PROBABIILISTIC DESIGN OF BREAKWATERS
THE ENNORE BREAKWATER PROJECT

Failure mechanisms
of a crested rubble mound breakwater

Saskia Plate
Hydraulic Engineering Department
Faculty of Civil Engineering
Delft University of Technology

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in co-operation with TUDelft and HASKONING

Team of supervisors

Prof.drs.ir. J.K. Vrijling
(chairman) Hydraulic Engineering Department
Faculty of Civil Engineering
Delft University of Technology

Prof.ir. K. d'Angremond
Hydraulic Engineering Department
Faculty of Civil Engineering
Delft University of Technology

Ir. F.R. Kalff
HASKONING Royal Dutch Consulting Engineers and Architects

Ir. W. Meermans
Hydraulic Engineering Department
Faculty of Civil Engineering
Delft University of Technology

1HASKONING Royal Dutch Consulting Engineers and Architects employs more than 650 persons in the Netherlands, and is represented in more than 40 countries in the rest of the world.
C'était bien dit à lui; j'approuve sa prudence:
Il était experimenté
Et savait que la méfiance
Est mère de la sûreté.

Jean de la Fontaine (1668)
Preface

This report is submitted in partial fulfilment of the requirements for a master’s degree of the Department of Hydraulic and Geotechnical Engineering of the Faculty of Civil Engineering at the Delft University of Technology. It was written under the supervision of HASKONING Royal Dutch Consulting Engineers and Architects, which, after sponsoring a three-month stay in India during which I participated in the design of the new harbour at Ennore, was so generous to give me the opportunity to use their facilities and write this report at the headoffice of HASKONING in Nijmegen.

Three reports were in fact written over the last nine months;
Report A: Design of breakwaters: a deterministic and probabilistic approach;
Report B: Disturbance of harbour basin tranquility: analysis of wave climate and failure mechanisms of breakwater;
Report C: Failure mechanisms of the armour layer of a crested rubble mound breakwater.

These reports have been integrated into the present master’s thesis. I chose to write several reports because they seemed a good way to control the development of the study. This approach also enabled my supervisors to follow the progress of my research, and gave them the occasion to intervene when necessary. I would like to take this opportunity to thank them: Professor Vrijling and Professor d’Angremond and my HASKONING supervisor F.R. Kalff for guiding me through the whole project, W. Meermans of the University of Technology for his skill in solving problems, and the many people at HASKONING (too numerous to name here) for their ‘constructive’ help during the last few months.

The master’s project required by the University of Technology is meant to prove one’s practical ability to apply scientific principles. It also serves as a proof that one can carry out independent research.

Whether or not I have shown myself capable of applying scientific principles as well as of working independently is not up to me to judge. All I can say is that the research done in a foreign environment and reported in this study has no pretension to be complete. Intended as a survey of the possible failure mechanisms of a crested rubble mound breakwater, it only aims to contribute to the construction of a lasting breakwater at Ennore.

I owe many thanks to my family and friends for supporting me, both financially and emotionally, during the course of my studies. I would especially like to thank Ruth for providing great mental support during these last few months and my adorable sister Liedeke, who proved that no ocean is too large to communicate.

Delft, 31 augustus 1995

Saskia E. Plate

S.E. Plate

June 1995
Summary

Breakwaters differ from each other by their functions or their type. The most important function of breakwaters is to allow the ships to be loaded and unloaded in calm water conditions and to provide dock or quay facilities. Several types of breakwaters can be distinguished, depending on their operation and design. The most common breakwater concepts are divided into two types: statically stable and dynamically stable breakwaters.

The site variables, set by the situation at Ennore, concern the geotechnical aspects, the construction material and the hydraulic boundary conditions. The hydraulic boundary conditions will be determined by the wave climate during extreme weather conditions, which is during cyclones. Cyclones arrive on the east coast of India usually with a return period of 2 years.

Wave motions in harbours can, under extreme conditions, endanger the handling of vessels at berth. Therefore a certain degree of tranquillity of the harbour basin is required.

The tranquillity of the harbour basin is essential for ships to load and unload in calm water conditions. Too high waves in the harbour basin will cause an unsafe handling of ships. Should the coal handling cease at Ennore, then the storage of coal will run out, leading to the breakdown of the power station.

The breakwater structure has to reduce the wave heights in the harbour by adequate layouts. By analysing the failure mechanisms of all the different parts of the breakwater, reliability functions can be formed and the probability of failure, according to the various mechanisms, can be calculated. These reliability functions, described by each failure mechanism, are mainly based on hydraulic and geotechnical parameters.

This study attempts to survey all imaginable failure mechanisms that may occur during the design, construction, and lifetime of the crested rubble mound breakwater at Ennore. In order to address the problems encountered systematically, a general fault-tree has been given.

The quantification of some failure mechanisms for the crested rubble mound breakwater at Ennore is effected for the following failure mechanisms:
- instability of a rock armour slope
- instability of an armour layer with ACCROPODE® elements
- slip circle of a rock armour layer
- horizontal displacement of the crest element.

The quantification of these failure mechanisms is a complex task which most important difficulty resides in the determination of the mean value and the standard deviation of the variables. After the calculation, an adjustment of these values may take place in order to improve the design.
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As part of the national policy of extending and improving the power supply to the region around Madras in India, Tamil Nadu Electricity Board started the construction of a new coal-fired power station at Ennore. To operate at full capacity, it has been assumed that the New North Madras Thermal Power Station will need some 16 million tons coal per year. This will be supplied partly by the Talcher coal fields in the state of Orissa. The coal will be transported by train from the Talcher fields to Paradip, a harbour city some 250km south of Calcutta (Figure 1.1). The coal will be shipped from Paradip to Ennore. Additional coal will be brought in from coal fields abroad.

Figure 1.1 coal port project: Talcher - Paradip - Ennore
The conclusion of a preliminary design study conducted by HASKONING led to a proposal for a new harbour for coal handling at Ennore. As proposed, the harbour layout with a Southern entrance will offer optimal conditions for coal import (Figure 1.2). In case the capacity of the harbour of Madras becomes too low, it should also provide good opportunities for the future development of terminals of various other cargoes. Ennore could then become a satellite harbour of Madras.

Figure 1.2 layout harbour and breakwaters with a Southern entrance at Ennore

Several special requirements are possible for the design of this new harbour. One of them is an optimum degree of tranquillity in the harbour basin. Ships entering the harbour and laying at berth must be protected against waves and swell coming from the sea during practically the whole year. Other requirements possible are related to ship manoeuvrability (for example the turning manoeuvre) and motions (stopping distance). To realise this, breakwaters are planned. Therefore the construction of breakwaters is indispensable at Ennore.
From the past, it is known that breakwaters have collapsed, especially in the Mediterranean Sea area (PIANC, 1989): the strengths of the structures could not resist the acting forces. The collapse of the breakwaters of Sines (Portugal) and of Tripoli (Libya), in the early eighties, are striking examples. To determine the possible causes that may lead to the collapse of breakwaters, a number of investigations have been carried out since (CIAD, 1985; Van der Meer, 1987; PIANC, 1989; etc.). Unfortunately, no conclusive reasons for the collapsing of breakwaters have been determined as yet.

The aim of this study is to provide a survey of the imaginable failure mechanisms that could lead to the collapse of a breakwater. These failure mechanisms will be described by their processes, so that engineers will be able to design new breakwaters while bearing these possible failure mechanisms in mind.

Figure 1.3 shows some of the possible failure mechanisms which will be worked out in this study.

![Figure 1.3 possible failure mechanisms](image-url)
This analysis is based on a literature study and is limited to a crested rubble mound breakwater. This approach has been chosen for two practical reasons: first, the study of all the possible types of breakwaters would require research beyond the scope of a master’s thesis. Secondly, and more pragmatically, the structure at Ennore will most probably be such a crested rubble mound breakwater.

The type of armour elements constitutes a further way of delimiting this study. Two possibilities for the armour layer will be considered:

1) rock slopes, provided from the selected quarry;

2) **ACCROPODE®** elements, made out of concrete, and developed by SOGREAH Ingénierie, a French Consulting Agency.

At the time of writing this report, the team of engineers based in Madras had not yet decided for which type of armour they would opt. Both types will therefore be considered in this report.

Finally, this study will be restricted to the Ultimate Limit States (U.L.S.) of the failure mechanisms of the breakwater. Considering only the U.L.S. means that the breakwater will be looked at under extreme wave conditions. The serviceability limit states of the structure will only be indicated.

The structure of this report is as follows: Chapter 2 describes breakwaters in general and the breakwater at Ennore in particular. Chapter 3 gives the boundary conditions for the U.L.S. for the situation of Ennore, whereas Chapter 4 deals with the requirements for the breakwaters. Chapters 5 to Chapter 9 describe the failure mechanisms of respectively the armour layer (Chapter 5), the toe structures and filter layers (Chapter 6), the crest element (Chapter 7), the core (Chapter 8) and the subsoil (Chapter 9) for a crested rubble mound breakwater. To get an overview of the potential problems involved in such a breakwater, the failure mechanisms will be represented by a fault-tree in Chapter 10. Calculations of a few probabilities of occurrence of the failure mechanisms which may occur for the breakwaters at Ennore are discussed in Chapter 11.
Chapter 2
Description of breakwater structures

Since Falconer’s recording of the use of "the hull of some vessel, sunk at the entrance of a small harbour, to diminish the forces of the waves" (qtd. in the Oxford English Dictionary), people have designed increasingly elaborate barriers to break the impact of waves at the front of a harbour. Section 2.1 of this chapter surveys the variety of contemporary breakwater structures, and will clarify the choices involved in HASKONING’s selection of a crested rubble mound breakwater for Ennore. Because of HASKONING’s choice for a crested rubble mound breakwater, this type of breakwater will be further worked out in this study. Section 2.2 will discuss the choice at more depth by providing a description of the structure’s main components.

2.1 Breakwater structures

Breakwaters differ in functions and in type. Following a discussion of the main functions of breakwaters, this section will describe the different types of breakwater structures.

2.1.1 Functions of breakwaters

Berth in unprotected or relatively exposed locations gives rise to a specific set of problems. Under unfavourable wind and/or swell conditions ships traditionally have to leave their berths to avoid damage from impacts with harbour structures or with other ships, and to preclude the breaking of mooring lines. Breakwaters provide ships with a safe berth; they shield harbours from wave action, and because they prevent excessive swell and currents, they fulfil a protective function (Figure 2.1).

![Diagram of a harbour and its breakwaters]

Figure 2.1 example of a harbour and its breakwaters
Chapter 2

- **Protection from waves**
  A very important function of a breakwater can be to break waves coming from deeper water. The reflection and the breaking of waves on a breakwater considerably reduces the energy in the harbour so that the vessels in the harbour receive the required protection. This applies to sailing vessels as well as to vessels at berth, which are particularly sensitive to high wave motions.

- **Influencing the sediment transport**
  Sediment transport takes place when the orbital movement of the waves brings the sediment particles in suspension while the sediment is transported by the longshore current. The sediment transport occurs mainly in the breaking zone of the waves where the turbulence is large enough to bring the particles in suspension.

  Breakwaters influence the sand transport within harbour areas because they modify the wave pattern between the deep-water area and the area beyond these breakwaters. Waves coming from deeper water have a certain amount of energy $E$, depending among others on the significant wave height $H_s$:

  $$E = \frac{1}{8} \cdot \rho_w \cdot g \cdot H_s^2 \quad (2.1)$$

  with $\rho_w$ mass density of water [kg/m$^3$]
  $g$ acceleration of gravity [m/s$^2$].

  The waves coming at the head of the breakwaters will be diffracted: part of the energy will be lost due to turbulence near the breakwater's head and part of the energy of the waves will be spread over a larger area, so that the amount of energy $E$ behind the breakwater will be less per square meter.

  The wave heights behind the breakwater are then reduced ($E$ decreases in Equation 2.1, so $H_s$ decreases also).

  Since wave heights are reduced behind the breakwater by spreading their energy by diffraction, the orbital movement of the waves will be less important and therefore there will be less particles in suspension: the transport capacity is reduced.
Such a reduction leads to the deposition of sediment at the lee-side of the breakwaters (Figure 2.2). In this example the breakwaters are constructed past the breaker zone, so that the currents with the sediment are obstructed by the breakwaters and a deposition of sediment may take place.

**Figure 2.2 accretion and erosion pattern near breakwaters protecting a harbour area**

An adequate design of a harbour’s area can limit the sedimentation problem. In designing a new harbour, siltation in the harbour or in the entrance channel of the harbour will be attempted to be avoided. To do this, a right orientation and dimensions will be given to the breakwaters of the harbour.

- **Guidance of currents**
  Vessels are difficult to manoeuvre in strong current gradients, especially when they approach a harbour and have to reduce speed to berth. Because of the lowering of the speed of the vessel, the captain might loose the grip on the rudder in strong current gradients. Because breakwaters reduce the strength of the currents within the harbour, they may be essential in the design of harbours.

- **Provision of dock or quay facilities**
  The requirement that a ship can safely enter and leave the harbour is as important to harbour efficiency as the availability of berth. The nautical operational limits can drastically effect harbour efficiency. These limits depend on the ship’s type and class, environmental conditions and the lay-out and dimensions of the harbour.
2.1.2 Types of breakwaters

Several types of breakwaters can be distinguished, depending on their principle of operation and design. The most common breakwater concepts, described in the following sections, are divided in two types of breakwaters, monolithic and non-monolithic breakwaters:

- monolithic breakwaters
  - conventional monolithic breakwaters
  - composite breakwaters

- non-monolithic breakwaters
  - rubble mound breakwaters
  - berm breakwaters
  - low-crested breakwaters

Monolithic breakwaters

* Conventional monolithic breakwaters

A monolithic breakwater is a massive structure that consists of a small number of very large elements that are usually stiff and immovable. Monolithic breakwaters can consist of concrete caissons, cellular sheet piling, stacked block walls, etc. Since its most striking feature is a vertical front wall, this type of breakwater is often referred to as a vertical wall breakwater (Figure 2.3).

![Vertical Wall Breakwater Diagram](image)

*Figure 2.3 cross-section of a caisson or vertical wall breakwater*

In the case of an impermeable vertical wall structure the wave energy is not absorbed but reflected. Because the structure is very sensitive to uneven settlement, the structure may end in a complete destruction and loss of function. To prohibit such disasters filter layers may be dumped. One of the reasons for the dumping of the filter layers is just to provide an equalization of the seabed.
* Composite breakwaters

Composite breakwaters consist of a rubble mound and a monolithic structure in one cross-section. When the water depth increases, the height of the caisson increases also, the placement of such a caisson on a large rock foundation (Figure 2.4) provides an economic alternative.

![Figure 2.4 cross-section of a vertical wall composite breakwater](image)

The monolithic and the composite breakwaters are particularly attractive for countries where insufficient good-quality rock for the construction of a conventional rubble mound breakwater is available.

Non-monolithic breakwaters

* Rubble mound breakwaters

Rubble mound breakwaters are structures built of quarry rock or other stone materials. Generally, the larger rock armour stones are used for the outer layer, which must protect the structure from wave attack.

![Figure 2.5 cross-section of a rubble mound breakwater](image)

The armour layer of the rubble mound breakwater structure is relatively porous and therefore absorbs a great part of the wave energy. The structure is also flexible: the blocks may displace in order to follow the uneven settlement of the subsoil.
* Berm breakwaters
In a berm breakwater the stones are dynamically stable. Dynamically stable breakwaters are structures in which a reshaping of the profile is allowed. Material around the still-water level may move during each run-up and run-down of the waves until the profile has reached its equilibrium.

In the case of a severe storm, the material is redistributed by the wave attack to form a natural profile as shown in Figure 2.6.

![Figure 2.6 cross-section of a berm breakwater](image)

* Low-crested breakwaters
Low-crested breakwaters (Figure 2.7) or submerged breakwaters, depending on the water level with respect to the crest elevation, are one of the breakwaters’ type which may either be statically or dynamically stable. The crest height is determined by the required wave reduction. Since wave energy is transmitted over the breakwater the armour layer has to extend on the lee-side as well.

![Figure 2.7 cross-section of a low crested breakwater](image)

An example of a low-crested dynamically stable breakwater is the reef breakwater.

The reef breakwater is a low structure, generally detached and parallel to the shore, with much overtopping (see Figure 2.8). It is composed of a mound of graded stones which is allowed to develop naturally into a dynamically stable profile (final profile), and thus contrasts with low - crested or submerged breakwaters, which are conventional statically stable rubble mound breakwaters.
2.2 The breakwater structures at Ennore

In 1989, during the preliminary design period, HASKONING made its choice of the breakwater structure for Ennore (HASKONING, 1990). After having considered different alternatives, the team of engineers based in Madras choose a classic type of breakwater: the crested rubble mound breakwater.

The crested rubble mound breakwater is the most common type of breakwater. Indeed, in Ennore, the rubble mound is quite easy to get, transport and dump, and it is much cheaper than other types of structures.

The crest element, on the top of the structure, is trusted above the sea level. This allows for transport over and inspection of the breakwaters which is one of the requirements of the structure at Ennore.
The different parts of the breakwater are presented in Figure 2.9. They will successively be described in the following sections.

![Diagram of breakwater structure]

**Figure 2.9 the different parts of the breakwater structure of Ennore**

### 2.2.1 Armour layers

The armour layers (or primary layers) protect the rubble structure with a cover layer of selected armour elements of either quarry stone or specially shaped concrete units. Each breakwater structure has two armour layers: one on the sea-side (outer armour layer) and one on the harbour-side (inner armour layer). Under the outer primary armour layer a second armour layer is often present.

The outer toe structure supports the armour layer on the sea-side of the structure. Figure 2.10 shows the place of the armour layers in the cross-section of a breakwater.

![Diagram of cross-section of breakwater]

**Figure 2.10 armour layers**

The armour layer protects a breakwater structure against wave attack.
In order to protect the crest element when the wave attack on it is too severe, armour elements are placed at the top of the armour layer (Figure 2.11).

Figure 2.11 top of the armour layer and crest element

Whereas the armour layer on the sea-side of the breakwater structure has to withstand waves coming from deeper water, the armour layer on the harbour-side has to withstand the wave overtopping. Overtopping occurs when the crest level is lower than the level of wave run-up; in this case, water will flow over the top of the breakwater.

The breakwater structures designed for the new harbour of Ennore will be high crested breakwaters, which means that overtopping should only occur a few times a year. A different design wave condition will thus be required for each side. The wave action, directly attacking one side of the breakwater, may be much less severe than that attacking the other side. Therefore the size of the stones of the armour layer on the sea-side will be larger than the size on the harbour-side of the breakwater.
The minimum armour layer thickness is usually two stones \((2D_{sD})\) for sufficient stability and geometrical tightness (Figure 2.12).

![Layer thickness diagram](image)

**Figure 2.12 layer thickness**

Armour elements in the cover layer may be placed in an orderly pattern to obtain good wedging or interlocking, or they may be placed at random.

Armour layers differ from each other primarily by their material: rock or concrete. Rock is obtained from the quarry, concrete units have to be produced. The concrete units (Tribar, ACCROPODE\(^{®}\), Tetrapods, Quadripods, Cubes, etc.) differ from each other by their shape, sharpness of edges and degree of interlocking obtainable in placement.

The team of engineers based in Madras for the design of the new breakwater structures for the harbour of the new thermal power station at Ennore compared several types of armour elements. A choice had to be made between rock and a combination of rock and ACCROPODE\(^{®}\) elements.

The armour layer with ACCROPODE\(^{®}\) elements is a single layer system developed in the early eighties by SOGREAH Ingénierie.
At the deepest point of the breakwaters for Ennore, the stability criterion for the ACCROPODE® elements requires an average weight of several tonnes. If the selected quarry is able to provide stones large enough (comparable to the required ACCROPODE® elements), then it will be cheaper to use only rock. If the quarry is unable to provide the required number of large stones, concrete elements will be considered.

At the time of writing, ACCROPODE® elements (Figure 2.13) have been selected among other concrete armour elements for several reasons. One of these reasons is because ACCROPODE® elements interlock quite easily and therefore give each other more support; they stay stable longer than other armour elements of the same weight. Settlement by the first small storms creates a compacted and strong armour layer.

Figure 2.13 ACCROPODE® elements
2.2.2 Toe structures and filter layers

A breakwater may have two toes, one at the foot of the outer slope (sea-side) and one at the foot of the inner slope (harbour-side). Toe structures are made of rock and they are permanently under water (Figure 2.14).

Filter layers separate the core of the breakwater from the seabed. They form the base for the toe protection and the layers are normally thicker under the toe than under the core. Filter layers are constructed in a way that prohibits smaller particles to be washed through the layer.

![Diagram of toe structures and filter layers](image)

**Figure 2.14 toe structures and filter layers**

2.2.3 Crest element

The crest element is the structure on top of the breakwater. Usually a road is made on the crest element. One of the purposes of the road is to facilitate transport over the breakwater (for example during construction). It further serves for maintenance and recreation.

Sometimes, crest elements have a wall at the sea-side. Such a wall leads to a substantial reduction of the amount of stones which would otherwise be needed for a comparable conventional design (Figure 2.15).
The crest element is made of concrete. The crest element is constructed in situ; each element is approximately 20 meters long and the elements are separated by expansion joints so as to enable the absorption of movements from shrinkage, differences in temperature and differential settlements.

Figure 2.15 crest element

Two situations may occur in front of the crest structure (Figure 2.16):
1) units of the armour layer are actually situated in front of the crest structure, reducing the wave impact on the crest element;
2) there are no units protecting the crest structure: the wall of the crest structure rises over the units and it faces the wave energy in full.

Figure 2.16 front of the crest structure

In the first case, most wave energy will be absorbed and wave reflection will be reduced.
In the second case, if the crest element is directly exposed to the wave attack, the incoming wave will be completely reflected.
2.2.4 Core

The core of a breakwater is usually composed of stones with a diameter smaller than that of the stones in the outer layer or in the secondary layer. These stones are provided from the quarry and are too small to be used in either armour layers. They are so to speak the crumbles of the quarry and are therefore called 'quarry run'. They will normally fill the inner space of the breakwater (Figure 2.17).

![core diagram]

*Figure 2.17 core structure*

2.2.5 Subsoil

The soil under the breakwater is called the subsoil and may consist of rock, gravel, sand, silt or clay. Because it forms the foundation of the structure, an investigation of its nature is indispensable; a detailed investigation will provide the information about the type and properties of the subsoil and the geological stratification required for the design of a reliable breakwater.
Chapter 3
Boundary conditions at Ennore

3.1 Introduction

Before the breakwaters of the coal harbour project at Ennore can be designed, the physical site conditions have to be considered. These conditions determine for a great part for example the layout of the harbour and its breakwaters, the height of the breakwaters and the size of the stones needed.

