Breakwaters on steep foreshores

The influence of the foreshore steepness on armour stability

Master of Science thesis

Martijn Hovestad

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Front page: artistic impression of small scale Xbloc units on a breakwater in a flume.

PREFACE

This document is the final report of my graduation project in order to fulfil the requirements for obtaining the Master of Science (Ir.) degree at Delft University of Technology (DUT), faculty of Civil Engineering and Geotechnics, department of Civil Engineering (CE). The work was carried out in cooperation with Delta Marine Consultants (DMC).

Executing a graduation project is an integral part of the education program at Civil Engineering and is regarded as the final work of the education in which the student shows his skills and knowledge obtained at the faculty. The focus of this project was on breakwaters on steep foreshores and a major part of the research was executed by experiments in the wave flume of the Stevin III-laboratory at Delft University. The topic of breakwaters is a part of the Coastal Engineering section, however, the laboratory is a part of the Fluidmechanics section. The work was supervised by a graduation committee with members from both DUT and DMC:

Prof. dr. ir. M.J.F. Stive	DUT, Civil Engineering, section of Coastal Engineering (Chairman)
Ir. H.J. Verhagen	DUT, Civil Engineering, section of Coastal Engineering
Dr. ir. H.L. Fontijn	DUT, Civil Engineering, section of Fluidmechanics
Ir. M. Klabbers	Delta Marine Consultants b.v.

I would like to thank the members of this committee for the fruitful discussions I had with them and for useful advice they gave concerning both the execution of the experiments and the interpretation of the results.

Many people at the laboratory have helped me in various ways during the execution of the experiments. This ranged from the construction of the foreshore to the measuring equipment and the processing software. I would like to thank all of these people for their support.

Further, I would like to thank Delta Marine Consultants b.v. for giving me the opportunity to do this research for them. At DMC, I would like to thank the people of the coastal engineering section, especially Markus Muttray and Bas Reedijk, for their useful input.

Next to that, I would like to thank my friends for their support and for the great times we had together. Finally, I need to give a special thank you to my parents, who have shown such a tremendous patience during my years in Delft. Without their enduring support, my graduation from this university would certainly not have been possible.

Martijn Hovestad
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SUMMARY

Designing breakwaters on steep foreshores faces some problems. At present, it is not known what the influence of the steepness of the bottom slope is. Design formulae for armour layers don't take the bottom steepness into account in a direct manner. In theory, in irregular waves, wave average heights in shallow water become higher as the bottom becomes steeper, but clear formulae don't exist.

In this report, an attempt has been made to investigate the influence of the slope of the sea bottom on the stability of an armour layer. The two main questions were:

- is there an influence of the foreshore slope at all?
- if yes, how large is this influence?

In a literature study, no answer could be found to these questions, so a model investigation has been done in order to measure the influence of the foreshore steepness. In the investigation, two foreshore steepnesses have been compared (1:30 and 1:8) and four different wave periods have been used. In most experiments, the armour existed of stone.

Two comparisons have been made: one was with the same wave conditions at the wave board, the other was with similar wave conditions close to the toe of the breakwater. The experiments showed a clear influence of the steepness of the foreshore:

- with equal offshore wave conditions, the damage is significantly higher at a steep foreshore.
- if the wave conditions near the breakwater are known, even then the damage on the steep foreshore is higher than on the mild foreshore.
- interlocking units showed more rocking.

As, for the last case, the wave heights and the wave spectra were equal at the toe, these cannot explain the differences. Therefore, another explanation has been tried to be found. It appeared that the wave shape, or more precise: the steepness of the wave front, changes. On the steep foreshores, the wave fronts are steeper. This may explain the larger damage levels at the steep foreshore.

At present, there are no wave theories that can calculate such a steepness of the wave front. Therefore, an empirical correction factor has been derived, which can take several values, depending on the situation. Until more research has been done on this topic, it is recommended to pay special attention to areas with steep foreshores if model tests are performed in the breakwater design process.

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LIST OF PARAMETERS

A	scaling factor (Goda)	[-]
A_e	erosion area	[m^2]
d	stone thickness or axial breadth	[m]
D	stone diameter	[m]
D_{15}	sieve size where 15% of the stones fits through	[m]
D_{50}	sieve size where 50% of the stones fits through	[m]
D_{85}	sieve size where 85% of the stones fits through	[m]
D_n	nominal stone diameter	[m]
D_{n50}	median of the nominal stone diameter	[m]
$D_{n50\text{-}core}$	median of the nominal stone diameter of the core	[m]
$E(f)$	spectral density	[m^2/Hz]
f	frequency	[Hz]
f_m	peak frequency	[Hz]
F_G	force of gravity	[N]
$F_{//}$	slope parallel gravity component	[N]
$F_{\perp}$	slope perpendicular gravity component	[N]
g	acceleration of gravity	[m/s^2]
h	water depth	[m]
h_b	water depth at the wave board	[m]
h_{br}	water depth at breaking point	[m]
H	wave height	[m]
H_0	deep-water wave height	[m]
$H_{1/3}$	average of the $^1/_3$ highest part of the waves	[m]
$H_{1/10}$	average of the $^1/_{10}$ highest part of the waves	[m]
$H_{1/250}$	average of the $^1/_{250}$ highest part of the waves	[m]
H_{br}	maximum wave height at the breaking point	[m]
H_{m0}	m_0-wave height	[m]
$H_{m0,0}$	H_{m0} at deep water	[m]
$H_{m0,b}$	H_{m0} at the wave board	[m]
$H_{m0,t}$	H_{m0} at the toe	[m]
$H_{m0,u}$	undecomposed H_{m0} at the toe	[-]
H_{max}	maximum wave height	[m]
H_s, H_{sig}	significant wave height	[m]
H_{spike}	wave peakedness descriptor	[m]
K_D	breakwater coefficient (Hudson)	[-]
K_s	shoaling coefficient	[-]
l	maximum axial stone length	[m]
L	wavelength	[m]
L_0	deep-water wavelength	[m]
L_0	deep-water wavelength	[m]
M	stone mass	[kg]
m_{-1}	-1^{st} moment of the spectrum	[m^2/Hz]
m_0	0^{th} moment of the spectrum	[m^2]
m_D	dry mass of a stone	[g]
m_n	n-th moment of the spectrum	[m^2/Hz^n]
m_U	underwater mass of a stone	[g]
N	number of waves	[-]
N_i	number of displaced stones (scaled)	[-]
N_L	scaling factor for length	[-]

N_{od}	number of displaced stones (normalised)	[-]
N_S	number of displaced stones	[-]
N_T	scaling factor for time	[-]
P	notional permeability (Van der Meer)	[-]
Q_1, Q_2, Q_3, Q_4	dimensionless surface elevation rising velocity descriptors	[-]
R_1, R_2, R_3, R_4	surface elevation rising velocity descriptors	[-]
s	wave steepness	[-]
s_0	deep water wave steepness (based on T_p)	[-]
S	damage level (Van der Meer)	[-]
$T_{m-1,0}$	wave period (Van Gent)	[-]
T_p	peak period	[s]
V_s	volume of a stone	[m^3]
W	stone weight	[N]
z	sieve size (side of smallest square hole where a stone fits through)	[m]
α	slope angle breakwater face	[-]
α	scaling factor (Pierson-Moskowitz)	[-]
β	bottom steepness (slope angle)	[-]
γ_0	scaling factor (Jonswap peak enhancement factor)	[-]
γ_{br}	breaker parameter	[-]
γ_H	foreshore steepness correction factor for Hudson-formula	[-]
γ_{HH}	foreshore steepness correction factor for Hudson-formula	[-]
γ_M	foreshore steepness correction factor for Van der Meer-formulae	[-]
γ_{MM}	foreshore steepness correction factor for Van der Meer-formulae	[-]
δt	time period between zero-upcrossing and top of a wave	[s]
$\delta\eta^+$	maximal surface elevation in a wave above mean water level	[m]
Δ	relative underwater density	[-]
η	surface elevation	[m]
ξ	Irribarren (breaker-) parameter	[-]
ξ_F	Irribarren parameter at the foreshore	[-]
ξ_m	Irribarren parameter (Van der Meer)	[-]
ρ_s	density of stone	[kg/m^3]
ρ_w	density of water	[kg/m^3]
σ, σ_a, σ_b	scaling factor (Jonswap peak enhancement factor)	[-]
Ψ	lump factor for Van der Meer formulae	[-]

1 INTRODUCTION

In this chapter, the problem will be described and briefly analysed. The research objective is then defined, followed by the outline of the report

1.1 INTRODUCTION

All structures in the coastal zone (but of course also many of them in inland waters), regardless of their function, have one thing in common: they must be able to withstand the forces caused by wave attack. Breakwaters and sea defences are no exception to this rule. As a matter of fact: the wave attack is one of the most important design parameters for these structures and they must be able to resist this attack well.

Breakwaters can be divided in two main types: the rubble-mound breakwaters and the monolithic breakwaters. Next to that, there are a few special types.

1.1.1 Rubble-mound breakwaters

Most breakwaters[1] in the world are of the rubble-mound type. They basically consist of a core with relatively fine, loose material (the so-called quarry-run), some kind of toe structure, one or more filter layers and an armour layer, see Figure 1-1. Usually, a filter layer is placed under the breakwater as well. Sometimes, a crown wall is added on the top of the breakwater, but this is not necessary. The armour layer can either be a closed layer or a granular layer. A closed layer usually consists of asphalt. A granular layer is an open layer and can consist of rocks or special concrete units. The choice for rock or concrete units depends on several factors, like the wave height.

[1] and sea defences as well. In the remainder of this text, sea defences will not be mentioned separately any more. Where it says "(a) breakwater(s)", the reader should read "(a) breakwater(s) and/or (a) sea defence(s)".

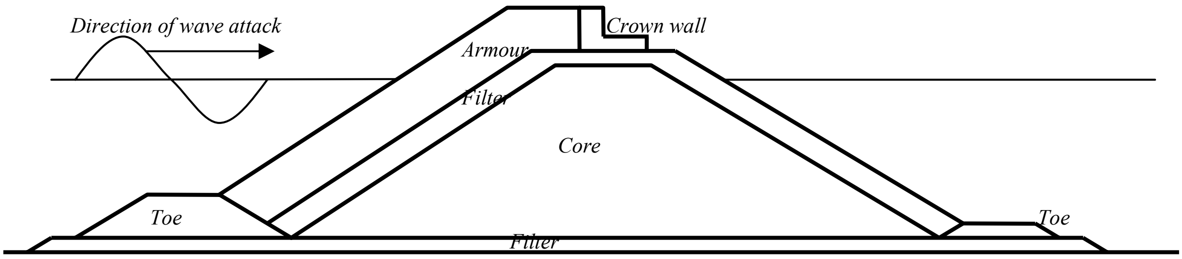


Figure 1-1: Basic rubble-mound breakwater lay-out

The main strength parameter for most types of breakwater is the type of armour and the weight of this armour. The interlocking between the units is important. Interlocking is the principle that units in a breakwater armour "work together". By doing so, they distribute the forces caused by waves to neighbouring units. This way, the mass of individual armour units can be reduced. Rocks and concrete cubes have relatively little interlocking whilst other units such as e.g. Dolos, Accropode and Xbloc are designed to have a high interlocking rate. Picture 1-1 shows an example of interlocking with Xbloc units.

Picture 1-1: Interlocking between Xblocs (scale model)

1.1.2 Other types of breakwater

Besides rubble-mound breakwaters, other types of strucutres are also used. An important type is the caisson breakwater, a monolithic type of breakwater and especially popular in areas with deep nearshore waters, e.g. in Japan. For this type of breakwater, completely different design considerations apply (and these will not be dealt with in the remainder of this report). Also combinations between rubble-mound and caisson type breakwater are possible.

