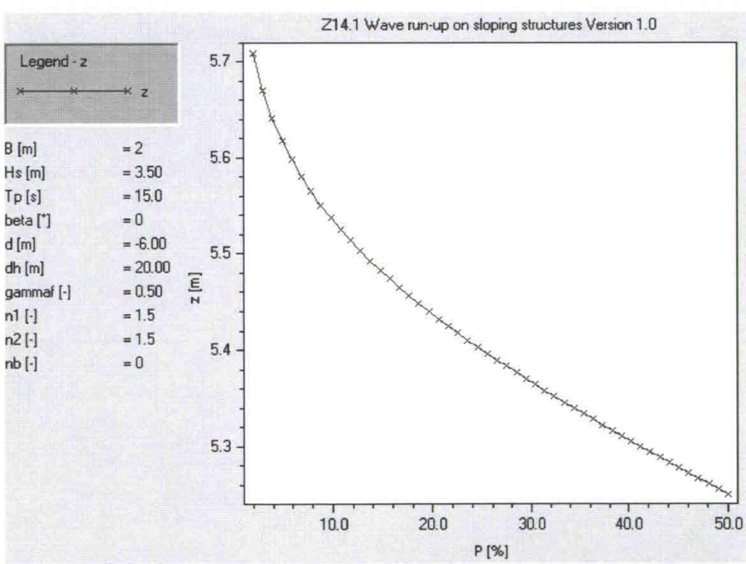


figure 9-3: exceedance probability varied in Cress;

Z [m]	ξ_{op} [-]	γ [-]	U_{max} [m/s]
5.71	6.68	0.5	15

table 9-9: results run-up calculation Cress;

9.3.3 Summary

Run-up of the New North Groyne takes place with the significant wave height. The crest of the groyne is 4.98 m above MSL. Because there is significant run-up on the slope, the groyne will be overtopped. The overtopping will be discussed in the following paragraph.

9.4 Wave overtopping

9.4.1 Introduction

Overtopping is defined as the quantity of water passing over the crest of a structure per unit of time. Because this quantity of water is over the length of the structure, it is expressed as a specific discharge per unit length [m^3/s/m].

When designing breakwaters the quantity of overtopping is important to determine the safety of people or installations on the crest of the breakwater. Although the assessment of these sometimes-subjective risks on the basis of model experiments is difficult, it is possible to derive a trend.

Wave overtopping occurs when the highest run-up levels exceed the crest freeboard R_c as illustrated in figure 9-4. The amount of allowable overtopping depends on the function of the particular structure. Certain functions put restrictions on the allowable overtopping discharge. For example berths for vessels and pedestrians on the groyne are overtopping design considerations. Overtopping is allowed because the groyne has the functions to protect the harbour entrance against waves, retain the sand behind the groyne. The groyne will be accessible for pedestrians.

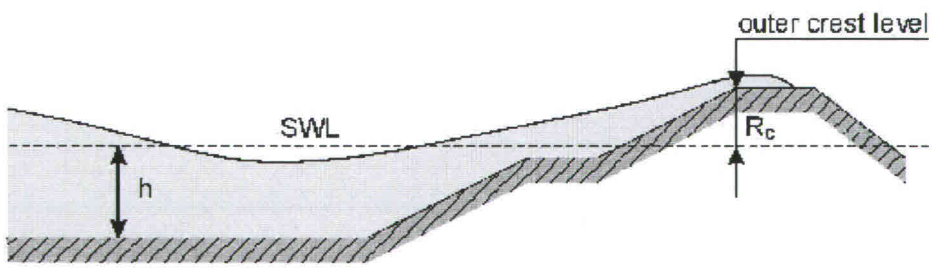


figure 9-4: overtopping;

9.4.2 Methods to calculated overtopping

There are various model studies preformed to simulate overtopping. As the conditions governing wave run-up, the quantity of overtopping is also largely influenced by the nature of the outer slope of the structure. An added effect is the influence of the shape and nature of the crest. Therefore, it is not possible to derive a generally applicable formula for overtopping. To estimate the overtopping the following methods will be used for general approximation:

- Bradbury (1988);
- Owen (1980);
- PC-overslag;
- Cress.

9.4.3 Bradbury

Bradbury (1988) is valid for structures without a crown wall and similar to figure 9-4. The equation is,

$$Q^* = a\left(R^*\right)^{-b},$$

where,

R^*	Crest freeboard $R^* = \left(\frac{R_c}{H_s}\right)^2 \sqrt{\frac{s_{om}}{2\pi}}$	[-],
Q^*	Specific discharge $Q^* = \frac{Q}{\sqrt{gH_s^3}} \sqrt{\frac{s_{om}}{2\pi}}$	[-],
R_c	Crest freeboard relative to SWL	[m],
H_s	Significant wave height	[m],
s_{om}	Deep water wave steepness	[-],
Q	Specific discharge	[m^3/s/m],
a,b	Empirical coefficients	[-].

In the table 9-10 the values are given for the New North Groyne.

R_c [m]	H_s [m]	s_{om} [-]	a [-]	b [-]	R^* [-]	Q^* [-]	Q [m^3/s/m]
4.89	3.5	0.015	$3.7.10^{-9}$	3.82	0.0954	$2.9.10^{-5}$	0.0123

table 9-10: overtopping: Bradbury (1988);

The calculated amount of overtopping using the Bradbury (1988) method is 12.3 l/s/m. This is a first approximation for the overtopping at the New North Groyne.

9.4.4 Owen (1980)

Owen (1980) produced typical values of the roughness coefficient based upon the relative run-up performance of alternative types of construction. These coefficients were originally derived for simple slopes but can also be conservatively applied to bermed slopes and roughness and armoured seawalls mean overtopping discharge.

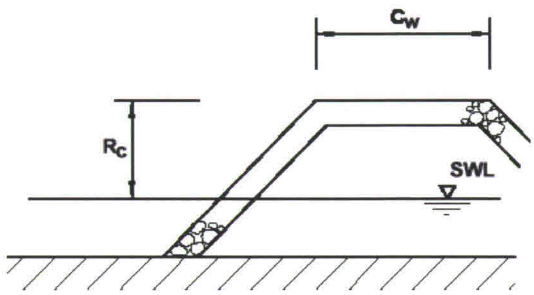


figure 9-5: overtopping crest width;

Owen's model uses the following equations to calculate the mean discharge,

$$R_* = R_c / T_m \left(gH_s\right)^{0.5},$$

$$Q_* = A\exp\left(-\frac{BR_*}{r}\right),$$

$$Q = Q_* T_m g H_s,$$

where,

T_m	Mean wave period at the toe of the seawall	[s],
r	Roughness coefficient	[-],
g	Gravitational constant	[m/s^2],
A, B	Empirical coefficients dependent on the cross-section of the seawall	[-].

The input and output values are shown in table 9-11.

R_c [m]	H_s [m]	T_m [s]	r [-]	A [-]	B [-]	R^* [-]	Q^* [-]	Q [m^3/s/m]
4.89	3.5	12	0.5-0.55	0.0088	19.9	0.0695	0.00071	0.293

table 9-11: input and output overtopping Owen (1980);

Owen's (1980) equations are valid for $0.05 < R^* < 0.30$. $R^* = 0.06954$ therefore the Owen (1980) equation can be used for the overtopping calculation. When a permeable crest is taken into account, reduction factor C_r is determined as follows,

$$C_r = 3.06\exp\left(-1.5\frac{C_w}{H_s}\right),$$

where,

C_w Crest width [m].

Note, when $\frac{C_w}{H_s} < 0.75$, C_r is assumed to be 1.

C_w [m]	H_s [m]	C_r [-]	Q [m^3/s/m]
20	3.5	0.00058	0.000169

table 9-12: reduced overtopping Owen (1980);

The amount of overtopping using the Owen (1980) equation is 0.17 l/s/m. This is a very low value. The reduction factor for the permeable crest is calculated far too low. The crest is only partial permeable. Water will therefore overtop instead of flow through the core.

9.4.5 PC overtopping

PC-overtopping is developed based on the technical rapport Wave run-up and wave overtopping on dikes (Ref. I14). The program uses several equations to calculate overtopping. The program is designed for dikes but the profile can be varied to match the profile of the groyne. In this case the input for PC-overslag is shown in table 9-13.

H_{m0} [m]	$T_{m-1.0}$ [s]	β [°]	SWL [m]	T_{sm} [s]	T_m [s]
3.5	13.63	0	20.1	43200	12

table 9-13: PC-overslag wave overtopping input;

where,

H_{m0}	Significant wave height	[m],
$T_{m-1.0}$	Spectral wave period	[s],
β	Angle wave attack	[°],
SWL	Sea water level	[m],
T_{sm}	Storm duration	[s],
T_m	Average wave period	[s].

The profile of the head of the New North Groyne is also required as input and shown in figure 9-6.

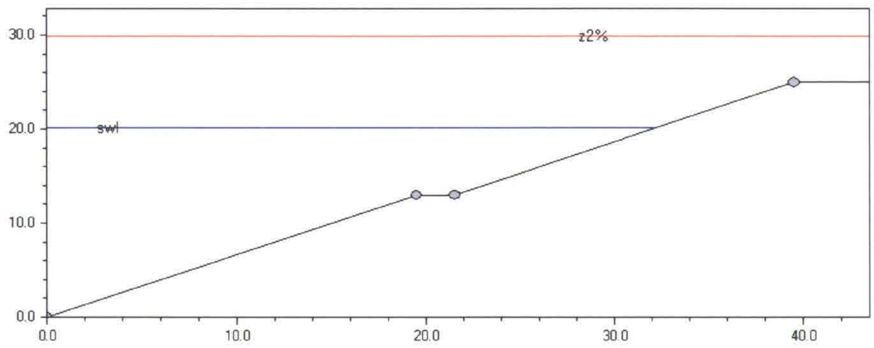


figure 9-6: profile of the New North Groyne for PC-overslag input;

In the figure 9-7 the measured points are given for different studies into wave run-up. The 2 % wave run-up is plotted against the Iribarren parameter. The line drawn is the average wave run-up for straight smooth slopes. The points below the line are reduced with reduction factors. These reduction factors are based on the width of the berm and the roughness of the slopes. The calculated situation is shown as a circled point in figure 9-7. The location shows that the calculated value is not closely related to the population of the researched situations.

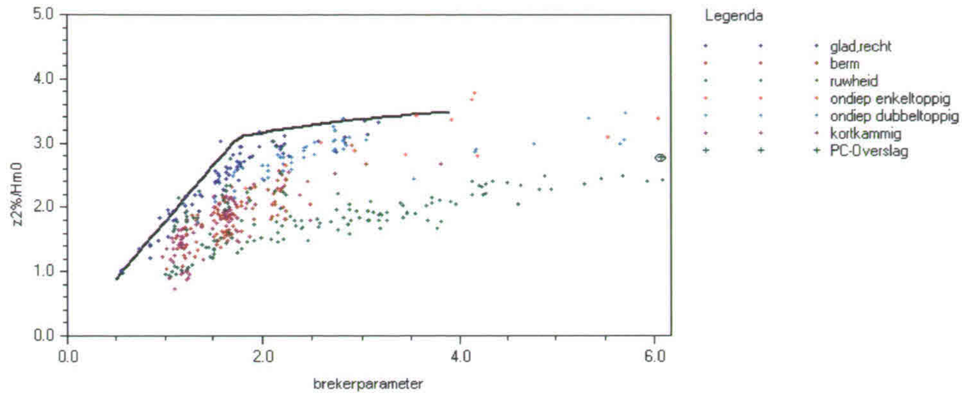


figure 9-7: wave run-up, circled point at breaker parameter 6.0;

The data shown in figure 9-8 is the dimensionless wave overtopping plotted against the dimensionless crest heights depending on the value of the Iribarren parameter. All the measured points under the line are not corrected for the influence of berms and slope roughness. In the figure the calculated point based on of the input data in PC-overslag is shown with a circle.

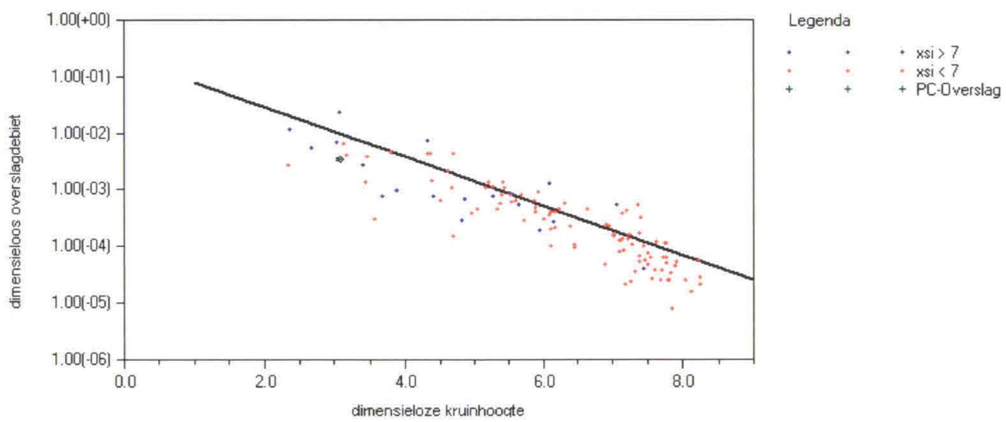


figure 9-8: dimensionless crest height plotted against dimensionless overtopping, circled point at crest height 3.0;

The cross-section calculation data is listed in table 9-14, the results are listed in table 9-15.

Parameter	Value
$z_{2\%}$ [m]	9.7
Overtopping [l/s/m]	77.4
H_{m0} [m]	3.5
T_{m0} [s]	13.6

K_{sio} [-]	6.1
L_0 [m]	290.2
γ_b [-]	1.0
γ_f [-]	0.8
$G_{\beta,run-up}$ [-]	1.0
$G_{\beta,overtopping}$ [-]	1.0
Water depth [m]	20.1
$\tan\alpha$ [-]	0.67
Iterations [-]	3

table 9-14: cross-section calculation;

Parameter	Value
$z_{2\%}$ [m]	9.7
$z_{2\%}$ +SWL [m]	29.8
Overtopping [l/s/m]	77.4
V_{max} [l/wave/m]	0
Comments	The 2 %-overtopping is higher than the groyne

table 9-15: results calculations;

The result of the PC-overslag program is an overtopping of 77 l/s/m. In the program there is no option for the input of a horizontal crest on top of the groyne. The crest width is important for the amount of overtopping. A wide crest has less overtopping than a narrow crest. Therefore, the calculated amount of overtopping is not accurate for the New North Groyne.