The most important site variables concern the geotechnical boundary conditions (Section 3.2), the construction materials (Section 3.3) and the hydraulic boundary conditions (Section 3.4). These conditions are fixed by the situation of Ennore.

3.2 Geotechnical boundary conditions

The investigations of the site where the breakwaters are to be constructed, have been executed during the Feasibility Study (HASKONING, 1994). The site has three layers (Figure 3.1): first a top layer of coarse dense sand, then a layer with loose fine silty sand. This layer covers a third layer with soft clay. This is important to know because these layers indicate if the bearing capacity of the sea bottom is sufficient for the breakwater structures.

![Diagram of layers](image)

**Figure 3.1 nearshore geotechnical investigations**

3.3 Construction materials

A breakwater structure of natural rock will require a large amount of stones of different sizes. The largest stones are required for the armour layer for the cross-section of the breakwater at CD-11.50m. The stones are quarried some 100km west of Ennore. From geotechnical tests, the mass density of rock ($\rho_r$) is estimated to be 2650kg/m$^3$.

If the nearby quarry is not able to produce the required amount and size of stones for the armour layer, concrete armour elements will be needed. The mass density of concrete ($\rho_c$) is estimated to be 2400kg/m$^3$.

For the calculations of the probability of occurrence of the failure mechanisms of the breakwaters, it is important to make a distinction between an armour layer of rock and an armour layer of concrete elements. For more information over these two different layers, the reader will be referred to Chapter 5.
3.4 Hydraulic boundary conditions

Every year, two monsoon periods are distinguished in the Bay of Bengal. The Southwest monsoon and the more vigorous Northeast monsoon are alternated with calm periods. During the Northeast monsoon, cyclones may reach the coast by Ennore.

From observations over the years, it appears that the most important threat to the breakwater is posed by the cyclonic conditions. Therefore, in this Section, a closer look will only be taken at cyclonic wave heights, cyclonic periods, and directions as well as at probabilities of occurrence of cyclones. Before these aspects will be analysed, different types of waves, as they may occur at Ennore during a cyclone, will be described.

3.4.1 Waves

In the following Section, the most important types of waves will be described, starting with waves with the shortest period and ending with waves with the longest period. These types of waves are consecutively: wind waves, swell, seiches and tides.

* Wind waves
As the wind blows over the sea, wind waves are generated in a variety of heights, lengths and periods. Wind waves are 'short waves' which means that they have just a short period (a few seconds to a few minutes) and thus a short wave length. Wind waves are usually quite easy to detect with a naked eye.

* Swell
When waves are generated by a storm, at some distance of Ennore, from the Roaring Forties, short, steep waves are transformed into relatively long waves, which reach the shore. Such waves, which have lengths from 30 to more than 500 times the wave height, are called 'swell'. Swell can be quite disturbing. Because of its wave length, which may be close to the length/width of a vessel, swell may disturb the navigation of vessels (unstable course deviation, roll ship motions, etc.).

* Seiches
During storms, gusts of wind may cause water level oscillations with periods of 20 to 40 minutes. These kind of oscillations causes a range of resonances in the harbour. This is important to bear in mind while designing the harbour layout.

* Tides
Tidal motions of water masses are a form of very long period wave motion that results in a rise and a fall of the water surface. These tides are caused by the attracting force of the moon and, to a lesser extent, of the sun and the planets.
These attracting forces, and the fact that the sun, moon and earth are always in motion relative to each other, causing waters of ocean basins to be set in motion. There are two tides a day at Ennore, which cause a constant change in the level at which waves attack the shore.

3.4.2 Cyclonic study

The cyclonic study is based on a deep water hindcast study carried out by OCEANWEATHER (OCEANWEATHER, 1994).

Unfortunately it is not possible to consult a cyclone-forecast study. However, in order to get some insight in the possible characteristics of cyclones, the results of the 'cyclonic hindcast study’ has therefore in this study been used.

In this Section, the following descriptions will be given:
- cyclonic wave height
- wave period during a cyclone
- cyclonic wave steepness
- water levels
- wave directions during a cyclone.

* Cyclonic wave height

In January 1995, Delft Hydraulics studied the cyclone-generated extreme winds and waves in deep water, based on the study of OCEANWEATHER.

For the study of Delft Hydraulics, the extreme wave conditions in deep water near Ennore were based on the statistical evaluation of hindcasted wave heights for selected historical storms affecting the east coast of India on a strip of land of 600km long and 50km wide. Assuming that the probability that cyclones reach the coast is uniformly distributed and considering that the width of the area with the largest wave heights in a cyclone is some 50km, the following values for Hₜ have been adopted (Table 3.1).
<table>
<thead>
<tr>
<th>storm number</th>
<th>start of the storm</th>
<th>end of the storm</th>
<th>$H_{s,max}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27-11-1952</td>
<td>30-11-1952</td>
<td>3.85</td>
</tr>
<tr>
<td>2</td>
<td>29-11-1955</td>
<td>01-12-1955</td>
<td>3.85</td>
</tr>
<tr>
<td>3</td>
<td>19-10-1963</td>
<td>21-10-1963</td>
<td>5.40</td>
</tr>
<tr>
<td>4</td>
<td>05-12-1967</td>
<td>08-12-1967</td>
<td>4.06</td>
</tr>
<tr>
<td>5</td>
<td>20-11-1972</td>
<td>22-11-1972</td>
<td>4.31</td>
</tr>
<tr>
<td>7</td>
<td>16-11-1977</td>
<td>19-11-1977</td>
<td>5.14</td>
</tr>
<tr>
<td>8</td>
<td>09-05-1979</td>
<td>12-05-1979</td>
<td>4.74</td>
</tr>
<tr>
<td>9</td>
<td>30-10-1979</td>
<td>01-11-1979</td>
<td>3.95</td>
</tr>
<tr>
<td>10</td>
<td>23-11-1979</td>
<td>25-11-1979</td>
<td>2.79</td>
</tr>
<tr>
<td>11</td>
<td>17-10-1982</td>
<td>18-10-1982</td>
<td>4.88</td>
</tr>
<tr>
<td>12</td>
<td>11-11-1984</td>
<td>14-11-1984</td>
<td>6.72</td>
</tr>
<tr>
<td>13</td>
<td>29-11-1984</td>
<td>01-12-1984</td>
<td>5.46</td>
</tr>
<tr>
<td>14</td>
<td>11-12-1985</td>
<td>14-12-1985</td>
<td>3.97</td>
</tr>
<tr>
<td>15</td>
<td>30-10-1989</td>
<td>03-11-1987</td>
<td>3.28</td>
</tr>
<tr>
<td>16</td>
<td>06-05-1990</td>
<td>09-05-1990</td>
<td>5.95</td>
</tr>
<tr>
<td>18</td>
<td>02-12-1993</td>
<td>04-12-1993</td>
<td>4.07</td>
</tr>
</tbody>
</table>

Table 3.1 cyclones affecting the coast of India - wave height analysis from 1952 to 1993

For the design of a new harbour of Ennore, wave heights in shallow water are needed, instead of wave heights in deeper water. Table 3.1 should therefore first be adapted. There are several ways to calculate wave heights in shallow water. One of these ways is to use the computer program ENDEC (acronym for ENergy DECay). The model takes into account shoaling, refraction, energy dissipation (generated by local winds) and bottom friction, gain of wave energy (due to local wind generation) and the water level (due to radiation stresses).

Another way is to assume that the wave heights in shallow water may be approximated by:

$$H_{shallow} = \gamma_r \cdot \gamma_s \cdot H_{deep} = 1.0 \cdot 0.6 \cdot H_{deep}$$

(3.1)

with $\gamma_r$ refracting coefficient [-]
$\gamma_s$ shoaling coefficient [-]
$H_{deep}$ significant wave height in deep water [m].

Table 3.2 gives the wave heights in shallow water of the cyclones affecting the coast by Ennore, using equation 3.1.
Table 3.2 wave height analysis in shallow water from 1952 to 1993

<table>
<thead>
<tr>
<th>storm number</th>
<th>start of the storm</th>
<th>end of the storm</th>
<th>$H_{s,\text{shallow}}$ [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27-11-1952</td>
<td>30-11-1952</td>
<td>2.31</td>
</tr>
<tr>
<td>2</td>
<td>29-11-1955</td>
<td>01-12-1955</td>
<td>2.31</td>
</tr>
<tr>
<td>3</td>
<td>19-10-1963</td>
<td>21-10-1963</td>
<td>3.24</td>
</tr>
<tr>
<td>4</td>
<td>05-12-1967</td>
<td>08-12-1967</td>
<td>2.44</td>
</tr>
<tr>
<td>5</td>
<td>20-11-1972</td>
<td>22-11-1972</td>
<td>2.59</td>
</tr>
<tr>
<td>7</td>
<td>16-11-1977</td>
<td>19-11-1977</td>
<td>3.08</td>
</tr>
<tr>
<td>8</td>
<td>09-05-1979</td>
<td>12-05-1979</td>
<td>2.84</td>
</tr>
<tr>
<td>9</td>
<td>30-10-1979</td>
<td>01-11-1979</td>
<td>2.37</td>
</tr>
<tr>
<td>10</td>
<td>23-11-1979</td>
<td>25-11-1979</td>
<td>1.67</td>
</tr>
<tr>
<td>11</td>
<td>17-10-1982</td>
<td>18-10-1982</td>
<td>2.93</td>
</tr>
<tr>
<td>12</td>
<td>11-11-1984</td>
<td>14-11-1984</td>
<td>4.03</td>
</tr>
<tr>
<td>13</td>
<td>29-11-1984</td>
<td>01-12-1984</td>
<td>3.28</td>
</tr>
<tr>
<td>14</td>
<td>11-12-1985</td>
<td>14-12-1985</td>
<td>2.38</td>
</tr>
<tr>
<td>15</td>
<td>30-10-1989</td>
<td>03-11-1987</td>
<td>1.97</td>
</tr>
<tr>
<td>16</td>
<td>06-05-1990</td>
<td>09-05-1990</td>
<td>3.57</td>
</tr>
<tr>
<td>18</td>
<td>02-12-1993</td>
<td>04-12-1993</td>
<td>2.44</td>
</tr>
</tbody>
</table>

Based on the data of the hindcast study, carried out by OCEANWEATHER for Delft Hydraulics (OCEANWEATHER, 1994) for these 18 cyclones, an attempt to fit a distribution on these available data was made with the computer program STATFIT.PAS, written in TurboPascal language. There are several ways possible to 'fit' a distribution on a data (Beem, 1992).

The significant wave height in shallow water seems best to follow a Gumbel distribution:

$$F(H_s/cyclone) = \exp \left(-e^{-\frac{(H_s-A)}{B}}\right) \quad (3.2)$$

with $A = 3.31$ and $B = 1.518$. 

---

*S.E. Plate 3 - 5 August 1995*
This distribution, depicted in Figure 3.2, is conditional on the occurrence of a cyclone.

Figure 3.2 *Gumbel distribution of the significant wave height* $H_s$ of cyclones
Since during this 41 year observation period, 18 cyclones occurred in the neighbourhood of Ennore, the probability of occurrence of a cyclone is approximately equal to:

\[ p(\text{cyclone}) = \frac{18 \text{ cyclones}}{41 \text{ years}} = 0.439 \quad [1/\text{year}] \quad (3.3) \]

The probability distribution of the significant wave height is consequently given by:

\[ F_H(H_s) = 1 - [1 - F(H|\text{cyclone})] \cdot p(\text{cyclone}) \quad (3.4) \]

\[ F_{H_s}(H) = 1 - 0.439 \cdot [1 - \exp(-e^{ \frac{H-3.31}{1.518} })] \quad (3.5) \]

The prediction of the extreme wave heights are given for different return periods by the following probability of occurrence of wave height per year:

\[ P(H_s > H) = 1 - F_{H_s}(H) = 0.439 \cdot [1 - \exp(-e^{ \frac{H-3.31}{1.518} })] \quad (3.6) \]
A graph of the probability of exceedance per year is given in Figure 3.3.

Figure 3.3 probability of exceedance of the significant wave height, supposing a Gumbel distribution
* Wave period during a cyclone

A hindcast study for the cyclones on the wave period was carried out by OCEANWEATHER (OCEANWEATHER, 1994). The results are shown in Table 3.3. These results are important for establishing a relationship between the wave period and the wave height. This relationship is significant for the determination of the wave steepness on deep water, which depends on the wave height and period, and influences the amount of dissipation of wave energy due to bottom friction and wave breaking in shallow water.

<table>
<thead>
<tr>
<th>storm number</th>
<th>start of the storm</th>
<th>end of the storm</th>
<th>$T_p$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27-11-1952</td>
<td>30-11-1952</td>
<td>9.39</td>
</tr>
<tr>
<td>2</td>
<td>29-11-1955</td>
<td>01-12-1955</td>
<td>8.69</td>
</tr>
<tr>
<td>3</td>
<td>19-10-1963</td>
<td>21-10-1963</td>
<td>10.61</td>
</tr>
<tr>
<td>4</td>
<td>05-12-1967</td>
<td>08-12-1967</td>
<td>9.92</td>
</tr>
<tr>
<td>5</td>
<td>20-11-1972</td>
<td>22-11-1972</td>
<td>10.65</td>
</tr>
<tr>
<td>6</td>
<td>25-11-1975</td>
<td>27-11-1975</td>
<td>8.34</td>
</tr>
<tr>
<td>8</td>
<td>09-05-1979</td>
<td>12-05-1979</td>
<td>7.96</td>
</tr>
<tr>
<td>9</td>
<td>30-10-1979</td>
<td>01-11-1979</td>
<td>9.18</td>
</tr>
<tr>
<td>10</td>
<td>23-11-1979</td>
<td>25-11-1979</td>
<td>6.29</td>
</tr>
<tr>
<td>11</td>
<td>17-10-1982</td>
<td>18-10-1982</td>
<td>10.31</td>
</tr>
<tr>
<td>12</td>
<td>11-11-1984</td>
<td>14-11-1984</td>
<td>9.77</td>
</tr>
<tr>
<td>13</td>
<td>29-11-1984</td>
<td>01-12-1984</td>
<td>10.75</td>
</tr>
<tr>
<td>14</td>
<td>11-12-1985</td>
<td>14-12-1985</td>
<td>10.87</td>
</tr>
<tr>
<td>15</td>
<td>30-10-1989</td>
<td>03-11-1987</td>
<td>10.58</td>
</tr>
<tr>
<td>16</td>
<td>06-05-1990</td>
<td>09-05-1990</td>
<td>12.18</td>
</tr>
<tr>
<td>18</td>
<td>02-12-1993</td>
<td>04-12-1993</td>
<td>10.05</td>
</tr>
</tbody>
</table>

Table 3.3 cyclones affecting the coast of India - wave period analysis from 1952 to 1993

For the analysis of the wave period, the best distribution curve was fitted on the 18 datapoints given in Table 3.3.

The wave period $T_p$ during a cyclone seemed to be best approximated by a Gumbel distribution function, with the following coefficients:

$$F_{T_p}(T) = \exp\left(-e^{-\frac{(T-A)}{B}}\right)$$

(3.7)

with $A = 8.969$
$B = 0.514$. 

---

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Figure 3.4 shows the distribution of the wave period at Ennore during cyclones.

Figure 3.4 *Gumbel distribution of the peak wave period $T_p$ of cyclones*

* Cyclonic wave steepness

The wave steepness may give some insight in the type of waves as they have been described in Section 3.4.1. For example: wave steepness of 1% or less may show that waves are coming from swell generated at a large distance of Ennore.
The determination of the wave steepness $S_p$ on shallow water depends among others on the wave height $H_e$ in shallow water and the peak wave period $T_p$, and may be described by the following equation:

$$S_p = \frac{H_{e,\text{shallow}}}{L_p} = \frac{H_{e,\text{shallow}}}{\frac{g \cdot T_p^2}{2\pi}} \quad [-]$$

(3.8)

with $L_p$ deep water wave length [m] (with period $T_p$),
g gravitational acceleration [m/s$^2$]

Table 3.4 gives for the wave steepness for the cyclonic hindcast study mentioned above:

<table>
<thead>
<tr>
<th>storm number</th>
<th>start of the storm</th>
<th>end of the storm</th>
<th>$S_p$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>27-11-1952</td>
<td>30-11-1952</td>
<td>0.017</td>
</tr>
<tr>
<td>2</td>
<td>29-11-1955</td>
<td>01-12-1955</td>
<td>0.019</td>
</tr>
<tr>
<td>3</td>
<td>19-10-1963</td>
<td>21-10-1963</td>
<td>0.018</td>
</tr>
<tr>
<td>4</td>
<td>05-12-1967</td>
<td>08-12-1967</td>
<td>0.016</td>
</tr>
<tr>
<td>5</td>
<td>20-11-1972</td>
<td>22-11-1972</td>
<td>0.015</td>
</tr>
<tr>
<td>6</td>
<td>25-11-1975</td>
<td>27-11-1975</td>
<td>0.018</td>
</tr>
<tr>
<td>7</td>
<td>16-11-1977</td>
<td>19-11-1977</td>
<td>0.011</td>
</tr>
<tr>
<td>8</td>
<td>09-05-1979</td>
<td>12-05-1979</td>
<td>0.029</td>
</tr>
<tr>
<td>9</td>
<td>30-10-1979</td>
<td>01-11-1979</td>
<td>0.018</td>
</tr>
<tr>
<td>10</td>
<td>23-11-1979</td>
<td>25-11-1979</td>
<td>0.027</td>
</tr>
<tr>
<td>11</td>
<td>17-10-1982</td>
<td>18-10-1982</td>
<td>0.018</td>
</tr>
<tr>
<td>12</td>
<td>11-11-1984</td>
<td>14-11-1984</td>
<td>0.027</td>
</tr>
<tr>
<td>13</td>
<td>29-11-1984</td>
<td>01-12-1984</td>
<td>0.018</td>
</tr>
<tr>
<td>14</td>
<td>11-12-1985</td>
<td>14-12-1985</td>
<td>0.013</td>
</tr>
<tr>
<td>15</td>
<td>30-10-1989</td>
<td>03-11-1987</td>
<td>0.011</td>
</tr>
<tr>
<td>16</td>
<td>06-05-1990</td>
<td>09-05-1990</td>
<td>0.015</td>
</tr>
<tr>
<td>17</td>
<td>12-11-1991</td>
<td>15-11-1991</td>
<td>0.018</td>
</tr>
<tr>
<td>18</td>
<td>02-12-1993</td>
<td>04-12-1993</td>
<td>0.015</td>
</tr>
</tbody>
</table>

Table 3.4 cyclones affecting the coast of India - wave steepness analysis (1952 - 1993)

The wave steepness during a cyclone seemed to be best approximated by a Gumbel distribution function, with the following coefficients:

$$F_S(S) = \exp\left[-e^{-\frac{(S-A)}{B}}\right]$$

(3.9)

with $A = 0.0295$

$B = 0.0051$. 

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Wave steepness also shows the correlation between the significant wave height $H_s$ and the wave steepness $S_p$. If there is no correlation between these two parameters, then there is no distinguishable link. This is shown in Figure 3.5:

**Figure 3.5** significant wave height $H_s$ versus wave steepness $S_p$

Figure 3.6 shows the density distribution of the wave steepness at Ennore during cyclones.
Figure 3.6 *Gumbel distribution of the wave steepness $\delta_s$ of cyclones*
• Wave directions during a cyclone

Wave directions are important to know because they determine for a great part the layout of the harbour.

The main wave direction for short period waves is strongly related to the wind direction. Results of the study carried out by OCEANWEATHER (OCEANWEATHER, 1994) show that the wave direction at the storm peak varies largely. However the most extreme wave heights are usually obtained when the centre of the cyclone is just south of Ennore, with wave directions from east to north-east.

• Water levels

Water levels may change due to wave set-up or due to seiches, tidal waves and seasonal variations and due to wind set-up and barometric pressure (Dekker, 1995).

These variations in the water levels will be described in the following.

- Wave set-up

Wave set-up occurs in the nearshore zone due to the breaking of waves by depth limitation. The dissipation of wave energy causes wave induced forces which lead to a gradual rise of the mean water level between the line where wave breaking starts and the shore. The wave set-up, depending on the significant wave height, has to be taken into account for the design height of the breakwaters.

Table 3.5 gives the increase of the water level due to wave set-up for several water depths.

<table>
<thead>
<tr>
<th>water depth [in m below MSL]</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>wave set-up [in m]</td>
<td>0.11</td>
<td>0.04</td>
<td>0.05</td>
<td>-0.03</td>
<td>-0.01</td>
</tr>
</tbody>
</table>

Table 3.5 increase of water level due to wave set-up for different water depths
- Water levels due to seiches, tidal waves and seasonal variations

As Ennore is situated close to Madras (+ 20km), the levels for Madras have been used for the water levels at Ennore. Because a storm may coincide with any stage of the tide, the level of mean high water spring (MHWS) at Madras was included in the design water level. This is CD + 1.1m (Dekker, 1995).

According to tidal tables, there is a seasonal change of the mean level along the east coast of India. For Madras, these seasonal variations are related to the general wind patterns: an onshore north-east monsoon causing a set-up at the coast during the months November to January, and an offshore-directed south-west monsoon causes a small set-down in the period June to September (Table 3.6).

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>+0.1</td>
<td>0.0</td>
<td>-0.1</td>
<td>-0.1</td>
<td>0.0</td>
<td>0.0</td>
<td>-0.1</td>
<td>-0.1</td>
<td>+0.1</td>
<td>+0.2</td>
<td>+0.2</td>
<td></td>
</tr>
</tbody>
</table>

Table 3.6 seasonal variations of mean sea level [in m]

- Wind set-up and barometric pressure

The wind set-up was calculated assuming the slope of the water surface is such that the hydrostatic forces are in equilibrium with the wind friction forces. The wave conditions with a return period of 50 years are generated by a wind speed of 30m/s. Wind set-up for different water depths are given in Table 3.7:

<table>
<thead>
<tr>
<th>water depth [in m below MSL]</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>wind set-up [in m]</td>
<td>0.29</td>
<td>0.24</td>
<td>0.15</td>
<td>0.11</td>
<td>0.08</td>
</tr>
</tbody>
</table>

Table 3.7 increase of water level due to wind set-up for different water depths

According to a study carried out by Delft Hydraulics (Dekker, 1995) the level rise, due to barometric pressure differences at Ennore is assumed to be 0.20m.
The total of all the changes in water levels, leads to the design water level above Chart Datum of the nautical chart. For the design of the breakwater, a High Water Level (H.W.L.) is required for a water depth of 10-15m. This maximum high water level depends on the following items (the wave set-up being negligible at these depths):

- Astronomical tide: +1.0m
- Seasonal variations: +0.1m
- Wind set-up: +0.2m
- Barometric pressure: +0.2m

Total: +1.5m.