Next to the main types of rubble-mound and monolithic, special solutions are also possible, like floating breakwaters. These, however, account for only a very small part of the world's breakwaters and will not be treated any further here.

1.1.3 Breakwater design

Designing a breakwater means finding a stable configuration for a breakwater, given the wave conditions. If the breakwater has to be stable, this, amongst many other things, means that the individual armour units must be stable. Thus the stability is often defined as a ratio between the unit size (be it a weight or a length scale) and the wave height. If such a unit cannot withstand the incoming wave, it can be removed from the breakwater profile by wave action.

If an armour unit has been removed, this doesn't mean a failure of the breakwater yet. The placement of units is a stochastic process and it is quite possible that a unit has already an unstable position right after the construction of the breakwater. It can then be removed in relatively light wave conditions. The point at which an occasional unit can be removed is called the 'start of damage'. This is normal for rubble-mound breakwaters and doesn't have to be a problem. Of course, the number of units that are removed this way should not be too large.

In severe storms, there is a risk of loss of more units. This is called damage, but as long as the remainder of the armour is still able to protect its underlayers, this is not considered failure. Depending on the damage level, a repair may be necessary after the storm.

If, however, the wave attack becomes that severe that such a large number of armour units is washed away that underlayers become damaged, this may quickly lead to the failure of the breakwater. In a well-designed breakwater, this should not happen in the design conditions.

Although many design parameters, including safety statistics, economic and practical considerations, apply, the design of a breakwater basically boils down to finding the correct weight of the armour units given the design wave conditions.

1.2 PROBLEM ANALYSIS

From engineering practice it is supposed that the steepness of the sea bottom in front of a breakwater or sea defence, in Figure 1-2 denoted as the angle β, may have influence on the stability of the armour units on these structures. In the past, some of these structures have failed and it is thought that the steepness of the foreshore may have had an influence on the failure.

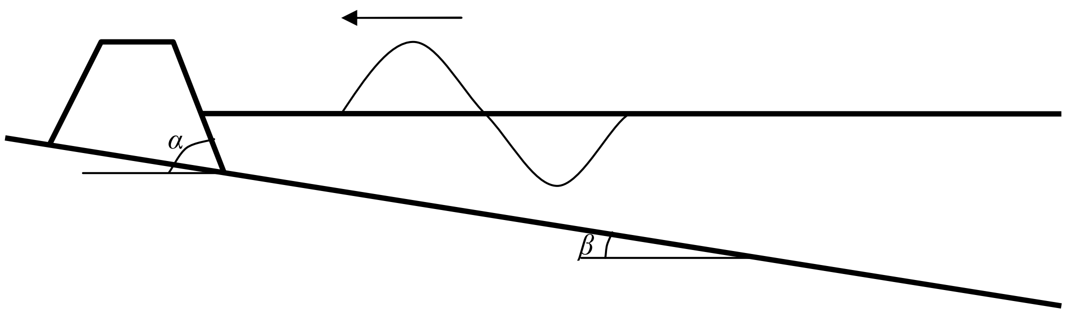


Figure 1-2: Definition of the foreshore slope (angle β)

1.2.1 Example: the damage at the Scarborough sea defence

A very recent example of damage to a coastal structure, where a steep foreshore is thought to have had an influence, is the damage of the sea defence of Scarborough in the United Kingdom.

The sea defence at Scarborough was constructed with Accropode-armour units; in the affected area, 6.3m^3 units were used. The sea defence features a rather unusual toe: during construction, a row of concrete piles was grouted into the sea bottom. The piles extend some distance above the sea bottom; this way, they are able to hold the armour layer in position.

Scarborough lies at a headland and the coastline has a rather convex shape. As such, it works as a "wave lens", focussing waves during storms. Below the water surface, near the shore, the bottom has a quite irregular shape. At many places, the bottom slope is relatively mild until quite some distance offshore. However, at an about 200-metre long stretch along the coast, there is a flat plateau along the coastline, at a short distance offshore. Between the plateau and the coastline, the sea bottom rises sharply, with inclinations of up to 1:7 (see Figure 1-3, depth contour lines lying close to each other). The design of the sea defence was the same in this section as in the adjacent sections.

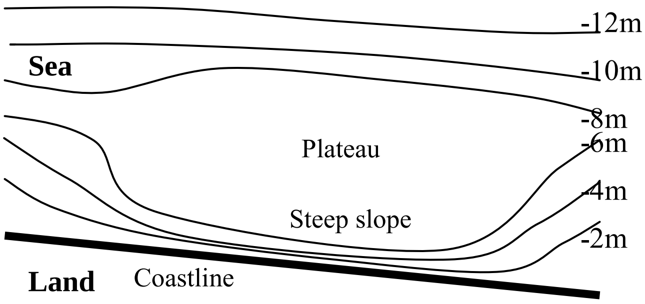


Figure 1-3: Part of the coastline and depth contours at Scarborough (illustrative, no real lines or values)

Although the investigation into the damage is still going on, it is striking that during the storm that caused the damage, the damage was done only to the section of the sea defence located along the steep part of the sea bottom, not to any other section. Many armour units were broken in two or more pieces, or parts were broken off, as can be seen in Picture 1-2 and **Error! Reference source not found.**.

Picture 1-2: Impact of high waves at the Scarborough sea defence

Picture 1-3: Damaged armour units after storm

A visual inspection showed that the toe of the breakwater did not collapse, so apparently the damage has directly been done to the armour layer.

Later storms further damaged the sea defence, also at adjacent sections, but the damage is concentrated at the section with the steep foreshore. Regarding the damage, the Scarborough case nourishes the assumption that the inclination of the sea bottom may have an (major) influence on the loads on a sea defence.

In spite of the cases like the one described in this chapter and similar cases, the physical process that influences the stability in steep-foreshore situations is very poorly understood.

1.2.2 Literature and existing theories

In the process of designing a breakwater, two things are the most important:
- calculating the wave height near the breakwater and
- calculating the stability of the armour layer, given the wave conditions.

In Appendix E, a more detailed description of the theory of waves and the design of armour layers is given, including a literature review. A brief overview will be presented here.

1.2.2.1 Wave formulae

For calculation of wave heights, a large range of formulae exists. The best-known theory is the linear wave theory, which, under a number of assumptions, can calculate wave heights at a given depth as a function of a given offshore wave height (or a wave height at another depth) and a wave period. Although it is widely used and works very well for many practical situations, this theory does not take the steepness of the bottom into account: it takes just the depths themselves and doesn't take into account what happens between the two locations. Furthermore, it has no inherent function for the breaking of waves, although it is known that the bottom steepness may influence the breaking, both for the way waves break and at which depth-height ratio they break. The absence of these properties in the linear wave theory means that it cannot be used to explain the (possible) differences in the behaviour of the armour layer on breakwaters on different bottom steepnesses.

Another well-known theory is the theory by GODA (2000). Although it has a parameter modified by the bottom steepness, the formula Goda gives is very complicated and does not give any physical insight in what happens at different bottom steepnesses. (For more on this formula: see Appendix C.)

Another type of formulae is the formulae that describe the wave height distribution. For rather deep water, some good theories exist which work well. For shallow water however, especially if waves are breaking, the situation is far more difficult. Many theories exist, many of them even taking the bottom steepness into account, but these models are either very complicated and not fit for practical use, or they rely on numerical calculations. The results of these numerical calculations are hard to interpret and cannot be used in the situations as intended in this research.

In summary: although some research has been done, and is still going on, a ready to use theory of waves in relatively shallow water, incorporating the bottom steepness, which gives a clear physical insight, is not available at this moment.

1.2.2.2 Armour calculation formulae

The two most well known formulae to design breakwater armours are the formulae by Hudson and Van der Meer (see e.g. D'ANGREMOND et al (2001)). The Hudson formula is the elder of the two and although easy to use, it relies heavily on an empirical coefficient, K_D, in which many influences are gathered. The Van der Meer formula takes more influences into account and it also incorporates a damage coefficient, an important improvement in comparison with the Hudson formula.

Neither the Hudson formula nor the Van der Meer formula takes the bottom steepness in front of the breakwater into account in a direct manner. A possibility is that it would be done by modifying the input parameter of the wave height, but as has been described in the previous paragraph, this is a very hard to calculate parameter.

Other theories, that maybe include the bottom steepness in some direct manner, have not been found during the literature research.

The uncertainty about the influence of the foreshore has led sometimes to design recommendations that are sometimes very strict for steep foreshores. For example, the Hudson K_D-stability coefficient is often used for indicating the stability of a certain armour unit. In case of a steep foreshore, this value sometimes drops to nearly half its value compared to its mild-foreshore value. It is not clear how these values are determined. They may be a good estimation, based on experience, but as, to the author's knowledge, no systematic research into the influence of the bottom slope has ever been conducted, so it may as well be that these values are incorrect.

It is remarkable that in many researches into breakwater stability as well as in design studies, if the bottom steepness is mentioned at all, this fact is merely mentioned and further seems to be fully neglected. Most model studies on breakwater stability are performed on flat bottoms or relatively mild bottom slopes, 1:30 maximum.

1.2.2.3 Breakwater design on steep foreshores

As has been mentioned before, design formulae for breakwaters do not take the steepness of the foreshore into account in a direct manner. It could however be done through the wave height: waves propagating over a sloped bottom change their properties. If the wave height is known and the design formulae are correct, then it should be possible to design a reliable armour layer. But as stated earlier: experience sometimes tells a different story.

All together, the engineering of breakwaters or sea defences on steep foreshores faces some uncertainties and it is very desirable to gain insight in the physical process, to check whether the influence of the foreshore is really important and if so, how big the influence is and what parameters are important.

1.2.3 Problem limitation

The foreshore can influence the stability of the breakwater in two different ways; see Figure 1-4. The first possibility is through the sea bottom (geotechnical influence), for example a sliding circle may develop which is less likely at a mild foreshore. The other possibility is through the motion of water on the foreshore (hydraulic influence), i.e. wave and flow properties change due to the altered bottom slope. The geotechnical influence will not be considered here or in any other part of this report; the focus will be on the results of the alteration of the water motion by the sea bottom.

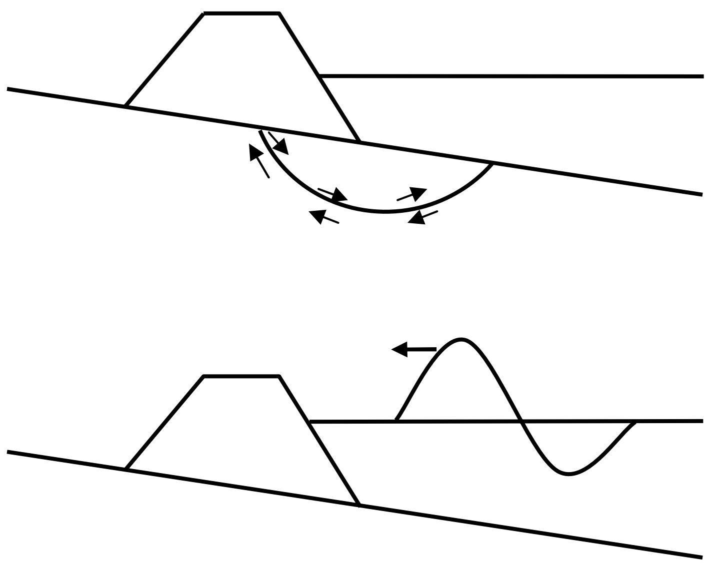


Figure 1-4: Geotechnical versus hydraulic influence

Within the area of hydraulic influence, the focus will be on the influence of short (wind-driven) waves, as they occur during a (design) storm. Long waves (e.g. tides, seiches), exceptional events (e.g. tsunamis) or (steady) flow phenomena (e.g. surf zone currents) will not be taken into account.