9.4.6 Cress

In the Cress program calculations are made for overtopping. The input of the program is shown in table 9-16 and the output in table 9-17.

Parameter	Value
H_s [m]	3.5
T_p [s]	15
h_c [m]	4.89

n_1 slope [cot]	1.5
n_2 slope [cot]	1.5
b berm width [m]	2
D_h depth berm [m]	7.1
β angle wave [°]	0
D depth [m]	20.1
n_b slope berm [cot]	0 (horizontal)
γ_f roughness [-]	0.5
P probability exceedance [%]	2-50

table 9-16: overtopping slope, Cress input;

q [l/s/m]	ξ_{op} [-]
21	6.68

table 9-17: overtopping slope, Cress output;

Using the Cress calculation, the overtopping of the New North Groyne is calculated to be 21 l/m/s with a probability exceedance of 2 %. The calculations have also been executed varying the probability of exceedance from 2 to 50 %. The results are shown in figure 9-9.

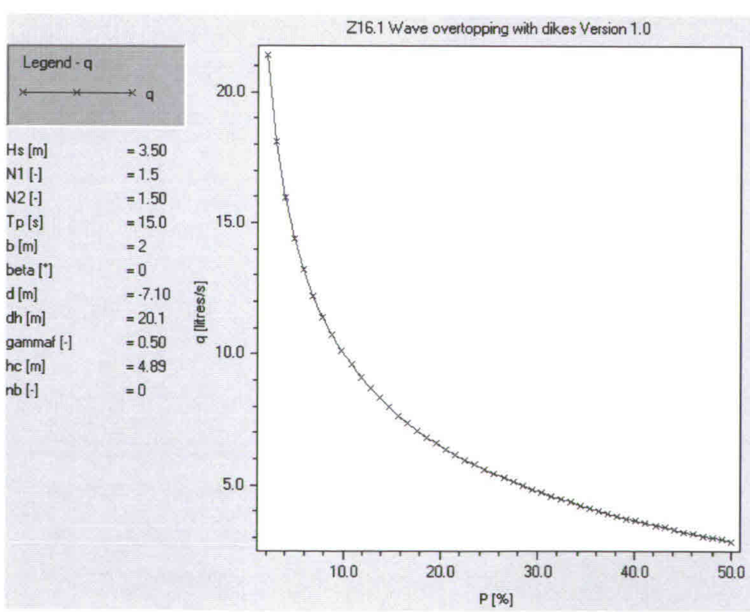


figure 9-9: probability of exceedance plotted against overtopping, Cress;

9.4.7 Summary

Overtopping of the New North Groyne must be taken into account. During severe storms with large wave heights wave overtopping of the groyne will take place. All the calculations were made for the head of the groyne, because the highest waves will attack the head. The results of the executed calculations are given in table 9-18.

Bradbury [l/s/m]	Owen [l/s/m]	PC-overslag [l/s/m]	Cress [l/s/m]
12	0.17	77	21

table 9-18: overview results overtopping calculations;

There is a big difference between the calculation methods. The result using the Owen (1980) equation is a small amount overtopping. This is mainly caused by the reduction factor used for crest permeability. PC-overslag calculates a fairly large overtopping of 77 l/s/m. The crest width is not taken into account using this method. The overtopping in PC-overslag is for impermeable structures. The results of the calculation methods of Bradbury (1988) and Cress give realistic values for the overtopping. Therefore, these methods were used to get an approximation for the overtopping. The calculated overtopping using Cress is the largest, 21 l/s/m. In further calculations a value of 21 l/s/m will be used with a significant wave height of 3.5 m.

It must be safe for pedestrians walking on the groyne (TR 16). Using figure 9-10 the groyne is not considered safe for pedestrians with an overtopping of 21 l/s/m. There is however, in case of the considered overtopping, no danger for the structural safety of the groyne. The head of the groyne must be closed in case of severe storms.

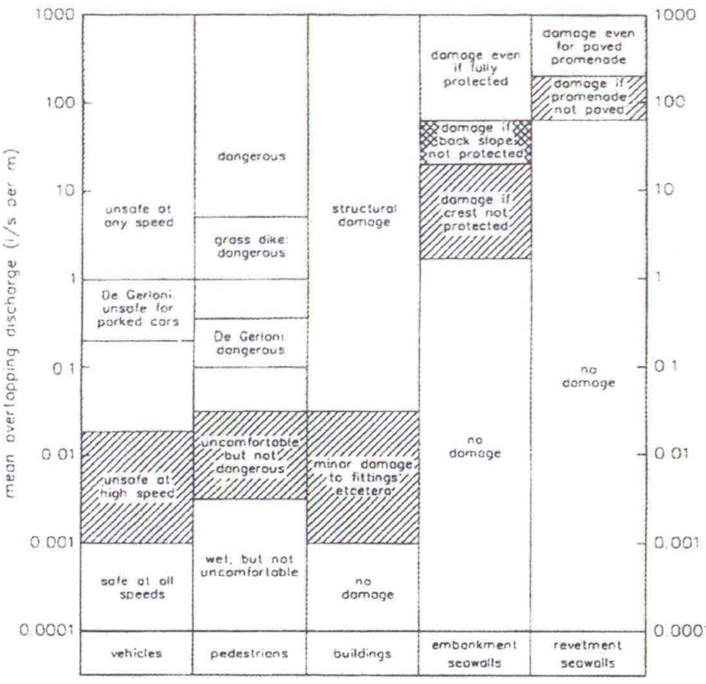


figure 9-10: overtopping discharge and its effects;

The maximum allowed overtopping when pedestrians are on the groyne is 0.5 l/s/m. In figure 9-11 the wave height is plotted against the overtopping discharge. Using this figure the head of the groyne must be closed when the significant wave height is above 2.5 m. The New North Groyne will have to be closed approximately once a year.

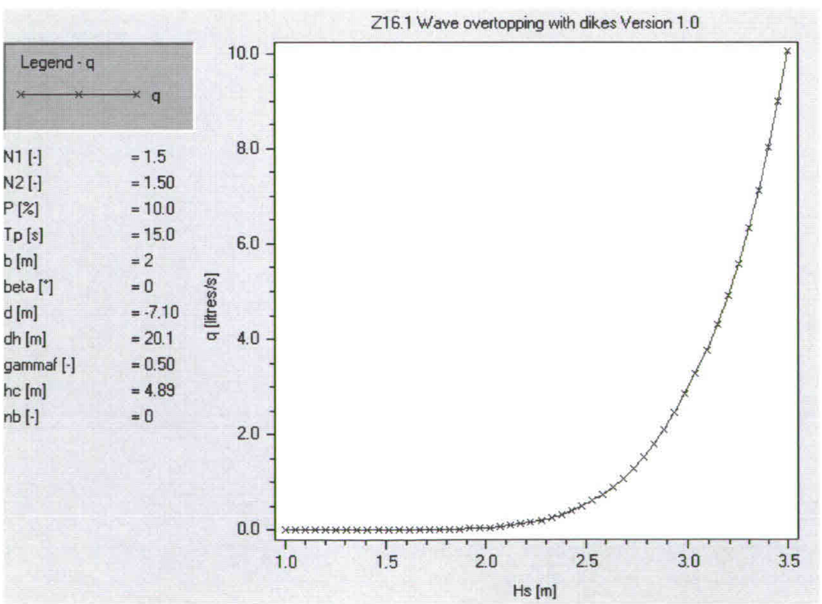


figure 9-11: significant wave height plotted against overtopping discharge;

9.5 Wave transmission

9.5.1 De Jong

Wave transmission is the phenomenon where wave energy passing over and through a breakwater creates reduced wave action in the lee of the structure. This will certainly happen when considerable amounts of water are overtopping the structure. Wave transmission is also possible, in situations where the core of the structure is very permeable and the wave period is relatively long.

De Jong derived a formula to calculate the transmission coefficient from a number of structural parameters of a breakwater. He assumed that the freeboard is dimensionless in relation to the incoming wave height,

$$K_t = a\frac{R_c}{H_s} + b,$$

where,

$$a = -0.4,$$

$$b = 0.64\left(\frac{B}{H_s}\right)^{-0.31}\left(1 - e^{-0.5\xi}\right).$$

The factor 0.64 is valid for permeable structures.

$$K_t = H_{trans} / H_s,$$

K_t	Transmission coefficient	[-],
H_s	Incoming significant wave height	[m,]
H_{trans}	Transmitted significant wave height	[m],
R_c	Crest freeboard relative to SWL	[m],
B	Crest width	[m],
s_{om}	Wave steepness	[-],
ξ_{om}	Iribarren parameter (T_m)	[-].

H_s [m]	ξ_{om} [-]	R_c [m]	B [m]	K_t [-]	H_{trans} [m]
3.5	5.34	4.89	20	0.1878	0.65

table 9-19: results De Jong calculation wave transmission;

The wave height that will be transmitted by the groyne is $H_{trans} = 0.65$ m, see table 9-19. This is a small wave height compared to the significant wave.

9.5.2 Cress

With the Cress program calculations are made for wave transmission through sloped structures. The following methods are used:

- Van der Meer (vdM);
- Hamer & Hamer (H&H);
- Powel (Pow);
- Goda & Seelig (G&S).

The input is shown in table 9-20 and the output in table 9-21.

Parameter	Value
H_s [m]	3.5
T_m [s]	12
D depth [m]	20.1
F [m]	4.89
B crest [m]	20
n slope cot [°]	1.5
P probability exceedance [%]	2-50

table 9-20: input Cress calculation wave transmission;

where,

F	Freeboard, crest of dam above SWL (in case of a submerged dam, is F negative),	[m],
B	Crest width	[m],
n	Slope of dam	[°],

h	Water depth	[m],
T	Incident wave period	[s].

H_t (vdM) [m]	B/L [-]	H_t (H&H) [m]	H_t (Pow) [m]	H_t (G&S) [m]
1	0.13	0.00	0.04	0.94

table 9-21: results Cress calculation wave transmission;

In figure 9-12 the probability of exceedance is plotted against the transmitted wave height, H_t. The probability is varied between 2 and 50 %.

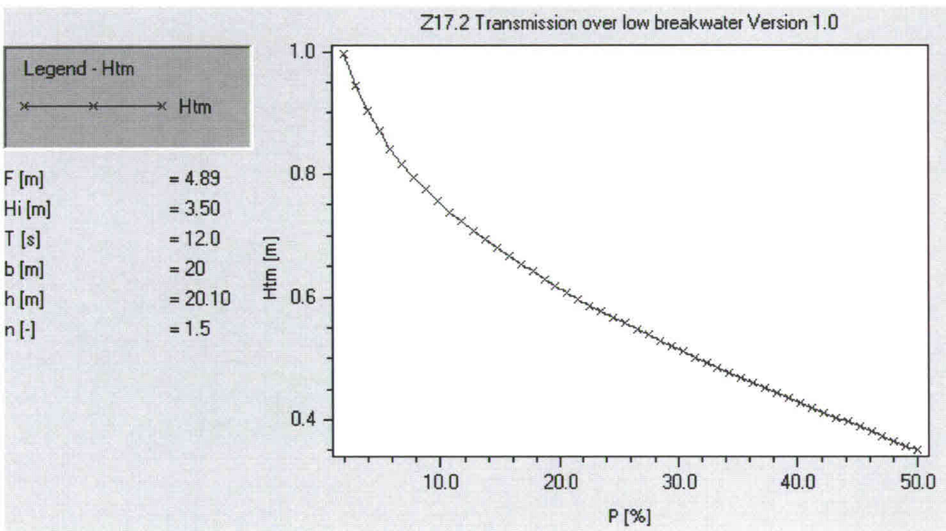


figure 9-12: wave transmission plotted against probability of exceedance;

9.5.3 Summary

The transmitted wave height is relatively small compared to the incoming wave height. The calculations were made with Cress and the transmission equation. The calculated value of the transmitted wave height is 0.65 m using the transmission equation and 1.0 m using Cress. The transmitted wave heights are significantly smaller than the waves already occurring in the channel due to shipping and wave penetration. Therefore the transmitted wave is not considered significant in respect to shipping safety.

9.6 Conclusion

Four wave-structure interaction mechanisms were assessed. The reflection of waves has no significant impact on ships in the channel. The groyne has a large run-up resulting in overtopping of the groyne. The maximum value of overtopping at the head of the groyne is 21 l/s/m. If the wave height in the channel exceeds 2.5 m the head of the groyne must be closed for the safety of the pedestrians on the groyne. This happens approximately once a year. The transmitted waves have a maximum height of 0.65 m. This has no significant influence on the ships in the channel.

10. SLOPE STABILITY

10.1 Introduction

This chapter deals with the stability of the channel slopes of the New North Groyne. First the relevant points of attention will be discussed and the situation will be described for determination of the geometry input. A Finite Element Method stability analysis will be made using Plaxis. The effects of liquefaction on the slope stability will also be discussed.

The aim of this chapter is to analyze the stability of the New North Groyne channel slopes during and after the widening and deepening of the channel.

10.2 Points of attention

10.2.1 Micro-Macro instability

A distinction needs to be made between micro and macro instability, see figure 10-1. Macro stability is defined as the stability of the structure as a whole. Macro instability can occur due to insufficient shear resistance caused by, for example, liquefaction or material properties. Micro stability considers the stability of the grains on the slope. Micro instability can occur due to seepage caused by passing waves or groundwater flow. Sediment transport caused by currents and waves is not considered micro stability but rather morphology.