For the purpose of this study, the maximum high water level (H.W.L.) will therefore be assumed to be equal to CD + 1.5m.
Chapter 4
Requirements for the breakwater structures at Ennore

4.1 Introduction

The aim of the design of the breakwaters is to ensure a sufficient harbour basin tranquillity of the new harbour at Ennore. The design of the breakwaters depends on the boundary conditions and on the necessary requirements. The boundary conditions have been described in Chapter 3. The necessary requirements will be the topic of this Chapter.

A description of harbour basin tranquillity will be given first (Section 4.2), followed by the requirements of the vessels at berth and in the harbour basin, needed to determine the degree of the harbour basin tranquillity (Section 4.3). Finally, this Chapter closes by giving a description of failure of the breakwaters and of the top-event, when the requirements needed are not fulfilled (Section 4.4).

4.2 Description of harbour basin tranquillity

Harbour basin tranquillity is needed in order to have sufficient safety. When there is not enough tranquillity, accidents might happen and vessels or harbour facilities might be damaged. For example, when the vessels are at berth and the waves are too high, the forces on the mooring lines may be too high so the vessels can hit the berth, which can result in some damage. All kinds of accidents may cause unsafe situations for sailors or harbour personal, environmental consequences and inconvenient delays, when there is not enough harbour basin tranquillity.

4.2.1 Disturbance in the harbour basin

The tranquillity of the harbour basin may be disturbed by the following phenomena:

- waves hitting the breakwaters which transfer part of the wave energy through the breakwater (wave transmission);
- waves hitting the breakwater which transfer part of their wave energy over the breakwater (wave overtopping);
- waves entering through the harbour entrance; these waves may disturb the ship's handling and navigation;
- waves reflected in the harbour basin;
- waves generated by local wind, wind set-up or vessel motions.
These phenomena are indicated in Figure 4.1.

Figure 4.1 potential causes for disturbance of the harbour basin tranquility

The pattern of wave penetration of these phenomena in the harbour basin, depends on among other things, the harbour basin layout and the wave climate. This pattern gives the contours of the wave height in percents of the wave height, coming from deeper water. This pattern is determined by the different situations created by the hydraulic conditions and for an important part by the position of the breakwaters. A correctly designed layout of the breakwaters may decrease the height of the waves at places in the harbour basin (berths, turning circle of the vessels) where it is needed.

The five causes for disturbance as they were mentioned at the beginning of this section will be described in the following sections:
1) wave transmission;
2) wave overtopping;
3) waves entering through the harbour entrance;
4) waves reflecting in the harbour basin;
5) waves generated by local wind, wind set-up or vessel motions.
4.2.2 Wave transmission

Because breakwater structures are permeable, incoming wave energy from deep water is transmitted through the breakwater. The passing of the wave energy through the breakwater is called wave transmission. Behind the breakwater the wave continues its way, but by passing the breakwater the wave looses energy. The transmitted wave height will therefore always be lower than the incoming wave height as is shown in Figure 4.2.

![Wave transmission diagram]

**Figure 4.2 wave transmission**

Transmission is expressed as a fraction of the incoming wave height:

$$ K_t = \frac{H_t}{H_i} $$  \hspace{1cm} (4.1)

with
- $K_t$ transmission coefficient [-]
- $H_t$ transmitted wave height [m]
- $H_i$ incoming wave height [m].

A recent series of experiments has led to a re-analysis of a design formula for wave transmission, including the effect of wave period or wave steepness and crest width (Van der Meer, d’Angremond, 1993). This re-analysis of a design formula led to the establishment of a linear relationship between the wave transmission coefficient $K_t$ (see equation 4.1) and the relative crest height $R_c/D_{\text{nom}}$. Here $R_c$ is the crest height above the water level and $D_{\text{nom}}$ the nominal diameter of the stones.

At this point, it would be interesting to investigate the relation between wave transmission and tranquillity in the harbour basin, in order to determine the importance of the disturbances for a safe handling and navigation of the vessels in the harbour of Ennore. This would be interesting because it seemed that wave transmission especially counts for waves with a long period because these waves pass easily through the breakwater structures.

An investigation of the relation between wave transmission and the tranquillity in the harbour basin will not be performed in this study, due to a lack of time.
4.2.3 Wave overtopping

An other disturbance from waves hitting the breakwater is wave overtopping. Wave overtopping is the passing of the waves over the breakwater, expressed in m³/s/m'. Wave overtopping occurs when breakwater structures are not high enough to withstand the waves during storms. Incoming waves from deep water may pass over the breakwater and disturb the harbour basin tranquillity (Figure 4.3).

Figure 4.3 wave overtopping

Formulae to calculate the degree of overtopping only, are difficult to establish because overtopping usually does occur simultaneously with wave transmission.

Within the framework of this study, wave overtopping will be assumed to be one of the most important causes for disturbance of the harbour basin tranquillity. In this study, failure of the breakwaters is therefore considered when the overtopping is excessive.

4.2.4 Waves entering through the harbour entrance

Waves entering the harbour basin through the harbour entrance may disturb the activities within the harbour basin too. A proper layout will have to take the main direction of waves into consideration so that a minimum number of waves will disturb the tranquillity of the harbour basin. The position of the breakwaters at Ennore (see Figure 4.1) has been planned so that the main high waves, occurring during the Northeast monsoon, will not enter directly in the harbour basin.

4.2.5 Waves reflecting in the harbour basin

Waves within the harbour basin may be reflected by the berth or the breakwaters. This reflection may enlarge the disturbance of the harbour basin tranquillity. The degree of reflection depends among others on the surface material from which the wave is reflected. For example, the wave will be totally reflected by the vertical concrete wall of the berth. This may cause nuisances to the vessels at berth.
4.2.6 Waves generated by local wind, wind set-up or vessel motions

Finally, waves generated by local winds, wind set-up or vessel motions may disturb the harbour basin tranquility.

Wind blowing over the harbour basin may develop waves and wind set-up. Because the harbour basin is not extreme long, the possibility for the wind, blowing over the harbour basin, to develop waves and wind set-up, is quite limited.

Vessel motions also create waves. But as the vessels will have low speed in the harbour basin, these waves will not be of importance.

4.3 Description of the vessels

In the previous section, the different causes of disturbance of the harbour basin tranquility were mentioned. In this section, the requirements of the vessels at berth and in the harbour basin will be described.

The harbour of Ennore will be visited by dry bulk carriers. Dry bulk carriers are designed for transport of commodities such as grain, coal, iron, bauxite, phosphate, cement. The loading and unloading of these bulk carriers is usually done by shore-based equipment. The most common method of loading and unloading is done by shore-based equipment. Another method of loading and unloading is by ship-borne equipment. Some geared vessels have this kind of equipment on board.

At the time of writing, a long debate between the Madras Port Trust and the Consultants regarding the pros and cons of geared or gearless vessels, led to the decision to use gearless vessels for the transport of the coal from Paradip to Ennore. It was decided, however, to design the coal terminal for both geared and gearless vessels to allow for import coal from abroad.

In the following section, the type of vessels entering the harbour of Ennore will be described (Section 4.3.1). Taking these vessels into consideration, a tranquillity optimization will be determined (Section 4.3.2).

4.3.1 Types of vessels entering the harbour

The berths at Ennore have been designed to accommodate, geared and gearless, Panamax bulk carriers of 65.000 DWT. Figure 4.4 shows an example of the dimensions of a gearless dry bulk carrier.

(LOA is the horizontal distance between two vertical lines: one tangent to the ship's bow and one to the ship's stern.)
Figure 4.4 typical gearless dry bulk carrier

The characteristic ship dimensions (loaded and ballasted draft, displacement) for a 65,000 DWT bulk carrier follows from the figures given in Table 4.1 below:

<table>
<thead>
<tr>
<th>type of ship</th>
<th>gearless bulk carrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>tonnage</td>
<td>65,000 DWT</td>
</tr>
<tr>
<td>displacement</td>
<td>780,000 kN</td>
</tr>
<tr>
<td>length over all (L&lt;sub&gt;OA&lt;/sub&gt;)</td>
<td>220 - 240 m</td>
</tr>
<tr>
<td>length (L)</td>
<td>220 - 235 m</td>
</tr>
<tr>
<td>beam (B)</td>
<td>32.0 - 33.5 m</td>
</tr>
<tr>
<td>draft loaded (D)</td>
<td>13.0 - 13.5 m</td>
</tr>
<tr>
<td>draft ballasted</td>
<td>7 - 8 m</td>
</tr>
</tbody>
</table>

Table 4.1 dimensions of a gearless dry bulk carrier
4.3.2 Tranquillity optimization

In order to be able to make a definition of a minimum tranquillity of the harbour basin, distinction must be made between two phases:

1) a construction phase, which stresses the importance of stability of the breakwater and workability of vessels and equipment during construction phases, and

2) an operational phase, which emphasizes the importance of the planned stability of the breakwater and of the overall tranquillity in the harbour.

In order to limit this study, it will be restricted to the operational phase. For the final design of the operational phase, the probability of occurrence of the design wave conditions is based on an optimization for the most economic lifetime of the breakwaters. To make it possible to calculate a design wave height for the breakwater stability in the final operation phase, a lifetime of 100 years has been chosen.

In the harbour of Ennore, three main activities during the operational phase are sensitive to wave disturbances:
1) loading and unloading vessels at berth;
2) manoeuvring of vessels in the harbour basin;
3) manoeuvring of tugboats at the entrance of the harbour basin when they start getting in position and when they tie up to assist the incoming vessels.

The harbour of Ennore has to meet the requirements of the Tamil Nadu Electricity Board for the new North Madras Thermal Power Station (NMTPS). Since this power station will be working 365 days a year, a continuous availability of coal is required. This in turn means an almost uninterrupted activity in the harbour, creating constraints on the tranquillity requirements.

In order to ensure a continuous availability of coal, two possible scenarios have to be kept in mind, should too high waves and/or waves with long periods penetrate the harbour:
1) ships may be unable to unload the coal, may even be unable to stay in the harbour, and
2) the harbour-master may prohibit the vessels from entering the harbour.

These two scenarios lead to the following tranquillity requirements, which are expressed here in significant wave heights (Hₜ) at berths and in the harbour basin.
Requirements for vessels at berth:
- \( H_s = 0.8 \text{m} \) (Velsink, 1993) shall not be exceeded during a certain number of days per year, which is set for Ennore at 35 days per year or 10 days consecutive. For example, significant wave heights shall not be higher than 0.8m for a total of 90% of the time.

Requirements in the harbour basin:
- \( H_s = 1.5 \text{m} \) \(^1\) shall not be exceeded during 10% of the time or for 10 consecutive days. For example, significant wave heights may not be higher than 1.5m for 35 days per year.

It depends on the admitted wave penetration pattern, whether the requirement at berth or in the harbour basin will be decisive.

4.4 Description of failure and of the top-event

Vessels need a certain degree of tranquillity in the harbour basin to navigate safely. This tranquillity can be disturbed by different causes, as is has been described earlier in this Chapter. In this study, an excessive overtopping will be assumed to be the leading cause for disturbances in the harbour basin. The breakwater structures will fail when an excessive overtopping passes over the breakwaters. This wave overtopping will create in the harbour basin, an other wave pattern. There are consequences for the loading and unloading of the vessels, when excessive overtopping occurs.

In general, failure is the exceedance of a limit state which occurs when the loading exceeds the strength. The limit state is a loading condition defined in relation to the resistance capacity of the structure (the performance) which is modelled in a reliability function. Two limit states can be distinguished: the Ultimate Limit State (U.L.S.) and the Serviceability Limit State (S.L.S.).

The U.L.S. occurs under extreme conditions when a structure not only fails, but also collapses.
The S.L.S. occurs under "normal" conditions, leading to failure without necessarily entailing the total collapse of the structure.

The requirements of a breakwater structure depend on the limit state under consideration. This report will focus on an analysis of one of the limit states: the Ultimate Limit State (U.L.S.). The U.L.S. will take only the hydraulic boundary conditions during cyclones (Chapter 3) into account.

---

\(^1\) It is difficult to say at which significant wave height the harbour-master prohibits the vessels to enter the harbour, in most of the literature a \( H_s \) of 1.5m is mentioned.
The failure mechanisms of the breakwaters can be structured as components of a fault-tree (Figure 4.5). Such a fault-tree will be used to schematize the relations between failure of a system and the not wanted final result.

For the purpose of this study, the top-event of the fault-tree for the failure mechanisms of the breakwater structures has been defined as follows:

the breakwater structures will fail in their functions if the wave overtopping is such, that the tranquillity of the harbour basin is disturbed for more than 35 days in a year or for more than 10 consecutive days per year.

This fault-tree will be worked out further in the following Chapters. Chapters 5 to Chapter 9 will describe the failure mechanisms of the armour layer (Chapter 5), the toe structures and filter layers (Chapter 6), the crest element (Chapter 7), the core (Chapter 8) and the subsoil (Chapter 9) for the crested rubble mound breakwater at Ennore respectively.
Chapter 5
Failure mechanisms of the armour layer

5.1 Introduction

In Chapter 4, a fault-tree has been given of the possible failure mechanisms which may lead to the top-event. A definition of the top-event has been given in Chapter 4. By some of these failure mechanisms, the armour layers are involved. This chapter will address the most important failure mechanisms of the armour layers, as they may occur within the framework of the Coal Project of Ennore. This will be done for the armour layer at the sea-side and for the armour layer at the harbour-side.

The main threat on the armour layer at the sea-side of the breakwater is the wave impact. For the armour layer at the harbour-side, the main threat lies in wave overtopping.

As described in Chapter 2, the armour layer will consist of rock or of ACCROPODE® elements. Depending on the type of stones of the armour layer, different failure mechanisms will have to be considered.

Unless mentioned explicitly in the text, no distinction between the armour layer at the sea-side and the armour layer at the harbour-side of the breakwater will be made. This will also be the case for an armour layer that consists of rock and an armour layer made of ACCROPODE® elements.

The main functions of the armour layers are (Figure 5.1):
1) protection of the core. If the core erodes excessively, the crest element will be unstable and loose height. This may lead to excessive wave overtopping and to failure of the breakwater;
2) protection of the crest element. To avoid that the crest element may undergo such a displacement that wave overtopping will be excessive.

When these functions are not sufficiently fulfilled, the crest element may become unstable, which may lead to excessive passage of water over the breakwater.

These functions will not be sufficiently fulfilled when one (or both) of two possible failure mechanisms, concerning the armour layer, occur. These two failure mechanisms are be the following:
1) instability of the stones of the armour layer (stones move individually);
2) slip circle of (a part of) the armour layer (stones move together).

The instability of the stones of the armour layer will be described in Section 5.2 and the slip circle of the armour layer in Section 5.3.
5.2 Instability of the armour layer

Instability of individual stones of the armour layer means that the stones do not stay in their place. The stones do not stay in place when the resulting force of the uplifting force and the gravity force surmounts the friction force (Figure 5.2).
The stones do not stay in place consequent on four main reasons (Figure 5.3):
1) hydraulic forces are too large (wave attack or wave overtopping) (Section 5.2.1);
2) insufficient support of the toe (Section 5.2.2);
3) slope of the armour layer is too steep (Section 5.2.3);
4) insufficient weight of the stones of the armour layer (Section 5.2.4).

Figure 5.3 fault-tree of the instability of the armour layer

5.2.1 Too large hydraulic forces
The hydraulic forces are to be determined by an analysis of waves in shallow water during extreme conditions. Hydraulic forces may be larger than expected because the analysis of the wave forces can not be reproduced exactly like in reality.

5.2.2 Insufficient support of the toe
The toe structure is there among other things to prevent sliding down of the stones of the armour layer. It is possible that the toe is not able to fulfil this function. The reasons herefore will be described in Chapter 6.

5.2.3 Too steep slope
The slope may become too steep by wave attacks during extreme conditions (reshaping of the slope). When this happens, stones of the armour layer may become unstable and roll down.

5.2.4 Too light stones of the armour layer
Stones of the armour layer may be too light consequent on two possible reasons (Figure 5.4):
A) the rock size of the armour units were designed too light for the wave forces they have to withstand (Section 5.2.4.1);
B) the stones lost weight due to degradation or fracture of the stones (Section 5.2.4.2).
Figure 5.4 fault-tree for too light stones

5.2.4.1 Design of the rock size of armour units

Many methods for the prediction of the rock size of armour units designed for wave attack that would prevent instability of the armour layer have been proposed in the past 50 years. In this report only the formulae derived by Van der Meer will be treated in detail. This will occur in three stages: first, some general remarks will be made. In a second stage, the formulae of Van der Meer for rock slopes will be described and an attempt for a reliability function for rock slopes will be made. Finally, the formulae of Van der Meer for an armour layer with ACCROPODE® elements will also be discussed. An attempt for a reliability function will also be carried out. The reliability functions are important because with these functions probabilistic calculations, in order to determine the probability of occurrence of instability due to a design fault for the rock size of the armour units, may be calculated.

The stability formulae of van der Meer for rock slopes as well as for armour layers with ACCROPODE® elements, are generally applied nowadays and are preferred to the formulae given by Hudson or Iribarren because they include the influence of wave period, the wave action on a slope, number of waves and permeability of the structure.
The permeability of the structure has, among other things, an influence on the instability of the armour layer. This depends on the particle size of the filter layers and of the core material and can be given by the permeability factor \( P \) (Van der Meer, 1988). Examples of the permeability factor \( P \) are shown in Figure 5.5:

\[
\begin{array}{c|c}
\text{P = 0.1} & \text{P = 0.4} \\
\hline
\text{Armour} & \text{Armour} \\
\text{Impervious} & \text{Filter} \\
\text{Dn50A / Dn50F = 4.5} & \text{Dn50A / Dn50F = 2} \\
& \text{Dn50F / Dn50C = 4} \\
\hline
\text{P = 0.5} & \text{P = 0.6} \\
\text{Armour} & \text{Armour} \\
\text{Core} & \text{No filter} \\
\text{Dn50A / Dn50C = 3.2} & \text{No core} \\
\hline
\text{Dn50A = nominal diameter of armour stones} \\
\text{Dn50B = nominal diameter of filter material} \\
\text{Dn50C = nominal diameter of core}
\end{array}
\]

**Figure 5.5** examples of the permeability factor \( P \) for various structures
The most adequate parameter describing the wave action on a slope, and some of its effects, is the surf similarity parameter, $\xi_{op}$, depending on the peak period and on the slope angle $\alpha$ and the wave steepness $s_{op}$:

$$\xi_{op} = \frac{\tan \alpha}{\sqrt{s_{op}}}$$  \hspace{1cm} (5.1)

with

$$s_{op} = \frac{2 \pi H_s}{g T_p^2}$$  \hspace{1cm} (5.2)

with \( g \) gravitational acceleration [m/s²]
\( H_s \) significant wave height [m]
\( T_p \) spectral peak period [s]

The breaker parameter $\xi$ (Battjes, 1974) is often used to describe the form of wave breaking on a structure (Figure 5.6):

\[\text{Figure 5.6 breaker types as a function of } \xi\]
In order to describe the damage caused by wave attack, a defined damage level parameter $S \, [\cdot]$ has to be introduced in the formulae. $S$ has been defined as follows (Van der Meer, Pilarczyk, 1987):

$$S = \frac{A}{D_{n50}^2} \quad (5.3)$$

where $A$ is the mean profile area from which material has been eroded, and $D_{n50}$ is the nominal median armour unit diameter defined using the median armour mass $M_{50}$ and material density, $\rho_r$:

$$D_{n50} = \left( \frac{M_{50}}{\rho_r} \right)^{1/3} \quad (5.4)$$

The "no damage" criterion is generally taken to be when $S$ is between 1 and 3. The advantage of this definition of damage is that it is not dependent on the length of the breakwater.

In the following pages, an investigation of the stability of a rock slope will be carried out first. Next an examination of the stability of an armour layer with ACCROPODE® will follow.

Armour layer with rock elements

Two formulae were derived by Van der Meer (Van der Meer, 1987) for plunging and for surging waves, respectively:

- for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2 \, P^{0.18} \left( \frac{S}{N} \right)^{0.2} \, \varepsilon^{-0.5} \quad (5.5)$$
- for surging waves:

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

(5.6)

where:

- \(H_s\) significant wave height [m]
- \(\Delta\) relative mass density [-]
- \(D_{n50}\) nominal diameter [m]
- \(\xi_{op}\) breaker parameter (depending on the peak period) [-]
- \(P\) permeability coefficient [-]
- \(S\) damage level [-]
- \(N\) number of waves [-]
- \(\alpha\) slope angle [-]

The transition from plunging to surging waves can be calculated using a critical value of \(\xi_{op,c}\):

\[
\xi_{op,c} = \left[ 6.2 \, P^{0.31} \sqrt{\tan \alpha} \right]^{1 \over P - 0.5}
\]  

(5.7)

Equations 5.5, 5.6 and 5.7 are based upon a series of model tests. More than 250 tests have been performed to ensure that all relevant variables are taken into account. As a result of this series of systematic investigations, the main shortcomings of the Hudson-type formulae have been eliminated.

Central to the probabilistic calculations is the reliability of the formulae itself. This reliability consists of a part due to the random behaviour of a rubble mound structure and a part due to curve fitting. The coefficients 6.2 and 1.0 in respectively equations 5.5 and 5.6 were treated as stochastic variables, which have a normal distribution, with 6.2 and 1.0 as mean values and a standard deviation of 0.4 and 0.08 respectively.