As a failure mechanism for the breakwater, only failure of the armour layer will be considered. Research into toe stability is an interesting topic in conjunction with steep foreshores, but will not be dealt with here, nor will any other possible failure mechanism.

Another choice to be made is the type of armour. The original question was to investigate the influence of the foreshore steepness on the stability of Xbloc-armour units alone. But as will be argued in paragraph 2.2.4, it makes sense to perform a major amount of tests on an armour layer built of loose stones. So the types of armour to be researched are stones and Xbloc unitss.

So in short: this report will focus on the influence of short waves, propagating on an sloping sea bottom, on the stability of a rubble-mound breakwater (with rock armour) and of a breakwater with an armour layer of concrete armour units.

1.3 PROBLEM DEFINITION

At present, the influence of a steep foreshore on the stability of breakwater armour units is unclear and some recommendations might lead to inaccurate designs. For practical engineering situations, it is considered desirable to have more insight into the stability of armour units on a breakwater, built on an inclining sea bottom. Although it is assumed that the steepness of the sea bottom may have an influence on the stability of the armour units of a breakwater, the exact influence remains unknown. No current theory includes the influence of the steepness of the sea bottom slope and therefore a research should be undertaken to assess this influence.

The literature does not give any clear answer to the question what makes the design of a breakwater armour on a steep foreshore different than a breakwater on a mild foreshore. All together, the engineering of breakwaters or sea defences on steep foreshores faces some uncertainties and it is very desirable to gain insight in the physical process, to check whether the influence of the foreshore is really important and if so, how big the influence is and what parameters are important. This leads to the following definition of the problem (see next page):

At present, there is no theory, calculation method or design guideline which clearly indicates the influence of the bottom steepness on the stability of an armour layer of a breakwater. As a steep foreshore is a situation that occurs frequently in engineering practice, more insight in the processes that occur at steep foreshores near breakwaters is desired.

1.4 RESEARCH OBJECTIVE

The present research aims at assessing the influence of the steepness of the sea bottom slope on the stability of armour units on a breakwater built on this sea bottom. This shall be done by a literature study and by experiments in a wave flume. The ultimate goal is to investigate whether there is an important influence at all. This shall be described in a qualitative way, and if possible, in a quantitative way, so to give an order of magnitude of this influence. A possible explanation for the influence of the foreshore shall be given as well. In summary, the research objective can be summarised as:

The objective of this research is to investigate whether there is a significant influence of the bottom steepness on the stability of breakwater armour layers and to give an order of magnitude of this influence.

1.5 REPORT OUTLINE

As existing theories do not give answer to the question what happens on a steep foreshore, a model research was proposed. This model research is the core of this report and it will be treated extensively. The next chapter, chapter 2, starts with a description of the set-up of the experiments and the equipment that was used. Chapter 3 describes the results of the experiments, while in chapter 4 the in-depth analysis of these results is done. Chapter 5 gives a few guidelines for design of breakwaters on steep foreshores, based on the findings in the experiments. Finally, chapter 6 gives the conclusions and the recommendations as they were found from this research.

2 EXPERIMENT SET-UP

In this chapter, the equipment that was used for the tests will be described, along with the configuration of the breakwater and the waves that were used. Further, the testing procedures are described.

2.1 GENERAL SET-UP OF THE EXPERIMENTS

The main goal of the present research is to investigate the difference in the behaviour of the armour layer of the breakwater at different steepnesses of the foreshore. As has been argued in the previous chapter, it became necessary to perform some experiments to investigate the influence of the foreshore steepness on the behaviour of the armour layer.

In the experiments, two different foreshores were therefore installed: a 1:30-foreshore and a 1:8-foreshore. For the rest, the configuration of the breakwater was kept the same.

The values for the steepnesses were chosen for practical reasons. First of all, it was desired that the values of the steepness were as far apart as possible, in order to create the biggest possible effect (if any). The 1:30-slope was chosen as the “mild” foreshore case, as a floor with this inclination is permanently fixed in the flume. The 1:8-slope on the other hand was the result of an optimisation: it needed to be as steep as possible, but making it too steep would result in the horizontal length of the slope becoming too short in relation to the wave lengths to be used. If it is too short, it may be that the waves don’t shoal correctly. Thus the steepness was chosen in such a way that the length of the slope was about as long as twice the wave length (the wave length in this case defined at the deepest part of the flume, based on the peak period). This resulted in the 1:8-foreshore, from now on referred to as “steep”.

In the following paragraphs, more details will be given on how the experiments were set up and how the tests were executed.

2.2 EQUIPMENT

In this section, all the equipment that was used is described.

2.2.1 Flume

The measurements were executed in the "Lange Speurwerkgoot" ("Long Research Flume") of the Fluid Mechanics Laboratory of the Faculty of Civil Engineering and Geosciences at Delft University of Technology. This flume is 42m long, 80cm wide and has a maximal depth of 100cm. Inside the flume, there is a semi-permanent inclining floor at a 1:30-grade. Between the beginning of this inclining floor and the wave board, the floor is flat. The slope starts at about 8.5m from the centre position of the wave board. For a layout of the flume, see Figure 2-1.

In the remainder of this research, the level of the flat floor between the wave board and the slope will be referred to as the reference level.

Along the complete length of the flume, a chain rail is installed along the upper side of one of the side-walls, which can be used for an automatic measuring carriage. Although not used in this experiment, the presence of the rail is important as it limits the maximum usable depth of the flume to 95cm.

Picture 2-1: View of the wave flume, towards the wave board

For the second stage of the measurements, an additional sloping floor was installed on top of the existing 1:30-floor. This was done in such a way, that the resulting slope had a 1:8-grade. This slope ended at a height of 50cm above the reference level, thus the slope was horizontally 4m long. After this slope, a short horizontal section was built in order to be able to build the breakwater.

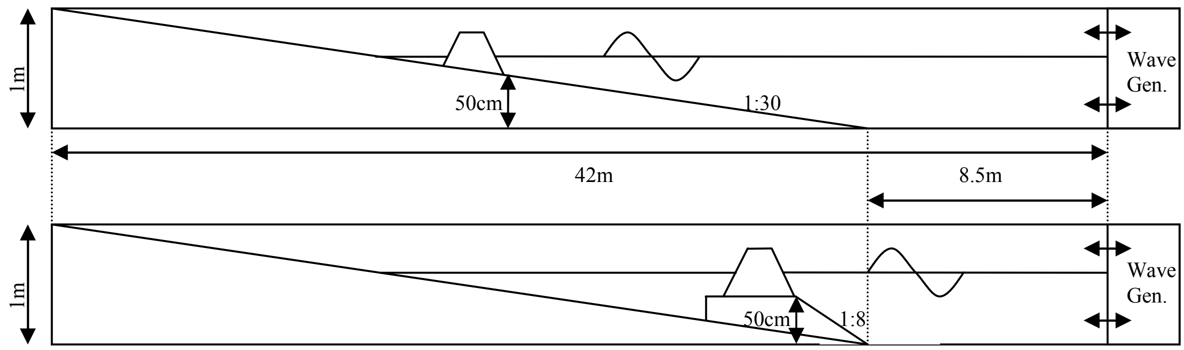


Figure 2-1: Basic wave flume lay-out (not to scale)

In most experiments, the water level was at 66cm above the reference level. As the toe of the breakwater was at 50cm above the reference level, the water level above the toe was 16cm. This level of 16cm at the toe was chosen as this would be the water level at which the waves would be close to their breaking point, so in the last stage of shoaling before breaking. A higher water level would lead to less shoaling, a lower water level to too much breaking.

The choice of the steepness of the new slope and the water depth to be used was largely a practical consideration. The water level could not be increased much beyond the 66cm, as irregular waves were to be used. The tops of the highest waves were not allowed to pass beyond the 95cm-level. Lower water levels would lead to a lowering of the complete breakwater, as it was desired to have 16cm of water at the toe. This would result in a shorter slope in case of the steep foreshore. This was undesired, as the slope had to have a length of at least one or two wavelengths in order to develop a good shoaling.

A similar consideration was applied for the steepness. It was desired to have a slope as steep as possible, in order to see slope effects as good as possible, but too steep a slope would again result in too short a slope. The 1:8-slope was found to be a good compromise.

2.2.2 Wave board

The waves were generated using a piston-type wave board. It is able to produce both regular and irregular waves. It has a 2-metre stroke and is equipped with a well-working Automatic Reflection Compensation (ARC) system, thus creating a real “sea boundary” at the wave board side of the flume, as it absorbs nearly all reflected wave energy.

The wave board is steered by a dedicated program, WL Wavegenerator Control. However, this program is not able to create the steering files by itself, so for creating the steering files the Delft Auke-software package was used. This program is able to create steering files for a large range of spectra, including Jonswap and Pierson-Moskowitz spectra.

2.2.3 Wave Measurements

For the measurements, four analogue wave gauges were used. These gauges measure the water level by measuring the voltage drop between the two poles. This can easily be translated to water levels easily, as the voltage drop behaves linearly to the water level by 10V::25cm. The range of the gauges is –10V to 10V, i.e. a maximal range for the water level variations of 50cm, which was sufficient for all tests. The sampling frequency of the gauges was set at 50Hz.

The gauges were grouped two by two. This is done in order to be able separate the waves that come from the wave board directly from the waves that are reflected by the breakwater. This process is called decomposition. Two gauges were placed on the deepest part of the flume, i.e. where the floor of the flume is still flat, just before the slope starts. This is done in order to measure the incoming waves from the wave board, which have not yet been affected by shoaling.

The other pair of gauges was placed just before the toe of the breakwater. The wave gauge closest to the breakwater was placed immediately in front of the bottom plate of the breakwater (see paragraph 2.3.2). The distance between two gauges in a pair was in all-but-one tests 30cm. The signals from the gauges were recorded using the DasyLab 5.03 32-bit software package. This programme creates a file with time series of the digitised measured voltages. The further processing of these files was done using Matlab, versions 12 and 13.

The decomposition programme was provided by Delft University of Technology and is based on the theory of ZELT and SKJELBREIA (1992). For two gauges, this theory is the same as the theory described by GODA (1976). Officially, this theory is valid for linear, non-braking waves only. However, at the toe, some wave breaking took place sometimes and it was evident that the waves were not always linear. Nevertheless, the programme was still used, as non-linear decomposition programmes are not available.

2.3 BREAKWATER

In the following section, the properties of the breakwater as it was constructed in the flume, will be described.

2.3.1 External properties

The breakwater was in all series constructed at 50cm above the reference level, so at about 23.5m from the centre position of the wave board in the first series of tests and at about 12.5m in the second series. The top of the breakwater just reached the chain-rail level, making it 45cm high. The front slope was constructed at a 1:2-grade, the rear slope had a 1:1.5-grade. The crest was 25cm wide.

The dimensions of the breakwater can be found in Figure 2-2, a photographic impression in Picture 2-2, Picture 2-3 and Picture 2-4.

It is to be noted, that the riprap armour did not extend all the way to the top of the breakwater. This was not necessary, as the wave impact occurred exclusively in the lower zone. Also the run-up hardly extended above the top level of the armour. In fact, the top of the breakwater is at an unnecessary high level, but as this was the way the breakwater was built, it is represented here that way.