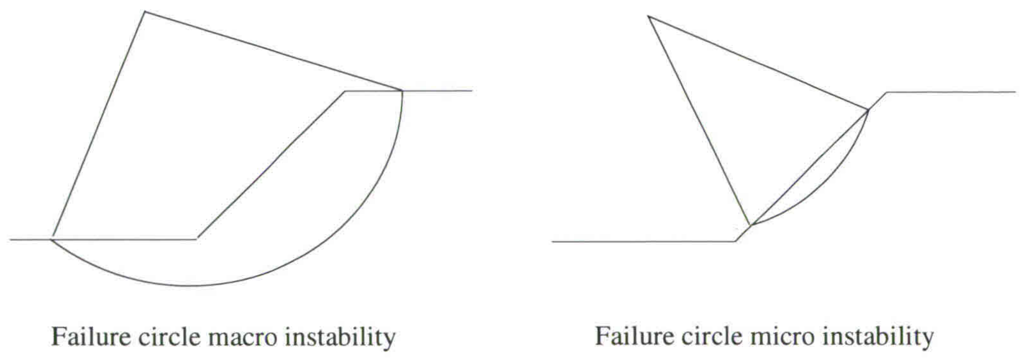


figure 10-1: macro and micro instability;

Macro instability will, most of the time, result in large movements of soil and failure of the slope. Micro instability will not necessarily lead to failure of the construction as a whole but large grain movements can eventually lead to macro instability.

10.3 Material Properties

10.3.1 Subsoil

A cross section of the subsoil through the length of the New North Groyne is given in figure 5-4. Because of lack of further geotechnical investigations this composition is assumed to be correct for the New North Groyne and the North Bank Revetment (A 4). The material properties of the subsoil are determined in chapter 5.4, and shown in table 5-3 (A 2).

10.3.2 New North Groyne

The New North Groyne will be schematised as a distributed load on the sand bed. This is under the assumption that the groyne itself is a stable structure and will not experience large deformations (A 19). The main focus of this analysis is the dredged slope between the channel and the groyne.

10.4 Geometry

10.4.1 Length section

For the slope stability calculations the length cross-section of the New North Groyne, as proposed in the chosen alternative 4, is considered. In this alternative the entire New North Groyne, head and trunk, is constructed on the existing seabed. The slopes are the longest and steepest compared to other alternatives. The New North Groyne itself is considered stable so only the sand slopes will be analysed for stability. The length cross-section is shown as a sketch in figure 10-2.

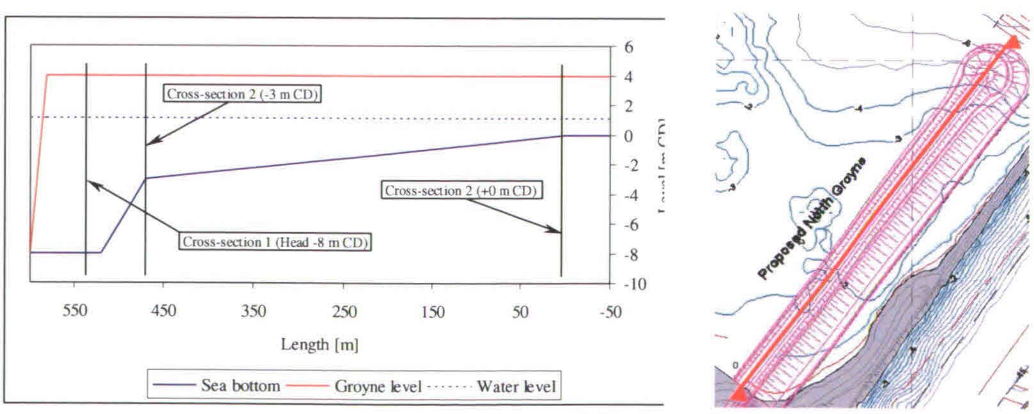


figure 10-2: length cross-section New North Groyne (alternative 4);

Three cross-sections are considered for slope stability:

- Head section with foundation level at –8 m CD;
- Trunk section with foundation level at –3 m CD;
- Trunk section with foundation level at +0 m CD.

For all cross-sections the top of the groyne is at a level of +4 m CD and the Mean Sea Level (MSL) +1.11 m CD. Different phases in the construction will be considered for each cross-section and the most unsafe cross-section will be determined.

10.4.2 Input geometry

A critical situation is considered during construction where no slope protection is placed on the channel side slopes. The slope is dredged directly between the toe of the New North Groyne and the bottom of the channel. This is a hypothetical situation but considered to be the most unfavourable situation possible during construction. The three cross-sections for the reference test are shown in figure 10-3, figure 10-4 and figure 10-5.

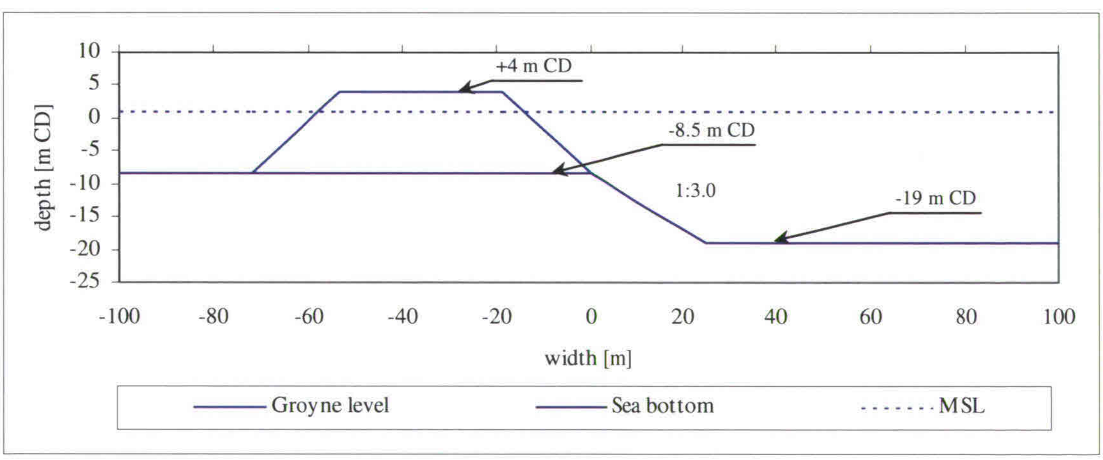


figure 10-3: cross-section 1 (head) reference test;

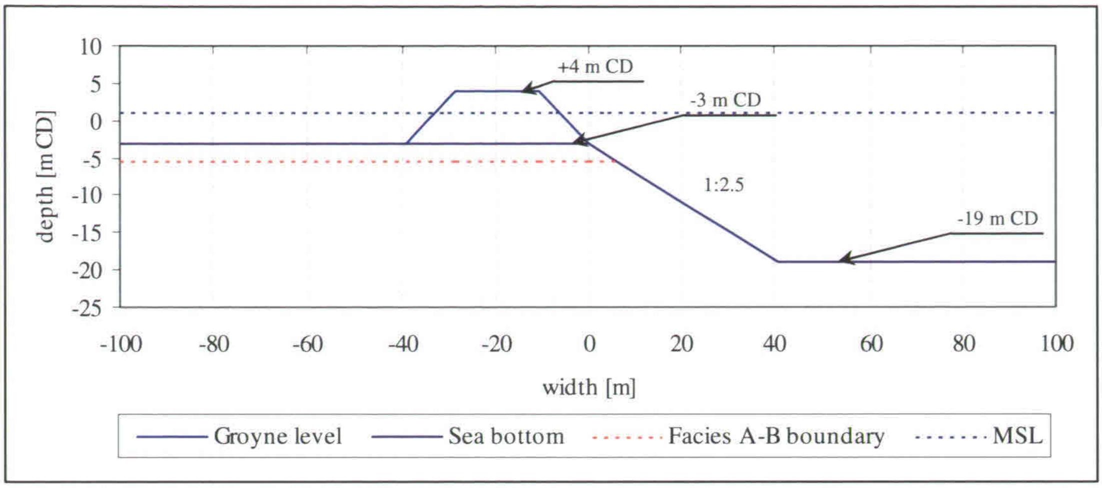


figure 10-4: cross-section 2 (trunk –3 m CD) reference test;

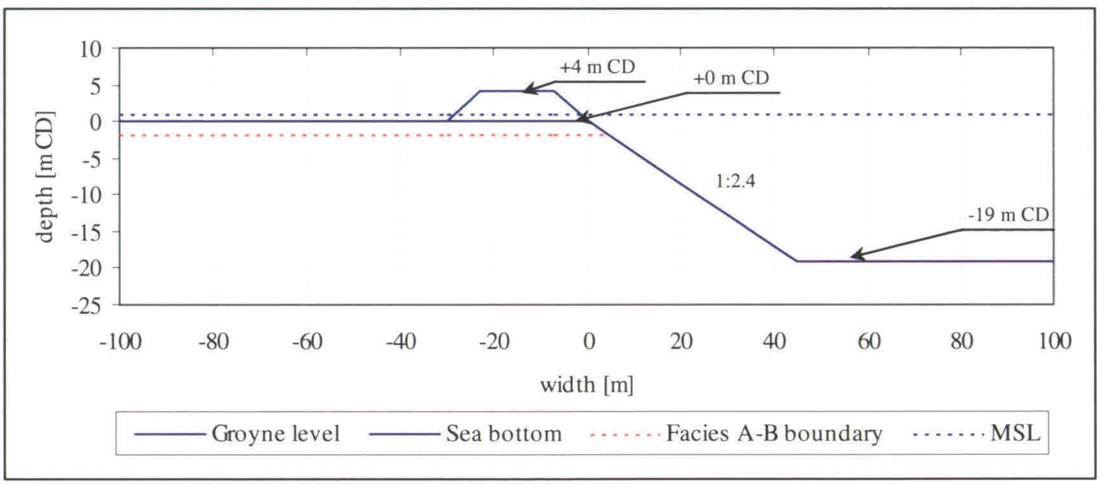


figure 10-5: cross-section 3 (trunk +0 m CD) reference test;

10.5 Calculations

10.5.1 Introduction

In this paragraph the calculations proposed in the last paragraph will be executed with the finite element model Plaxis. First an explanation of the program and material model will be given. Next different tests will be executed. Finally the results will be evaluated. The next paragraph will contain the conclusions and recommendations based on these tests.

10.5.2 Plaxis

Plaxis is a finite element program for geotechnical applications in which soil models are used to simulate the soil behaviour. Using this program, modelling of the problem, the understanding of the soil models and their limitations and the selection of the model parameters must be taken into account.

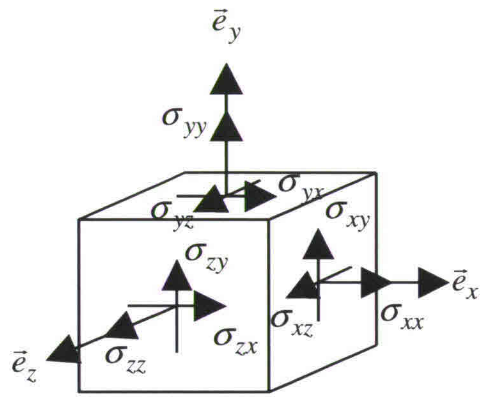
Material model

A material model is a set of mathematical equations that describes the relationship between stress and strain in a material. In the following section it is described how stresses and strains are defined in Plaxis (Ref. A6). After this the model to be used, the Mohr-Coulomb model, will be discussed (Ref. A7).

Stress

Stress is a tensor that can be represented by a matrix in Cartesian coordinates:

$$\underline{\sigma} = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix}$$



In the standard deformation theory, the stress tensor is symmetric such that $\sigma_{xy} = \sigma_{yx}$, $\sigma_{yz} = \sigma_{zy}$ and $\sigma_{zx} = \sigma_{xz}$. In this situation, stresses are often written in vector notation, which involve only six different components:

$$\underline{\sigma} = \left(\sigma_{xx}\ \sigma_{yy}\ \sigma_{zz}\ \sigma_{xy}\ \sigma_{yz}\ \sigma_{zx}\right)^T.$$

In plane strain condition, however:

$$\sigma_{yz} = \sigma_{zx} = 0.$$

According to Terzaghi's principle, stresses in the soil are divided into effective stresses, $\underline{\sigma}'$, and pore pressures, $\underline{\sigma}_w$:

$$\underline{\sigma} = \underline{\sigma}' + \underline{\sigma}_w .$$

Water is considered not sustain any shear stresses. As a result, effective shear stresses are equal tot total shear stresses. Positive normal stress components are considered to represent tension, while negative normal stress components indicate pressure.

It is often useful to use principal stresses rather than Cartesian stress components when formulating material models. Principal stresses are the stresses in such a coordinate system direction that all shear stress components are zero. Principal stresses are the eigenvalues of the stress tensor and can be determined in the following way:

$$Det(\underline{\underline{\sigma}}' - \sigma' \underline{\underline{I}}) = 0 \quad \rightarrow \quad \sigma'^3 - J_1 \sigma'^2 + J_2 \sigma' - J_3 = 0 ,$$

$$J_1 = tr(\underline{\underline{\sigma}}') = \sigma'_{xx} + \sigma'_{yy} + \sigma'_{zz} \ ,$$

$$J_2 = \tfrac{1}{2}\left(J_1^2 - tr(\underline{\underline{\sigma}}' \cdot \underline{\underline{\sigma}}')\right) = \sigma'_{xx}\sigma'_{yy} + \sigma'_{yy}\sigma'_{zz} + \sigma'_{zz}\sigma'_{xx} - \sigma_{xy}^2 - \sigma_{yz}^2 - \sigma_{zx}^2 ,$$

$$J_3 = \det(\underline{\underline{\sigma}}') = \left(-J_1^3 + 3J_1J_2 + tr(\underline{\underline{\sigma}}' \cdot \underline{\underline{\sigma}}' \cdot \underline{\underline{\sigma}}')\right) =$$

$$= \sigma'_{xx}\sigma'_{yy}\sigma'_{zz} + 2\sigma'_{yy}\sigma_{yz}\sigma_{zx} - \sigma'_{xx}\sigma_{yz}^2 - \sigma'_{yy}\sigma_{zx}^2 - \sigma'_{zz}\sigma_{xy}^2 .$$

Where $\underline{\underline{I}}$ is the identity matrix and J_1, J_2 and J_3 are the invariants of $\underline{\underline{\sigma}}'$. Because $\underline{\underline{\sigma}}'$ is symmetrical, the solution of this so-called characteristic equation yields three real eigenvalues for $\underline{\underline{\sigma}}'$ (σ'_1, σ'_2 and σ'_3), which are called the principal effective stresses. In addition to principal stresses it is also useful to define invariants of stress, which are stress measures that are independent of the orientation of the coordinate system.
Two useful stress invariants are:

- Isotropic or mean stress: $p' = -\dfrac{\sigma'_{xx} + \sigma'_{yy} + \sigma'_{zz}}{3} = -\dfrac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3}$;
- Equivalent shear stress: $q = \sqrt{-3J_2} =$

$$= \frac{1}{\sqrt{2}}\sqrt{(\sigma'_{xx} - \sigma'_{yy})^2 + (\sigma'_{yy} - \sigma'_{zz})^2 + (\sigma'_{zz} - \sigma'_{xx})^2 + 6\sigma_{xy}^2 + 6\sigma_{yz}^2 + 6\sigma_{zx}^2} .$$

Principal stresses can be written in terms of invariants:

$$-\sigma'_1 = p' + \frac{2}{3} q \sin(\theta - \frac{2}{3}\pi),$$

$$-\sigma'_2 = p' + \frac{2}{3} q \sin(\theta),$$

$$-\sigma'_3 = p' + \frac{2}{3} q \sin(\theta + \frac{2}{3}\pi).$$

In which θ is referred to as Lode's angle, which is defined as:

$$\theta = \tfrac{1}{3} \arcsin\left(\frac{27 J_3}{2 q^3} \right).$$

Strain

Strain is a tensor that can be represented by a matrix with Cartesian coordinates as:

$$\underline{\varepsilon} = \begin{bmatrix} \varepsilon_{xx} & \varepsilon_{xy} & \varepsilon_{zx} \\ \varepsilon_{yx} & \varepsilon_{yy} & \varepsilon_{yz} \\ \varepsilon_{zx} & \varepsilon_{zy} & \varepsilon_{zz} \end{bmatrix}.$$

According to the small deformation theory only the sum of complementing Cartesian shear strain components ε_{ij} and ε_{ji} result in shear stress. The sum is denoted as the shear strain γ. Hence, instead of ε_{xy}, ε_{xz}, ε_{yx}, ε_{yz}, ε_{zy} and ε_{zx} the shear strain components γ_{xy}, γ_{yz} and γ_{zx} are used respectively. Under the above conditions, strains are often written in vector notation, which involve only six different components:

$$\underline{\varepsilon} = \begin{bmatrix} \varepsilon_{xx} & \varepsilon_{yy} & \varepsilon_{zz} & \gamma_{xy} & \gamma_{yz} & \gamma_{zx} \end{bmatrix}^T,$$

for plane strain conditions,

$$\varepsilon_{zz} = \gamma_{xz} = \gamma_{yz} = 0.$$

In analogy to the invariants of stress, it is also useful to define invariants of strain. A strain invariant that is often used is the volumetric strain, ε_v, which is defined as the sum of all normal strain components,

$$\varepsilon_v = \varepsilon_{xx} + \varepsilon_{yy} + \varepsilon_{zz} = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 .$$

The volumetric strain is defined as negative for compaction and as positive for dilatancy. For elastoplastic models, as used in Plaxis program, strains are decomposed into elastic and plastic components,

$$\underline{\varepsilon} = \underline{\varepsilon}^e + \underline{\varepsilon}^p .$$

Elastic strains

Material models for soil and rock are generally expressed as a relationship between infinitesimal increments of effective stress ("effective stress rates") and infinitesimal increments of strain ("strain rates"). The simplest material model in Plaxis is based on Hooke's law for isotropic linear elastic behaviour. Hooke's law can be given by the equation,

$$\underline{\underline{\sigma}}' = \frac{E'}{1+\nu'}\left[\underline{\underline{\varepsilon}} + \frac{\nu}{1-2\nu'}\varepsilon_v \underline{\underline{I}}\right] \quad \Leftrightarrow \quad \underline{\underline{\varepsilon}} = \frac{1+\nu'}{E'}\left[\underline{\underline{\sigma}} - 3p'\frac{\nu'}{1+\nu'}\underline{\underline{I}}\right],$$

$$\begin{bmatrix} \sigma'_{xx} \\ \sigma'_{yy} \\ \sigma'_{zz} \\ \sigma'_{xy} \\ \sigma'_{yz} \\ \sigma'_{zx} \end{bmatrix} = \frac{E'}{(1+\nu')(1-2\nu')} \begin{bmatrix} 1-\nu' & \nu' & \nu' & 0 & 0 & 0 \\ \nu' & 1-\nu' & \nu' & 0 & 0 & 0 \\ \nu' & \nu' & 1-\nu' & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2}-\nu' & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{2}-\nu' & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{2}-\nu' \end{bmatrix} \begin{bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{bmatrix},$$

where,

E'	the effective Young's modulus	[kPa],
ν'	the effective Poisson's ratio	[-].

In the remaining part of this report effective parameters are denoted without the dash ('), unless a different meaning is explicitly stated. The relationship between Young's modulus E and other stiffness moduli, such as the shear modulus G, the bulk modulus K and the oedometer modulus E_{oed}, is given by,

$$G = \frac{E}{2(1+\nu)} \qquad \text{[kPa]},$$

$$K = \frac{E}{3(1-2\nu)} \qquad \text{[kPa]},$$

$$E_{oed} = \frac{(1-\nu)E}{(1-2\nu)(1+\nu)} \qquad \text{[kPa]}.$$

The Mohr-Coulomb model (perfect-plasticity)

Plasticity is associated with the development of irreversible strains. In order to evaluate whether or not plasticity occurs in a calculation, a yield function, f, is introduced as a function of stress and strain. A yield function can often be presented as a surface in principal stress space. A perfectly-plastic model is a constitutive model with a fixed yield surface, i.e. a yield surface that is fully defined by model parameters and not affected by (plastic) straining. For stress states represented by points within the yield surface, the behaviour is purely elastic and all strains are reversible.

For Mohr-Coulomb type yield functions, the theory of associated plasticity leads to an over prediction of dilatancy. Therefore, in addition to the yield function, a plastic potential function g is introduced. The case $g \neq f$ is denoted as non-associated plasticity. In general, the plastic strain rates are written as:

$$\underline{\varepsilon}^{p} = \lambda \frac{\partial g}{\partial \underline{\sigma}'}.$$

Where λ is the plastic multiplier. For purely elastic behaviour λ is zero, whereas in the case of plastic behaviour λ is positive,

$$\lambda = 0 \quad \text{for:} \quad f < 0 \qquad \text{Elasticity},$$

$$\lambda > 0 \quad \text{for:} \quad f = 0 \qquad \text{Plasticity}.$$

The Mohr-Coulomb yield condition is an extension of Coulomb's friction law to general states of stress. In fact, this condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition consists of six yield functions in terms of principal stresses,

$$f_{1a} = \tfrac{1}{2}(\sigma'_2 - \sigma'_3) + \tfrac{1}{2}(\sigma'_2 + \sigma'_3)\sin\varphi' - c'\cos\varphi' \le 0,$$

$$f_{1b} = \tfrac{1}{2}(\sigma'_3 - \sigma'_2) + \tfrac{1}{2}(\sigma'_3 + \sigma'_2)\sin\varphi' - c'\cos\varphi' \le 0,$$

$$f_{2a} = \tfrac{1}{2}(\sigma'_3 - \sigma'_1) + \tfrac{1}{2}(\sigma'_3 + \sigma'_1)\sin\varphi' - c'\cos\varphi' \le 0,$$

$$f_{2b} = \tfrac{1}{2}(\sigma'_1 - \sigma'_3) + \tfrac{1}{2}(\sigma'_1 + \sigma'_3)\sin\varphi' - c'\cos\varphi' \le 0,$$

$$f_{3a} = \tfrac{1}{2}(\sigma'_2 - \sigma'_1) + \tfrac{1}{2}(\sigma'_2 + \sigma'_1)\sin\varphi' - c'\cos\varphi' \le 0,$$

$$f_{3b} = \tfrac{1}{2}(\sigma'_2 - \sigma'_1) + \tfrac{1}{2}(\sigma'_2 + \sigma'_1)\sin\varphi' - c'\cos\varphi' \le 0.$$

The two plastic model parameters appearing in the yield functions are the well-known friction angle φ and the cohesion c. These yield functions together represent a hexagonal cone in principal stress space as shown in figure 10-6.

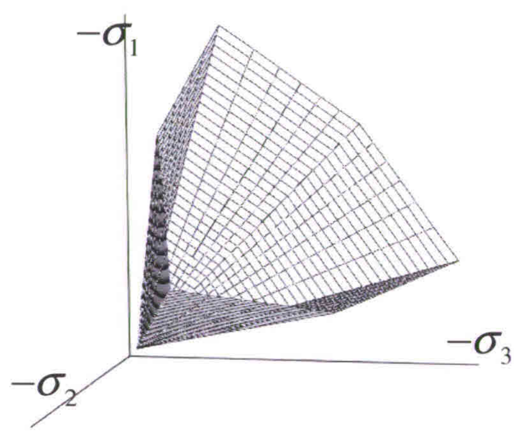


figure 10-6: The Mohr-Coulomb yield surface;

In addition to the yield functions, six plastic potential functions are defined for the Mohr-Coulomb model,

$$g_{1a} = \tfrac{1}{2}(\sigma'_2 - \sigma'_3) + \tfrac{1}{2}(\sigma'_2 + \sigma'_3)\sin\psi,$$

$$g_{1b} = \tfrac{1}{2}(\sigma'_3 - \sigma'_2) + \tfrac{1}{2}(\sigma'_3 + \sigma'_2)\sin\psi,$$

$$g_{2a} = \tfrac{1}{2}(\sigma'_3 - \sigma'_1) + \tfrac{1}{2}(\sigma'_3 + \sigma'_1)\sin\psi,$$

$$g_{2b} = \tfrac{1}{2}(\sigma'_1 - \sigma'_3) + \tfrac{1}{2}(\sigma'_1 + \sigma'_3)\sin\psi,$$

$$g_{3a} = \tfrac{1}{2}(\sigma'_1 - \sigma'_2) + \tfrac{1}{2}(\sigma'_1 + \sigma'_2)\sin\psi,$$

$$g_{3b} = \tfrac{1}{2}(\sigma'_2 - \sigma'_1) + \tfrac{1}{2}(\sigma'_2 + \sigma'_1)\sin\psi.$$

The plastic potential functions contain a third plasticity parameter, the dilatancy angle ψ. This parameter is required to model positive plastic volumetric strain increments (dilatancy) as actually observed for dense soils.

Basic parameters of the Mohr-Coulomb model

The Mohr-Coulomb model requires a total of five parameters, which are generally familiar to most geotechnical engineers and which can be obtained from basic tests on soil samples,

E	Young's modulus	[kN/m^2],
ν	Poisson's ration	[-],
φ	friction angle	[°],
c	cohesion	[kN/m^2],
ψ	dilatancy angle	[°].

Abstract:

The material behaviour in this report is represented by the Mohr-Coulomb model. This model represents a "first-order" approximation of soil or rock behaviour. The use of this model will be to consider a first analysis of the problem. Because of the lack of sufficient soil data, there is no sense in using more advanced analysis models.

10.5.3 Calculation 1

As mentioned before the test considers an unprotected slope between the toe of the New North Groyne and the bottom of the channel. Calculations are executed with Plaxis using the Mohr-Coulomb material model. The groyne is schematised as a distributed load depending on the volume of the cross-section. Four phases are considered in the test (see figure 10-7):

- **Initial phase:** seabed without loads or excavations;
- **Groyne construction phase:** The load is placed on the seabed in a period of 14 days;
- **Consolidation phase:** The soil consolidates until almost 100% of the excess pore pressures has dissipated;
- **Dredging the channel phase:** The channel is dredged in a period of 4 weeks (28 days).

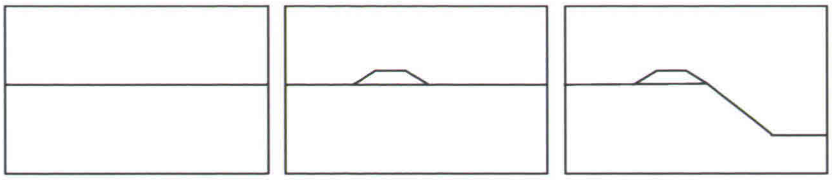


figure 10-7: finite element method analysis, phasing calculation 1;

It should be noted that due to the high permeability of the Facies A and B consolidation does not take more than a couple of days in each cross-section.

For each of the three considered sections the safety factor is calculated after the dredging phase and again after the consolidation phase. This is done using the phi/c reduction method incorporated in the Plaxis software. In this method a gradual decrease of the friction angle and the cohesion results in a measure of the safety defined as,

$$\text{Safety factor} = \frac{S_{\text{maximum available}}}{S_{\text{needed for equilibrium}}},$$

where S represents the shear strength.

Introducing the standard Coulomb condition the safety factor is obtained,

$$S_f = \frac{c - \sigma_n \tan \varphi}{c_r - \sigma_n \tan \varphi_r},$$

where,

c, φ	input strength parameters	[kPa],[°],
σ_n	actual normal stress component	[kPa].

The subscript r means the strength parameters are reduced just enough to maintain equilibrium.

10.5.4 Evaluation calculation

The Plaxis generated plots of safety factor determination, the deformed finite element mesh and the total displacements after consolidation can be found in appendix W. This appendix also contains a graphical representation of the incremental shear strains measured during the phi/c reduction process. Although the values are irrelevant (due to the massive reductions) the picture gives a nice estimation of the occurring failure mechanism.

The difference between the cross-sections lies mainly in the value of the distributed load and height difference and steepness of the slope. Because the crest of the groyne is always located at +4 m CD and the channel bottom always at -19 m CD, the foundation level of the groyne determines the distributed load and the height difference and steepness of the slope. The distributed load is the highest for the head section and lowest for the trunk section with a foundation level at 0 m CD while the height difference of the slope is highest for this section and the smallest for the head section.

For all the considered cross-sections the consolidation time is relatively short, a few days after the load is placed almost all excess pore pressures have dissipated. The high permeability of the sand causes a fast dissipation of the excess pore pressures generated by the distributed load. With the assumed permeability value no long consolidation period need to be taken into account before dredging can start.