In general, a reliability function \(Z\) can be expressed as a function consisting of two components: the strength of the structure, expressed by 'résistance' \(R\), and the load, described by 'solllicitation' \(S\), to which it is subjected.
For formulae 5.5 and 5.6, a distinction for the different parameters between strength and load has been made as follows:

<table>
<thead>
<tr>
<th>Strength</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{e50}$</td>
<td>$H_s$</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>$T_p$</td>
</tr>
<tr>
<td>$P$</td>
<td>$N$</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>slope angle</td>
</tr>
<tr>
<td>$A$</td>
<td>erosion area in a cross-section [m$^2$]</td>
</tr>
</tbody>
</table>

The reliability functions $Z$ may be established by rearranging equations 5.5:

- for plunging waves:

$$6.2 \ P^{0.18} \ \left( \frac{S}{\sqrt{N}} \right)^{0.2} \ \xi_{op}^{-0.5} \ \Delta D_{e50} - H_s = 0 \quad (5.8)$$

$$\begin{align*}
6.2 \ P^{0.18} \ \left( \frac{A}{D_{e50}} \right)^{0.2} \ \frac{1}{\sqrt{\tan \alpha}} \ \left( \frac{2\pi H_s}{gT_p^2} \right)^{0.25} \ \Delta D_{e50} - H_s \cdot N^{0.1} &= 0 \quad (5.9)
\end{align*}$$

Rewriting equation 5.9 may lead to the following reliability function $Z$ for plunging waves:

$$Z = R - S \quad (5.10)$$

$$Z = 6.2 \left( \frac{2\pi}{g} \right)^{0.25} \ P^{0.18} \ A^{0.2} \ \frac{1}{\sqrt{\tan \alpha}} \ \Delta D_{e50}^{0.6} - H_s^{0.75} \cdot N^{0.1} \cdot T_p^{0.5} \quad (5.11)$$
- for surging waves:

\[
\frac{H_s}{\Delta D_{n50}} = 1.0 \cdot P^{-0.13} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \cdot \xi_{op}^P
\]  
\[ \text{(5.12)} \]

\[
1.0 \cdot P^{-0.13} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot \alpha} \cdot \xi_{op}^P \cdot \Delta D_{n50} - H_s = 0
\]  
\[ \text{(5.13)} \]

\[
1.0 \cdot P^{-0.13} \left( \frac{A}{D_{n50}^2} \right)^{0.2} \sqrt{\cot \alpha} \cdot \frac{1}{\sqrt{\tan \alpha}} \cdot \left( \frac{2\pi H_s}{gT_p^2} \right)^{0.5} \cdot \Delta D_{n50}
\]  
\[ - H_s \cdot N^{0.1} = 0 \]  
\[ \text{(5.14)} \]

Rewriting equation 5.14 may lead to the following reliability function Z for surging waves:

\[
Z = 1.0 \cdot P^{-0.13} \left( A^{0.2} \sqrt{\cot \alpha} \cdot (\tan \alpha)^{P} \cdot \left( \frac{g}{2\pi} \right)^{0.5P} \cdot \Delta D_{n50}^{0.6}
\]  
\[ - H_s^{(1+0.5P)} \cdot N^{0.1} \cdot T_p^{-P} \]  
\[ \text{(5.15)} \]

**Armour layer with ACCROPODE® elements**

As for the derivation of formulae for the stability of rock layers, several model tests have been executed to formulate new models for the stability of ACCROPODE® elements. This has been largely investigated by I.W. Van der Meer (Van der Meer, 1988).

In his investigation, Van der Meer compares the stability of rock with that of concrete elements (Cubes, Tetrapods and ACCROPODE®) for breakwater structures under random wave attack.

The results of the experiments showed two horizontal lines (Figure 5.6). This suggests no influence of the wave period.
The results are shown in Figure 5.7 for no damage \((N_0=0)\) and severe damage \((N_0>0.5)\).

![Figure 5.7 stability of ACCROPODE® elements](image)

**Figure 5.7 stability of ACCROPODE® elements**

From Figure 5.7, a number of conclusions can be drawn. The stability for start of damage for \(N_0=0\) is very high. This is caused by the settlement of the slope during the bedding as a "blanket" where each unit contacts several adjacent ones.

Start of damage \((N_0=0)\) and severe damage of failure \((N_0>0.5)\) are very close: this means that the initial stability of ACCROPODE® is very high, but that the structure fails in a progressive way.

Since storm duration and wave period have no influence on the stability of ACCROPODE®, and since the "no damage" and "failure" criteria are very close, the stability can be described by the following formulae:

\[
\text{Start of damage, } \quad N_0 = 0: \quad \frac{H_z}{\Delta D_{n50}} = 3.7 \quad (5.16)
\]

\[
\text{Failure, } \quad N_0 > 0.5: \quad \frac{H_z}{\Delta D_{n50}} = 4.1 \quad (5.17)
\]
The reliability of these formulae for probabilistic design has been established by considering the coefficients 3.7 and 4.1 in equation 5.16 and 5.17 respectively as stochastic variables. From analysis (Van der Meer, 1988), it followed that the standard deviation for these both coefficients (assuming a normal distribution) amounted to $\sigma = 0.25$.

For the instability of the armour layer with ACCROPODE® elements, an analysis of the ultimate limit state will be carried out. Total collapse of the armour layer will occur when $H_s/\Delta D_{50} = 4.1$.

The reliability function $Z$ may be given by:

$$Z = 4.1 \Delta D_{50} - H_s$$  \hspace{1cm} (5.18)

### 5.2.4.2 Degradation/fracture of the stones

Analysis of some recent failures of large breakwaters has revealed that degradation and/or fracture of a rock or a concrete armour element is one of the most important reasons for decreasing strength of the armour layer (PIANC, 1989).

Degradation occurs when the element has lost its ability to retain its physical properties in engineering service due to a physical attack on its durability. For an armour element, durability reflects the rate at which the breakwater structures wears down during service conditions.

Fracture of an element takes place when the impact from movement are larger than the strength capacities of the element.

Degradation and/or fracture have the effect of reducing block integrity, average weight and angularity, and thus modify the weight and interlocking properties of the armour. These degradation processes can significantly impair the performance of the armour layer during the design life of the structure and the onset of failure under less severe storm conditions than those of the "design storm" could result.

Degradation and fracture of an element, whether rock or ACCROPODE®, may occur during the production or construction phase as well as during the lifetime of the breakwaters. This report will consider only the degradation and fracture possibilities during the lifetime of the breakwater structures; i.e. it is supposed that the elements are 'intact' when they have been placed. This will occur in two stages: firstly, for a rock armour layer and secondly for an armour layer with ACCROPODE® elements.
Rock armour layer

There are two types of degradation/fracture of stones (CUR-CIRIA):
1) degradation/fracture into major pieces along flaws;
2) degradation/fracture along new lines usually resulting in edges and corners being knocked or sheared off.

In general, the possibility of major flaws (type 1) is increased in the case of large pieces of rock. Their overall ability to resist degradation/fracture is therefore likely to be reduced, the intact strength being less significant. The presence of flaws is linked to production methods: their formation takes place during the blasting process in the quarry.

Type 2 degradation/fracture in armour stone can result in the rounding of blocks, giving a reduced interlock and porosity on the armour layer. This effect may be reduced by using rocks with higher degradation resistance, higher tensile and compressive strengths.

The analysis of both types of degradation/fracture is quite complicated. Because the likelihood of occurrence of such a failure mechanism is low, this analysis will not be carried out further in this study.

Armour layer with ACCROPODE® elements

Several causes may lead to a degradation or fracture of an ACCROPODE® element. The main causes may be:
1) fracturing of the concrete as a result of the development of heat during the hardening of the concrete. The hardening of the concrete depends mainly on the form of the element and the temperature of the concrete mixture (Horden, 1986);
2) forces of adjacent elements leading to uneven stresses in the element;
3) rocking of an element: the flowing water causes the unit to rotate and leaves it in a position from which it falls back into a rest position.

The degree of rocking is a measure of the probability of degradation/fracture. Passed a certain degree, the unit will break and the armour layer will fail.
For a complete analysis of degradation/fracture of an ACCROPODE® element, all causes leading to this failure mechanism have to be considered. This failure mechanism will not be further worked out in this study. However an investigation of the probability of occurrence of degradation or fracture of an ACCROPODE® element will be recommended.

5.3 Slip circle of the armour layer

Armour layers may become unstable and erode when they have become too steep by other influences such as waves or scouring. The layer is then no longer able to withstand the gravitational forces and slides: a slip surface occurs. Soil moves side- and downwards.

Two types of slip circles of the armour layer can occur (see also Figure 5.8):  
1) slip circle on the sea-side of the structure, due to excessive wave actions;  
2) slip circle on the harbour-side of the structure, due to excessive wave overtopping.

![Diagram of slip circles](image)

**Figure 5.8 examples of slip circles of the armour layer**

A number of methods is available for the determination of the stability of slip circles (Fellenius, Taylor, Bishop, Morgenstern-Price, Spencer, etc.). Most of these methods divide the part of the structure to be analysed in a set of vertical slices (Figure 5.9).

Simplified Bishop uses a shear stress \( \tau \). This shear stress is present along the slip circle.
The shear stress $\tau$ [N/mm$^2$] is equal to (the safety factor $F$ is assumed to be equal for each slice):

$$\tau = \frac{1}{F} \left( c + \sigma'_n \tan \phi \right)$$  \hspace{1cm} (5.19)

where $F$ safety factor [-]
$c$ rubble cohesion$^1$ [N/mm$^2$]
$\sigma'_n$ effective stress in rubble [N/mm$^2$]
$\phi$ angle of internal friction of the rubble [degree].

Equilibrium along the slip circle will be reached when (Figure 5.9):

$$\Sigma \left( \gamma \ h \ b \ R \sin \alpha \right) = \Sigma \left( \frac{\tau b R}{\cos \alpha} \right)$$  \hspace{1cm} (5.20)

with $\gamma$ unit of weight of the slice [N/m$^3$]
$h$ height of the slice [m]
$b$ width of the slice [m]
$R$ radius of the circle [m]
$\alpha$ slope angle.

The angle of internal friction $\phi$ of the rubble depends on particle size distribution, shape and surface roughness, applied strain field (triaxial, plane) and strain rate, intact particle strengths and stress field.

![Diagram](image)

**Figure 5.9** slip circle calculations

---

$^1$ the rubble cohesion is related to the fact that the removal of a particle at zero effective stress requires some effort. For rock and artificial armour units, the cohesion is related to the interlocking.
If every slice has the same width \((b_1 = b_2 = b_3 \text{ etc.})\) then equations 5.19 and 5.20 lead to a new equation:

\[
F = \frac{\Sigma \left(\frac{c + \sigma_n' \tan \alpha}{\cos \alpha}\right)}{\Sigma \gamma h \sin \alpha}
\]  
(5.21)

From the vertical equilibrium, the following equation can be written:

\[
\gamma h = \sigma_n' + p + \tau \tan \alpha
\]  
(5.22)

with \(p\) pore water pressure [N/mm²].

Rewriting equation 5.22 leads to the following equation:

\[
\sigma_n' = \gamma h - p - \tau \tan \alpha
\]  
(5.23)

Inserting equations 5.19 and 5.20 in 5.21 gives the following equation:

\[
F = \frac{\Sigma \left[\frac{c + (\gamma h - p) \tan \alpha}{\cos \alpha (1 + \tan \alpha \tan \phi)}\right]}{\Sigma (\gamma h \sin \alpha)}
\]  
(5.24)

In equation 5.24, \(F\) will have to be determined iteratively. If the safety factor \(F\) is larger than 1, the structure will be safe. The state of failure is determined by \(F < 1\), so that the reliability function \(Z\) could be considered:

\[
Z = F - 1
\]  
(5.25)

or with equation 5.24:

\[
Z = \frac{\Sigma \left[\frac{c + (\gamma h - p) \tan \alpha}{\cos \alpha (1 + \tan \alpha \tan \phi)}\right]}{\Sigma (\gamma h \sin \alpha)} - 1
\]  
(5.26)
For the project of Ennore, ir. H. Laboyrie of HASKONING developed a simplified algorithm. This algorithm considers the slope angle, the crest level, the soil cohesion and the angle of internal friction. It does not, however, take the following parameters among others into account: still water level, wave height, crest width, seepage force, turbulent flow, wave period and nominal diameter of the armour and of the core. Although HASKONING's algorithm has these shortcomings, HASKONING's approach will none the less be used for convenience's sake.

5.4 Conclusions

Failure of the armour layer will lead to an excessive overtopping over the breakwater and/or a rapid disintegration of the structure. To anticipate such excessive overtopping, the possible failure mechanisms have to be borne in mind. Of those, two probable failure mechanisms have been examined, viz.:

1) instability of the armour layer
2) slip circle of the armour layer.

For the instability of the armour layer, a number of experiments have been carried out on scale-models in flumes by Van der Meer. Since the resulting empirical formulae are usable, they will be applied for the calculations of the breakwaters at Ennore.

Degradation or fracture of the armour element has not to be restricted to external forces acting on the element. Tension in the element, as a result of unequal hardening of the concrete during the construction of the element, have to be taken into account.

The study of the degradation and fracture of rock and of ACCROPODE® elements would none the less be recommended. As the analysis of the fracturing of tetrapods elements has shown (Horden, 1986), the occurrence of such fractures does affect the performance of breakwaters.

Slip circles have been analysed among others by Barends. From his study, it appeared that quite a few variables are influencing the occurrence of a slip circle. Because of the complexity of Barends' model, Bishop's simpler model has been used here for the calculations of the failure of slip circles. Therefore it needs to be pointed out, that because of the chosen approach, it is possible that some factors, which are not addressed by the simplified Bishop model, may further affect the serviceability of breakwaters.
height of crest element is too low

rotation, vertical and/or horizontal displacement

loss of height and horizontal displacement

loss of height of the crest element

excess of the core

washing-out of the core

settlement of the subaill

leakage of the subaill

horizontal displacement of the core

loss of height of the crest element

concretion of the core

washing-out of the core

settlement of the subaill

leakage of the subaill


the arrow refers to the Chapter
Chapter 6
Failure mechanisms of the toe structures and of the filter layers

6.1 Introduction

In Chapter 4, a general fault-tree has been described. In this fault-tree, the most important failure mechanisms, involving the toe structures and the filter layers, are indicated.

Further investigations on these mechanisms, which may occur within the framework of the breakwaters at Ennore, will be the topic of this Chapter. The author decided to analyse the failure mechanisms of the toe structures and of the filter layers together in one chapter because toe structures have much in common with filter layers. Sometimes even so much that they are quite undistinguishable from one each other.

One of the most important functions of the toe structures is prevent the washing-out of the filters through the armour layers. An other function is to provide a sufficient support to the armour layer. When the toe structure will not give enough support, the armour layer may be instable and will not be able to give enough support to the crest element. The crest element may loose height: the wave overtopping may then be too excessive (Chapter 4).

The most important function of the filter layers is to protect the area for and under the breakwater (toe and core). When the filter layers do not fulfil their function, the toe may not be able to fulfil on its turn its function too.

There are four failure mechanisms where the toe structures and the filter layers may be involved, leading to an insufficient support of the armour layer (see also the fault-tree in Figure 6.1):
1) instability of the armour stones of the toe (Section 6.2);
2) slip circle of the toe (Section 6.3);
3) foreshore erosion (Section 6.4);
4) erosion of the filter (Section 6.5).

![Figure 6.1 failure mechanisms of the toe structures and of the filter layers](image-url)
Another possible failure mechanism where instability of the toe structure may be involved, is the sliding of the foreshore (see main fault-tree in Chapter 4). This failure mechanism will be discussed in Chapter 9 (failure mechanisms of the subsoil).

The toe structure at the outer side of the breakwater will generally be exposed to stronger currents and higher waves than the toe structure at the inner side of the breakwater. The failure mechanisms of the inner toe structure are the same as the failure mechanisms at the outer slope, but they will only occur during extreme conditions, for example when waves overtop the breakwater structure. For convenience, only the failure mechanisms of the outer toe structure will be analysed.

6.2 Instability of the armour stones of the toe

Each wave approaching the breakwater produces a turbulent current. When this current is strong enough, armour stones may undergo an uplifting force (Figure 6.2). If the resulting force of the vertical component of the uplifting force and the gravity force surmounts the friction force, a displacement of the stones will take place.

![Diagram of forces on a stone of the toe structure](image)

**Figure 6.2** forces working on a stone of the toe structure
The armour stones of the toe may become instable when excessive displacement of the stones has taken place (Figure 6.3).

Figure 6.3 instability of the toe structure

When the stones of the toe do not provide enough support to the armour layer, the armour layer may not stay in its place. In this case, the toe structure is not fulfilling its function any more.

Three main causes of instability of the toe can be defined:

1) the geometry of the toe may be incorrect; the slope of the toe structure can be too steep, the stones will then roll down the slope.

2) the currents in the vicinity of the toe structure are higher than expected.

3) the weight of the stones is insufficient, so that the vertical component of the uplifting force is greater than the gravity force of the stones. Three causes are possible for this result:
   a) the stones, coming from the quarries, are lighter than expected;
   b) the durability of the stones diminishes during the lifetime of the breakwater;
   c) a calculation error was made during the design period.
ad a)
The expected weight of the rocks is not as expected, due to for example: inadequate materials, primitive facilities, poor controls.

ad b)
The durability depends on chemical corrosion (oxidization of the iron minerals, dissolving of calcium), fatigue of the material (ease of splitting or rupture) and physical corrosion (wearing capacity).

ad c)
The design of the weight of the stones at the outer side of the breakwater has been the subject of a study carried out by Delft Hydraulics. The stability of armour stones on the toe berm, placed at a level \( h_t \) below the water level on a structure, including the damage level \( N_{nd} \) (Figure 6.4), may be estimated from the following equation (Van der Meer, 1995):

\[
\frac{H_s}{\Delta D_{s50}} = (0.24 \frac{h_t}{D_{s50}} + 1.6)N_{nd}^{0.15}
\]

with \( N_{nd} = 0.5 \) start of damage, \( N_{nd} = 2 \) some flattening out and \( N_{nd} = 4 \) complete flattening out of the toe. The damage level \( N_{nd} \) is the actual number of stones removed, related to a width (along the longitudinal axis of the structure) of one nominal diameter \( D_{s50} \) (Van der Meer, 1995).

![Figure 6.4 definition sketch of Van der Meer's formula](image)

With equation 6.1, a reliability function may be defined. With this function, the probability of occurrence of this failure mechanism may be calculated. This will actually fall outside the scope of this study.
6.3 Slip circle of the toe structures

Slip circle of the toe structure is another possible failure mechanism. Stones of the toe may slide down inlumps; the support for the armour layer will disappear. For more information over slip circles, the reader will be referred to Chapter 5.

The most critical slip circle is calculated by considering the moment of the weight and the shear resistance along the sliding surface (Bishop). The soil mass is divided into several slices. The mechanical equilibrium of each of them is considered (see Figure 6.5). Inter-slices forces between adjacent slices are disregarded.

Figure 6.5 principle of Bishop's method (CUR/CIRIA)

The most important factors for slip stability are pore pressures and the water table. Both may be controlled by adequate drainage and must be take into account in a reliability function.

Since geotechnical instability along a slip circle may occur in very poor conditions of the subsoil and since the seabed at Ennore seems to be strong enough, the probability of occurrence of failure of the breakwater structures, due to a slip circle at the toe of the structure, is quite low. Therefore the slip circle of the toe structure will not be further be analysed.
6.4 Foreshore erosion

The breakwaters of Ennore are founded on a seabed of sand (see Chapter 3). When the combined effects of waves and currents exceed a threshold level, bed material may be eroded from areas with high shear stresses/differences in pressure. Wave and current velocities may act upon to the toe structure, leading to an increased bed movement. The bed movement appears as a local erosion in front of or alongside the structure. The erosion may in turn intensify the general reduction which may lead to changes in the seabed level. Prevention of local erosion should therefore be a principal objective.

Foreshore erosion is an excessive erosion of the subsoil in front of the structure, which leads to instability of the toe structure (see Figure 6.6). This erosion may take place on an unprotected or on an insufficient protected foreshore.

![Diagram of foreshore erosion]

Figure 6.6 foreshore erosion

Erosion takes place when the outgoing sediment volume is larger than the incoming sediment volume, due to a combined effect of current and oblique wave attack. This means that the gradient of sedimentation transport in time or with respect to the place is less than zero:

$$\frac{dS}{dt} < 0 \; ; \; \frac{dS}{dx} < 0$$  \hspace{1cm} (6.2)

Transport depends, among other things, on the properties of the seabed material properties (grain size, mass density, cohesion, etc.) and the wave direction.
6.5 Erosion of the filter layers

Filters are made to prevent foreshore erosion (as described in Section 6.4) which can cause instability of the toe structure. When erosion of the filter layers occurs, the toe structure will not be stable any longer and it will not be able to support the armour layer any more. This will lead to the instability of the crest element and to the top-event as described in Chapter 4.

Erosion of the filter layers may occur when the hydraulic loads, due to a combined effect of current and wave attack, are higher than expected: too strong currents, wave turbulence and pore-pressure build-up are mainly responsible for the instability of the filter layers. A pore-pressure build-up in a rock structure may occur due to external time-dependent loads and local deformation of the granular skeleton. It dissipates by porous flow and unloading. Because excess pore pressures cause a decrease of the effective stresses and consequently of a reduction in the shear strength of the granular skeleton, it is important to consider these pressures in a stability analysis.

If the properties of the filter material are insufficient, excessive erosion will destabilize the toe (Figure 6.7). They are insufficient for example when smaller particles beneath the filter are washed through the layer and the filter stones are washed through the armour or if the weight is too low.

![Figure 6.7 erosion of the filter layers](image-url)
6.6 Conclusions

In this Chapter, the most important failure mechanisms involving the toe structures and the filter layers have been described. These mechanisms are:
- instability of the armour stones of the toe structure;
- slip circle of the toe structure;
- foreshore erosion;
- erosion of the filter layers.

Experiences in flumes have shown that instability at the toe may be an important failure mechanism (Van der Meer, 1995).

Slip circle of the toe structures has been briefly described. As the situation at Ennore may not easily allow slip circles to happen, a further analyse of this failure mechanism has been skipped.

Foreshore erosion and erosion of the filter layers may occur also when design faults are involved. The design faults of both failure mechanisms must actually be very large for the occurrence of failure of the breakwater itself. In other words, foreshore erosion and erosion of the filter layers may occur, but the possibility that these erosions are so excessive that failure of the breakwater is involved, remains low.

This Chapter described the most important failure mechanisms involving the toe structures and the filter layers, the following Chapter will describe the most important failure mechanisms involving the crest element.
Chapter 7
Failure mechanisms of the crest element

7.1 Introduction

This Chapter will focus on the failure mechanisms of the crest element. Because it is a vast subject in itself, the analysis of these mechanisms has been limited in this Chapter by selecting one single type of crest element as object of study: that one chosen for the breakwater at Ennore. A crest element with a wall at the seaside of the element was chosen.

The failure mechanisms involving the crest element will be examined in consideration of three main mechanisms (Figure 7.1). The common point of these mechanisms is that the crest element looses height so that more overtopping over the breakwater will occur than is allowed for sufficient basin tranquillity (Chapter 4). The three main mechanisms are:

1) loss of height of the crest element by a vertical displacement. This displacement may occur due to different possible causes (compaction and washing-out of the core, settlement of the subsoil and liquefaction). Because these causes involve the core and the subsoil, they will be discussed in Chapters 8 and 9 respectively.

2) loss of height of the crest element by a vertical and horizontal displacement or by a rotation and a vertical displacement. This failure mechanism will be described in Section 7.3.

3) loss of height of the crest element by fracturing of the element. This failure mechanism will be briefly looked at in Section 7.4.