The armour with Xbloc units did extend all the way to the top, as in the tests with Xblocs higher waves, with proportionally higher run-up levels, were used.

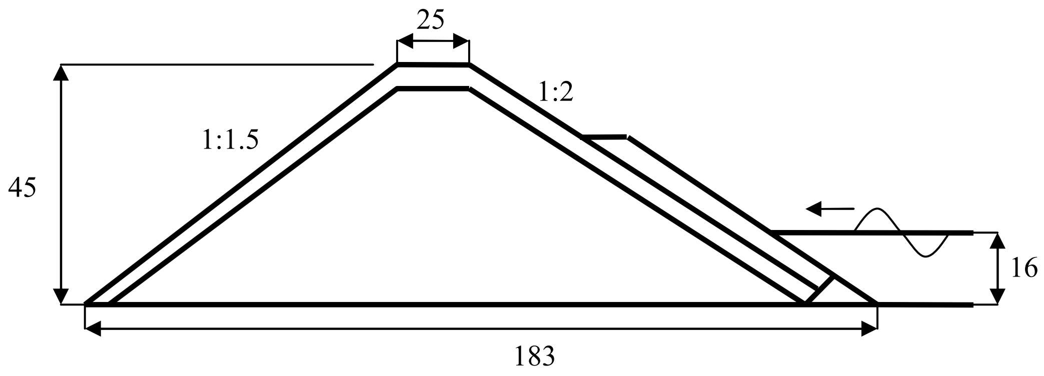
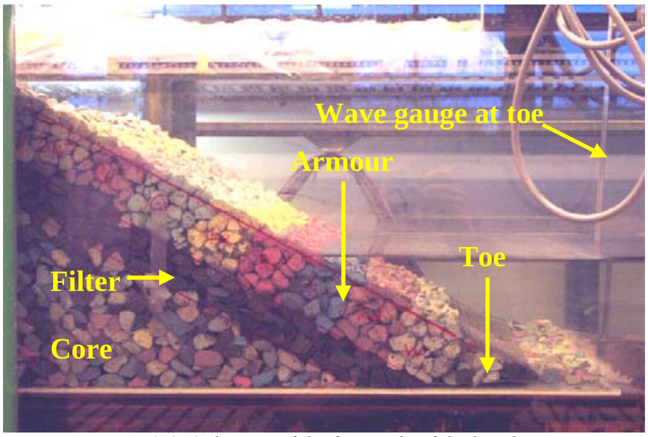


Figure 2-2: Breakwater dimensions in cm



Picture 2-2: Side view of the front side of the breakwater

Picture 2-3: Front view of the breakwater

Picture 2-4: Front view of breakwater with Xbloc units

2.3.2 Bottom layer

As the floor on which the breakwater was constructed consists of solid material, there was no need to construct a bottom layer or filter under the breakwater to prevent the wash out of bottom material. However, in order to prevent possible sliding of the complete breakwater or parts of it, it was necessary to build an adhesive layer. This was done by sticking stones with silicone kit onto a large plastic plate, which was as broad as the width of the flume; see Picture 2-5. The plate was fixed to the sloping bottom. This plate extended a few centimetres to the front and the rear of the actual breakwater.

Picture 2-5: The adhesive bottom plate

During the tests, a few stones were removed from the plate in front of the breakwater. This was not considered a problem, as the toe remained in place.

2.3.3 Toe

The purpose of this research was to investigate the influence of the foreshore on the stability of the armourlayer under wave attack. As toe research was not the purpose of this experiment, it was necessary to construct a strong toe, which would not fail under the test circumstances. Although a few design formulae exist for toes (see D'ANGREMOND et al., 2001), the uncertainty in them is still quite high. Also, given the water levels and wave heights that were planned, these formulae yielded rather large numbers for the required stone size, which would create more problems than it solved.

Sometimes, in researches like these, a small wooden bar is placed in front of the breakwater, to take over the function of a toe. However, this was regarded undesired for this research, as there were concerns on the permeability of this type of toe. It may be possible that wave pressures build up behind this bar during the receding of the water, which in their turn may push out the armour stones so the toe needed some possibility to drain.

Therefore it was decided to construct a kind of "gabion-toe": a prism-shaped toe (see Figure 2-3 and Picture 2-6) was constructed from mesh wire, which was filled with the same type stones as the core. The wire mesh extended about 10cm below the filter layer, so that the weight of the filter fixed the toe on the backside. The stones gave in the toe gave it enough mass to remain in place during all tests and by the nature of the construction, the toe had the same permeability as the layers above and behind it, so no artificial pressure build-up could occur.

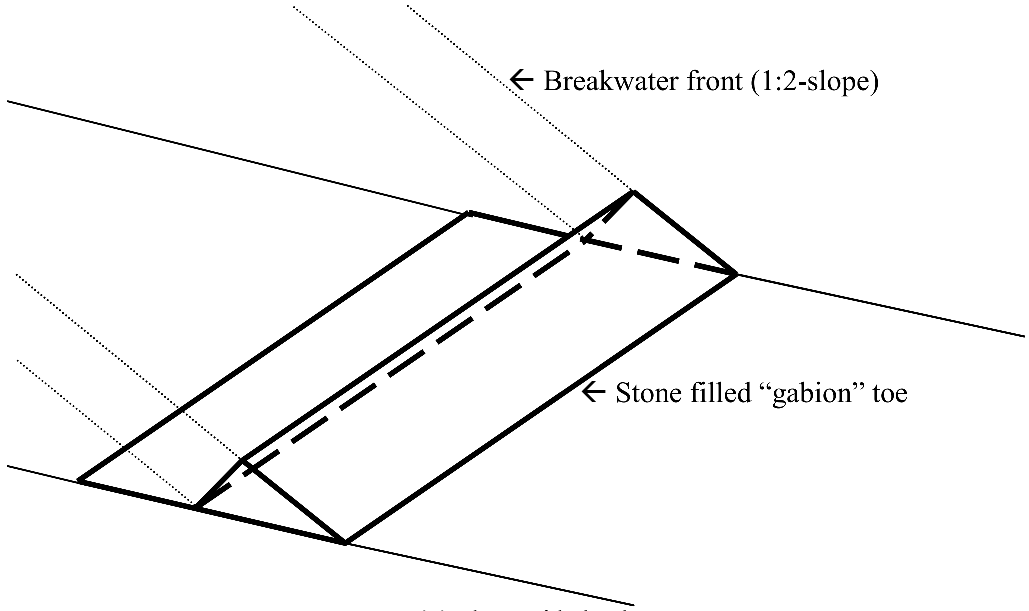
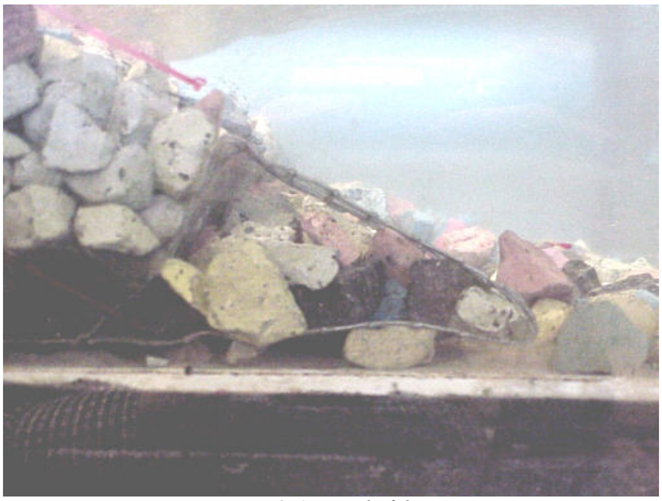


Figure 2-3: The toe of the breakwater



Picture 2-6: Detail of the toe

It is to be noted that the cross-section of this toe was triangular, not trapezoid. In a few tests in the beginning, a trapezoid cross-section was used, but it appeared that stones that were removed from the armour during the test would settle on top of this toe. After that, other stones settled on these stones again, thus creating a large mound of armour stones on the toe, which was undesired, as it may protect the armour layer by breaking the waves before they hit the armour. In order to prevent this, the triangular cross-section was constructed, of which the front was just in line with the armour layer of the breakwater.

Because of corrosion, it is not very feasible to use a construction like this in real-life breakwaters, but it appeared to work very well in this laboratory experiment.

On the back slope, no toe was constructed, as this side was not exposed and the adhesive bottom layer provided enough support for the back armour.

2.3.4 Armour

Originally, this research was intended to find the influence of the foreshore on the stability of Xblocs. However, it was decided to do a more conceptual research and the Xblocs were replaced by riprap for most of the experiments. The reasoning behind this is that interlocing armour units, Xbloc included, face a problem in model tests.

Tests in a wave flume on breakwaters are performed with gradually increasing wave heights until the armour fails. Failure is defined as a certain level of the damage. Riprap has a very gradual damage curve. Even below the design wave height, there will always be some damage as stones are removed. As the wave height increases, the damage will increase, with only a small statistical deviation.

Interlocking units on the other hand have a less clear damage curve. With wave heights below the design strength, damage will be minor, if there is damage at all. If an armour unit is removed, the ones surrounding it will relocate somewhat, so that they can work together again. As the wave height grows beyond the design value, still there won't be much damage until at a certain point, suddenly a large damage occurs, which is usually considered failure. This can happen because at this point, all armour units are working together at their maximal potential. If at this stage one block leaves the profile, a progressive collapse occurs, as the other blocks cannot take over any more loads. The point at which this happens is, unfortunately, difficult to determine in a statistical way: the deviation is large. This does not mean that inerlocking units are unsafe: the design value is always well below this collapsing value. Xbloc for example, has a start-of-damage wave height of around 140% of the design height and a failure wave height of at least 160% of the design wave height, while in some tests in the past they did not even fail at 200%.

So if Xbloc-units were to be used, that would either mean that the results would be hard to interpret, or a large number of tests had to be performed, in order to overcome the statistical problem. In order to overcome this, it was chosen not to use the Xbloc-units in most tests, but to use riprap. This way, it is possible to detect subtle changes between the behaviour on the mild and the steep slope. A few tests were done on Xblocs, illustrative, in order to compare the results with the riprap armour.

2.3.4.1 Armour stones

The selection of the stones was in a first step done by simply measuring a sample of stones with a slide gauge. Often, the following definition of the stone dimension is used (see also CUR, 1994):

- z sieve size (i.e. the side of smallest square hole where a stone fits through)
- l maximum axial length
- d thickness or axial breadth

This definition was also used in selecting the stones. This means that of every stone three sizes had to be measured.

The CUR (1994) advises to limit the amount of stones with a ratio l/d>3 to an amount of 3 to 5 percent.

After a sort of stones was found that appeared to have the correct properties, a more extensive sample of 100 stones was taken. However, from this sample, it turned out that more than 40% of the stones had the l/d-ratio of more than three, which is not acceptable, so during a visual inspection, a large amount of stones that looked more or less good was taken from the crate. From these stones, again a 100-stone sample was taken and this revealed that no more stones had an l/d-ratio of three or more, and for only a few stones (19), this ratio was more than two, indicating that on average, the stones had quite cubical dimensions.

From the measurements, it turned out that the D_{50} of the stones (i.e. the sieve size through which 50% of the stones passes) was 2,02cm. Furthermore, the D_{85} was 2,22cm and the D_{15} was 1,78, yielding a D_{85}/D_{15}-ratio of 1,25, indicating a (very) narrow grading. The sieve curve of the stones is given in figure 3.3.