There is a noticeable difference between the soil displacements of the different sections. The biggest displacements in sections 2 and 3 are in the form of heave caused by the dredging (unloading) of the channel. The largest displacements in section 1 are caused by settlement of the sand directly under the distributed load. The settlement of approximately 25 cm under the groyne head will have to be taken into account during and after the construction of the groyne. The settlement is mainly dependent on the soil stiffness. Larger deformations can be expected when a smaller stiffness is measured in the field than assumed during this calculation.

No large differences between the calculated safety factors occur. The safety factor has a value of approximately 1.9 which is considered to be safe (TR 13). Because the Facies A and B are almost cohesionless the main parameter determining the safety factor of the slope is the friction angle.

10.5.5 Calculation 2

A large amount of swelling is calculated during the dredging for all cross-sections. This is a fallacy of the Mohr-Coulomb model where the unloading stiffness is presumed equal to the loading stiffness, which results in larger than realistic swelling value. This error can be avoided by using another soil model, for example the Hardening Soil model, but, as mentioned before, not enough soil data is available to justify the use of such a model. It is therefore decided to make an extra calculation without the excavation step. Now 3 phases are considered, see figure 10-8:

- **Initial phase:** the slope is already dredged;
- **Groyne construction phase:** The load is placed on the seabed in a period of 14 days;
- **Consolidation phase:** The soil consolidates until almost 100% of the excess pore pressures have dissipated.

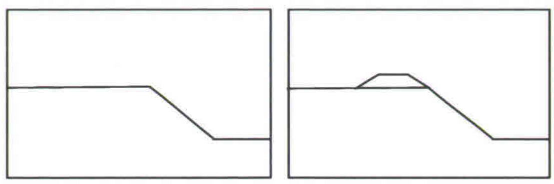


figure 10-8: finite element method analysis, phasing calculation 2;

The results for the three cross-sections can be found in appendix W.

10.5.6 Evaluation calculation 2

As in calculation 1 the consolidation time is relatively short for all the considered cross-sections, a few days after the load is placed almost all excess pore pressures have dissipated.

The only displacements calculated are caused by the distributed load, no heave due to unloading occurs. The settlements of the soil directly under the distributed load (the New North Groyne) in calculation 2 are equal to settlements calculated in calculation 1, as could be expected. The "Displacement after consolidation" image in appendix W now gives a clearer view of the displacements caused by the distributed load because it is not disturbed by the unloading effect caused by the dredging. Although in reality some heave due to unloading will take place.

This calculation allows for the determination of the safety factor in three of the construction phases. The results are shown in table 10-1. In the initial situation the safety factor is the highest because no load is placed yet. It can also be seen that the difference in safety factor between the construction phase and after consolidation is small. This is because most of the consolidation has already taken place during construction.

	Dredging	Initial situation	Construction	Consolidation
	Calculation 1	Calculation 2		
Section 1: head (see figure 10-3)	1.9	2.2	1.9	1.9
Section 2: trunk on –3 m CD (see figure 10-4)	1.8	2.2	1.9	1.9
Section 3: trunk on +0 m CD (see figure 10-5)	1.9	2.0	1.8	1.9

table 10-1: safety factors calculations;

10.5.7 Conclusions

The safety factors calculated using the two different methods are almost the same, see table 10-1. It is therefore concluded that the channel side slopes can be considered safe during construction with the assumed parameters. The maximum occurring settlement under the head is approximately 25 cm and under the trunk approximately 15 cm. The consolidation period, using these parameters, is no more than 3 weeks.

10.6 The cyclic liquefaction potential

10.6.1 Introduction

Earthquakes and water waves cause pore pressure changes in cohesionless deposits. This affects the soil strength. This strength reduction is a major cause of collapse of offshore and coastal constructions. During the stability analysis of the New North Groyne the reduction of the soil strength caused by the waves in the channel will be taken into account.

Most literature on liquefaction of sandy soils refers to earthquake-induced liquefaction. Because the Durban area is not a seismic active region (A 20), this liquefaction potential is not a subject in this report. Wave-induced liquefaction differs from earthquake-induced liquefaction because the number of load cycles is much larger (unfavourable) and the load frequency is much lower (favourable). It will be shown that drainage and compaction often play an important and favourable role in wave-induced liquefaction.

The concept of liquefaction is related to dilatancy (or better: contractancy), expressing the volumetric change of a grain skeleton under pure shear loading. Usually the attention is focused on the uni-axial cyclic loading of seabed sands. Hence the term liquefaction, the situation that the effective stresses become zero, is somewhat misleading. The situation can better be addressed by the term "transient instability". Whether it is dangerous or not, is a practical question depending on other simultaneous occurring mechanisms (scour, gravity, etc.) and on the local geometry (submerged slope).

10.6.2 Basic equations

Tests show that uni-axial laterally confined cyclic loading of loose sand deposits may generate a pore-pressure build up. An explanation could be local shear-induced dilatancy effects in uni-axial deformation. Suddenly applied constant (un)loading may give similar effects, leading to triggered liquefaction. Under cyclic loading slight residual deformations may change the stiffness and the permeability of sand. This is also referred to as preloading or pre-shearing and it includes the effect of structural changes and densification. A simple approximate method is presented to include these effects. Consider the uni-axial linear storage equation,

$$n\beta p_{,t} + \varepsilon_{,t} = (k/\gamma) p_{,zz} ,$$

where,

p	Excess pore pressure	[kPa],
ε	Volume strain	[-],
β	Fluid compressibility	[kPa^{-1}],
n	Porosity	[-],
k	Permeability	[m/s],
γ	Volumetric weight of water	[kPa/m].

The linear stress-strain law leads to,

$$p_{,t} - m\sigma_{,t} = c_v p_{,zz} ,$$

where,

σ Boundary total stress (cyclic) [kPa],

m Compressibility of the soil: [-],

$$m = \frac{1}{1 + \frac{n\beta}{\alpha}},$$

α Uni-axial soil compressibility [kPa^{-1}],

c_v Consolidation coefficient: [m^2/s],

$$c_v = \frac{km}{\gamma\alpha}.$$

In the approximate approach the problem is split into two parts:

1. $p_{,t} = m\sigma_{,t} + \psi_0$;
2. $p_{,t} - \psi = c_v p_{,zz}$ with $\psi = \psi(z, t, \psi_0)$.

The first part determines the "pore pressure rate ψ_0" for the undrained situation, which is described by a cyclic (loading) and a steady term. The pressure shows a cyclic increase with time. The (undrained) pore pressure rate ψ_0 that is referred to as the liquefaction potential, can be measured in the laboratory by undrained triaxial tests. The second part is the drainage in the system, or the excess pore pressure dissipation. This is achieved by solving the storage-drainage equation with the liquefaction potential ψ_0. This equation resolves a local excess pore pressure (the hydrostatic and cyclic loading part is not considered) induced by the dilatancy rate ψ. To obtain the real pore pressure, the hydrostatic part and the cyclic part, caused by cyclic loading σ, must be added to this local excess pore pressure. The excess pore pressure will cause changes in effective stress and thus effects on the local strength. The effect of dilatancy induced excess pore pressures is strongly damped in semi-saturated sands. When no air entrainment occurs, the pore fluid will be relatively incompressible,

$$\beta = 0 \rightarrow m = 1 \rightarrow c_v = \frac{k}{\gamma\alpha}.$$

The liquefaction potential ψ_0 depends on the initial effective stress σ'_0. In laboratory tests ψ_0 is a constant, but in the field ψ_0 is a function of the depth z (local effective stress) and a function of time (pre-shearing). From now on ψ_0 is assumed constant in depth.

A simple approach is assumed by considering a pore pressure averaged with respect to the layer thickness d, vulnerable to liquefy. The choice of d is not trivial, since the layer thickness is not known beforehand. In this approach the question is rather, whether a layer of certain thickness d is vulnerable to liquefaction or not. By changing d the sensitivity with respect to practical requirements can be established. The average pore pressure P in this layer is considered,

$$P = \int_0^d p dz / d .$$

The corresponding storage equation becomes, ψ constant for z and assuming $p(z)$ is a parabola, dependent on t,

$$p_{,t} - \psi = c_v p_{,zz} ,$$

$$\int \left(p_{,t} - \psi = c_v p_{,zz} \right) dz \rightarrow \int \left(p_{,t} \right) dz - \int \psi dz = \int \left(c_v p_{,zz} \right) dz ,$$

$$\int \left(p dz \right)_{,t} - \psi \int dz = c_v \int dp_{z} \rightarrow P_{,t} d - \psi d = -c_v \left(p_{,z} \big|_{\downarrow 0} - p_{,z} \big|^{\uparrow d} \right),$$

$$= -c_v (0 + 3P / d) \rightarrow P_{,t} - \psi = -\theta P \qquad \text{with} \qquad \theta = \frac{3c}{d^2} .$$

Where P is the average excess pore pressure in a layer of thickness d. The generation rate ψ is defined by,

$$\begin{aligned} &t < 0 \rightarrow \psi = 0 \\ &t > 0 \rightarrow \psi = \psi_0 \exp(-\delta t) \end{aligned} .$$

Equivalent time is expressed as $t = NT_c$ (N number of cycles and T_c cycle period). The constant δ characterises the pre-shearing effect, expressing that due to densification and structural changes the porous skeleton becomes gradually less sensitive to liquefaction. Thus, the average pore-pressure generation can be characterised by four parameters:

- P_0 Initial set-up [Pa];

- ψ_0 Dilitant behaviour [Pa/s];
- θ Drainage [s^{-1}];
- δ Pre-shearing [s^{-1}].

Now the average pore pressure $P(t)$ can be found this will be done in appendix X. The result is,

$$P = \frac{\psi_0 (\exp(-\delta t) - \exp(-\theta t)}{\theta - \delta}.$$

It reveals a pore-pressure variation with an initial rate of ψ_0, reaching a maximum at,

$$t_m = \frac{\ln(\theta) - \ln(\delta)}{(\theta - \delta)},$$

and thereafter diminishing to zero, see figure 10-9.

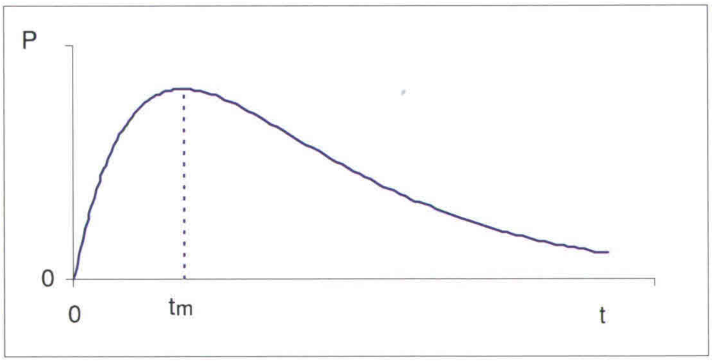


figure 10-9: pore pressure development due to dilatancy in time;

The average pore pressure equation describes that the loading starts to generate compaction (dilatancy). As the process takes place the porosity decreases slightly, and the skeleton becomes less sensitive to liquefaction. A liquefaction potential decreasing with time can simulate this phenomenon. It is referred to as the effect by preloading or pre-shearing. The value of δ can be determined by measuring the maximum densification, or the maximum porosity change. If the initial porosity is identified with n_0 and the final porosity by n_1, then the process of gradual densification under cyclic loading can presumable proceed as presented in figure 10-10.

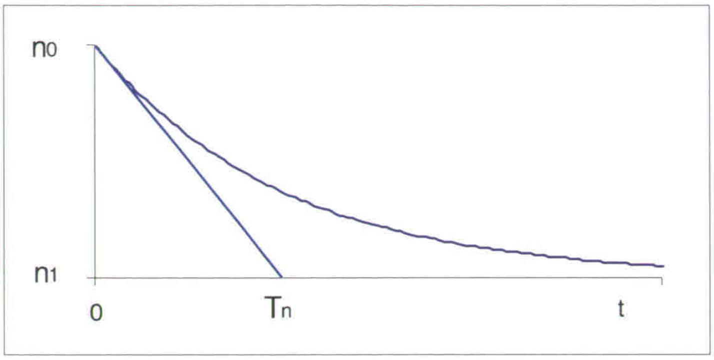


figure 10-10: pre-shearing effect in time;

The variation of n with time can be expressed by,

$$n = n_1 + (n_0 - n_1)\exp\left(\frac{-t}{T_n}\right) = n_1 + \Delta n \exp\left(\frac{-t}{T_n}\right),$$

where T_n is some reference time. Elaboration yields,

$$n_{,t} = \frac{\Delta n}{T_n}\exp\left(\frac{-t}{T_n}\right).$$

On the other hand the relation between the porosity and the liquefaction potential can be worked out as follows, keeping in mind that in the undrained test the pore pressure increase reflects the porosity decrease in the drained situation,

$$n_{,t} = (1-n)\varepsilon_{,t} = -(1-n)\alpha' p_{,t} = -(1-n)\alpha'\psi = -(1-n)\alpha'\psi_0 \exp(-\delta t).$$

This gives,

$$\frac{\Delta n}{T_n} = -(1-n)\alpha'\psi_0 \qquad \text{and} \qquad \delta = \frac{1}{T_n} = \frac{(1-n)\alpha'\psi_0}{\Delta n}.$$

This expression can be used to describe pre-shearing. This equation holds for both contractancy and dilatancy. The parameters ψ_0, α' and n_1 must be determined by laboratory tests. In this report, $\alpha' = \alpha$ is used. The establishment of n_1, which is not necessarily equal to the so-called critical density, requires special experience. The value n_0 is to be determined in the field with special density measuring equipment; this is the most expensive and essential parameter. During the process of compaction the permeability and compressibility will change. This effect can be included in the present approach by increasing δ.