The probability of occurrence of these failure mechanisms depends on the strength of the element and on the loadings on this element. The loadings will be represented by the hydraulic boundary conditions. The hydraulic boundary conditions will be given in the following Section.
Figure 7.1 fault-tree of the failure mechanisms of the crest element

7.2 Hydraulic boundary conditions

In order to be able to analyse the failure mechanisms of the crest element as they have been mentioned in Section 7.1, a definition of a framework for the hydrostatic and quasi-static pressures which may act on the crest element, will be given first.

Generally, wave pressures by non-breaking waves are related to the run-up and run-down of the waves. Such an approach implies that the wave attack has the form of a sinus function. It also implies that when waves are getting longer (wave period increases), the sinus form will gradually disappear. The wave attack may then be treated as a load with a static and a quasi-static component on the structure. In this study, the dynamic loads of waves will not be taken into consideration, though they probably are very important. The situation, taken into consideration, is therefore a simplified one.

To express the wave attack on the crest element in terms of wave run-up and wave run-down, requires knowledge of the breaker parameter $\xi$ and hence requires the angle of the slope structure and the wave steepness (see also Chapter 5).
These determinations are difficult to obtain and difficult to work with, because there is not really one formula for the run-up, but there are various different formulae with coefficients (which are based on experiments in flumes), that have to be determined for each individual case.

In this Chapter, another formula for the wave force on the crest element will be used. Although this new formula should prove more easy to handle, it has also its drawbacks. To begin with, it deals exclusively with non-breaking waves. This restriction implies that breaking waves fall outside its purview.

In Section 7.2.1 the static and quasi-static pressures, acting on the crest element, will be defined. With these pressures, the horizontal and vertical forces will be determined in Section 7.2.2.

7.2.1 Static and quasi-static pressures

During extreme conditions severe wave attack will occur at the crest structure and, within the framework of this study, it will be assumed that only high hydrostatic ($p_{hw}$) and quasi-static pressures ($p_{qsw}$) will develop.

In this study, only the following situation will be analysed (Figure 7.2): the High Water Level (H.W.L.) lies half way the crest element and the waves overtop the crest element.
Figure 7.2 gives an overview of the parameters encountered by the definition of static and quasi-static pressure for this special situation:

![Diagram showing H.W.L., H_s, d, d_{sta}, quasi-static pressure, and hydrostatic pressure.]

**Figure 7.2 hydrostatic and quasi-static pressures**

Hydrostatic pressure, $p_{sta}$, is given by (Figure 7.3):

$$p_{sta} = \rho_w g \ d_{sta} \ [N/m^2] \quad (7.1)$$

in which $\rho_w$ mass density of water [kg/m$^3$]

$g$ acceleration due to gravity [m/s$^2$]

$d_{sta}$ water height between the maximum high water level of the tide (H.W.L.) and the bottom of the crest structure [m].

The hydrostatic pressure $p_{sta,f}$ at the foot of the breakwater (Figure 7.3) is given by:

$$p_{sta,f} = \rho_w g \ d \ [N/m^2] \quad (7.2)$$

with $d$ depth from high water level (H.W.L.) to bottom of the breakwater structure [m]
Quasi-static pressure $p_{qsta,f}$ at the foot of the breakwater is given by: (SPM, 1984)

$$p_{qsta,f} = \frac{(1 + \chi)}{2} \rho_w g \cdot \frac{H_s}{\cosh(\frac{2\pi d}{L})} = \alpha_1 \cdot \rho_w g H_s \quad [N/m^2]$$  \hspace{1cm} (7.3)

with $\alpha_1$, wave pressure factor [-]:

$$\alpha_1 = \frac{(1 + \chi)}{2} \cdot \frac{1}{\cosh(\frac{2\pi d}{L})}$$  \hspace{1cm} (7.4)

with $\chi$, wave reflection coefficient [-];

$H_s$, significant wave height [m];

$L$, wave length [m].

Since it is important to know what the total pressure at the bottom of the crest structure is, a new parameter is introduced: the resultant pressure $p_b$. Because at both sides of the crest element the static pressure will be the same, $p_{sta}$ at the harbour-side of the breakwater will also be used in the following equations.

The resultant pressure $p_b$ (Figure 7.3) for the situation at the bottom of the crest structure can be written as:

$$p_0 = \left(\frac{1}{2} \frac{H_s + d_{sta}}{H_s + d}\right) \cdot (p_{sta} + p_{qsta,f}) - p_{sta} \quad [N/m^2]$$  \hspace{1cm} (7.5)

Inserting equation 7.1, 7.2 and 7.3 in 7.5 gives the following equation:

$$p_0 = \left(\frac{1}{2} \frac{H_s + d_{sta}}{H_s + d}\right) \cdot (\rho_w g d + \alpha_1 \rho_w g H_s) - \rho_w g d_{sta} \quad [N/m^2]$$  \hspace{1cm} (7.6)

Rewriting equation 7.6 gives the following equation:

$$p_0 = \left(\frac{\alpha_1 H_s + d}{\frac{1}{2} H_s + d}\right) \cdot (\rho_w g) \cdot \left(\frac{1}{2} H_s + d_{sta}\right) - \rho_w g d_{sta} \quad [N/m^2]$$  \hspace{1cm} (7.7)
The resultant pressure $p_0$ is assumed to be equally distributed on the crest element and is given by rewriting equation 7.7:

$$p_0 = \rho g \cdot (\alpha_0 \left( \frac{1}{2} H_s + d_{sat} \right) - d_{sat}) \text{ [N/m}^2\text{]}$$  \hspace{1cm} (7.8)

with $\alpha_0$ wave pressure coefficient [-]

$$\alpha_0 = \frac{\alpha_1 H_s + d}{\frac{1}{2} H_s + d}$$  \hspace{1cm} (7.9)

---

**Figure 7.3** definitions of pressures and schematisation
7.2.2 Horizontal and vertical forces

Horizontal and vertical forces may be determined from the hydrostatic and quasi-static pressures (Figure 7.4):

![Diagram of forces](image)

**Figure 7.4 horizontal and vertical forces**

The total wave force $F_{\text{tot}}$, acting on the crest element, depends on the hydrostatic part $F_{\text{sta}}$ and on the quasi-static part $F_{\text{qsta}}$ of the pressure.

For the total horizontal force:

$$F_{\text{tot, hor}} = F_{\text{sta, hor}},_1 + F_{\text{qsta, hor}} - F_{\text{sta, hor}},_2 = F_{\text{qsta, hor}} \quad [N/m']$$ (7.10)

For the total vertical force:

$$F_{\text{tot, ver}} = F_{\text{sta, ver}} + F_{\text{qsta, ver}} \quad [N/m']$$ (7.11)
The total horizontal force $F_{\text{tot,hor}}$ may be approximated by ($F_{\text{stu,hor,1}} = F_{\text{stu,hor,2}}$):

$$F_{\text{tot,hor}} = p_0 \cdot d' = \rho_w g \left( \sigma_0 - \left( \frac{1}{2} H_s + d_{\text{sta}} \right) - d_{\text{sta}} \right) \cdot d' \quad [N/m'] \quad (7.12)$$

with $d'$ height of the crest element on which the wave pressure acts [m] (Figure 7.4).

The hydrostatic vertical force $F_{\text{stu,ver}}$ per running meter, can be derived from equation 7.1:

$$F_{\text{stu,ver}} = p_{\text{sta}} \cdot L_c = \left( \rho_w g d_{\text{sta}} \right) \cdot L_c \quad [N/m'] \quad (7.13)$$

with $L_c$ width of the crest element [m].

The quasi-static vertical force $F_{\text{qsta,ver}}$ per running meter, with an assumed triangular distribution of the uplift force, is given by:

$$F_{\text{qsta,ver}} = \frac{1}{2} \cdot p_0 \cdot L_c \quad [N/m'] \quad (7.14)$$

The total resulting vertical force, deduced from equation 7.14, is:

$$F_{\text{tot,ver}} = p_{\text{sta}} \cdot L_c + \frac{1}{2} \cdot p_0 \cdot L_c \quad [N/m'] \quad (7.15)$$

### 7.3 Loss of height of the crest element

The hydraulic boundary conditions are explained in the previous Section. In the following Section, the second failure mechanism of the crest element (loss of height of the crest element by a vertical and horizontal displacement or by a rotation and a vertical displacement) will be described. In order to be able to describe this failure mechanism, the horizontal displacement (Section 7.3.1), the vertical displacement (Section 7.3.2) and the rotation of the crest element (Section 7.3.3) will be explained individually.
7.3.1 Horizontal displacement of the crest element

A horizontal displacement of the crest element occurs when the horizontal forces of the waves are higher than the forces resulting from the friction between the crest element and its foundation (Figure 7.5).

![Figure 7.5 resulting horizontal force](image)

The following section will only consider forces of non-breaking waves.

Using equation 7.12 gives the total horizontal wave force $F_{\text{tot, hor}}$:

$$F_{\text{tot, hor}} = \rho_w g \left( a_0 \left( \frac{1}{2} H_s + d'_{\text{m}} \right) - d_{\text{slo}} \right) \cdot d' \quad [N/m'] \quad (7.16)$$

![Figure 7.6 forces acting on the crest element](image)
For the friction force, the weight $G$ of the crest structure and the uplift force of the hydrostatic pressure $F_{u_p}$ per running meter have to be calculated (Figure 7.6). The weight $G$ of the crest has been determined as:

$$G = \rho_{mat} \cdot \Omega \cdot g \ [N/m']$$  \hspace{1cm} (7.17)

with $\rho_{mat}$ mass density of the material of the crest structure [kg/m$^3$],

$\Omega$ area of the cross-section of the crest structure [m$^2$],

$g$ acceleration due to gravity [m/s$^2$].

The force of the hydrostatic pressure $F_{u_p}$ has been defined as:

$$F_{u_p} = F_{sta,ver} + F_{qua,ver}$$

$$F_{u_p} = L_c \rho_w g (d_{sta} + \frac{1}{2}d'\left(\alpha_0\left(\frac{1}{2}H_s + d_{sta}\right) - d_{sta}\right)) \ [N/m']$$  \hspace{1cm} (7.18)

Therefore, the friction force $F_{fr}$ per running meter will be:

$$F_{fr} = f (G - F_{u_p})$$

$$F_{fr} = fg\left[(\rho_{mat} \cdot \Omega) - L_c \rho_w (d_{sta} + \frac{1}{2}d'\left(\alpha_0\left(\frac{1}{2}H_s + d_{sta}\right) - d_{sta}\right))\right] \ [N/m']$$  \hspace{1cm} (7.19)

with $f$ friction coefficient [-].

The crest element will remain in place when:

$$F_{fr} - F_{tot,hor} = 0$$  \hspace{1cm} (7.20)

Inserting equations 7.16 and 7.19 in equation 7.20 gives:

$$fg\left[(\rho_{mat} \cdot \Omega) - L_c \rho_w (d_{sta} + \frac{1}{2}d'\left(\alpha_0\left(\frac{1}{2}H_s + d_{sta}\right) - d_{sta}\right))\right] - \rho_w g d'(\alpha_0\left(\frac{1}{2}H_s + d_{sta}\right) - d_{sta}) = 0$$  \hspace{1cm} (7.21)

Rewriting equation 7.21 leads to the following reliability function $Z$:

$$Z = R - S$$

$$Z = \frac{f\rho_{mat} \cdot \Omega - \rho_w d_{sta}[fL_c + \frac{1}{2}fL_c \alpha_0 d' - \frac{1}{2}fL_c d' + \alpha_0 d' + d']}{\rho_w \alpha_0 d'(\frac{1}{4}L_c + \frac{1}{2})} - H_s$$  \hspace{1cm} (7.22)
Arrived at this point of the study, it is interesting to compare the above mentioned model with comparable investigations. Jensen presented some results of measurements of forces on a crest element (Jensen, 1983, 1984), derived from various site specific studies. These results have been presented as graphs in the following pattern:

\[
\frac{F_{H,\text{max}}}{\rho g h_f L_p} = a + b \left( \frac{H_s}{A_e} \right)
\]  \hspace{1cm} (7.23)

with \( F_{H,\text{max}} \) force of the maximum wave height \((H_{\text{max}} = 2 \cdot H_s)\) [N/m²]
\( \rho \) mass density of water [kg/m³]
\( g \) acceleration of gravity [m/s²]
\( h_f \) depth of water [m]
\( L_p \) peak wave length [m]
\( a, b \) dimensionless coefficients [-]
\( H_s \) significant wave height [m]
\( A_e \) armour crest freeboard [m²].

As check point, a calculation of a force following equation 7.12 has been carried out:

\[
F_{t,\text{hor}} = \rho_w g \left( \alpha_0 \left( \frac{1}{2} H_s + d_{\text{sta}} \right) - d_{\text{sta}} \right) \cdot d' \hspace{1cm} (7.24)
\]

\[
F_{t,\text{hor}} = \rho_w g \left( 0.364 \cdot \left( \frac{1}{2} \cdot 2.31 + 13.0 \right) - 13 \right) \cdot 1.44 \hspace{1cm} (7.24)
\]

\[
F_{t,\text{hor}} = \rho_w g \cdot 11.30
\]

Filling equation 7.23 gives:

\[
\frac{F_{H,\text{tot}}}{\rho_w g h_f L_p} = \frac{\rho_w g \cdot 11.30}{\rho_w g \cdot 11.5 \cdot 99.74} = 0.009852 \approx 0.01 \hspace{1cm} (7.25)
\]

Further is:

\[
\frac{H_s}{A_e} = \frac{2.31}{5.40 - 1.50} = 0.5923 \approx 0.6
\]
An example of these results is presented in Figure 7.7.

Figure 7.7 results of measurement of forces on a crest element (Jensen, 1983, 1984)
7.3.2 Vertical displacement of the crest element

Vertical displacement of the crest element occurs when the vertical (uplift) force is greater than the weight of the crest element (Figure 7.8).

![Diagram of vertical displacement of the crest element](image)

**Figure 7.8 vertical displacement of the crest element**

The force $G$ resulting from the weight of the crest element is given by equation 7.27:

$$G = \rho_{\text{mat}} \cdot \Omega \cdot g \quad [N/m']$$  \hspace{1cm} (7.27)

with

- $\rho_{\text{mat}}$ mass density of the material of the crest structure [kg/m$^3$]
- $\Omega$ cross-section of the crest structure [m$^2$]
- $g$ acceleration of gravity [m/s$^2$].

The crest element will remain in place when:

$$G - F_{\text{tot, ver}} = 0$$  \hspace{1cm} (7.28)

Filling equations 7.15 and 7.27 in 7.28 gives the resulting vertical force $F_{\text{res}}$:

$$F_{\text{res}} = G - F_{\text{tot, ver}} = 0$$

$$F_{\text{res}} = \rho_{\text{mat}} \cdot \Omega \cdot g - [L_c \cdot \rho_w \cdot g (d_{\text{m}} + \frac{1}{2} d'[\alpha_0 \frac{1}{2} H_s + d_{\text{m}} - d_{\text{m}}])] = 0$$  \hspace{1cm} (7.29)

Rewriting equation 7.29 gives the following reliability function $Z$: 

---

*S.E. Plate* 7 - 13 August 1995
\[ Z = R - S \]
\[ Z = \frac{\rho_{\text{mat}} Q - L_c \rho_w d_{s\alpha} - \frac{1}{2} L_c \rho_w d'_{s\alpha}}{d'_{s\alpha} \rho_w a_0} - \frac{1}{2} d_{s\alpha} \]  
(7.30)

7.3.3 Rotation of the crest element

The crest element may rotate when an excessive erosion of the inner slope (the slope on the harbour-side) occurs or when the crest element is insufficiently supported by its foundation (Figure 7.9). Insufficient support of the crest element often means an unequal distribution of the weight of the crest element on its foundation, and therefore some elements might not be able to stay in their intended position.

![Diagram of crest element rotation](image)

**Figure 7.9 rotation of the crest element**

An excessive erosion of the inner side of the slope will occur when wave overtopping is higher than expected. Another cause for an excessive erosion is when the properties of the stones are insufficient (weight too low, instability due to too much steepness of the angle of the breakwater). Waves will run over the breakwater, create an instability at the top of the inner slope's armour layer, and wash away some stones (Figure 7.10).
Because of this loss of part of its foundation, the crest element will tend to rotate.

Figure 7.10 erosion of the inner slope due to excessive wave overtopping

Due to erosion over a distance $L$, the weight of the shaded part is not directly counteracted by a vertical support reaction. The crest element will rotate and will slide down from the top of the breakwater when erosion trespasses point $P$, which is situated below the centre of gravity of the crest element (Figure 7.11).

Figure 7.11 definition of rotation about point $P$

The loading parameter $S$ for the reliability function $Z$ comprises the significant wave height $H_s$; the strength parameter $R$ includes, among others, the weight of the stones and their material density.
7.4 Fracture of the crest element

In this study, fracture of the crest element will only be described when hydraulic forces occur and not by temperature induced stresses within the element.

Forces exerted by waves running up against the crest structure, will create large bending and shear forces inside the crest structure. These forces may cause the fracturing of the crest element. This may lead to a loss of height of the crest element, wave overtopping will be easier to occur which will lead to the failure of the breakwater.

To analyse all the fracture possibilities of the crest element, a definition for the different parts of the crest structure is required (Figure 7.12):

![Figure 7.12 definition of a crest element](image)

Crest elements made from concrete, allow small tensile forces caused by shear forces and bending moments. Different situations may be determined: excessive shear stresses in wall and base plate, excessive bending stresses in wall and base plate, etc.

In the following, only excessive shear stresses in the wall of an unreinforced structure will be analysed.

The shear stress $\tau_d$ in the cross-section A (with $A = b \cdot d$), due to the shear force $V_d$ is equal to (TGB, 1990) (Figure 7.13):

$$\tau_d = \frac{V_d}{b \cdot d} \quad [N/mm^2] \quad (7.31)$$
Figure 7.13 definitions of terms

Figure 7.14 gives the schematisation of the shear force $V_d$ for the situation described in Section 7.2.1. De factor $\beta$ is a reduced factor because the forces $F_{sta}$ and $F_{qsta}$ are reduced. This reduction is showed in Figure 7.14: the forces are not represented as squares, but they have a reduced area.

Figure 7.14 shear stress resulting from a shear force
Within the concrete wall of the crest element a certain degree of shear stress $\tau_a$ is acceptable. The design shear stress $\tau_a$ for unreinforced concrete depends on the quality of the concrete (TGB, 1990):

$$\tau_a = 0.4 \cdot f_b \quad [N/mm^2] \quad (7.32)$$

with $f_b$ tensile stress of the concrete $[N/mm^2]$.

The wall of the crest element will just not crack when:

$$\frac{V_d}{bd} = 0.4 \cdot f_b \quad \Rightarrow \quad V_d = 0.4 \cdot f_b \cdot bd \quad [N/m^1] \quad (7.33)$$

The magnitude of the shear force $V_d$ (see also equation 7.18) may be determined by:

$$V_d = \beta \cdot F_{vohor} \quad [N/m^1] \quad (7.34)$$

with $\beta$ the shear force coefficient for this situation. This coefficient will have to be experimentally determined in a more profound study. This will actually fall outside the scope of this study.

7.5 Conclusions

In this Chapter, the failure mechanisms of the crest element, based on the design selected for the breakwater structures at Ennore, is described. The three most important failure mechanisms of the crest element are:

1) loss of height of the crest element by a vertical displacement;
2) loss of height of the crest element by a vertical and horizontal displacement or by a rotation and a vertical displacement;
3) loss of height of the crest element by fracturing of the element.

In order to limit the complexity of the analysis of these mechanisms, some delimitations were set: even though it has the form of a sinus function, the wave attack which may occur at the level of the crest structure has been assumed to be static.

Waves, attacking the breakwater structures, have been taken to be non-breaking. They have been estimated to have the same hydrostatic and quasi-static pressures as the waves in deeper water approaching the breakwater structure. This means that the waves are not shoaling by the slope of the breakwater.
This simplification may have some significant consequences and should therefore be subjected to careful analysis. Such an analysis, however, will have to fall outside the scope of this study.

For the horizontal and vertical displacement of the crest element, a formulation of reliability functions was proposed. With these reliability functions, calculations of the probability of occurrence will be effected in Chapter 10.

Although the rotation and the fracture of the crest element will not be worked out any further here, future studies ought to look more closely into the aspects involved in these processes.
Chapter 8  
Failure mechanisms of the core

8.1 Introduction

In Chapter 4, a general fault-tree has been given in which two possible causes involve the core: compaction and washing-out of the core. These causes may lead to two different failure mechanisms:
1) loss of height of the crest element;
2) erosion of the slope.

ad 1) Loss of height of the crest element (see also Section 7.1). The causes for this loss is due to (Figure 8.1):
   - compaction of the core (Section 8.2);
   - washing-out of the core (Section 8.3);
   - settlement of the subsoil (Chapter 9);
   - liquefaction (Chapter 9).

![Figure 8.1 fault-tree of the loss of height of the crest element](image)

ad 2) Erosion of the slope. The causes for this erosion is due to (Figure 8.2):
   - insufficient weight of the stones of the armour layer (Chapter 5);
   - insufficient support by the toe (Chapter 6);
   - washing-out of the core (Section 8.3).

![Figure 8.2 fault-tree of the erosion of the slope](image)

Two possible failure mechanisms of the core will be explained in this Chapter. In Section 8.2, a description of the compaction of the core will take place. The washing-out of the core will be described in Section 8.3.
8.2 Compaction of the core

An excessive movement may take place under gravity force (under the weight of the adjacent elements), earthquake and wave loads. This phenomenon is known as compaction.

Although most of the compaction of the core occurs during construction, further compaction will take place during the lifetime of the breakwater: some rocks will move into a more stable position (principally in a vertical direction), and some stones will suffer from fracture.

As a result of compaction of the core, a lowering of the crest and horizontal deformation will occur (Figure 8.3).

![Figure 8.3 compaction of the core](image)

Compaction may also occur due to fracture of core material. This fracture depends on the durability properties of the material, the weight of the adjacent elements and the wave and/or earthquake forces acting on the stones. Core material may deteriorate during the lifetime of the breakwater by physical and/or chemical corrosion as well as by fatigue.
8.3 Washing-out of the core

To prevent internal washing-out of the particles in the core, it is necessary to ensure that the breakwater layers are properly designed as a filter. It should be taken into consideration that it might also be necessary to introduce filter layers between the seabed and the structure, and between the structure and any fine material deposited as fill against its lee side. Core material will pass through the layer immediately above it, unless passage is made impossible. This holding of the particle can be effected through the use of proper design rules.

When armour units are large, or when very fine core material is used, it may be necessary to design a multi-layer filter system. This filter system will protect the core, which in turn is protected by a second one, and so on, until the top layer is coarse enough not to wash through the armour’s voids.

Two causes of washing-out of the core material need to be distinguished:
1) the protection by the outer layer is insufficient
2) the currents through the breakwater are too strong and the voids too wide.