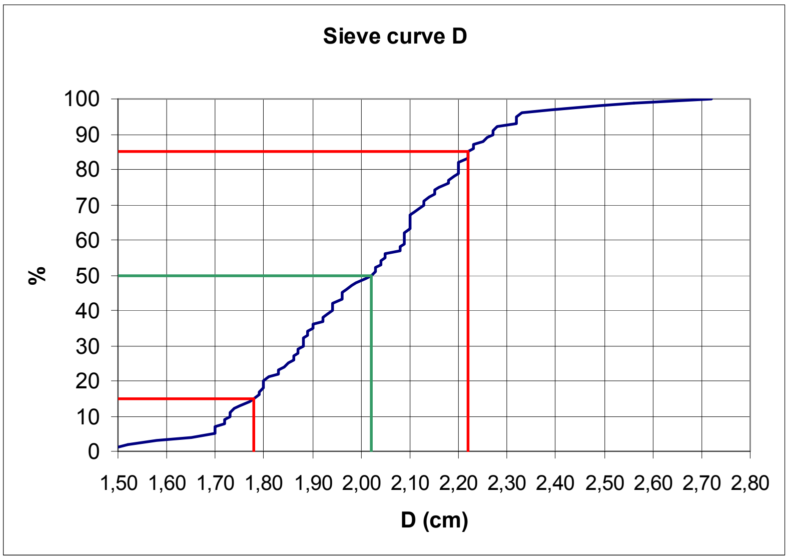


Figure 2-4: Sieve curve for the armour stones

After this, the volume and the density of stones were measured. This was done by carefully weighing them dry first. Then the stones were laid under water for a weekend and after this, they were weighed under water. From these weights, both D_{n50} and the stone density can be calculated:

Define:

V_s	volume of a stone	[cm^3]
ρ_s	density of the stone	[g/cm^3]
ρ_w	density of water	[g/cm^3]
m_D	dry mass of the stone	[g]
m_U	underwater mass of the stone	[g]

It can easily be seen that:

$$\rho_s = \rho_w \frac{m_D}{m_D - m_U}$$

and

$$D_n = \sqrt[3]{\frac{(m_D - m_U)}{\rho_w}}$$

ρ_w was taken constant at 1000kg/m^3.

The stones that were chosen, had a D_{n50} (i.e. a median D_n) of 1,57cm. It is also possible to create a sieve curve of D_n, see figure 3.4.

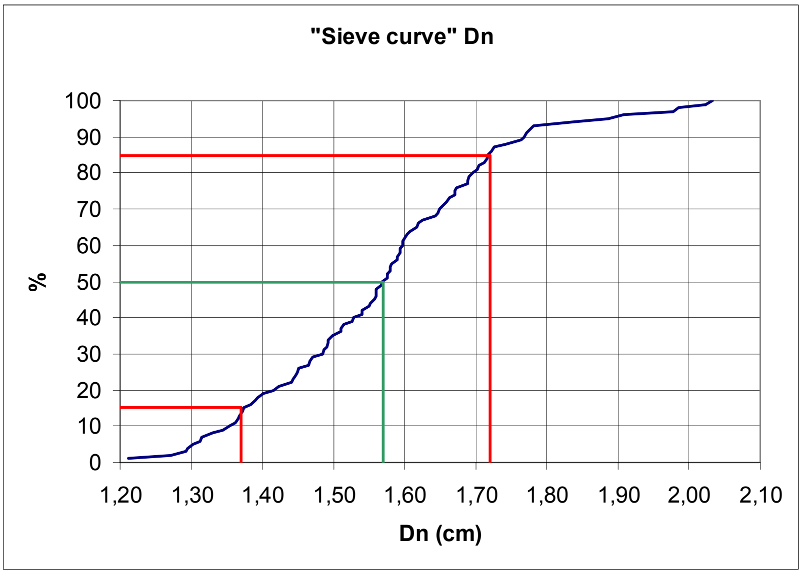


Figure 2-5: "Sieve curve" of the nominal diameter of the armour stones

In the density measurement it was found out that the stones consisted of two sorts, occurring in about the same numbers of stones. One sort had a density around (on average) 2630kg/m^3 and one sort had a density of around (on average) 2980kg/m^3. The weighed average was thus around 2780kg/m^3, see figure 3.5. From the outside, there was no possibility telling one stone sort from another.

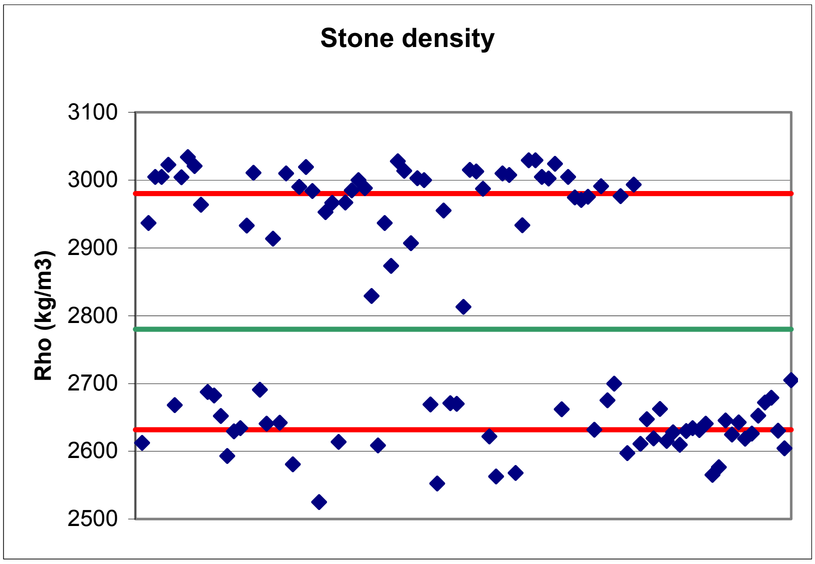


Figure 2-6: Density of the sample stones

The fact that these different densities occur may influence the damage measurements, as the stability of stones is heavily influenced by the density. Although the stones were well-mixed, it is still possible that the lighter stones are removed first. As a result, the remainder of the (heavier) stones may become unstable as well, causing them to be removed from the profile as well.

The stones were already painted in different colours, so they were sorted in order to be able to construct different lines in the armour layer, for detecting stone displacement.

Regarding the thickness of the armour layer, the CUR advises to use a layer thickness of al least $2d_{n50}$. As large damage numbers were projected to occur in this research, the thickness of the armour was doubled, i.e. it was 6cm thick.

2.3.5 Core

Scaling core material for laboratory research is always a bit difficult, as a clear laboratory effect may take place here, caused by a contradiction between the Froude scaling criterion and the Reynolds scaling criterion. The derivation of these scaling criteria can be found in Appendix D, but the result is as follows:

$$\begin{cases} N_T = \sqrt{N_L} & \text{(Froude)} \\ N_T = N_L^2 & \text{(Reynolds)} \end{cases}$$

In these formulae,

N	scaling factor	[-]
sub *T*	time	
sub *L*	length	

One can see that it is impossible to fulfil both criteria at one time with any scaling factor ≠1. Usually the Reynolds criterion is neglected, provided that the Reynolds-number is large enough, i.e. the flow should be turbulent. Scaling then takes place based on the Froude-scaling criterion.

The core of a breakwater usually consists of quarry run with a relatively small stone size. If geometric scaling is applied, this brings up a problem in the Reynolds scaling criterion.

As had been said before, in research practice, the Reynolds scaling criterion is often reduced to stating that in the model the Reynolds number should be large enough. In the case of quarry run, this requirement can't be met any more: the core material becomes too fine, so the hydraulic resistance increases and the flow velocities fall. The Reynolds numbers become so low, that the flow in the core becomes viscous. This changes the hydraulic behaviour of the core dramatically. For example, this may cause, during wave impact, internal pressure reflections between armour and core, which may push out armour units. As this is a purely artificial effect that will not happen in a prototype breakwater, this has to be prevented.

The easiest solution is to make the core material coarser. This way, the permeability will rise and the hydraulic resistance will drop, the flow becomes faster and shifts back to non-viscous flow. This procedure will not fundamentally change other breakwater properties, as the core is merely there for support to the upper layers and is not attacked directly by the waves.

BURCHARTH et al. (1999) have proposed a theory for scaling the core material in breakwaters. This theory states that scaling is done correctly if the hydraulic gradients in prototype and model are the same. Supposing a scaling factor of 1:50, and the other parameters as given in the previous paragraphs, the application of this theory yields that the stone size d_{n50} should be at least 1.2cm.

Although this result looks reasonable, larger stones were chosen: the same stones as were used in the armour layer, so d_{n50}=1.57cm. The reasoning behind this was that this research was not a model research of a prototype breakwater, but a purely conceptual research. The core doesn't have to be scaled like model of a prototype. The method of BURCHARTH et al. is sensitive for the scaling factor, so a different number could as well have been chosen. So in order to prevent all internal reflections, it was best to use the same stones as in the armour layer.

2.3.6 Filter

If the core and the armour layer have the same properties, a filter layer is strictly speaking not necessary. A filter was constructed in this research with the same stones as in the core and the armour, but with just black stones, in order to create a clear visual contrast between the armour and the core.

The thickness of this layer was $2d_{n50}$, i.e. 3cm.

2.3.7 *Xbloc units*

A limited number of tests was performed using Xbloc armour units. For these tests, the other properties of the breakwater were not altered. The Xbloc units have a sizes d=4,3cm and d_n=2,9cm. They were applied as a single layer, as they should be. Delta Marine Consultants also prescribes a staggered placement grid. The horizontal distance between the stones should be around 1,3d, the vertical distance between the lines around 0,65d. This way, any Xbloc is supported by two Xbloc units under it, but is also supports the two blocks above it.

For the toe in these tests, the gabion-toe was used, which performed also with Xbloc units well again.

2.4 WAVES

In this section, the wave programmes that were used to perform the tests are described.

2.4.1 *Spectra*

In all tests, irregular waves were used. This was done in order to simulate a "real" sea state in front of the breakwater, so that the results of this research can be translated to prototype situations. The waves were generated according to the standard Jonswap-spectrum. This spectrum describes a young sea state and was preferred over the Pierson-Moskowitz spectrum. That spectrum describes a fully developed sea state, but this is a situation that will hardly ever occur in nature.

The standard Jonswap-spectrum is described by:

$$E(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp\left(-\frac{5}{4}\left(\frac{f}{f_m}\right)^{-4}\right)\gamma_0^{\exp\left(-\frac{1}{2}\left(\frac{f-f_m}{\sigma f_m}\right)\right)}$$

with:

E	spectral energy density	[m^2/Hz]
α	scaling parameter (Pierson-Moskowitz)	[-]
f	frequency	[Hz]
f_m	peak frequency	[Hz]
γ_0	scaling parameter (Jonswap peak-enhancement factor)	[-]
σ	scaling parameter (Jonswap peak enhancement factor)	[-]

The last part of this equation is called the peak enhancement factor.

The value σ changes according to the frequency:
$\sigma=\sigma_a$ if $f<f_m$
$\sigma=\sigma_b$ if $f>f_m$

For the standard Jonswap spectrum, the values in the peak-enhancement factor are:

γ_0=3,3
σ_a=0,07
σ_b=0,09

These values were also used in all the experiments. More information on spectra can be found in e.g. BATTJES (1992).

2.4.2 Wave height, length and steepness

The spectrum has to be scaled according to the desired peak frequency and wave height. To do this, some representative wave height and wave period have to be defined.