The liquefaction potential is related to the change in total stress due to volume changes when the granular skeleton compacts. In this respect an undrained triaxial test experiment has to be performed on the material. Since the test is undrained, all volume changes caused by densification under the cyclic loading will come on account of a pore pressure increase. For the test careful attention has to be paid to the effect of air, which can influence the pressure significantly. The number of cycles required to cause a complete liquefaction (N_{liq}) is counted. The sensitivity to pore pressure generation under cyclic loading is measured and put into a graph normalized with respect to the initial effective stress σ'_0 and the cycle number N_{liq} for which liquefaction is observed, see figure 10-11. The trend for various densities is represented by the normalized generation rate ψ'; it shows an initial jump, then a more or less regular pattern, and finally a kind of acceleration. This acceleration is not representative, because in this stage the sample collapses with local shear bands and the pore pressure distribution in the sample is not homogeneous any more. In the field a similar behaviour can be observed. The undrained excess pore pressure rate or the liquefaction potential becomes,

$$\psi_0 = p_t = \frac{d(p/\sigma'_0)}{d(NT_c/N_{liq})}\frac{\sigma'_0}{N_{liq}} = \frac{r\psi'\sigma'_0}{T_c N_{liq}} = \frac{f_n r\psi'\sigma'_0}{N_{liq}},$$

where,

f_n	Loading frequency	[s^{-1}],
r	Stress correction factor	[-],
ψ'	Normalized rate	[-],
σ'_0	Test stress level	[kPa],
N_{liq}	Liquefaction cycle	[-].

The test usually shows a gradual pore pressure increase, which can be represented in a form as shown in figure 10-11. This example is given to illustrate the calculation of the liquefaction potential using the parameters in table 10-2. The slope of the resulting line is the normalized liquefaction potential ψ'.

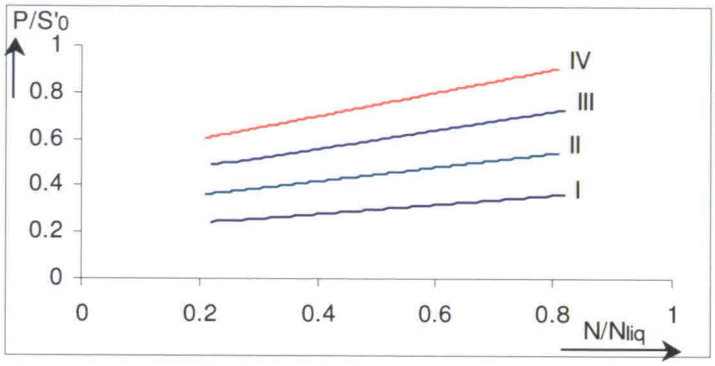


figure 10-11: liquefaction potential example;

Test	I	II	III	IV
σ'_0 [kPa]	20	20	20	20
N_{liq} [-]	100	200	100	200
n_0 [-]	0,4	0,4	0,42	0,42
T_c [s]	12	12	12	12
f_N [1/s]	0,083	0,083	0,083	0,083
ψ' [-]	0,2	0,3	0,4	0,5
ψ_0 [kPa/s]	0,0033	0,0025	0,0067	0,0042

table 10-2: liquefaction potential example;

The stress correction factor r is a factor relating the triaxial stress configuration to the uni-axial one and has a value of $r = 0.65$. The liquefaction cycle number N_{liq} depends on the loading intensity characterized by the stress ratio $\tilde{\tau}/\sigma'_0$ ($\tilde{\tau}$ is the amplitude of shear loading). For wave generation over an ocean floor sand bed, two-dimensional numerical analyses (Biot theory) show that the stress ratio $\tilde{\tau}/\sigma'_0$ is approximately constant for a sand bed on a relatively stiff layer. It implies that N_{liq} measured in the test can be applied in the field at any depth in the sand bed. Tests often show that in the early stage the generation rate is significantly larger. A physical explanation can be that sand becomes more mobile after surpassing a kind of static structural threshold. This is referred to as the apparent initial set-up

p_0 / σ'_0. It is important to include this aspect, since it causes initial pore pressures to develop faster. This initial set-up can be used to simulate static liquefaction. It is included in the present theory by a jump P_0 in the excess pore pressure at time $t = 0$. The effect of such a set-up in the field will decay due to drainage (excess pore pressure dissipation). The new pore pressure equation will be become,

$$P = P_0 \exp(-\theta t) + \frac{\psi_0(\exp(-\delta t) - \exp(-\theta t)}{\theta - \delta}.$$

The moment of maximum pore pressure under drainage/storage and pre-shearing becomes,

$$t_m = \frac{\ln(\omega\theta) - \ln(\delta)}{(\theta - \delta)} \quad \text{with} \quad \omega = 1 - P_0 \frac{\theta - \delta}{\psi_0}.$$

In this approach the process of drainage/storage and pre-shearing are uncoupled. The fact is that pre-shearing cannot develop when locally the induced pore pressure has had no opportunity to dissipate. Therefore, if the drainage/storage coefficient θ is small, such that the drainage time (hydrodynamic period) is significantly larger than the pre-shearing capacity ($1/\delta$), pre-shearing is limited by the pore volume change that is allowed for the consolidation. In most sands the hydrodynamic period is relatively short, except when the drainage path is long due to the specific geological situation (for example, a sand layer partly sealed off by a clay layer). It is in that case suggested to use the undrained pore pressure generation, with $\delta = 0$. The average excess pore pressure generation for $\theta t << 1$ becomes,

$$P = P_0 \exp(-\theta t) + \frac{\psi_0(1 - \exp(-\theta t)}{\theta} = P_0 + \psi_0 t.$$

The three described situations, the pore pressure generation undrained, drained and pre-loaded, are illustrated in figure 10-12. During the process the permeability and the compressibility will change. These effects, which are estimated to make a difference of 20 to 50 %, are not included in the presented approach. Moreover, the effect of the initial stress state is important.

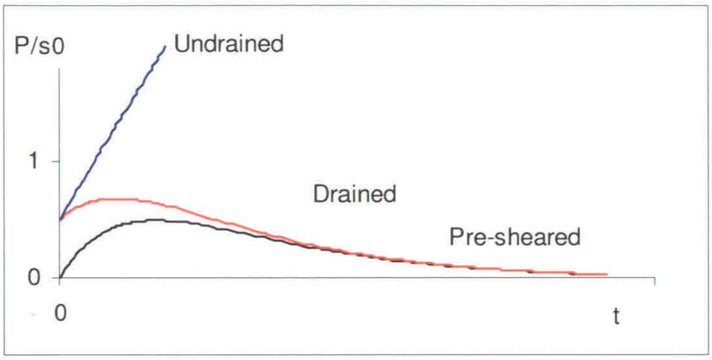


figure 10-12: pore pressure generation in time;

10.6.3 Calculation of the liquefaction potential

Using above-mentioned formulas, the liquefaction potential in the Durban Harbour Entrance case can be estimated. Important to note is that all the soil parameters, field observations and laboratory measurements, are estimated.

Laboratory measurements:

Undrained triaxial test in the lab shows: normalized generation rate: $\psi' = 0.3$

Thus the liquefaction potential becomes,

$$\psi_0 = p_t = 0.3\frac{\sigma'_0}{T_c N_{liq}}.$$

The value of r is not taken into account.

Critical storm:

Number of cycles: $N = 2000$

Cycle period: $T_c = 12$ s

Number of cycles for which liquefaction is observed: $N_{liq} = 100$

Critical porosity: $n_c = 0.38$

Submerged volumetric soil weight: $\gamma' = 10$ kPa/m

Permeability: $k = 10^{-4}$ m/s

Compressibility: $\alpha = 10^{-3}$ kPa^{-1}

Field observation:

Actual porosity: $n = 0.4 \quad \rightarrow \quad \Delta n = n - n_c = 0.02$

Depth: $d = 4$ m

Average $\sigma'_0 = 20$ kPa (at z = 2 m)

Follows:

Duration of the critical storm: $0 < t < NT_c = 24000$ s

$$c_v = \frac{k}{\gamma\alpha} = 0.01 \text{ m}^2/\text{s},$$

$$\theta = \frac{3c_v}{d^2} = 1.875\ 10^{-3}\ \text{s}^{-1},$$

$$\psi_0 = 0.3\frac{\sigma'_0}{T_c N_{liq}} = 5{,}0 \text{ Pa/s},$$

$$\delta = \frac{(1-n)\psi_0\alpha}{\Delta n} = 1{,}5\cdot 10^{-4}\ \text{s}^{-1},$$

$$t_m = \frac{\ln(\theta/\delta)}{\theta-\delta} = 1464 \text{ s}.$$

thus the maximum P_m is obtained within the duration of the storm and has a value of,

$$P_m = \frac{\psi_0(\exp(-\delta t) - \exp(-\theta t))}{\theta - \delta} = 2141 \text{ Pa}.$$

This is 11 % of the average effective stress of 20 kPa. The total pore pressure generation is illustrated in figure 10-13.

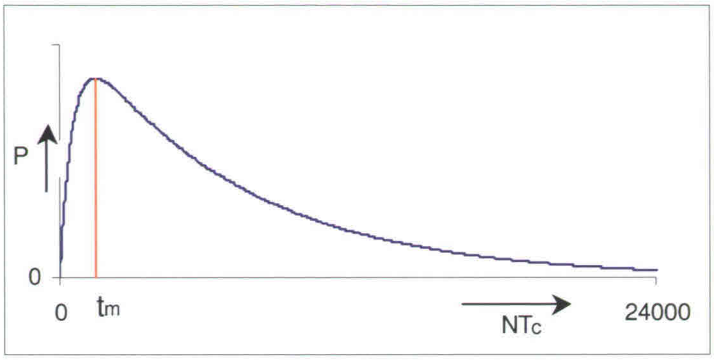


figure 10-13: pore pressure generation;

Conclusion

Liquefaction will not occur, but the shear strength is reduced by 11 %.

10.6.4 The cyclic stability of a submerged sand bed

The reaction to cyclic loading (total stresses) on a saturated submerged sandbed comprises effective stresses and pore pressures. Differences in pore pressures (pore-pressure gradients) cause flow, which in turn change the pore pressures with time. Effective stresses will also vary to compensate for the change in pore pressures. Stiffness and strength corresponding to the effective stresses are, therefore, indirectly related to the porous flow. A simple concept is available to superimpose the internal stresses (Terzaghi's effective stress rule),

$$(\sigma, \tau) = (\sigma' + p, \tau),$$

where,

σ	Total stress	[kPa],
σ'	Effective stress	[kPa],
τ	Shear stress	[kPa],
p	Pore pressure	[kPa].

The material response depends on effective stresses and pore pressures. Mohr-Coulomb's failure criterion states,

$$\tau_m = c' + \sigma' \tan(\varphi) = c' + (\sigma - p)\tan(\varphi),$$

where,

c'	Cohesion	[kPa],
φ	Internal friction angle	[°],
p	Actual pore pressure	[kPa].

The cohesion c' and the internal friction angle φ are material parameters to be measured. The actual pore pressure p is related to the actual flow field. The fact that p appears together with the total stress (loading) clarifies the importance of the flow field for the critical state.

When pore pressures are generated through density changes induced by contractancy, the shear strength is affected. A simple approach is adopted to estimate the failure potential of a submerged loosely packed non-cohesive sand bed. The dilatant behaviour is considered independent of the actual state of normal stress, if the density changes are caused by relatively small changes. Two situations are considered: the flow slide of an inclined sand bed and the shear strength of a sand bed supporting the New North Groyne.

10.6.5 Flow slide

A long slope with inclination ς is subjected to cyclic loading, see figure 10-14. The pore pressure generation due to dilatant effects is presented by p, described in the following equation,

$$p_{,t} - \psi = c p_{,zz} \quad \text{with} \quad \psi = \psi(z, t, \psi_0),$$

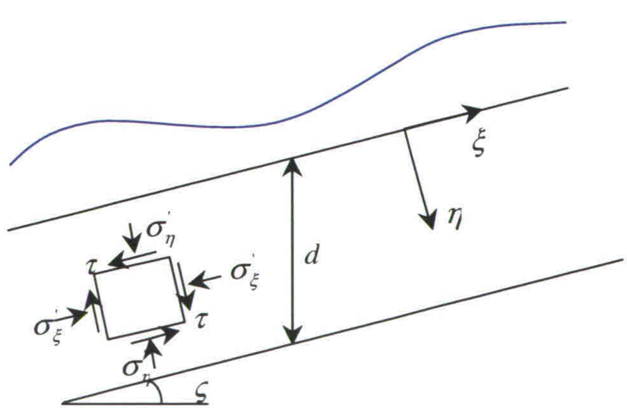


figure 10-14: cyclic loading on long slope;

The drainage of the pore water is considered perpendicular to the slope surface. The equilibrium of the slope, including the submerged weight is γ', is described by,

$$(\sigma'_\xi + p)_{,\xi} + \tau_{,\eta} + \gamma' \sin(\varsigma) = 0,$$
$$\tau_{,\xi} + (\sigma'_\eta + p)_{,\eta} - \gamma' \cos(\varsigma) = 0.$$

Where σ'_ξ, σ'_η, τ represents the effective stress tensor and (ξ, η) is the coordinate system along the slope. For a relative long slope the field state is uniform in ξ. Elaboration yields,

$$\tau = -\gamma' \sin(\varsigma)\eta,$$
$$\sigma'_\eta = \gamma' \cos(\varsigma)\eta - p.$$

The Mohr-Coulomb model describes the critical state. The mobilized shear strength is defined by $-\tau / \sigma'_\eta$ (negative sign is due to the orientation of the Mohr circle). The residual strength can be presented by a safety factor F, maximum shear strength over mobilized shear strength,

$$F = \frac{\tan(\varphi)}{-\tau / \sigma'_\eta},$$

which yields,

$$F = \frac{\left(1 - \frac{p}{\eta\gamma' \cos(\varsigma)}\right)\tan(\varphi)}{\tan\varsigma}.$$

Since it is the purpose to describe the situation in an approximate manner, the pore pressure p is simplified in an average way, such that $\int (p - \sigma'_0) dz = 0$.

Hence,

$$\frac{p}{\gamma'\eta} = \frac{2bP}{\sigma'_0 \cos(\varsigma)} \qquad \text{with} \qquad \sigma'_0 = \gamma' d,$$

and the critical state becomes,

$$F = \frac{\left(1 - \frac{2bP_m}{\sigma'_0 \cos^2(\varsigma)}\right)\tan(\varphi)}{\tan\varsigma},$$

where $P_m = Max(P[t_\infty], P[t_m])$ is the maximum value of the average pore pressure (t_∞ is the end of the loading period). The coefficient b is dependent on the ratio of drainage/storage capacity and generation rate. It appears that $b=1$ both for drained (parabolic distribution) and for undrained situations (uniform distribution). The stress state is, in fact, taken at $2d/3$ for drained situation and at $0.5d$ for undrained. By taking various values for d and adapting the current ψ_0 value for the actual average initial stress level, one may obtain an indication of the residual shear strength.