In the case of insufficient protection of the outer layer, (for instance, when the voids in the cover layer are larger than the grains in the filter layer underneath), the sublayer may be washed out through the voids of the coverlayer.

For larger voids the flow velocities of the outflowing water decrease as a result of the large cross-section and of the reduction of the gradient over the cover layer.

The principle of washing-out of particles is shown in Figure 8.4.

![Figure 8.4](image-url)  
*Figure 8.4 washing-out of the filter layer though the cover layer*
8.4 Conclusions

Compaction and washing-out of particles in a core may be caused by design faults. The probability of occurrence of these design faults, however, is likely to be low, in comparison to other failure mechanisms.

Because of their relatively low probability of occurrence, the failure mechanisms of the core will not be further examined. Investigation of these mechanisms in the light of the design of breakwaters, however, would surely repay study.

The failure mechanisms, involving the subsoil of the breakwater structure, will be the topic of the following Chapter.
Chapter 9
Failure mechanisms of the subsoil

9.1 Introduction

In this Chapter, the failure mechanisms involving the subsoil will be described with the aid of the fault-tree in Chapter 4. In this fault-tree, three possible causes involve the subsoil:
1) settlement of the subsoil
2) liquefaction of the subsoil
3) slide plane/squeezing of the subsoil.

These three causes may lead to the failure by the following mechanisms:
1) loss of height of the crest element (see also Chapter 7 and 8);
2) insufficient support of the toe (Chapter 6).

ad 1) Lost of height of the crest element (only a vertical displacement) due to the following causes (see also the fault-tree in Figure 9.1):
- compaction of the core (Chapter 8);
- washing-out of the core (Chapter 8);
- settlement of the subsoil (Section 9.2);
- liquefaction (Section 9.3).

Figure 9.1 fault-tree of the loss of height of the crest element
ad 2) Insufficient support of the toe structure may occur by the following causes (see also the fault-tree in Figure 9.2):
- instability of the stones of the toe (Chapter 6);
- slip circle of the toe (Chapter 6);
- foreshore erosion (Chapter 6)
- erosion of the filter layers (Chapter 6);
- slide plane/squeezing of the subsoil (Section 9.4).

Figure 9.2 fault-tree of the insufficient support of the toe structure

Three possible failure mechanisms will be discussed in the following Sections. In Section 9.2, the settlement of the subsoil will be described. A description of liquefaction will be given in Section 9.3 and the slide plane of the subsoil will be described in Section 9.4.

9.2 Settlement of the subsoil

Settlement of the subsoil consists of two main failure mechanisms:
- excessive consolidation (Section 9.2.1)
- collapse of cavities (Section 9.2.2).
9.2.1 Excessive consolidation of the subsoil

When a breakwater is set on a compressible soil layer, the subsoil may settle (Figure 9.2).

![Diagram of a breakwater settling](image)

**Figure 9.2 settlement of the subsoil**

Settlements cause uneven surfaces, which increase instability of the core and hence undermine the support for crest structures. In reaction to such settlement of the subsoil, the crest level settles too and the structure's capability to limit overtopping under conditions of high water levels and wave attack is reduced.

When designing the height of the breakwater structure, it must therefore be taken into consideration that the desired levels will be maintained after settlement of the structure and of the subsoil.

A number of theories describe the consolidation of soils (Verruyt, 1990). The theory of Terzaghi is one of the most frequently applied. All coastal structures standing suffer from consolidation of the soil.

9.2.2 Collapse of cavities

Large cavities may cause unexpected settlement, particularly when environmental conditions are changed by human activities, such as drilling for oil or petroleum.

Two types of collapse of cavities can be identified: formation of sinkholes (sudden settlement) and formation of dolines (slow subsidence). These formations may occur among other things because sub-surface erosion takes place because of the presence of the cavities. Unfortunately cavities are difficult to detect.
9.3 Liquefaction

Liquefaction refers to a situation in granular subsoils where intergranular contact is lost. The whole medium then loses its shear strength and behaves like a thick fluid. Under these circumstances any shear load causes instability of the structure.

Liquefaction can be caused by cyclic loads. These loads generate excess pore pressure if the deformation resulting from these loads cause compaction and if they are combined with an enhanced drainage capacity due to dissipation of the increase in pore pressure.

9.4 Slide plane

Slide plane occurs when the slope of the seabed is so steep that it slides under external loads. This failure mechanism will not be worked out any further.

9.5 Conclusions

Settlement of the subsoil may be caused by a consolidation of the subsoil or by the collapse of cavities. Rules of thumb exist for the calculation of the consolidation of the subsoil. The probability of occurrence of making a design fault will be relatively low, when taken these rules into account.
The possibility, that cavities will collapse, will be omitted from this study for two reasons: firstly, it rarely happens and secondly, the new harbour of Ennore will not be located in a region where such cavities are present.

Liquefaction is also a failure mechanism which has probably a very low probability of occurrence in the context of the situation of Ennore. Ennore is not located in a 'hot' place where the probability of occurrence of liquefaction is high.

In consequence, the failure mechanisms of the subsoil mentioned in this Chapter will not be described in this study in further detail.
Chapter 10
Quantification of some failure mechanisms of the breakwater structure at Ennore

10.1 Introduction

It is in principle possible to calculate the probability of occurrence of the top-event of the total fault-tree for the breakwater at Ennore, given in Chapter 4. Herefore it is necessary to know the probability of occurrence of every failure mechanism of the tree. To know these probabilities, a reliability function for each failure mechanism should be available.

Within the framework of this study, it will not be possible to calculate the probability of the top-event of the fault-tree of Ennore. This is because not all the mechanisms are known, there is no reliability function available, data is lacking, etc.).

This study will therefore be restricted to the quantification of four failure mechanisms for the situation of Ennore:
1) instability of a rock armour layer
2) instability of an armour layer with ACCROPODE® elements
3) slip circle of the armour layer
4) horizontal displacement of the crest element

These quantification will be calculated with a probabilistic approach.

ad 1 and 2) The instability of a rock armour layer and of an armour layer with ACCROPODE® elements have been described in Chapter 5. With the formulae of Van der Meer (equations 5.5 and 5.6 for rock and 5.17 for ACCROPODE® elements), reliability functions have been determined. With these functions, calculations of the probability of occurrence will be carried out in Section 10.3.2.

ad 3) For the quantification of the slip circle of the armour layer, the simplified Bishop formula has been used (Chapter 5). The calculation of the probability of occurrence will be carried out in Section 10.3.3.

ad 4) The horizontal displacement of the crest element has been described in Chapter 6.
The calculation of the probability of occurrence of this failure mechanism will be carried out in Section 10.3.4.

This Chapter will first describe the computer model which has been used to carry out the calculations of the probability of occurrence of the failure mechanisms of the breakwaters (Section 10.2). Then the data used as input for the different failure mechanisms will be define and finally, an analysis of the results of the calculations of the probabilities will conclude this Chapter (Section 10.3).
10.2 Description of the computer model

There are many different level-II methods. In this study, the probabilities will be calculated with an Approximate Full Distribution Approach (AFDA). This method has been chosen because it uses the design point for an expansion of the reliability function \( Z \). A design point is defined as the point on the failure boundary \( (Z=0) \), where the density of the probability of \( Z \) has its maximum. This computer-program is useful because it takes both the sensibility and the probability of each variable into account.

For the calculation of the probability of occurrence of the failure mechanisms of the breakwater, an existing AFDA computer-program ('source-file'), written in TurboPascal computer-language, has been used.

Each failure mechanism has two files: the first file ('formula-file') contains the reliability function of the failure mechanisms and the second file ('data-file') contains the mean values and the standard deviation of the variables, the type of distributions and their parameters.

The 'source-file' and the 'formula-file' are compiled to an executable file with which the data can be processed. During the reading of the data-file, the computer checks the distribution of the variables. If there is a distribution which is not Gaussian, then the computer will convert it into a normal (Gaussian) one.

The computer calculates the design point value of every variable by iteration, starting with values which have been given in the data-file (mostly the mean values of the variables). The computer also calculates the value of the reliability index \( \beta \) which gives the distance from the mean value of the reliability function \( \mu_z \) to the design point.

The program gives among others: the contribution of each variable to the uncertainty and, most importantly, the probability of failure.

10.3 Interpretation of the input and analysis of the results

10.3.1 Introduction

The determination of the numerical value of the parameters of the distributions of the variables is a complex task and can be seen, within the framework of this study, as an (iterative) economical optimization. Indeed, the best way to find these values would be to carry out a statistical analysis. For such a statistical analysis, a large amount of data, collected in different experiments would be necessary. This, however, would involve several years of research and is therefore not feasible here.
Therefore, a first guess for the parameters of the distributions of the variables in this study has been made, based on the best judgement of the engineer. These parameters will be controlled and, if necessary, adjusted if they seem inadequate for applications. This control will be carried out by several calculations with different input values.

The framework for this report is defined in Chapter 4: failure of the breakwater structure at Ennore occurs when (a part of) the breakwater collapses. The Ultimate Limit State (U.L.S.) is what occurs during extreme conditions. Translated to the project of Ennore, the U.L.S. will happen during cyclonic storms.

10.3.2 Instability of an armour layer

Rock armour layer

Parameters involved

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<th>symbol</th>
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<th>B - coefficient</th>
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<td>$\rho_i$</td>
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<td>0.5</td>
</tr>
</tbody>
</table>

Table 10.1 parameters of Gumbel and normal distributions

With the following formula expressing the Gumbel distribution:

$$F(x) = \exp(-e^{-\frac{(x-A)}{B}})$$

and the normal density function:

$$f(x) = \frac{1}{\sqrt{2\pi}} \cdot \frac{1}{B} \cdot e^{-\frac{(x-A)^2}{2B^2}}$$
Interpretation of the input

* Significant wave height ($H_s$)
  The wave height may be assumed to be best approximated by a Gumbel distribution with the following coefficients (see also Chapter 3):
  $A = 3.31$
  $B = 1.518$.

* Uncertainty ($fH_s$) in the wave height
  Uncertainty in the wave height distribution reflects the deviations of the actual wave height from the wave height as predicted by the selected distribution and is due to three sources of uncertainty:
  - intrinsic uncertainty;
  - model uncertainty and the so-called
  - statistical uncertainty.

Two ways to take this uncertainty into account present themselves: by multiplying or by adding the uncertainty to the wave height.

In the first case, the distribution of the uncertainty in the measured wave height increases concomitantly to the increase in predicted wave height, while in the second case the distribution of the uncertainty stays equal (Figure 10.1).

![Diagram of wave height uncertainty](image)

*Figure 10.1 definition sketch of wave height uncertainty*
In this study, the second definition of the model uncertainty will be used because it is assumed that there is no reason for an increasing uncertainty when the wave height increases. An approximation of the mean value of the uncertainty of the wave height will be 0.0 ($\mu(fH) = 0.0$) and of the standard deviation will be 0.8 [m] ($\sigma(fH) = 0.8$). This value is relatively high because the amount of available data is small and is therefore somewhat unreliable.

* Highest water level ($hw_{\text{max}}$)

The sum of all the changes in water levels leads to the design water level above Chart Datum of the nautical chart: the highest water level (Chapter 3). This maximum water level depends on the following:
- astronomic tide
- seasonal variations
- wind and wave set-up
- barometric pressure.

For the purpose of this study, the mean highest water level is taken to be equal to CD + 1.50m (mean value is $\mu(hw_{\text{max}}) = 1.50$) and to be normally distributed. All the uncertainties in the data collected on the items listed above have been gathered and a standard deviation of 0.3m ($\sigma(hw_{\text{max}}) = 0.3m$) will be used here.

* Level of the bed ($bed_{\text{max}}$)

The instability of a rock armour layer is assumed to occur at the deepest point of the breakwater (at CD-11.5m). Since the movement of sand changes the bed level, this level is bound to have a standard deviation of 0.67m ($\sigma(\text{bed}_{\text{max}}) = 0.67m$). It will be assumed to be normally distributed.

* Slope angle ($N_{\text{coop}}$)

The cotangents of the slope of the Northern breakwater at Ennore is supposed to be normally distributed and is equal to 3.5. This is an average angle of a dynamic stable profile of a rock slope. The standard deviation is set equal to 0.3.

* Damage level ($S$)

For an ultimate limit state, a complete collapse of the breakwater is assumed. As the stability at damage of ACCROPODE® is very high compared to rock structures, it is essential to take a high value for $S$ (for example $S = 12$). The distribution will be assumed to be normal and the standard deviation for the damage level will be set equal to 0.3 ($\sigma(S) = 0.3$).
* **Mass density of rock \((\rho_0)\)**
  The mean value for the mass density of rock is estimated to be 2650kg/m\(^3\) with a standard deviation of 50kg/m\(^3\). Both distributions will be assumed to be normal.

* **Permeability factor (P)**
  For the breakwater structure at Ennore, the mean value for the permeability will be equal to 0.55. This is because the structure has a relatively permeable core. The distribution is assumed to be normal. The standard deviation has been fixed at 0.05.

* **Armour unit weight \((W_{ao})\)**
  The armour weight has been determined at the most decisive spot in the breakwater structure: the deepest point (CD-11.5m). The mean value of the weight of rock stone is set equal to 7 tons. The standard deviation will be 0.35 ton. Both distributions are supposed to be normal.

* **Reliability factor for plunging waves (C1) and for surging waves (C2)**
  The reliability factor for plunging waves and for surging waves proves the shortcomings of Van der Meer’s formulae. This is because the measured data do not, among other things, fit the data from the predicted formulae (intrinsic uncertainty of the parameters. The standard deviation for plunging waves is equal to 0.4. For surging waves, it is equal to 0.08. Both distributions are supposed to be normal.

### Calculations and analysis of the results

Calculations of the probabilities of the failure of the armour layer according to various mechanisms have been carried out with a level-II computer program.

In order to improve understanding of the reliability of the breakwater structures, different calculations of the probability of occurrence have been carried out. An important result of this probabilistic analysis is the quantification of the contributions of the various variables involved, to the probability of occurrence of the instability of the armour layer.

For more details of the calculations, the reader is referred to Annex 2.
Probability of failure of the armour layer (instability): 0.0073 per year.

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<th>original B - coeff.</th>
<th>changes B - coeff.</th>
<th>%</th>
<th>new probability of failure</th>
<th>%</th>
</tr>
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<tbody>
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<td>3.641 (+0.331)</td>
<td>2.979 (-0.331)</td>
<td>1.518</td>
<td>+ 10%</td>
<td>0.009</td>
<td>+ 23.3%</td>
</tr>
<tr>
<td>wave steepness</td>
<td>S_p</td>
<td>10.6%</td>
<td>2.95</td>
<td>3.245 (+0.295)</td>
<td>2.655 (-0.295)</td>
<td>0.514</td>
<td>+ 10%</td>
<td>0.0041</td>
<td>+ 43.8%</td>
</tr>
<tr>
<td>uncertainty in wave height [-]</td>
<td>s_H_s</td>
<td>3.1%</td>
<td>0.00</td>
<td>--</td>
<td>0.8</td>
<td>1.2 (+0.4)</td>
<td>+ 50%</td>
<td>0.0083</td>
<td>+ 13.7%</td>
</tr>
<tr>
<td>damage level [-]</td>
<td>S</td>
<td>0.1%</td>
<td>12</td>
<td>13.2 (+1.2)</td>
<td>10.8 (-1.2)</td>
<td>0.3</td>
<td>+ 10%</td>
<td>0.0054</td>
<td>- 26.0%</td>
</tr>
<tr>
<td>permeability factor [-]</td>
<td>P</td>
<td>0.9%</td>
<td>0.55</td>
<td>0.605 (+0.055)</td>
<td>0.495 (-0.055)</td>
<td>0.05</td>
<td>+ 10%</td>
<td>0.0056</td>
<td>- 23.3%</td>
</tr>
<tr>
<td>armour unit weight [ton]</td>
<td>W_m</td>
<td>0.9%</td>
<td>70 tons</td>
<td>77.0 (+7.0)</td>
<td>63.0 (-7.0)</td>
<td>3.5</td>
<td>+ 10%</td>
<td>0.0044</td>
<td>- 39.8%</td>
</tr>
<tr>
<td>reliability factor plunging waves [-]</td>
<td>C1</td>
<td>14.5%</td>
<td>6.2</td>
<td>--</td>
<td>0.4</td>
<td>0.6 (+0.2)</td>
<td>+ 50%</td>
<td>0.012</td>
<td>+ 64.4%</td>
</tr>
<tr>
<td>reliability factor surging waves [-]</td>
<td>C2</td>
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<td>1.0</td>
<td>--</td>
<td>0.08</td>
<td>0.12 (+0.04)</td>
<td>+ 50%</td>
<td>0.0000</td>
<td>- 32.9%</td>
</tr>
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</table>

August 1995
<table>
<thead>
<tr>
<th>name variable</th>
<th>symbol</th>
<th>uncertainty %</th>
<th>original mean value</th>
<th>changes mean value</th>
<th>changes std. dev.</th>
<th>%</th>
<th>new probability of failure</th>
</tr>
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<tr>
<td>Level of the bed [m]</td>
<td>bed&lt;sub&gt;m&lt;/sub&gt;</td>
<td>3.3%</td>
<td>-11.50</td>
<td>-11.385 (+0.115)</td>
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<td>0.0067</td>
<td>-8.2%</td>
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<tr>
<td>11.615 (-0.115)</td>
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<td>9.6%</td>
<td>+35.6%</td>
<td></td>
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</tr>
<tr>
<td>-10.925 (+0.575)</td>
<td>+5%</td>
<td>0.0047</td>
<td>20.7%</td>
<td>+61.6%</td>
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<td>0.011</td>
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<td>+119.2%</td>
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</tr>
<tr>
<td>-10.35 (+1.15)</td>
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<td>0.0023</td>
<td>0.0%</td>
<td>+1.4%</td>
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<td></td>
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</tr>
<tr>
<td>-12.65 (-1.15)</td>
<td>-10%</td>
<td>0.016</td>
<td>0.0%</td>
<td>+2.7%</td>
<td></td>
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<tr>
<td>+10%</td>
<td>0.0074</td>
<td>+5%</td>
<td>0.0073</td>
<td>+1.4%</td>
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<td>0.0073</td>
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<td>0.0075</td>
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<td>+10%</td>
<td>0.0072</td>
<td>+5%</td>
<td>0.0083</td>
<td>+32.9%</td>
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<td>0.0067</td>
<td>+5%</td>
<td>-8.22%</td>
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<tr>
<td>Slope angle [°]</td>
<td>N&lt;sub&gt;sw&lt;/sub&gt;</td>
<td>6.3%</td>
<td>3.5</td>
<td>3.535 (+0.035)</td>
<td>+1%</td>
<td>0.0068</td>
<td>-6.8%</td>
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<td>3.465 (-0.035)</td>
<td>-1%</td>
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<td>9.6%</td>
<td>+32.8%</td>
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<td></td>
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<tr>
<td>3.675 (+0.175)</td>
<td>+5%</td>
<td>0.0049</td>
<td>50.7%</td>
<td>+56.2%</td>
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<td>3.325 (-0.175)</td>
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<td>0.011</td>
<td>119.2%</td>
<td>+5.5%</td>
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<tr>
<td>3.85 (+0.35)</td>
<td>+10%</td>
<td>0.0032</td>
<td>2.7%</td>
<td>+31.1%</td>
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<td>3.15 (-0.35)</td>
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<tr>
<td>+10%</td>
<td>0.0075</td>
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<td>0.0072</td>
<td>+1.4%</td>
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<td>+10%</td>
<td>0.0077</td>
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<td>0.0095</td>
<td>+20.1%</td>
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<td>+10%</td>
<td>0.0062</td>
<td>+5%</td>
<td>-15.1%</td>
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<tr>
<td>Mass density of rock µ (g/cm³)</td>
<td>ρ&lt;sub&gt;i&lt;/sub&gt;</td>
<td>1.9%</td>
<td>26.50</td>
<td>26.765 (+0.265)</td>
<td>+1%</td>
<td>0.006</td>
<td>-17.8%</td>
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<tr>
<td>26.355 (-0.265)</td>
<td>-1%</td>
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<td>23.3%</td>
<td>+64.4%</td>
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</tr>
<tr>
<td>27.825 (+1.325)</td>
<td>+5%</td>
<td>0.0026</td>
<td>160.3%</td>
<td>+88.1%</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>25.175 (-1.325)</td>
<td>-5%</td>
<td>0.019</td>
<td>543.8%</td>
<td>+8.2%</td>
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<td></td>
<td></td>
</tr>
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<td>29.15 (+2.65)</td>
<td>+10%</td>
<td>0.00057</td>
<td>0.0%</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>23.85 (-2.65)</td>
<td>-10%</td>
<td>0.0047</td>
<td>0.0%</td>
<td>+4%</td>
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<tr>
<td>0.525 (+0.025)</td>
<td>+5%</td>
<td>0.0074</td>
<td>0.0%</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.475 (+0.025)</td>
<td>-5%</td>
<td>0.0073</td>
<td>0.0%</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.55 (+0.05)</td>
<td>+10%</td>
<td>0.0074</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.45 (-0.05)</td>
<td>-10%</td>
<td>0.0073</td>
<td>+4%</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>0.75 (+0.25)</td>
<td>+50%</td>
<td>0.0079</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.25 (-0.25)</td>
<td>-50%</td>
<td>0.0079</td>
<td>+4%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New probability of failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Explanations of Table 10.3 (pages 10-7 and 10-8)

a) an increase and a decrease of 10% on the mean value have been set in order to see how the probability of failure changes when that value changes;

b) an increase and a decrease of 50% on the standard deviation have been set in order to detect the influence of the standard deviation on the probability of failure;

c) in the last column of the table, a '+' gives an increase and a '-' gives a decrease of the probability of occurrence of the failure mechanisms;

d) because of the large influence of the level of the bed, the slope angle and the density of rock, an increase and a decrease of 1% and 5% have also been taken on their mean value and standard deviation.