In many theories, the (deep water) wave steepness is an important parameter. The wave steepness in general is defined by:

$$s = \frac{H}{L}$$

with:

s	wave steepness	[-]
H	wave height	[m]
L	wave length	[m]

Usually, the wave steepness is defined on deep water, denoted as s_0. For the first series of tests, four different wave steepnesses were used: s_0=0.030, 0.044, 0.058 and 0.086. It turned out in after a few tests, that an deep water wave height $H_{m0,0}$ of around 12cm gave the best results. In this, H_{m0} indicates the wave height as calculated from the spectrum:

$$H_{m0} = 4\sqrt{m_0}$$

m_0 being the zero-th moment of the spectrum:

$$m_0 = \int_0^\infty E(f)df$$

The value of $H_{m0,0}$ was altered according to the linear wave theory to a value needed at the wave board: $H_{m0,b}$, the index b indicating the wave board. Using the already defined wave steepness, the corresponding wave period can be calculated. This period is to be used as the peak period, T_p. Details on the exact values used in the experiments can be found in Appendix A.

As will be explained later, the values of $H_{m0,0}$ had partially to be adapted for the second series of measurements. As a result, the values for s_0 don't correspond exactly to their counterparts from the first series. Also these values are printed in Appendix A.

2.4.3 Water level

The water level at the wave board, h_b, was in most experiments 66cm. As the toe of the breakwater was at 50cm above the reference level, the water level at the toe, h_t, was 16cm. Only in a few tests with Xblocs in the first series, the water level was raised, up to 74cm above reference level. These tests are indicated in Appendix A.

2.5 MEASURING PROCEDURE

The measurements were split in two phases: one series of measurements on the mild foreshore (1:30) and one series on the steep foreshore (1:8).

2.5.1 Mild foreshore

On the mild slope, first a series of preparatory measurements were executed, using the riprap armour. These measurements were done to find the correct wave height for the rest of the measurements, the correct repair method for the breakwater and to test for the effect of gradually increasing wave height.

After this, the principal measurements were done, also on riprap. The breakwater was tested for four different wave steepnesses, all with almost the same (theoretical) deep water wave height. Most experiments were executed with 1000 waves; however, a few were done with 2000 waves. All these tests started immediately with the target wave height, so no gradual build up of wave height was performed any more.

Finally in this series, some tests were performed with an Xbloc-armour layer. These were done with gradually increasing wave heights and for two different wave steepnesses. In this part of the experiments, the water level was increased during some tests, in order to be able to create higher waves at the toe of the breakwater.

2.5.2 Steep foreshore

On the steep foreshore, first, the experiments on the riprap armour layer were repeated. Most of these tests were done twice. First, the same spectra were used at the *wave board* as in the first series of the experiments. After that, in a few tests, lower waves were used with the wave steepness remaining constant, thus creating a longer wave period. As will be described in the next chapter, this yielded unsatisfactory measurements, so after that, the wave period was kept constant and the wave height was lowered so as to create the same spectra (or an as good fit as possible) at the *toe of the breakwater* as in the first series of experiments.

At the end, the Xblocs were tested again, with increasing wave heights and two different steepnesses, but not any more with increased wave height.

The details of all wave periods, steepnesses, heights, water levels and test durations can be found in Appendix A.

2.6 RECORDING

Of course, during all tests, the wave heights were recorded at the different gauges in order to reconstruct the spectra later on.

For determining the damage levels after waving, all stones, that weren't in their original part of the profile (coloured bands) anymore, were counted. This causes a problem if comparing these results with the Van der Meer-formulae. Van der Meer defines the damage level as the area of the erosion in the cross section of the breakwater, divided by the square of the stone size:

$$S = \frac{A_e}{d_{n50}^2}$$

With:

S	damage level	[-]
A_e	Erosion area	[m^2]

Figure 2-7: Definition of damage (schematised cross-section of an armour)

In Figure 2-7, a schematised part of a cross-section of a breakwater is drawn. On the left hand side, the layer is still intact. On the right hand side, stones are removed; the layer is damaged.

If a stone is removed from its original band, but remains in the erosion area (i.e. the lower yellow stone), it is counted as damage if just the stones are counted. However, according to the definition by Van der Meer, which uses the erosion area in the cross-section, this is no damage. As a result, the "counting stones"-method will structurally overestimate the damage level and the Van der Meer-formulae cannot be applied easily. This is not a problem within this experiment, but may become important if it tried to compare the results to the Van der Meer-formulae.

Finally, recording took place by making many pictures and some films during the experiments.

The parameters of each test, including the wave program that was used, can be found in Appendix A and B.

3 PERFORMED TESTS AND RESULTS

This chapter describes the measurements and their results. The complete results of all tests can be found in Appendix B. The reader is also referred to that Appendix if test numbers are mentioned.

3.1 MEASUREMENTS ON THE MILD FORESHORE

In the beginning, all tests on the mild foreshore, i.e. the foreshore with the 1:30 inclination, were performed.

3.1.1 Introductory measurements

The very first series of tests (T00001-T00160) were done in order to find the correct configuration for the breakwater and to find the best wave height. (The complete results can be found in Appendix B, the wave programmes in Appendix A.) Tests T00001-05 were a series of very first tests of the breakwater. Tests T00010-26 were tests with increasing wave heights each test, with an average wave steepness $s_0 \approx 0.057$ to 0.059.

3.1.1.1 Toe structure

Special attention in this phase was paid to the toe. This toe was right from the beginning constructed like a kind of gabion: it consisted of a wire mesh, filled with stones, see figure 4.1. This was done for constructing a toe as realistic as possible, as in this manner, it was permeable, but it could not fail following the loss of stones.

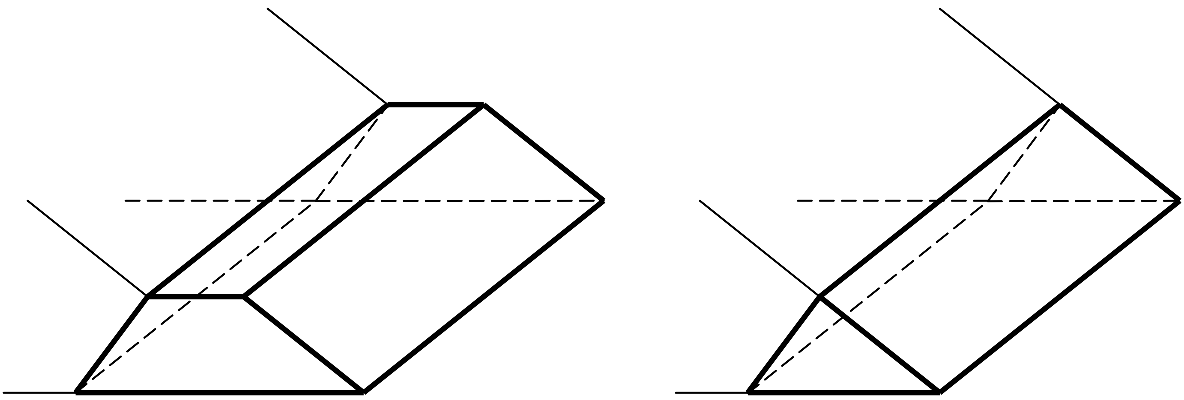


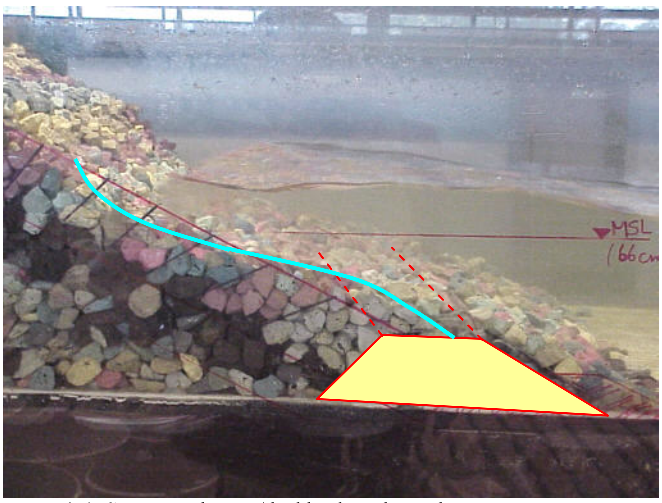
Figure 3-1: Trapezoidal vs. triangular cross-section of toe

At first, the toe had a trapezoidal cross-section (test numbers up to T00026). Although it stayed in place and seemed to support the armour well, there was one problem: the stones that were removed from the armour layer found a stable position on the top of the trapezium. The stones even showed some interlocking with the wire mesh. This way, the stones accumulated at this location instead of being transported away further downwards. To solve this problem, from test number T00100 onwards, the cross-section of the toe was altered to a triangular shape, with the front staying in line with the front of the breakwater. By doing so, the stones settled more in front of the breakwater and accumulation of stones on the toe did not take place any more. This can be seen by comparing for example the test numbers T00014/24 to T00104/14/24 (see Table 3-1). In all these tests, the same wave program was used and the wave history before was the same as well.

Type of toe	Test number	N_S [-]
Trapezoidal cross-section	T00014	902
	T00024	839
Triangular cross-section	T00104	1581
	T00114	1480
	T00124	1617

Table 3-1: Damage levels with different toes

The table shows the damage levels, in which N_S is the number of stones removed from the profile. This clearly demonstrates that the trapezoidal toe did reinforce the armour too much.



Picture 3-1: Stones on the toe (the blue line shows the accretion-erosion-area)

3.1.1.2 Repair strategy

Also, two different ways of repairing the breakwater were evaluated. (Test numbers T00150/52 and T01010-12, see Table 3-2.) One way was, more of less, by simply placing the stones back in the armour layer (test numbers T00150 and 52). The packing rate of the stones was improved by gently drilling them with a wooden bar. The other method implied taking away the complete armour layer and rebuilding it (tests T01010-12). Although the first method was by far the faster method, it showed a significant difference in performance with the second: for the tested circumstances, the damage was about 17% larger.

Repair strategy	Test number	N_S [-]
Repair damage only	T00150	1141
	T00152	1164
Rebuild armour	T01010	948
	T01011	990
	T01012	1011

Table 3-2: Difference in damage with different repair strategies

The table shows a clear difference, which can be explained by the fact that the armour layer after repair does not work as one entire layer any more. Apparently, some kind of internal reinforced plane remains present, with which the stones that are brought back to the profile do not interlock. As a result, the stones above this reinforced layer are removed easier, giving higher damage levels.

The difference the methods showed was considered far too large. As it is necessary to know the starting point of the measurement, the first method of repair was rejected for further tests. As a consequence, the measurements took much more time and, given the available time, fewer measurements could be executed.

3.1.1.3 Wave height

The most important issue in this phase was finding the most suitable wave height for the tests to come. This was done by performing a series of tests and increasing the wave height for each test. The most suitable wave height was found to be the wave heights $H_{m0,b}$ (i.e. at the wave board) between 11.4cm and 12.1cm. (For more details, see Appendix A.)

By means of these tests, also the influence of the wave height on the damage could be assessed. The damage assessment was done by counting the stones that were completely removed from the breakwater and making a good estimation of the number of stones that were still on the breakwater, but outside their original colour band. These stones could not be counted exactly, because removing them from the profile in order to count them exactly was undesired as they are still part of the strength of the armour. According to the Van der Meer- and Van Gent-formulae, the damage should grow with the 5^{th} power of the wave height. Both formulae have a term $H \propto S^{0.2}$ or reversely $S \propto H^5$.

This testing of the influence of the wave height was done in tests T00100-124. These were three series of five tests, with equal wave conditions. After each test, the wave height was increased a little, until the black filter layer became visible. The result of one series is plotted in Figure 3-2. The diamonds indicate the measured data.