10.6.6 Apparent friction calculation

With the generated equation a direct answer can be provided if the question is which slope ς is critical for a layer of thickness d, drained at the surface,

Friction angle: $\varphi = 35^\circ$

Mean pore pressure increase: $P_m / \sigma'_0 = 0.11$ (see example pore pressure generation)

Safety factor: $F = 1.10$

$b = 1$

$$F = \frac{\left(1 - \frac{2bP_m}{\sigma'_0 \cos^2(\varsigma)}\right)\tan(\varphi)}{\tan\varsigma} \quad \rightarrow \quad \varsigma = 25^\circ$$

The values of ς can also be considered as an apparent friction angle, reflecting the residual shear strength. This represents a reduction of the friction angle of 29 %.

10.6.7 Shear strength

The stability of this marine structure depends on its foundation, which is a sand bed. A failure state is simulated by the choice of a critical kinematic system assuming continuing deformation surfaces (slip surfaces), and comparing the mobilized force with the exposed one.

Key factors are internal friction φ and cohesion c'. The stress state along the slip surfaces is determined by wave-induced pore pressures and a simplified effective stress field (Bishop's method).

Local excess pore pressures (liquefaction) can be accounted for by excess pore pressures or a reduction of the friction angle. A simple method is by the reduction of the internal friction φ: the apparent friction method. In the slip-circle approach one assumes the initial submerged stress state, while the pore pressure effect is compensated for by an apparent friction angle φ_a as follows,

$$\tau_m = c' + \sigma'(t)\tan(\varphi) = c' + \sigma'_0 \tan(\varphi_a).$$

Apparent friction angle calculation

In this calculation the slip circle method will be applied to estimate the apparent friction angle. The excess pore pressure is increased by 11 % of initial effective stress: $p = 0.11\sigma'_0$, as has been calculated in the pore pressure generation example. The internal friction angle of the sand is 35 degrees and the cohesion is not taken into account.

$$\tau_m = \sigma'(t)\tan(\varphi),$$

$$\rightarrow (\sigma'_0 - p)\tan(\varphi) = 0.89\sigma'_0 \tan(\varphi) = \sigma'_0 \tan(0.91\varphi) = \sigma'_0 \tan(\varphi_a) \rightarrow \varphi_a = 31{,}9^\circ.$$

The apparent friction angle becomes $\varphi_a = 31{,}9^\circ$, representing a reduction of 9 %. The friction angle in the flow slide calculation is reduced by 29 %. The slip circle method gave a reduction of 9 %. This is because a submerged slope is much more sensitive for liquefaction.

10.6.8 Safety factor using the reduced friction angle

Using the reduced friction angle the safety factor in comparison to the critical calculated safety factor in the FEM analysis can be calculated. Where S represents the shear strength. Introducing the standard Coulomb condition the safety factor is obtained,

$$S_f = \frac{c - \sigma_n \tan\varphi}{c_r - \sigma_n \tan\varphi_r},$$

where,

c, φ	Input strength parameters	[kPa],[°],
σ_n	Actual normal stress component	[kPa].

The subscript r means the strength parameters are reduced just enough to maintain equilibrium. The input friction angle becomes the apparent friction angle, which has a value of $\varphi = \varphi_a = 25^\circ$. The new safety factor becomes: $S_f = 1.2$ which is 33 % lower than the calculated safety factor without reduction. The slope is still safe during construction (TR 13).

10.6.9 Conclusions

Accurate determination of relevant field parameters is important to get realistic data using the described equations. The question whether liquefaction could be a case in the Durban Harbour Entrance project could not be answered reliably in this phase of the project. The apparent friction angle will be reduced by 29 %. This critical parameter is chosen as friction angle in the slope stability calculations. The new safety factor has a value of 1.2, which is still safe during construction.

10.7 Conclusions and recommendations

10.7.1 Conclusions

The finite element method analysis of the slope stability of the New North Groyne has proven the slope to be safe in all cross-sections during the critical construction phase. This is also the case when cyclic loading, which reduces the slope stability, is taken into account.

10.7.2 Recommendations

By using the Mohr-Coulomb model in the calculations it was impossible to correctly model the construction order of the New North Groyne slopes. It is therefore advised to re-evaluate the slope stability using a more advanced model like the Hardening Soil Model. To correctly simulate the soil properties testing is required. Based on the required soil parameters of the Hardening soil model it is recommended to at least do some Oedometer tests and some UU (Unconsolidated Undrained) and CU (Consolidated Undrained) Triaxial test on undisturbed soil samples at different depths.

The same problem occurs for the soil profile. Based on the geological surveys and the vibrocores taken in the area a soil profile was assumed, see figure 5-5. More geotechnical surveys will have to be executed to determine the real soil profile. Based on the preliminary survey and experience clay layers of limited thickness can be present in the area. The relatively low shear resistance of these layers draws potential failure circles. When such a layer is located new calculations will have to be made to determine the slope stability.

The stability of the slopes is affected by the occurring tides. As mentioned before micro instability can be caused by flow through the surface of the slope. Depending on the permeability of the soil and the water level difference between flood tide and ebb tide, a head difference can occur between the groundwater level in the slope and the water level in the channel. If this difference is too large the flow generated will dislodge grains and cause micro instability. The water level outside and inside the slope is also a load and it has to be determined which level is most unfavourable for macro instability calculations. Tidal information is shown in table 5-5 (TR 7).

11. FALLING APRON

11.1 Introduction

In chapter 8 a falling apron is chosen as the optimal option as revetment of the channel side slope. This apron has to protect the sand base from scouring caused by the current through the channel. The apron has to work as a filter on the slope and must withstand the harsh conditions of the wave climate. It is vital that the slope is well protected so no damage can occur to the construction because the New North Groyne is already in place when the falling apron is executed.

The aim of this chapter is to design a safe falling apron for the head and the trunk of the New North Groyne.

11.1.1 Execution

The falling apron is effectively an amount of stones placed at the toe of the New North Groyne after completion of the construction. When the construction is finished an instable steep slope, starting at the channel boundary, will be dredged. When the scour starts to develop and the slope is getting less steep (erosion), the slope reaches the stones. When the advancing slope reaches the falling apron the material will fall onto the developing slope. The loose elements are assumed to cover the slope to a sufficient thickness to retain most of the bed material. Maintenance is required because the created protection is an open filter.

11.1.2 Material properties

The properties of the sand at the location of the New North Groyne are derived from the vibrocore measurements and are shown in table 11-1.

D_{n50} [mm]	Sorting Ave [mm]	Dry density [tonne/m^3]	Porosity [-]
0.43	0.76	1.53	0.42

table 11-1: sand properties Facies A;

Detailed sand properties were acquired at the sediment hopper station. These properties are shown in table 11-2. It is assumed that this dredged sand from the Southern Breakwater area has approximately the same properties as the sand in the New North Groyne area (A 21). In the calculations the material properties from table 11-2 will be used.

D_{50} [mm]	D_{n50} [mm]	D_{b85} [mm]	D_{b15} [mm]
0.311	0.261	0.721	0.184

table 11-2: sediment hopper station sand properties;

11.2 Falling apron rock type

11.2.1 Head

From the design chapter (8.4.3) follows that the head has to be pre-dredged down to -8.5 m CD so the dolosse can be placed down to the required -5.0 m CD. After placing the dolosse the falling apron can be placed to protect the channel slope against scouring. Because the dolosse are placed down to the -5.0 m CD (> 1.5 H_s), the apron will not have to be designed to withstand the wave attack but rather the occurring currents. To design the optimal falling apron the flow velocities will have to be evaluated. There are several types of current affecting the apron. They will be discussed one by one.

Tidal Current

The tidal difference drives a tidal velocity through the entrance channel. This current can be calculated using the threshold of motion equation. The depth averaged velocity u = 0.131 m/s (appendix V) and the critical velocity $u_* = 0.015$ m/s (Shields equation).

Long shore current

This current was found to remain relatively uniform through the water column, having a median speed of 0.2 m/s. The 1 % exceedance values range from approximately 0.60 m/s at a depth of 10 m to 0.73 m/s near the surface.

Return current ships

The return current, u_r can be found by iteration,

$$\frac{u_r}{\sqrt{gh}} = \left[\frac{1}{1 - A_s / A_c - z / h} - 1\right] \cdot \frac{V_d}{\sqrt{gh}}.$$

The result is $u_r = 1.58$ m/s.

Due to eccentricity this becomes,

$$u_{r-ecc} = (1+\frac{y}{b}) \cdot u_r = 2.0 \text{ m/s}.$$

Orbital movement

The waves generate an orbital movement in the water column. The depth of the apron during mean low water springs is 5.71 m. The waves generate a maximum flow velocity of 3.90 m/s at this level; this is a second order effect. This calculation was made using Cress. The friction of the bed and slope is not taken into account.

Conclusion

The orbital movement generates the strongest velocity and therefore this velocity determines the size of the rocks to be used. The calculations for the rock size are done using Schiereck (Ref. I4),

$$D_{n50} = 2.15 \cdot \frac{\hat{u}_b^{2.5}}{\sqrt{T} \cdot (\Delta g)^{1.5}} \qquad \text{and} \qquad W_{50} = (D_{n50})^3 \cdot \rho_a .$$

The results are represented in table 11-3.

$\hat{u}_b$ [m/s]	T [s]	D_{n50} [m]	ρ_a [tonne/m^3]	W_{50} [kg]
3.90	15	0.256	2.65	44.49

table 11-3: rock size falling apron, head;

The grading of the rock is 10-60 kg. This grading has a D_{n50} = 0.26 m and is used to ensure a save apron. This results in a D_{50} of 0.31 m. The grading graphs show that D_{f15} = 0.23 m, which results in a D_{f85} of 0.39 m.

11.2.2 Trunk

In the chosen alternative the New North Groyne will be constructed on the existing seabed (-3.5 m CD). At this depth the falling apron will, in the final situation, be susceptible to wave attack because wave influence reaches approximately to $1.5H_s$ = -3.75 m CD. The falling apron must therefore be designed to cope with these attacks. The critical location is there where the rocks are closest to the water level, at the toe of the groyne. In this area the largest influence of the wave attack can be expected. For the calculation of the required rocks the Van der Meer equation is used,

$$\xi = \frac{\tan\alpha}{\sqrt{s}},$$

with,

$$s = 2\pi \cdot \frac{H}{g \cdot T^2}.$$

For surging waves,

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

where,

H_s	Significant wave height	[m],
ρ_s	Density of armour material	[tonne/m^3],
Δ	Relative density	[-],
	$\Delta = \frac{(\rho_s - \rho_w)}{\rho_w}$	
S	Damage level	[-],
P	Permeability coefficient	[-],
D_{n50}	Nominal diameter, 50% value of sieve curve	[m],
α	Angle of the seaward slope of a structure	[°].

The design storm is 2000 waves. The accepted damage level is relatively low because loss of slope protection can cause stability problems for the New North Groyne. The results of de Van der Meer calculation are shown in table 11-4.

H_s [m]	N [-]	S [-]	T_m [s]	Porosity [-]	D_{n50} [m]	Weight [tonne]
2.5	2000	4	12	0.4	0.89	1.86

table 11-4: results Van der Meer equation;

From table 11-4 follows that the rock median required for a stable apron is 1000-3000 kg. During the dredging the sand under the apron will be removed and the rocks will fall and settle on a stable slope of approximately 1:2. The falling apron and the final situation are illustrated in figure 11-1.

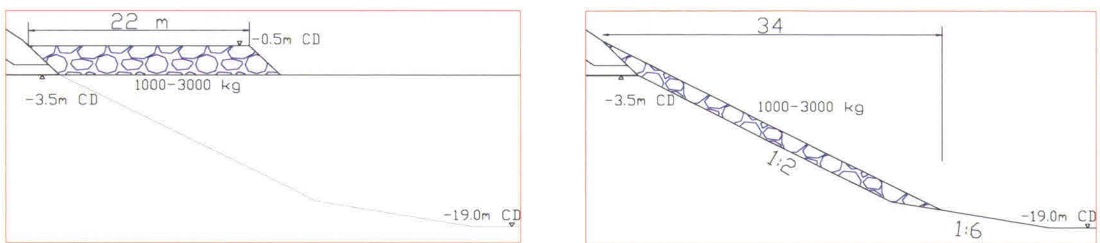


figure 11-1: falling apron placement and final protected slope;

Because of the rock size required for the protection function of the apron and the execution in the form of only one layer it is impossible to construct a closed filter that can protect the slope. The falling apron is therefore a geometrically open filter (see box 5). With the size of the chosen rocks it is almost impossible to prevent large sediment transport through the falling apron. In the next paragraph an alternative construction method will be discussed to limit this transport. An important parameter in the determination of the rock size is the water depth, therefore a pre-dredging alternative is considered in the next paragraph.

Box 5: Filters

In a **geometrically closed filter** the space between packed grains is much smaller than the grains themselves. In a **geometrically open filter** the grains of the base layer can erode through the filter layer, but the occurring gradient has to be below the critical value to prevent large erosion.

11.2.3 Trunk pre-dreging

A lower foundation level of the trunk will result in it not having to be designed on the wave attack but rather on the occurring currents. There are several currents affecting the apron that will be discussed.

Tidal Current

The tidal difference drives a tidal velocity through the entrance channel. This current can be calculated using the threshold of motion equation. The depth averaged velocity u = 0.131 m/s (appendix V) and the critical velocity $u_* = 0.015$ m/s (Shields equation).

Long shore current

The currents were found to remain relatively uniform through the water column, having a median speed of 0.2 m/s. The 1 % exceedance values range from approximately 0.60 m/s at a depth of 10 m to 0.73 m/s near the surface.

Return current ships

The return current, u_r is found by iteration,

$$\frac{u_r}{\sqrt{gh}} = \left[\frac{1}{1 - A_s / A_c - z / h} - 1\right] \cdot \frac{V_d}{\sqrt{gh}}.$$

This gives u_r =1.58 m/s,

Due to eccentricity this becomes,

$$u_{r-ecc} = (1 + \frac{y}{b}) \cdot u_r = 2.0 \text{ m/s.}$$

Orbital movement

The waves generate an orbital movement in the water column. The depth of the apron during mean low water springs is 4.21 m. The waves generate a maximum flow velocity of 4.67 m/s at this level; this is a second order effect. This calculation was made using Cress. The friction of the bed and slope is not taken into account.