Results

From Table 10.3, the following conclusions can be drawn:

1) the significant wave height $H_s$ has the largest portion of uncertainty in the reliability function (58.2%), the model factor for plunging waves $C_1$ (14.5%) and for the wave steepness $S_p$ (10.6%) also have a significant influence;

2) the largest corrections in the probability of occurrence occur by a change in the mean value and standard deviation of the level of the bed ($\text{bed}_u$), the angle of the slope ($N_{\text{slope}}$) and the mass density of rock ($\rho_v$);

3) a decrease of 1% in the mean value of the mass density of rock ($\rho_v$) leads to an increase of 23.3% of the probability of occurrence of failure of the armour layer;

4) an increase of 50% in the standard deviation of the slope angle ($N_{\text{slope}}$) leads to an increase of 32.9% in the probability of failure;

5) an increase of 50% in the standard deviation of the reliability factor for plunging waves ($C_1$) leads to an increase of 64.4% in the probability of failure;

6) increases and decreases of 50% in the standard deviation of the damage level $S$ and the armour unit weight ($W_{\text{uo}}$) barely influence the probability of failure.
From these results the following recommendations may be made:

a) the precise determination of the crest level, angle of the slope and mass density of rock is required: special attention must be given to the monitoring of these variables during the design and construction periods. Accurate back-hoes or other machines for the shaping of the slope seem to be a good investment, and reliable research on the rock could be very useful;

b) a flatter slope and larger stones would be recommended should a decrease in the probability of occurrence of failure be needed;

c) it would be advisable to do more investigations with a rock slope and the attack of plunging waves in order to find a lower standard deviation for the coefficients C1 and C2 in the formula of Van der Meer.

**Armour layer with ACCROPODE® elements**

**Parameters involved**

<table>
<thead>
<tr>
<th>name variable</th>
<th>type of distribution</th>
<th>symbol</th>
<th>A - coefficient</th>
<th>B - coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>significant wave height</td>
<td>G</td>
<td>$H_s$</td>
<td>3.31</td>
<td>1.518</td>
</tr>
<tr>
<td>wave steepness</td>
<td>G</td>
<td>$S_s$</td>
<td>2.95</td>
<td>0.514</td>
</tr>
<tr>
<td>uncertainty in wave height</td>
<td>N</td>
<td>$fH_s$</td>
<td>0.0</td>
<td>0.8</td>
</tr>
<tr>
<td>high water level</td>
<td>N</td>
<td>$h_{w,l}$</td>
<td>1.50</td>
<td>0.3</td>
</tr>
<tr>
<td>level of the bed</td>
<td>N</td>
<td>$b_{d,l}$</td>
<td>-11.50</td>
<td>0.67</td>
</tr>
<tr>
<td>mass density of concrete</td>
<td>N</td>
<td>$\rho_c$</td>
<td>24.00</td>
<td>0.5</td>
</tr>
<tr>
<td>armour unit weight</td>
<td>N</td>
<td>$W_{\infty}$</td>
<td>150</td>
<td>7.5</td>
</tr>
<tr>
<td>model uncertainty</td>
<td>N</td>
<td>$X_m$</td>
<td>4.1</td>
<td>1.0</td>
</tr>
</tbody>
</table>

**Table 10.5 parameters of Gumbel and normal distributions**

**Interpretation of the input**

* Significant wave height ($H_s$)

The wave height may be assumed to be best approached by a GUMBEL distribution with the following coefficients (See also Chapter 3):
- $A = 3.31$
- $B = 1.518$. 
**Uncertainty (\(H_u\)) in the wave height**  
(See Section 10.3.2)

**Highest water level (\(hw_{\text{max}}\))**  
(See Section 10.3.2)

**Level of the bed (\(bed_{\text{alt}}\))**  
The instability of armour elements is assumed to happen at the deepest point of the breakwater (at CD-11.5m). This level is bound to have a standard deviation of 0.67m (\(\sigma(bed_{\text{alt}})=0.67\text{m}\)). The level of the bed will be assumed to be normally distributed.

**Slope angle (\(N_{\text{slope}}\))**  
The cotangents of the slope for the breakwater at Ennore with ACCROPODE\(^\text{®}\) elements is supposed to be normally distributed and is set to be equal to 3.5. The standard deviation will be set to be equal to 0.3.

**Mass density of concrete (\(\rho_c\))**  
The mean value for the mass density of concrete is estimated to be 2400kg/m\(^3\) with a standard deviation of 50kg/m\(^3\). Both distributions will be assumed to be normal.

**Armour unit weight (\(W_{\text{uo}}\))**  
The armour weight has been determined at the most decisive spot in the breakwater structure: the deepest point (CD-11.5m). The mean value of the weight of ACCROPODE\(^\text{®}\) elements is set to be equal to 15 tons. The standard deviation will be 0.75 ton. The armour weight is supposed to be normal.

**Model uncertainty for the stability of ACCROPODE\(^\text{®}\) (\(X_{\text{uo}}\))**  
Important wave impact may damage the armour layer of a breakwater. In order to compare the damage to an armour layer constructed with ACCROPODE\(^\text{®}\) elements and the damage to an armour layer made of rock, one must consider the total collapse of the structure. For armour layers with ACCROPODE\(^\text{®}\) elements, total collapse means a damage level of 4.1 (see also equation 5.17).

As described in Chapter 5, the formula for the stability of ACCROPODE\(^\text{®}\) has some shortcomings. The model uncertainty takes these shortcomings into account. In his article "Stability of cubes, tetrapods and ACCROPODE\(^\text{®}\)" (Van der Meer, 1988), Van der Meer considered the coefficient 4.1 as a stochastic parameter. From analysis, it followed that the standard deviation (assuming a normal distribution) was equal to 0.2. This standard deviation belongs to the data resulting from that particular experiment, which was carried out on a scale model in a flume.
Because little is known about armour layers with ACCROPODE® elements, a relatively high standard deviation ($\sigma(X_m)=1.0$) of the reliability factor for armour layers with ACCROPODE® will be applied in this study.

**Explanations of the Table 10.6**
(See also the explanations of Table 10.3).

Because of the large influence of the mass density of concrete ($\rho_c$), an increase and a decrease of 5% have also been taken on its mean value.

For more details of the calculations, the reader is referred to Annex 3.

**Results**

From Table 10.6, the following conclusions can be drawn:

1) the model uncertainty $X_m$ (65.1%) has the largest contribution to the uncertainty in the reliability function; the significant wave height $H_s$ (31.5%) has also a significant influence;

2) the largest corrections in the probability of occurrence occur by a change in the mean value of the mass density of concrete ($\rho_c$), the significant wave height ($H_s$) and the level of the bed ($\text{bed}_0$);

3) a decrease of 5% in the mean value of the mass density of concrete ($\rho_c$) leads to an increase of 38.9% of the probability of failure of the armour layer;

4) a decrease of 50% in the standard deviation of the weight of the armour unit ($W_{so}$) leads to a decrease of 61.1% in the probability of failure;

5) an increase of 50% in the standard deviation of the model uncertainty ($X_m$) has a negligible influence on the probability of failure.

From these results the following recommendations may be made:

a) an exact determination of the mass density of concrete is required: special attention must be given to the monitoring of this variable;

b) higher mass density of concrete and larger elements would be recommended should a decrease in the probability of occurrence of this failure mechanism be needed.
Probability of failure of the ACCROPODE® armour layer (instability): 0.018 per year.

<table>
<thead>
<tr>
<th>name variable</th>
<th>symbol</th>
<th>uncertainty %</th>
<th>original A - coeff.</th>
<th>changes A - coeff.</th>
<th>original B - coeff.</th>
<th>changes B - coeff.</th>
<th>%</th>
<th>new probability of failure %</th>
</tr>
</thead>
<tbody>
<tr>
<td>significant wave height</td>
<td>$H_s$</td>
<td>31.5%</td>
<td>3.31</td>
<td>3.641 (+0.331)</td>
<td>2.979 (-0.331)</td>
<td>1.518</td>
<td>+10%</td>
<td>0.021</td>
</tr>
<tr>
<td>wave steepness</td>
<td>$S_p$</td>
<td>0.0%</td>
<td>2.95</td>
<td>3.245 (+0.295)</td>
<td>2.655 (-0.295)</td>
<td>0.514</td>
<td>+10%</td>
<td>0.018</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>name variable</th>
<th>symbol</th>
<th>uncertainty %</th>
<th>original mean value</th>
<th>changes mean value</th>
<th>original std. dev.</th>
<th>changes std. dev.</th>
<th>%</th>
<th>new probability of failure %</th>
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</thead>
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<td>$f_{H_s}$</td>
<td>2.3%</td>
<td>0.00</td>
<td>--</td>
<td>0.8</td>
<td>0.8</td>
<td>+50%</td>
<td>0.019</td>
</tr>
<tr>
<td>high water level</td>
<td>$h_{w_m}$</td>
<td>0.0%</td>
<td>1.50</td>
<td>1.65 (+0.15)</td>
<td>1.35 (-0.15)</td>
<td>0.3</td>
<td>+10%</td>
<td>0.018</td>
</tr>
<tr>
<td>level of the bed</td>
<td>$b_{ed_m}$</td>
<td>0.1%</td>
<td>-11.50</td>
<td>-10.35 (+1.15)</td>
<td>-12.65 (-1.15)</td>
<td>0.67</td>
<td>+10%</td>
<td>0.016</td>
</tr>
<tr>
<td>mass density of concrete</td>
<td>$\rho_c$</td>
<td>0.3%</td>
<td>24.00</td>
<td>25.2</td>
<td>22.8</td>
<td>26.4</td>
<td>+5%</td>
<td>0.013</td>
</tr>
<tr>
<td>armour unit weight</td>
<td>$W_{so}$</td>
<td>0.1%</td>
<td>150 tons</td>
<td>77.0 (+7.0)</td>
<td>63.0 (-7.0)</td>
<td>7.5</td>
<td>+10%</td>
<td>0.015</td>
</tr>
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<td>model uncertainty</td>
<td>$X_m$</td>
<td>65.1%</td>
<td>4.1</td>
<td>--</td>
<td>1.0</td>
<td>(+)</td>
<td>+50%</td>
<td>0.061</td>
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10.3.3 Slip circle of the armour layer

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<th>symbol</th>
<th>A - coefficient</th>
<th>B - coefficient</th>
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<td>slope angle</td>
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<td>$N_{\text{slope}}$</td>
<td>3.5</td>
<td>0.3</td>
</tr>
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<td>angle of internal friction</td>
<td>N</td>
<td>$\phi$</td>
<td>25.0</td>
<td>2.0</td>
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<tr>
<td>soil cohesion</td>
<td>N</td>
<td>$c$</td>
<td>10.0</td>
<td>1.0</td>
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<tr>
<td>crest level</td>
<td>N</td>
<td>$\text{crest}_{\text{cd}}$</td>
<td>5.20</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Table 10.7 parameters of normal distributions

Interpretation of the input

* Slope angle ($N_{\text{slope}}$)
  The cotangents of the slope of the Northern breakwater at Ennore is normally distributed and is equal to 3.5. This is an average angle of a dynamically stable profile of a rock slope. The standard deviation is equal to 0.3.

* Angle of internal friction of the rubble ($\phi$)
  For rock/stone media, the friction angle depends on various material conditions as well as on the actual effective stress level. The mean value is equal to 25.00 [-] and its standard deviation is equal to 2.0.

* Rubble cohesion ($c$)
  The cohesion is related to the fact that the removal of a particle at zero effective stress requires some effort. For rock and artificial armour units, the cohesion is related to the interlocking. For the interlocking of rock the use of an apparent cohesion has been suggested (SOGREAH, 1994), in particular when a Bishop stability analysis is applied. A value of 10 KN/m² for rock will be used for the rubble cohesion. The standard deviation for the cohesion will be 1.0 KN/m².

* Level of the crest ($\text{crest}_{\text{cd}}$)
  The mean level of the crest at CD-11.5m is equal to 5.20m. Its standard deviation is guessed to be 0.10m.

Calculations and analysis of the results

For more details of the calculations, the reader is referred to Annex 4. Extra calculations have been carried out for the crest level ($\text{crest}_{\text{cd}}$) in order to analyse the influence of the crest level better: an increase and a decrease of 1% have been taken on its mean value. Because of the large influence of the slope angle ($N$), an increase and a decrease of 10% have also been included in its standard deviation.
Results

From Table 10.8, the following conclusions can be drawn:

1) a change in all the involved variables lead to great increase or decrease in the probability of failure;

2) the slope angle $N_{\text{slope}}$ has the largest share in the uncertainty of the reliability function (64.7%);

3) the largest changes in the probability of failure is caused by a change in the mean value of the crest level ($\text{crest}_{\text{m}}$) and a change in the standard deviation of the slope angle ($N_{\text{slope}}$);

4) an increase of 1% in the mean value of the crest level ($\text{crest}_{\text{m}}$) leads to an increase of 31.3% of the probability of occurrence of failure of the slip circle;

5) an increase of 10% in the standard deviation of the slope angle (N) leads to an increase of 131.3% in the probability of failure.

From these results the following recommendations may be made:

a) a scrupulous determination of level of the crest element and of the slope angle are required: special attention should be given to the monitoring of these variables;

b) a higher crest level and a flatter slope will be recommended when decrease in the probability of occurrence of this failure mechanism is needed;

c) an investigation of the reliability of this approach of the slip circle is advisable.
Probability of failure of a slip circle for an armour layer with ACCROPODE® elements: 1.6 \times 10^{-4} per year.

<table>
<thead>
<tr>
<th>name variable</th>
<th>symbol</th>
<th>uncertainty %</th>
<th>original mean value</th>
<th>changes mean value</th>
<th>original std. dev.</th>
<th>changes std. dev.</th>
<th>%</th>
<th>new probability of failure</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>slope angle</td>
<td>N</td>
<td>64.7%</td>
<td>3.50</td>
<td>3.85 (+0.35)</td>
<td>3.15 (-0.35)</td>
<td>0.3</td>
<td>+ 10%</td>
<td>3.3 \times 10^{-4}</td>
<td>- 97.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 10%</td>
<td>4.2 \times 10^{-4}</td>
<td>+ 250.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 1%</td>
<td>3.7 \times 10^{-4}</td>
<td>+ 131.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 1%</td>
<td>6.3 \times 10^{-3}</td>
<td>- 60.6%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+ 50%</td>
<td>4.3 \times 10^{-2}</td>
<td>+ 258.7%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 50%</td>
<td>1.8 \times 10^{-4}</td>
<td>- 98.9%</td>
</tr>
<tr>
<td>angle of internal friction of the rubble</td>
<td>φ</td>
<td>18.2%</td>
<td>25.00</td>
<td>27.50 (+2.5)</td>
<td>22.50 (-2.5)</td>
<td>2.0</td>
<td>+ 10%</td>
<td>2.1 \times 10^{-3}</td>
<td>- 86.9%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 10%</td>
<td>1.2 \times 10^{-4}</td>
<td>+ 650.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+ 50%</td>
<td>7.2 \times 10^{-4}</td>
<td>+ 350.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 50%</td>
<td>6.4 \times 10^{-3}</td>
<td>- 60.0%</td>
</tr>
<tr>
<td>soil cohesion</td>
<td>c</td>
<td>15.4%</td>
<td>10.00</td>
<td>11.0 (+1.0)</td>
<td>9.0 (-1.0)</td>
<td>1.0</td>
<td>+ 10%</td>
<td>3.5 \times 10^{-3}</td>
<td>- 78.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 10%</td>
<td>7.0 \times 10^{-4}</td>
<td>+ 337.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+ 50%</td>
<td>5.4 \times 10^{-3}</td>
<td>+ 237.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 50%</td>
<td>7.0 \times 10^{-3}</td>
<td>- 56.3%</td>
</tr>
<tr>
<td>crest level</td>
<td>crest_t</td>
<td>1.8%</td>
<td>5.20</td>
<td>5.252 (+0.052)</td>
<td>5.148 (-0.052)</td>
<td>0.10</td>
<td>+ 1%</td>
<td>2.1 \times 10^{-4}</td>
<td>+ 31.3%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 1%</td>
<td>1.2 \times 10^{-4}</td>
<td>- 25.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+ 10%</td>
<td>1.8 \times 10^{-4}</td>
<td>+ 1025.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 10%</td>
<td>8.0 \times 10^{-4}</td>
<td>- 95.0%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+ 50%</td>
<td>1.9 \times 10^{-4}</td>
<td>+ 18.8%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- 50%</td>
<td>1.5 \times 10^{-4}</td>
<td>- 6.3%</td>
</tr>
</tbody>
</table>
10.3.4 Horizontal displacement of the crest element

Parameters involved

<table>
<thead>
<tr>
<th>name variable</th>
<th>type of distribution</th>
<th>symbol</th>
<th>A - coefficient</th>
<th>B - coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>significant wave height</td>
<td>G</td>
<td>(H_s)</td>
<td>3.31</td>
<td>1.518</td>
</tr>
<tr>
<td>friction coefficient</td>
<td>N</td>
<td>(f)</td>
<td>0.9</td>
<td>0.15</td>
</tr>
<tr>
<td>mass density of material</td>
<td>N</td>
<td>(\rho_m)</td>
<td>2650</td>
<td>50</td>
</tr>
<tr>
<td>mass density of water</td>
<td>N</td>
<td>(\rho_w)</td>
<td>1020</td>
<td>5</td>
</tr>
<tr>
<td>wave pressure coefficient</td>
<td>N</td>
<td>(\alpha_0)</td>
<td>0.364</td>
<td>0.05</td>
</tr>
<tr>
<td>width of crest element</td>
<td>N</td>
<td>(L_c)</td>
<td>9.0</td>
<td>0.1</td>
</tr>
<tr>
<td>cross-section of crest element</td>
<td>N</td>
<td>(\Omega)</td>
<td>10.5</td>
<td>0.1</td>
</tr>
<tr>
<td>height for wave pressure</td>
<td>N</td>
<td>(d')</td>
<td>1.44</td>
<td>0.05</td>
</tr>
<tr>
<td>model uncertainty</td>
<td>N</td>
<td>(X_m)</td>
<td>1.0</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Table 10.1 parameters of Gumbel and normal distributions

Interpretation of the input

* Significant wave height (\(H_s\))
  The wave height may be assumed to be best modelled by a GUMBEL distribution with the following coefficients (see also Chapter 3):
  \(A = 3.31\)
  \(B = 1.518\).

* Friction (f)
  The friction between the crest element and the rough underlayer may be approximated by 0.9. The standard deviation is to be equal to 0.15.

* Mass density of the crest element (\(\rho_{mat}\))
  The mean value for the mass density of the crest element is estimated to be 2650kg/m\(^2\), with a standard deviation of 50kg/m\(^2\). Both distributions will be assumed to be normal.

* Mass density of sea water (\(\rho_w\))
  The mass density of sea water has a mean value of 1020kg/m\(^2\). The standard deviation will be equal to 5 kg/m\(^2\). The distribution is assumed to be normal.
* Wave pressure coefficient ($c_p$)
  The mean value of the wave pressure coefficient is set to be equal to 0.364 (see also Chapter 7), its standard deviation is 0.05. Both are guessed to be normally distributed.

* Width of the crest element ($L_c$)
  The width of the crest element is 9.0m at CD-11.5m. The standard deviation will be set to be equal to 0.1.

* Area of the cross-section of the crest element ($\Omega$)
  The area of the cross-section of the crest element will be equal to 10.5m$^2$. The standard deviation will be assumed to be 0.1.

* Height on which the wave pressure acts on the crest element ($d'$)
  In the context of the situation at CD-11.50m, the height on which the wave pressure acts on the element should be set to be equal to 1.44m. Its standard deviation will be assumed to be 0.05. Both distributions will be assumed to be normal.

* Model uncertainty for the horizontal displacement of the crest element ($X_{d'c}$)
  The mean value for the model uncertainty for the horizontal displacement of the crest element is equal to 1.0 and its standard deviation is assumed to be 0.2.

For more details of the calculations, the reader is referred to Annex 5.
## Probability of failure of a horizontal displacement: $7.9 \times 10^3$ per year.

<table>
<thead>
<tr>
<th>Name Variable</th>
<th>Symbol</th>
<th>Uncertainty %</th>
<th>Original Value</th>
<th>Changes A - coeff.</th>
<th>Original B-coeff.</th>
<th>Changes B - coeff.</th>
<th>%</th>
<th>New Probability of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height</td>
<td>$H_s$</td>
<td>64.3%</td>
<td>3.31</td>
<td>$3.641 (+0.331)$</td>
<td>$2.979 (-0.331)$</td>
<td>$1.518$</td>
<td>$1.6698 (+0.1518)$</td>
<td>$1.3662 (-0.1518)$</td>
</tr>
<tr>
<td>Friction coefficient</td>
<td>$f$</td>
<td>0.6%</td>
<td>0.9</td>
<td>$0.99 (+0.09)$</td>
<td>$0.81 (-0.09)$</td>
<td>$0.15$</td>
<td>$0.225 (+0.075)$</td>
<td>$0.075 (-0.075)$</td>
</tr>
<tr>
<td>Unit weight of material</td>
<td>$\rho_{\text{mat}}$</td>
<td>0.1%</td>
<td>2.65</td>
<td>$2.915 (+0.265)$</td>
<td>$2.385 (-0.265)$</td>
<td>$0.05$</td>
<td>$0.075 (+0.025)$</td>
<td>$0.025 (-0.025)$</td>
</tr>
<tr>
<td>Unit weight of water</td>
<td>$\rho_w$</td>
<td>0.0%</td>
<td>1.02</td>
<td>$1.122 (+0.102)$</td>
<td>$0.918 (-0.102)$</td>
<td>$0.005$</td>
<td>$0.0075 (+0.0025)$</td>
<td>$0.0025 (-0.0025)$</td>
</tr>
<tr>
<td>Wave pressure coefficient</td>
<td>$a_0$</td>
<td>6.3%</td>
<td>0.364</td>
<td>$0.4004 (+0.0364)$</td>
<td>$0.3276 (-0.0364)$</td>
<td>$0.05$</td>
<td>$0.075 (+0.025)$</td>
<td>$0.025 (-0.025)$</td>
</tr>
<tr>
<td>Width of crest element</td>
<td>$L_w$</td>
<td>0.0%</td>
<td>9.0</td>
<td>$9.9 (+0.9)$</td>
<td>$8.1 (-0.9)$</td>
<td>$0.1$</td>
<td>$0.15 (+0.05)$</td>
<td>$0.05 (-0.05)$</td>
</tr>
<tr>
<td>Area of cross-section of the crest element</td>
<td>$A$</td>
<td>0.0%</td>
<td>3.5</td>
<td>$11.55 (+1.05)$</td>
<td>$9.05 (-1.05)$</td>
<td>$0.1$</td>
<td>$0.15 (+0.05)$</td>
<td>$0.05 (-0.05)$</td>
</tr>
<tr>
<td>Height on which the wave pressure acts</td>
<td>$d'$</td>
<td>0.0%</td>
<td>1.44</td>
<td>$1.584 (+0.144)$</td>
<td>$1.296 (-0.144)$</td>
<td>$0.05$</td>
<td>$0.075 (+0.025)$</td>
<td>$0.025 (-0.025)$</td>
</tr>
<tr>
<td>Model uncertainty</td>
<td>$X_m$</td>
<td>28.5%</td>
<td>1.0</td>
<td>$-$</td>
<td>$-$</td>
<td>$0.2$</td>
<td>$0.3 (+0.1)$</td>
<td>$0.1 (-0.1)$</td>
</tr>
</tbody>
</table>
Results

From Table 10.11, the following conclusions can be drawn:

1) the significant wave height ($H_s$) and the model uncertainty ($X_m$) contribute most to the uncertainty in the reliability function (64.3% and 28.5% respectively);

2) the largest corrections in the probability of occurrence occur by a change in the mean value of the area of the cross-section of the crest element ($\Omega$), the unit weight of material ($\rho_{mat}$), the wave pressure coefficient ($\alpha_0$) and the unit weight of water ($\rho_w$);

3) the largest change in the probability of occurrence occurs by a transformation of the standard deviation of wave pressure coefficient ($\alpha_0$);

4) an increase of 10% in the mean value of the unit weight of material and of the area of the cross-section of the crest element ($\Omega$) lead to an increase of 77.2% of the probability of occurrence of failure of the horizontal displacement of the crest element;

5) an increase of 50% in the standard deviation of the wave pressure coefficient ($\alpha_0$) leads to an increase of 26.6% in the probability of failure.