The line in the figure uses the damage level at the lowest wave height as the reference and subsequent values are calculated by:

$$N_S = N_{S,ref}\left(\frac{H_{m0,t}}{H_{m0,t,ref}}\right)^5$$

in which

N_S damage level

$N_{S,ref}$ reference damage level (i.e. the damage level measured at the lowest wave height)

$H_{m0,t}$ wave height at the toe

$H_{m0,t,ref}$ reference wave height (i.e. the lowest wave height)

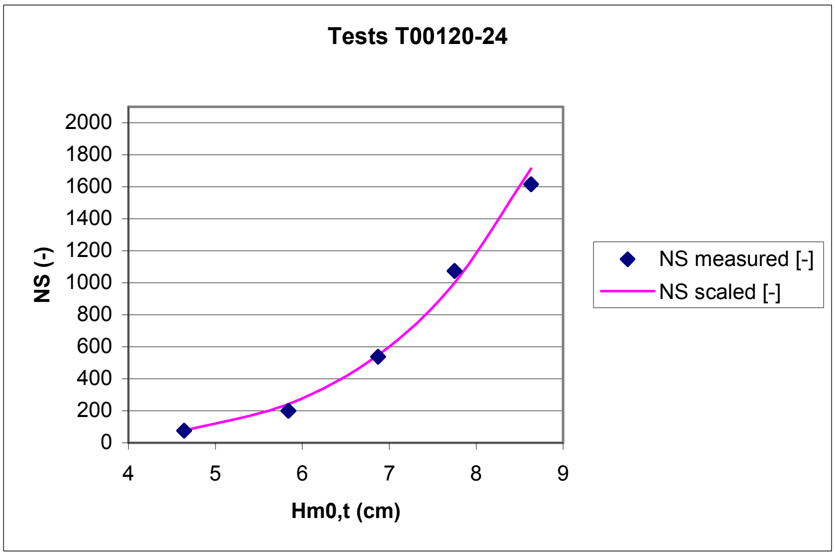


Figure 3-2: Test of wave height vs. damage

These diagrams support the theory of Van der Meer, which states that the damage done to the breakwater is proportional to the fifth power of the incident wave height.

3.1.2 First series of measurements: mild foreshore

The first series of measurements was performed on the 1:30-slope. The complete results can be found in Appendix B, at test numbers T01000-T01934, where the tests T010xx were executed with riprap and the tests T019xx with Xblocs.

3.1.2.1 Riprap Armour

In the tests on riprap, the target wave height was immediately used, so no gradual build-up of wave heights was performed any more. This was done in order to be able to take out all stones that were removed from their original colour band as to count them, as well as to save some time on the tests by reducing the number of them. As a result, the initial settlement of stones that usually takes place in calm wave conditions, did not take place and so a higher damage level may be expected than from the Van der Meer-formulae. This however, does not have to be a problem: the results can anyway not be compared exactly due to the difference in definition of damage. Furthermore, the purpose of the tests is to compare the results for mild and steep foreshores, not to perform an exact validation of the Van der Meer-formulae. (See 4.5.)

The tests with the riprap armour showed a strong influence of the wave steepness, so of the wave period, as can be expected from the Van der Meer-formulae. Here, the Van Gent-formula deviates, as it does not take the wave period nor the wave steepness into account. With the lowest steepnesses, i.e.

the longest waves, the damage was significantly larger that with the high wave steepnesses, i.e. the shortest waves. Almost all tests were executed thrice and the results showed a remarkable precision: in each sub-series, the difference between the smallest and the largest damage was just 6 to 7%. Just in one test (T01022) showed a somewhat larger difference (10%), but this was the result of the accidental selection of a slightly different wave program and the result from this test was therefore not used in the further analysis.

All the tests were done using N=1000 waves. This number of waves is used often in breakwater research. It is assumed that after 1000 waves, the highest waves, which account for the largest part of the damage, will have occurred a few times.

Further, for one wave steepness (s_0=0.058, test numbers T01040-01042), the influence of the length of the wave program, i.e. the number of waves, was tested. The number of waves was increased to N=2000. From the Van der Meer-formulae, it is expected that the damage will increase proportionally with $\sqrt{N}$. On average, in the tests with 1000 waves, the number of stones removed from the profile, N_S, was 670. If this would be extrapolated to 2000 waves, this number would be 948. The tests with 2000 waves, this average became 1031.

N	Test number	N_S [-]
1000	T01020	653
	T01021	664
	T01023	694
2000	T01040	1072
	T01041	1056
	T01042	1005

Table 3-3: Damage vs. number of waves

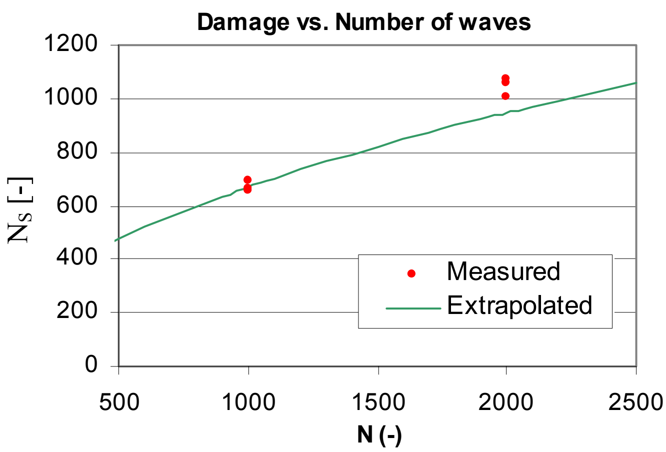


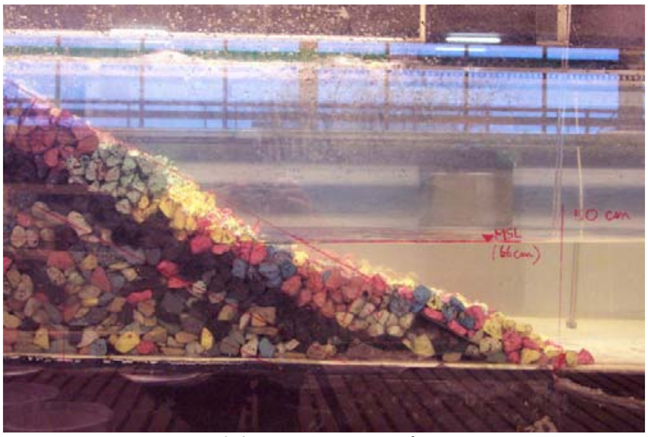
Figure 3-3: Damage vs. number of waves

As can be seen table 4.3 and also from figure 4.3, there is a slight deviation from the extrapolated $\sqrt{N}$ -curve. This slight difference may be explained again from the fact that the definition of damage Van der Meer uses is different from the definition of damage used in this report and by the non-uniform density of the stones. Also, like most breakwater design formulae, the Van der Meer-formulae are a result from empirical research and curve-fitting, which implies that some statistical deviations are possible, which may also explain a part of the difference.

3.1.2.2 Resumé of measurements with riprap

The majority of the measurements in this series were the measurements that would have to be compared to the measurements on the steep foreshore. They were executed for four different wave steepnesses: s_0=0.030, 0.044, 0.058 and 0.086. They were all done on a fixed water level of 66cm above the reference level, i.e. 16cm above the toe. The wave height at the board varied slightly, between $H_{m0,b}$=11.4cm and $H_{m0,b}$=12.1cm.

Each test was performed thrice, in order to check the statistic reliability and repeatability of the measurements.



Picture 3-2: Riprap armour after testing

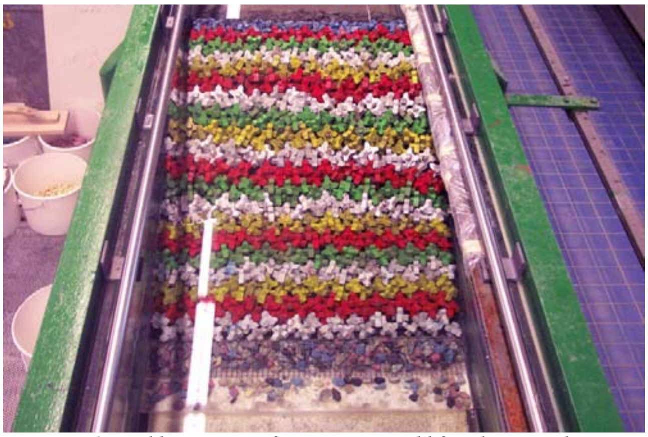
The results from these measurements can be found in Appendix B and will be evaluated further in the next chapter.

3.1.2.3 Xbloc armour

All but one tests on the Xblocs were also performed with N=1000 waves, but in this case, a gradual build-up of wave height was performed, as the initial settlement of armour units is of paramount importance for the strength of an armour layer with interlocking armour units. These tests were done only for the two lowest wave steepnesses: s_0=0.030 and 0.044 (See test numbers T01900-T01934).

Picture 3-3: Xbloc armour layer ready for testing

Initially, the water depth was the same as in the tests with the riprap armour: 66cm above reference level. However, this way it appeared that it was not possible to remove one single Xbloc from the armour layer. As the wave board produced higher waves, the waves broke at locations further away from the breakwater, so the waves became depth-limited. Therefore, the water depth was increased in two steps (till 72 and 74cm above reference level) in order to have higher waves at the breakwater. But even at the highest water level, with waves occasionally spilling from the flume, it was not possible to inflict any damage to the Xbloc-layer. Only minor rocking took place in higher waves.



Picture 3-4: Xbloc armour after testing on mild foreshore: no damage

Concluding, it can be said, that in these circumstances, it was not possible damage the Xbloc breakwater. Partially, this may be explained from the fact that the 1:2-breakwater face is milder than usual for interlocking units like Xbloc. They are usually applied on an 1:1.5 or 3:4-slope. The increased gravity component may give them a higher stability. However, another effect, which may cancel the advantage of an increased gravity component, comes into play here, which will be treated further in paragraph 4.3.

3.2 MEASUREMENTS ON THE STEEP FORESHORE

This series of tests was performed on an 1:8-foreshore. The complete results can be found again in Appendix B, test numbers T02000-T02993

3.2.1 Introductory measurements

On the steep foreshore, no more introductory measurements were performed any more. All the necessary parameters were already determined, based on the mild foreshore and the measurements on the steep foreshore needed to have an as good as possible agreement, so testing e.g. another type of toe wasn't necessary any more.

3.2.2 Second series of measurements: steep foreshore

The second series of measurements was performed on the 1:8-slope. The compelte results can be found in Appendix B, at test numbers T02000-T02944, where the tests T020xx and T021xx were executed with riprap and the tests T029xx with Xblocs.

3.2.2.1 Riprap

In this series, again the tests on a riprap armour were done first. In a first step, the same wave programs as in the first series were used (test numbers T02000-30). This resulted in higher damage levels compared to the mild foreshore, as can be seen in table 4.4. In the last column of this table, the differences in terms of percents are given. The results are not really surprising, as it was known already, from e.g. the Goda-formulae, that waves can grow higher on steep foreshores before breaking, so the average wave height (whether be it H_{m0} or H_s) grows as well and with the wave height, the damage level rises.

Wave steepness s_0	Mild foreshore Test number	N_S	Steep foreshore Test number	N_S	Difference (avg. %)
0.030	1000 1001 1002	1482 1417 1386	2000	1786	+25.0
0.044	1010 1011 1012	948 990 1011	2010 2011	1455 1394	+44.9
0.058	1020 1021 1023	653 664 694	2020	1102	+64.4
0.086	1030 1031 1032	284 295 277	2030	632	+121.5

Table 3-4: Comparison between steep and mild foreshores with equal waves at the wave board

The larger wave height can also be shown by the spectra. In Figure 3-4, the spectra for the tests at s_0=0.044 are shown. One can clearly see the higher values of the spectral energy density for the steep foreshore cases.