Conclusion

The orbital movement generates the strongest velocity and therefore this velocity determines the size of the rocks to be used. The calculations for the rock size are done using Schiereck (Ref. I4),

$$D_{n50} = 2.15 \cdot \frac{\hat{u}_b^{2.5}}{\sqrt{T} \cdot (\Delta g)^{1.5}} \quad \text{and} \quad W_{50} = \left(D_{n50}\right)^3 \cdot \rho_a .$$

The results are represented in table 11-5.

$\hat{u}_b$ [m/s]	T [s]	D_{n50} [m]	ρ_a [tonne/m^3]	W_{50} [kg]
4.67	15	0.402	2.65	171.84

table 11-5: rock size falling apron, trunk;

The grading of the rock is 60-300 kg this grading has a D_{n50} = 0.43 m and is used to ensure a save apron. This results in a D_{50} of 0.15 m. The grading graphs show that D_{f15} = 0.40 m, which results in a D_{f85} of 0.60 m.

To prevent the waves from directly affecting the falling apron, pre-dredging to a depth of -7.0 m CD will have to be executed. The trunk will be constructed on a pre-dredged slope (north to south) from -3.5 m CD to -7.0 m CD. The placement of the falling apron and the final situation is illustrated in figure 11-2.

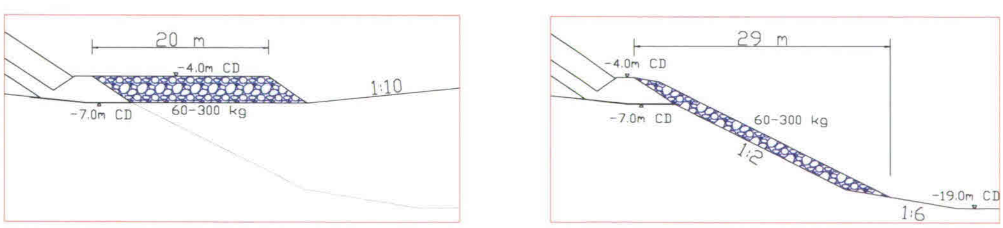


figure 11-2: falling apron and final situation, pre-dredging;

This alternative requires more dredging and more material.

11.3 Layer thickness

11.3.1 Head

The velocity and turbulence decreases inside the rock layer, but remains constant for a distance of 1.5 times D_{50} inside the layer (Ref. I2). Because of the constant value in the lower part of the rock layer, the filter protection against scour of a thicker layer does not follow from the decrease of the turbulent velocities inside the filter. The explanation is found in the longer path of the sand grains through the rock layer. In tests with the top layer directly placed on the bed material, it was observed that the bed material was transported through the top layer when the latter was less than 1.5 times the stone diameter.

To determine the thickness of the falling apron the Wörman (1989) equation is used. Although this formula is only valid for a riprap protection around bridge piers. As the protection around a bridge pier was meant to consist only of one thick single layer the formula can be used for the calculation of the thickness of a falling apron which also exists of only a single layer,

$$t = 0.16 \cdot \frac{\Delta_f}{\Delta_b} \cdot \frac{n_f}{1-n_f} \cdot \frac{D_{f85} \cdot D_{f15}}{D_{b85}},$$

where,

t	Layer thickness	[m],
0.16	Dimensionless constant which was found in experiments	[-],
Δ_f	Relative density of the filter material	[-],
Δ_b	Relative density of the bed material	[-],
n_f	Porosity of the filter material	[-],
D_{f85}	Characteristic diameter of the filter material (85 % smaller)	[m],
D_{f15}	Characteristic diameter of the filter material (15 % smaller)	[m],
D_{b85}	Characteristic diameter of the bed material (85 % smaller)	[m].

The result of the calculation is shown in table 11-6.

D_{b85} [m]	D_{f15} [m]	D_{f85} [m]	n_f [-]	Δ_f [-]	Δ_b [-]	t [m]
0.00072	0.23	0.39	0.4	1.59	1.59	13.3

table 11-6: results Wörman calculation, head;

The calculated layer thickness of 13.3 m is construction technical not feasible. It is therefore accepted that some scouring will occur. The required armour layer thickness is calculated using the following formula from the Shore Protection Manual,

$$t = n \cdot k_t \cdot D_{n50},$$

The results using the formula are given in table 11-7.

k_t [-]	n [-]	D_{n50} [m]	t [m]
1	2	0.26	0.52

table 11-7: layer thickness, head;

11.3.2 Trunk

For the trunk the same calculations are executed, see table 11-8.

D_{b85} [m]	D_{f15} [m]	D_{f85} [m]	n_f [-]	Δ_f [-]	Δ_b [-]	t [m]
0.00072	0.40	0.60	0.4	1.59	1.59	36

table 11-8: results Wörman calculation, trunk;

The calculated layer thickness of 36 m is construction technical not feasible. It is therefore accepted that some scouring will occur. The required armour layer thickness is calculated using the following formula from the Shore Protection Manual,

$$t = n \cdot k_t \cdot D_{n50}.$$

The results using the formula are given in table 11-9.

k_t [-]	n [-]	D_{n50} [m]	t [m]
1	2	0.43	0.86

table 11-9: layer thickness, trunk;

Conclusion

A layer thickness of 1.5 m is chosen for calculation purposes (Ref. AH). More calculations will have to be done to determine the erosion using this layer thickness. An outflow of sand can be acceptable but too much can cause stability problems of the fallen apron or the New North Groyne.

11.4 Material volume

11.4.1 Head

The foundation level of the falling apron is at -8.5 m CD. The head of the New North Groyne will be a round shape. Because of this curvature, the length of the edge will increase during setting: this is termed 'out-fanning'. In figure 11-3 this out-fanning is schematized. The quantity of rock stored in the apron must be adapted to this different distribution of material.

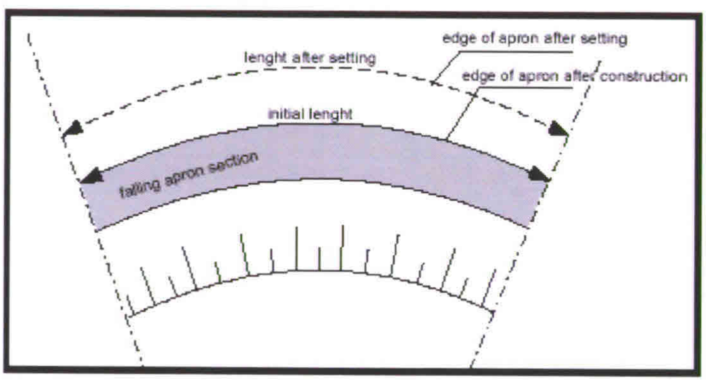


figure 11-3: curved part head;

The apron on the head will consist of 55 m^3/m rocks in the curved sections. The slope length is 26 m with a layer of 1.5 m this results in 40 m^3/m + 15 m^3/m safety and for 'out-fanning'. To ensure a save apron and to compensate for loss of rocks a volume of 65 m^3/m will be used.

11.4.2 Trunk

The height of the 1:2 slope is 15 m which gives a slope length of 33.5 m although this varies a bit over the length of the trunk. The total volume of rock needed for the slope, with a thickness of 1.5 m, is 50.0 m^2. To ensure a save apron and to compensate for loss of rocks a volume of 60 m^3/m will be used.

11.5 Conclusion and recommendations

11.5.1 Conclusions

The pre-dredged alternative requires a smaller rock size and is therefore expected to cause less erosion of the sand bed, although the precise effects have to be determined. Based on this advantage over the alternative without pre-dredging it is decided to pre-dredge both the head and the trunk of the New North Groyne.

11.5.2 Recommendations

The amount of scouring occurring through the falling apron has to be determined. Because it is almost certain at least some scouring will occur maintenance of the filter bed is required. Due to the lack of experience it is advisable to simulate the falling apron technique, preferable with a physical model.

12. CONCLUSIONS AND RECOMMENDATIONS

12.1 Introduction

This chapter will give the conclusion of the report. This conclusion will provide a description of the extent in which the objective of the project has been reached. Recommendations for future study and research required to reach the objective will be given.

The problem definition and objective, given in chapter 3, are repeated below:

12.1.1 Problem definition

The absence of a definitive design and optimal phasing of the construction of the New North Groyne in the Durban Harbour Entrance Widening and Deepening Project.

12.1.2 Objective

To create a definitive design and optimal phasing of the construction of the New North Groyne in the Durban Harbour Entrance Widening and Deepening Project.

12.2 Conclusions

Using the design process described in chapter 4 an optimal phasing and definite design have been made in chapter 7 and chapter 8.

The construction will start with the construction of the trunk of the New North Groyne while the existing groyne is still in place. During construction of the New North Groyne trunk, space problems for marine equipment can therefore occur due to the closeness of the existing North Groyne. After the construction of the trunk, the construction of the New North Groyne head commences. The slope of the New North Groyne, head and trunk, must be provided with a protection against erosion and scouring during and after dredging of the channel. This will be done using a falling apron. To properly place this falling apron some pre-dredging will be done at the head and trunk area prior to construction. During construction of the New North Groyne the partial removal of the existing North Groyne can take place (land based) using backhoes and bulldozers. This is only possible to a level upon which the groyne can accomplish its main function: prevention of the ingress of sand into the channel. Therefore the first 330 meter will be removed to a level of +1.0 m CD, the other part of the North Groyne, which is not located in the surf zone can removed to a level of -4 m CD or to its foundation depth, whichever is higher. Breaking up and removing of concrete sections, concrete units and rock have to be done in order to re-use this material in the new groyne. After finishing the New North Groyne, the remaining parts of the existing North Groyne top part will be removed followed by the removal of the remaining core construction (waterborne or land based) down to the foundation level. The waterborne removal will be done using a hydraulic backhoe dredger. The obtained material will be sorted and stored at the Point area. This material will be used to construct the New North Groyne core of the head part. The Point area offers enough space to sort and temporarily store this material. After the completion the New North Groyne, the channel can be dredged to the desired channel depth of –19 m CD, the falling apron is then executed and protects the channel side slope from scouring. The dredging will be done using a trailer suction dredger. The material will be dumped offshore in two designated areas, located a few miles outside the coast. Polluted material cannot be dumped offshore; this material has to be treated to prevent damage to the environment.

The armour of the head consists of 11t dolosse in a radius of 10 metres at the top. The underlayer consists of 300-1000 kg rock and has a thickness of 1.5 m. For the core, the quarry run is used and rocks and old rubble removed from the existing North Groyne. Rocks with a grading of 3–6 t are required for the armour layer of the trunk. The underlayer and core material will be the same as for the head. Because calculations determine the required armour depth at -5.5 m CD, it is possible to place the dolos armour before the existing North Groyne is removed. Under the base of the armour layer, a 2 m high toe and a 1 m underlayer using a geotextile construction are required for foundation and filtering. A depth of –8.5 m CD has to be dredged in the head area. The radius of the head at –8.5 m CD will be 35.3 m. This solution is far easier to realise and more safe to construct than placing the dolosse on a fallen apron. A falling apron is still required to protect the channel slope against scouring. On the head of the New North Groyne this apron will be placed at a depth of -8.5 m CD, this will require some pre-dredging. A volume of 65 m^3/m of 60-300 kg rocks will be placed. For the trunk a volume of 60 m^3/m will be placed at a depth of -7.0 m CD. An AutoCAD drawing of the final design can be found in appendix Z.

Some failure mechanisms were discussed. No significant wave reflection or wave transmission is calculated. Wave run-up will occur; the extreme calculated overtopping is 21 l/s/m. The finite element method analysis of the slope stability of the New North Groyne has proven the slope to be safe in all cross-sections during the critical construction phase. This is also the case when cyclic loading, which reduces the slope stability, is taken into account.

In conclusion the New North Groyne can be build as designed in this report.

12.3 Recommendations

12.3.1 Construction method and detailed design

Research will have to be done to ensure the stability of the Durban tourist beaches. During the construction of the New North Groyne the existing hopper station will be removed and the sediment supply to the beaches will be temporarily stopped. A temporary hopper station can be build or a buffer of sediment can be placed on the beaches. This buffer might affect the sediment transport into the channel during construction and its effects will have to be evaluated.

Construction alternatives for the North Bank Revetment are not discussed in this report. It is assumed that the revetment will be an extension of the New North Groyne trunk revetment. Other solutions are nevertheless also possible. A sheet pile quay wall instead of a slope can be a land saving and viable solution. The evaluation and design of these possible alternatives might be worthwhile to consider.

No sufficient data is available on the occurring currents in the channel at the location of the head and trunk of the New North Groyne. For verification of the chosen dimensions it is recommended to obtain this information.

A risk analysis is highly recommended for the design of the falling apron. This analysis is not included in the report and therefore assumptions have been made during the multi criteria analyses. Because there is little experience with the design and execution of the falling apron in these kind of situations the risk of damage might be too high to be accepted by the client. In that case the alternative without the use of the falling apron can become viable. A risk analysis can help to illustrate the chance of damage and the extent of this damage.

The amount of scouring occurring through the falling apron has to be determined. Because it is almost certain at least some scouring will occur maintenance of the filter bed is required. Due to the lack of experience it is advisable to simulate the falling apron technique, preferable with a physical model.

12.3.2 Failure mechanisms

By using the Mohr-Coulomb model in the calculations it was impossible to correctly model the construction order of the New North Groyne slopes. It is therefore advised to re-evaluate the slope stability using a more advanced model like the Hardening Soil Model. To correctly simulate the soil properties testing is required. Based on the required soil parameters of the Hardening soil model it is recommended to at least do some Oedometer tests and some UU (Unconsolidated Undrained) and CU (Consolidated Undrained) Triaxial test on undisturbed soil samples at different depths in the subsoil of the New North Groyne area.

The same problem occurs for the soil profile. Based on the geological surveys and the vibrocores taken in the area a soil profile was assumed, see figure 5-5. More geotechnical surveys will have to be executed to determine the real soil profile. Based on the preliminary survey and experience clay layers of limited thickness can be present in the area. The relatively low shear resistance of these layers draws potential failure circles. When such a layer is located new calculations will have to be made to determine the slope stability.

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