From these results the following recommendations may be made:

a) in order to get more insight in its standard deviation, the wave pressure coefficient should be investigated more thoroughly (see also preview Section);

b) a larger crest width will be recommended when decrease in the probability of occurrence of this failure mechanism is needed;

c) it is advisable to examine the reliability of this approach to the horizontal displacement of the crest element in order to reduce the standard deviation of the crest element and to verify the hypothesis of this approach.
10.4 Conclusions

From the quantification of four failure mechanisms for the crested rubble mound breakwater at Ennore, it appears that the most important difficulty is the determination of the mean value and the standard deviation of the variables. It also appears that in order to optimize the breakwater, an adjustment of these values has to take place after the calculations of the probabilities of failure according to the different mechanisms.

Because time, money and the prodigious size of civil projects put severe constraints on what kind of research is possible, a simpler approach would be to stick to the proposed level-II method analysis and to carry out a number of investigations designed to confirm (and, possibly, rectify) the reliability of this approach to the various individual failure mechanisms identified in this study.
Chapter 11
Conclusions and recommendations

Failures of parts of the breakwater will lead to an excessive overtopping over the breakwater and/or a rapid disintegration of the structure. To anticipate such eventualities, the possible failure mechanisms have to be borne in mind during the design and construction periods of the breakwater. To provide a survey of what might cause the failure of a crested rubble mound breakwater at Ennore, this study analysed a large amount of different failure mechanisms.

For each failure mechanism, the process is described and a possible fault-tree is given. These fault-trees are then collected into one fault-tree which leads to the top-event of the tree: the failure of the breakwater structure.

For what appeared some of the most important failure mechanisms for the situation at Ennore, reliability functions have been determined. Designed to compare the strength of the structure with the loads working on it, these reliability functions were calculated with a level-II computer program, and used an input data supplied by a study conducted by HASKONING:
- instability of a rock armour layer
- instability of an armour layer with ACCROPODE® elements
- slip circle of an armour layer with ACCROPODE® elements
- horizontal displacement of the crest element

A sensitivity analysis was then carried out to see which parameter influences most the probability of occurrence of those failure mechanisms.

For the instability of a rock armour layer, the following recommendations can be made:
- during the design and construction periods, special attention must be given to the monitoring of the height of the crest, the angle of the slope and the density of rock. Accurate backhoes or other machines for the shaping of the slope appear to be a good investment, and reliable research on the rock would probably prove very useful;
- for a decrease of the probability of occurrence of this failure mechanism, flatter slope and larger stones will be recommended.

For the instability of an armour layer with ACCROPODE® elements, the following advice may be given:
- because of the large influence of the standard deviation of Van der Meer's formula, it would be advisable to do more investigations with a rock slope and the attack of plunging waves;
- a small diminution of the mass density of concrete leads to an increase in the probability of occurrence of the failure mechanisms: control of the concrete during the manufacturing of the elements is therefore relatively important;
- higher mass density of concrete and larger elements will be recommended when decrease in the probability of failure is needed.
For the slip circle of the armour layer with ACCROPODE® elements, the following remarks can be made:
- precision in the determinations of the crest element and of the slope angle are required in order to limit a possible increase in the probability of occurrence of failure of the breakwater;
- a higher crest level and a flatter slope will be recommended when decrease in the probability of occurrence of this failure mechanisms is demanded;
- because of its simplification, Bishop's simplified model may not take all the possible parameters into account. More investigations are recommended on the reliability of this approach of the slip circle.

For the horizontal displacement of the crest element, an approach identical to that to the fracturing of an ACCROPODE® element was used. The same remark regarding the wave pressure coefficient is therefore valid. Furthermore, the following observations can be made:
- a larger crest width will be recommended when decrease in the probability of occurrence of failure is needed;
- in order to reduce the standard deviation of the crest element and to verify the hypothesis of this assumed approach, more investigations are recommended.

The aim of this study was to provide a survey of the imaginable failure mechanisms that could lead to the collapse of the breakwater at Ennore. In order to get an insight in the probability of failure of the top-event, all these imaginable failure mechanisms should be quantified. These quantification can be the topic of a following study.
References


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SOGREAH, "Strength of the ACCROPODE® - Finite Element Study", Grenoble.


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Annex 1

Drawing of the cross-section of the breakwater structure at Ennore at CD-11.5m
Annex 1

Legend:

A - Primary Armour Layer Sea Side: Accropode 15 Ton
B - Secondary Armour Layer Sea Side: Rock 1.5 Ton
C - Top Layer: Rock 0.5 Ton
D - Upper Armour Layer Port Side: Rock 5 Ton
E - Lower Armour Layer Port Side: Rock 1.5 Ton
F - Toe Layer Seaside: Rock 2 Ton
G - Second Filter Layer Sea Side: Rock 0.5 Ton
H - First Filter Layer Sea Side: Rock 10-50 Kg
I - First Filter Layer Port Side: Rock 10-50 Kg
J - Core: Quarry Run
K - Road: Concrete

Notes:
1. All measurements and levels are in meters
2. Levels are related to chart datum

Madras Port Trust
Ennore Coal Port Project

KBC Haskoning

CROSS-SECTION OF BREAKWATER WITH
ACCROPODE ARMOURING

Scale: 1:200
Sheet: 20-001
Annex 2

Calculations of the probability of occurrence
of the instability of a rock layer
Function Z( X : ARY; Switch : BOOLEAN ) : REAL;
{
rock breakwater stability armour layer Ennore 1995
formulae of Van der Meer
}

CONST
rho_w = 10.25;  { kN/m3 }

VAR
H_s,
fHs,
H_so,
H_smax,
S_p,
h_water,
h_w_lvl, bed_lvl,
L_p, L_shore,
T_p, T_z,
Gamma,
C1, C2, Hulp,
N_slope,
S,
P,
Delta_D,
rho_r,
Delta,
D_n50,
W_50 : REAL;

PROCEDURE Exchange;
BEGIN
H_so := X[1];
fHs := X[2];
S_p := X[3];
h_w_lvl := X[4];
bed_lvl := X[5];
N_slope := X[6];
S := X[7];
rho_r := X[8];
P := X[9];
W_50 := X[10];
C1 := X[11];
C2 := X[12];
END;

PROCEDURE Riprap(Hs, Tz, N_slope, P, S, C1, C2 : REAL;
VAR Delta_D : REAL);
CONST
N = 3000;
G = 9.81;

VAR
Br_Type: STRING[10];
L_Z, ksi_z, ksi_criterium : REAL;
\( n_{\text{stab}} : \text{REAL}; \)

\[
L_z := G \cdot \text{SQR}(Tz)/(2\pi) ;
\]
\[
k_{\text{i}} := (1/N_{\text{slope}})/\text{SQR}T(Hs/L_z);
\]
\[
\kappa_{\text{i}\_z} := \text{Power}(C1\cdot\text{Power}(P,0.31)/\text{SQR}T(N_{\text{slope}}), 1/(P+0.5));
\]
\[
\text{IF } k_{\text{i}\_z} < \kappa_{\text{i}\_\text{criterium}} \text{ THEN}
\]
\[
\text{BEGIN}
\]
\[
N_{\text{stab}} := C1 \cdot \text{Power}(P,0.18)\cdot\text{Power}(S/\text{SQR}T(N),0.2)/\text{SQR}(k_{\text{i}\_z});
\]
\[
\text{Br\_Type} := 'plunging';
\]
\[
\text{END}
\]
\[
\text{ELSE}
\]
\[
\text{BEGIN}
\]
\[
N_{\text{stab}} := C2 \cdot \text{Power}(P,-0.13)\cdot\text{Power}(S/\text{SQR}T(N),0.2)\cdot\text{SQR}(N_{\text{slope}})\cdot\text{Power}
\]
\[
\text{Br\_Type} := 'surging';
\]
\[
\text{END};
\]
\[
\Delta_D := Hs/N_{\text{stab}};
\]
\[
D;
\]

\text{procedure wavelength(T,H:real; VAR L1 :REAL);} 

\[
R
\]
\[
0,L2,X,A : \text{REAL};
\]

\[
\text{NST}
\]
\[
G = 9.8;
\]
\[
C0= 1.56;
\]

\[
\text{GIN}
\]
\[
L0:=C0 \cdot \text{SQR}(T);
\]
\[
\text{IF } H>L0/2 \text{ THEN } L1:=L0
\]
\[
\text{ELSE IF } H<L0/25 \text{ THEN } L1:=T^\ast\text{SQR}(G\ast H)
\]
\[
\text{ELSE}
\]
\[
\text{BEGIN}
\]
\[
L1:=L0;
\]
\[
A:=2^\ast\Pi^\ast H;
\]
\[
L2:= 0;
\]
\[
\text{END;
}\]
\[
\text{WHILE } \text{ABS}(L1-L2)>0.01 \text{ DO}
\]
\[
\text{BEGIN
}\]
\[
X :=A/L1;
\]
\[
L2:=\text{TANH}(X)\ast L0;
\]
\[
L1:=(2^\ast L2+L1)/3;
\]
\[
\text{END;}
\]
\[
\text{D;}
\]
\[
\{\text{end procedure length}\}
\]

\text{procedure breakerheight(L,H :REAL; VAR Hs :REAL);} 

\[
\text{NST}
\]
\[
\beta = 0.092;
\]

\[
\text{AR}
\]
\[
K : \text{REAL};
\]

\[
\text{GIN}
\]
\[
K := 2 \ast \pi / L ;
\]
\[
Hs := \beta \ast \text{TANH}(K \ast H) \ast L;
\]
\[
D;
\]
Exchange:
H_so := H_so + fHs;
IF S_p <0 THEN S_p := 0.01;
IF H_so <0.1 THEN H_so := 0.1;

L_p := 100* H_so/S_p;
T_p := SQRT(L_p/1.56);
T_z := T_p/1.25;

h_water := h_w_lvl - bed_lvl;

{ Breaking }
Wavelength(T_z, h_water, L_shore);
Breakerheight(L_shore, h_water, H_smax);
IF H_so > H_smax THEN H_s := H_smax ELSE H_s := H_so;

{ Rayleigh correction }
Hulp := h_water/H_s;
IF Hulp < 4 THEN Gamma := 1 - 0.03 * ( 4 - Hulp )ELSE Gamma := 1.0;

H_s := Gamma * H_s;
Riprap( H_s , T_z , N_slope, P , S, C1, C2, Delta_D);

Delta := rho_r/rho_w -1;
D_n50 := Power(W_50/rho_r , 0.33);
Z := Delta * D_n50 - Delta_D;

if switch = TRUE THEN
BEGIN
WRITE(LST,'h_water = ',h_water:6:2,,' HS_toe = ',H_s:6:2);
WRITELN(LST,' gamma = ',Gamma:6:2);
END
DATA FILE

INSTABILITY FOR BREAKWATER ROCK UNDER HURRICANE WAVE ACCORDING TO VAN DER MEER

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>H_sign</td>
<td>H_s</td>
<td>S</td>
<td>h_w_lvl</td>
<td>bed_lvl</td>
</tr>
<tr>
<td>G</td>
<td>3.31</td>
<td>1.518</td>
<td>0.00</td>
<td>7.0</td>
</tr>
<tr>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>G</td>
<td>2.95</td>
<td>0.514</td>
<td>0.00</td>
<td>3.00</td>
</tr>
<tr>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.5</td>
</tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>-11.50</td>
</tr>
<tr>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>3.5</td>
</tr>
<tr>
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</tr>
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<td>0.00</td>
<td>0.00</td>
<td>70.00</td>
</tr>
<tr>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>6.2</td>
</tr>
<tr>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

| H_sign_100G | 9.25 | 1.518 | 0.00 | 7.0 | 0.2 | max. 50 hurricanes |
Annex 3

Calculations of the probability of occurrence of the instability of an armour layer with ACCROPODE® elements
Function Z( X : ARY; Switch : BOOLEAN ) : REAL;

{ break water accropode stability Ennore 1995 }

CONST
  G = 9.81;
  rho_w = 10.25;

VAR
  H_so,
  fHs,
  H_smax,
  S_p,
  L_p,L_shore,
  T_p, T_z,
  Gamma,
  h_water,
  h_w_lvl,bed_lvl,
  Cl, Hulp,
  rho_c,
  Delta,
  D,
  W_accr : REAL;

PROCEDURE Exchange;

BEGIN
  H_so := X[1];
  fHs := X[2];
  s_p := X[3];
  h_w_lvl := X[4];
  bed_lvl := X[5];
  rho_c := X[6];
  W_accr := X[7];
  Cl := X[8];
END;

procedure Wavelength (T,H:real; VAR L1 :REAL);

VAR
  L0,L2,X,A : REAL;

CONST
  C0 = 1.56;

BEGIN
  L0 := C0 * SQRT(T);
  IF H > L0/2 THEN L1 := L0
  ELSE IF H <= L0/25 THEN L1 := T * SQRT(G * H)
  ELSE
    BEGIN
      L1 := L0;
      A := 2 * PI * H;
      L2 := 0;
      WHILE ABS(L1 - L2) > 0.01 DO
        BEGIN
          X := A / L1;
          L2 := TANH(X) * L0;
          L1 := (2 * L2 + L1) / 3;
        END;
      END;
    END;
END;
END;
{einde procedure lengte}

procedure Breakerheight(L,H :REAL; VAR Hs :REAL);

CONST
  beta = 0.1;

VAR
  K : REAL;

BEGIN
  K := 2 * pi / L;
  Hs := beta * TANH( K * H ) * L;
END;

BEGIN

Exchange;
  H_so := H_so + fHs;
  IF S_p < 0 THEN S_p := 0.01;
  IF H_so < 0.1 THEN H_so := 0.1;

  L_p := 100 * H_so / S_p;
  T_p := SQRT(L_p/1.56);
  T_z := T_p / 1.25;

  h_water := h_w_lvl - bed_lvl;

  { Breking }
  Wavelength(T_z, H_water, L_shore);
  Breakerheight(L_shore, h_water, H_smax);
  IF H_so > H_smax THEN H_s := H_smax ELSE H_s := H_so;

  { rayleigh correction }
  Hulp := h_water / H_s;
  IF Hulp < 4 THEN Gamma := 1 - 0.03 * SQR( 4 - Hulp ) ELSE Gamma := 1.0;

  H_s := Gamma * H_s;

  Delta := rho_c / rho_w - 1;

  D := Power(W_accr / rho_c, 0.33);

  Hulp := Delta * D * C1;

  Z := Hulp - H_s;

  IF switch = TRUE THEN
    BEGIN
      WRITE('Hs_toe = ', H_s:6:2);
      WRITELN('gamma = ', Gamma:6:2);
    END

END;
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value1</th>
<th>Value2</th>
<th>Value3</th>
<th>Value4</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_sign_1</td>
<td>3.31</td>
<td>1.518</td>
<td>0.00</td>
<td>7.00</td>
<td>0.1 [m]</td>
</tr>
<tr>
<td>f_H_s</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.8 [m]</td>
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<tr>
<td>s_p</td>
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<td>0.514</td>
<td>0.00</td>
<td>3.00</td>
<td>0.51 [-]</td>
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<td>h_w_lvl</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>1.57</td>
<td>0.30 [m]</td>
</tr>
<tr>
<td>bed_lvl</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>-11.50</td>
<td>0.67 [m]</td>
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<tr>
<td>rho_concr</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>24.00</td>
<td>0.50 [kN/m3]</td>
</tr>
<tr>
<td>W_accur</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>150.0</td>
<td>7.5 [kN]</td>
</tr>
<tr>
<td>Coef</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>4.1</td>
<td>1.00 [-]</td>
</tr>
<tr>
<td>H_sign_100G</td>
<td>9.25</td>
<td>1.518</td>
<td>0.00</td>
<td>7.00</td>
<td>0.2 [-]</td>
</tr>
</tbody>
</table>

1 hurricane

100 hurricanes
Annex 4

Calculations of the probability of occurrence of the slip circle
function Z(X : ARY; Switch: BOOLEAN): REAL;

VAR
  N_slope, Phi, Cohes, Crest_lvl, Safety_fac, Safety, AA, BB
    : REAL;

PROCEDURE Exchange;
BEGIN
  N_slope := X[1];
  Phi    := X[2];
  Cohes  := X[3];
  Crest_lvl := X[4];
  Safety_fac := X[5];
END;

BEGIN
  Exchange;

  AA := (-0.00135*Cohes - 0.00328)*Phi + 0.026197*Cohes + 0.0986;
  BB := (0.000647*Cohes + 0.0117)*Phi + 0.008475*Cohes - 0.0668;

  Safety := (BB * N_slope + AA) * (5.2/crest_lvl);

  Z := Safety - Safety_fac;
END;

SLIP CIRCLE STABILITY ACCORDING TO BISHOP

<table>
<thead>
<tr>
<th>S</th>
<th>10</th>
<th>0.0</th>
<th>30</th>
<th>UIT</th>
</tr>
</thead>
<tbody>
<tr>
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<td>N</td>
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<td>0.00</td>
<td>0.00</td>
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<tr>
<td>Phi</td>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Cohes</td>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Crest_lvl</td>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>Saf_crit</td>
<td>N</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>
Annex 5

Calculations of the probability of occurrence of the horizontal displacement of the crest element
HORIZONTAL DISPLACEMENT OF THE CREST ELEMENT

$S \quad = \quad 2.412$

Probability of failure $\quad = \quad 7.9E-0003$

<table>
<thead>
<tr>
<th>NAME</th>
<th>TYPE</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>( \mu )</th>
<th>( \sigma )</th>
<th>X#</th>
<th>Alpha</th>
<th>( \frac{dz}{dx} )</th>
<th>( \epsilon )</th>
</tr>
</thead>
<tbody>
<tr>
<td>f</td>
<td>N</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.900</td>
<td>0.150</td>
<td>0.812</td>
<td>0.006</td>
<td>2.291</td>
<td>2.00</td>
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<tr>
<td>rho_mat.</td>
<td>N</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>2.450</td>
<td>0.050</td>
<td>2.645</td>
<td>0.001</td>
<td>3.325</td>
<td>8.80</td>
</tr>
<tr>
<td>rho_w</td>
<td>N</td>
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<td>0.000</td>
<td>0.000</td>
<td>1.020</td>
<td>0.025</td>
<td>1.020</td>
<td>0.000</td>
<td>-4.623</td>
<td>-8.80</td>
</tr>
<tr>
<td>alpha_0</td>
<td>N</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.364</td>
<td>0.050</td>
<td>0.394</td>
<td>0.063</td>
<td>-22.309</td>
<td>-8.80</td>
</tr>
<tr>
<td>L_c</td>
<td>N</td>
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<td>0.000</td>
<td>0.000</td>
<td>9.000</td>
<td>0.130</td>
<td>9.004</td>
<td>0.000</td>
<td>-0.755</td>
<td>-6.80</td>
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<tr>
<td>OMEGA</td>
<td>N</td>
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<td>0.000</td>
<td>0.000</td>
<td>10.500</td>
<td>0.100</td>
<td>10.495</td>
<td>0.000</td>
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<tr>
<td>d_acc</td>
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<td>0.000</td>
<td>0.000</td>
<td>1.440</td>
<td>0.050</td>
<td>1.442</td>
<td>0.000</td>
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<td>-2.00</td>
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<tr>
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<td>G</td>
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<td>0.000</td>
<td>1.914</td>
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<td>8.797</td>
<td>0.643</td>
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<td>X_m</td>
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<td>0.000</td>
<td>0.000</td>
<td>1.000</td>
<td>0.200</td>
<td>0.742</td>
<td>0.285</td>
<td>11.850</td>
<td>8.80</td>
</tr>
</tbody>
</table>

\[ Z(X#) \quad = \quad 0.0000 \quad \text{Number of iteration} \quad = \quad 11 \quad \text{Calculation time} \quad = \quad 1.48 \text{ s.} \]

(This is a reliability function)

(for the horizontal displacement)

(of a crest element)

function Z ( X : ARY, Switch : BOOLEAN) : REAL;

VAR

f : \{ friction coefficient \}
rho_mat. : \{ unit weight of material \}
rho_w : \{ unit weight of water \}
alpha_0 : \{ wave pressure coefficient \}
L_c : \{ width of crest element \}
OMEGA : \{ area of the cross section of the crest structure \}
d_acc : \{ height of crest element \}
H_s : \{ significant wave height \}
X_m : \{ model uncertainty \}
S : \{ strength descriptor \}

REAL.

PROCEDURE Exchange;
BEGIN
f := X[1];
rho_mat. := X[2];
rho_w := X[3];
alpha_0 := X[4];
L_c := X[5];
OMEGA := X[6];
d_acc := X[7];
H_s := X[8];
X_m := X[9];
END;
This is a reliability function for the horizontal displacement of a crest element.

function Z(X : ARY, Switch: BOOLEAN) : REAL;

R :=
      rho_mat,  (! friction coefficient !)
     rho_mat,  (! unit weight of material !)
    rho_w,    (! unit weight of water !)
   pha_0,    (! wave pressure coefficient !)
L_c,       (! width of crest element !)
OMEGA,     (! area of the cross section of the crest structure !)
d_acc,     (! height of crest element !)
H_s,       (! significant wave height !)
V_m,       (! model uncertainty !)
l,         (! strength descriptor !)
3,         (! loading descriptor !)
:REAL;

PROCEDURE Exchange;
BEGIN
  t := X[1];
  rho_mat := X[2];
  rho_w := X[3];
  alpha_0 := X[4];
  L_c := X[5];
  OMEGA := X[6];
  d_acc := X[7];
  H_s := X[8];
  X_m := X[9];
END;

BEGIN
  Exchange;

  R := (t*rho_mat*OMEGA) / (rho_w*alpha_0*(0.5*t*L_c + 0.8*d_acc)) * X_m;
  S := H_s;
  Z := R - S;
END;