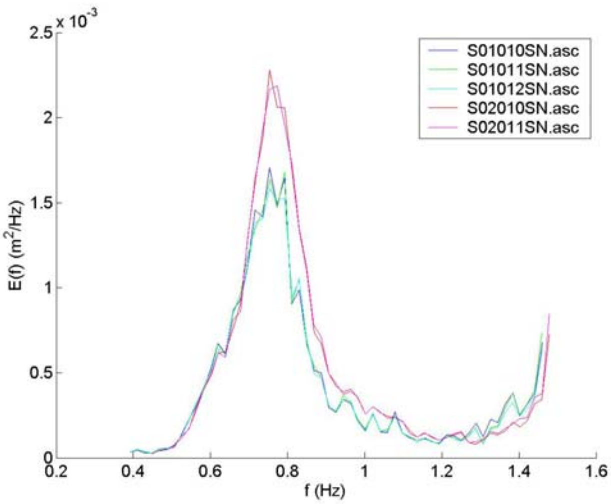


Figure 3-4: (Decomposed) spectra at the toe with s_0=0.044

In the second step, a somewhat lower wave height at the wave board was used, in order to try to get the same H_{m0}-values at the toe of the breakwater (see test T02050/51). Although this also reduced the wave height at the toe of the breakwater, it was still not considered a good result. The problem was that the wave steepness was kept constant, so by selecting a lower wave height, a shorter wave period was automatically selected. As a result, the spectra at the toe were not the same, see Figure 3-5, as the peak frequencies shifted to a lower value, so it was still not possible to fully compare the results with their mild foreshore counterparts.

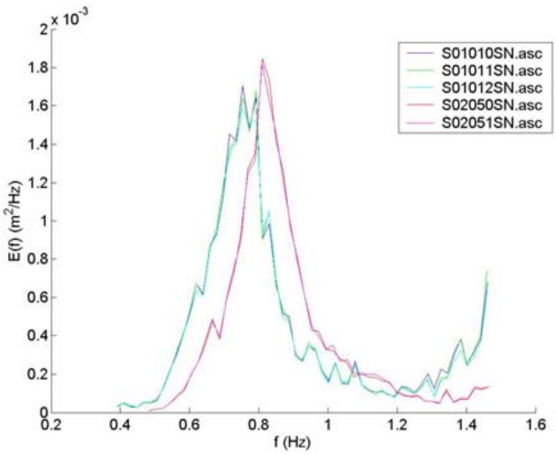


Figure 3-5: Spectra with adjusted wave program

In a third step, the wave peak period was kept constant and the wave height was reduced. (Test numbers T02060 and onwards). Although this alters the deep-water wave steepness, the aim was to have the same spectra at the toe of the breakwater. After some trying, the match between the resulting spectra at the toe was very good, as was the m_0-value of the spectra. So, having the same spectra at the toe, it might be expected on the basis of the existing armour design formulae that the damage levels would be more or less the same. The result however was amazing: the damage levels were up to more than 30% higher, see Table 3-5.

Wave period T_p [s]	Mild foreshore Test number	N_S	Steep foreshore Test number	N_S	Difference (avg. %)
1.60	1000 1001 1002	1482 1417 1386	2090 2091	1390 1466	±0.0
1.31	1010 1011 1012	948 990 1011	2070 2071	1082 1106	+11.3
1.13	1020 1021 1023	653 664 694	2080 2081	915 890	+34.6
0.92	1030 1031 1032	284 295 277	2110 2111	395 359	+32.1

Table 3-5: Comparison between steep and mild foreshores with equal wave conditions at the toe

For the longest wave period, the difference between the damage on the mild and the steep foreshore is almost zero. Also with the same conditions at the wave board, the difference in this case was the smallest. This may have been caused by the fact that the slope is too short in comparison to the wave length and the waves don't have enough time to adapt well: the wave length was, based on T_p, 3.36m at the deepest part of the flume, while the slope was just 4m long.

For the shorter wave periods, the difference is more pronounced. For the shortest waves, the differences were the largest, but the spectra for this situation was rather ill-developed (see Appendix F), so for a major part of the analysis in the next chapter, only the two intermediate wave periods were used. An example is shows again in Figure 3-6. Although the spectra are not exactly the same, the values are quite close to each other.

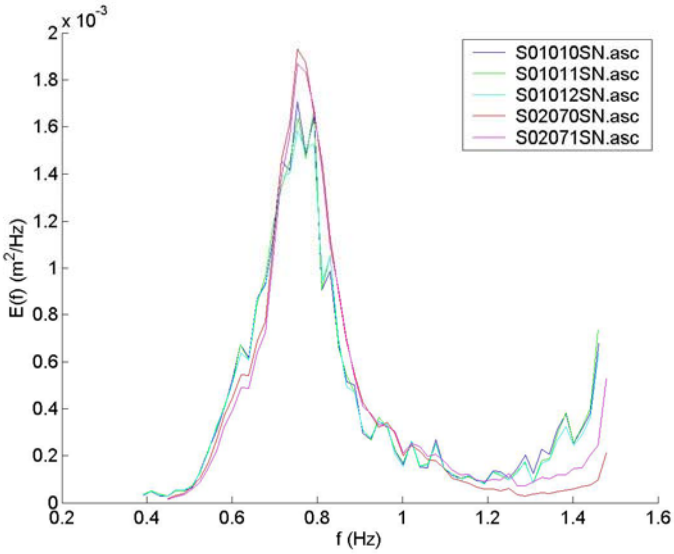


Figure 3-6: (Decomposed) spectra with adjusted waves at Tp=1.31s

As can be seen from the plots of the spectra, the spectra were almost the same for the steep and the mild foreshore. However, as the decomposition method of ZELT and Skjelbreia is valid only for (more or less) linear waves, the results from the decomposition may be unreliable. Non-linear decomposition methods do not exist (yet). So for another indication for the similarity of the wave conditions, the raw results from the wave height meters were compared as well. This means that the wave registration shows a mixed signal of the incoming and reflected waves.

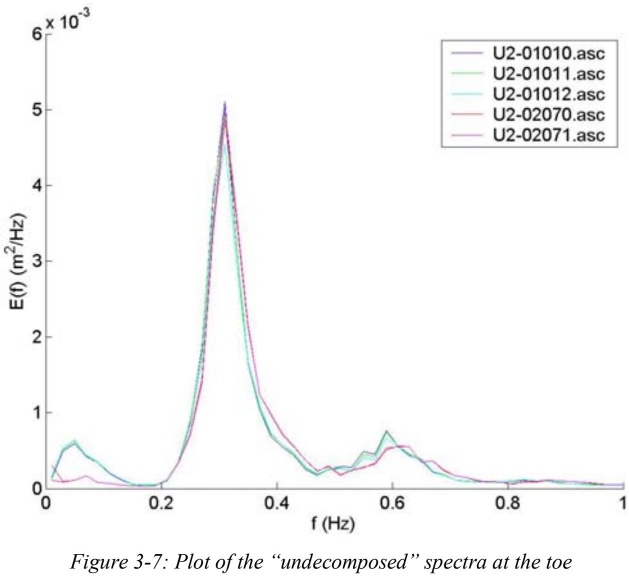


Figure 3-7: Plot of the "undecomposed" spectra at the toe

The plot of the "raw" spectra in Figure 3-7 shows a good agreement between the mild and the steep foreshore case. KLOPMAN and VAN DER MEER (1999) pointed out, that although waves are still propagating, a node-antinode pattern will develop just in front of a breakwater. This means that the raw wave heights vary significantly, according to the distance of the measuring location from the breakwater. The wave height meters were in exactly the same position during all tests, with one wave height meter just in front of the toe. This means that they will be severely influenced by this pattern, but as they are influenced the same way, the signals should be able to be compared. This comparison yielded a good agreement between the mild and the steep foreshore cases, so one can conclude on basis of the spectra, that more or less the wave conditions at the toe were found.

3.2.2.2 Xbloc armour

The last measurements were the measurements with Xbloc armour units on the breakwater. Al these tests were executed with a water level at 66cm above the reference level, i.e. 16cm above the toe.

In these tests in became clear from visual observation, that the load on the armour is higher in the case of higher waves. The depth limitation of the waves was less present here, so higher waves could get to the breakwater. The rocking of the armour was much more present compared to the mild foreshore case and in some tests; it was even possible to damage the armour layer. Sometimes, the damage started developing at the boundary of the flume, which may be an artificial effect. It is important to notice, that it was possible at all to damage the breakwater, as the mild foreshore tests didn't succeed in that.

For an impression of the damage development, see Picture 3-5.

Picture 3-5: Damage to the Xbloc layers

Besides the measured data, it was found that the sound of the impact of the waves on the breakwater was different. Although this is a subjective judgement, this also indicates, that the impact of the waves is different.

In these tests, it could also be seen, that the wave may hit the toe severely, as can be seen in Picture 3-6: High plunging waves attacking the toe below. This behaviour hasn't been observed in the mild foreshore tests. This indicates, that it was a good idea to construct the toe as a heavy gabion, but it also indicates the need to do further investigation in the future to the stability of the toe, focussed on steep foreshores.

Picture 3-6: High plunging waves attacking the toe

A further analysis of the wave data will be done in the next chapter.

4 ANALYSIS OF THE EXPERIMENTS

In the previous chapter, the results of the measurements have been treated. In this chapter, it will be tried to explain the differences with a more in-depth analysis. This analysis will be done for one case, the analysis of the other cases is similar and can be found in Appendix F.

4.1 WAVE DESCRIPTION

For the description of the waves, it is a usual procedure to establish a spectrum of the waves that occurred during the tests. This spectrum has been established in all tests at different locations and in different ways.

As a breakwater is a (weakly) reflective structure, it is necessary to separate the incoming waves from the reflected waves. This process is called decomposition and can be done if there are two or more wave gauges close to each other. As in this research two groups of two wave gauges were used, it is possible to calculate the (incoming and reflected) spectra at two different places: one set was at "deep" water, i.e. just in front of the sloping seabed, the other set was just in front of the breakwater.

For the decomposition, the method of ZELT and SKJELBREIA (1992) was used. This method, like any other method for wave composition, may however give unreliable results if the waves are strongly non-linear or breaking. As this was the case at the set of gauges close to the toe, it was decided that another method to estimate the wave conditions was necessary. This was done by establishing a wave spectrum of the data of the wave gauge that was the closest to the toe of the breakwater. As decomposition cannot be done here, this has been called the undecomposed spectrum, in order to clearly indicate that the decomposition has not been performed.

As the undecomposed spectrum does not distinguish between the incoming and the reflected waves, it is not possible to use these results for in-depth calculations. They can only be used in order to compare whether the sea conditions at the toe were more or less the same in front of the toe. It is supposed that if these undecomposed spectra were more or less the same, the incoming wave conditions were also more or less the same. By applying them this way, these spectra proved to be a useful tool.

4.2 RIP-RAP: IN-DEPTH ANALYSIS

All tests on rip-rap have been performed for four wave periods. They were T_p=1.60s, T_p=1.31s, T_p=1.13s and T_p=0.92s. The case of the waves at T_p=1.31s will be taken here as an example for the evaluation. The results for all other cases can be found in Appendix F.

Two types of comparison have been made. In the first comparison, the wave created conditions at the wave board were the same for the tests with the mild and the tests with the steep foreshore. The resulting spectra as well as the damage levels of the armour layers have been compared.

After the tests on the steep foreshore had been completed, it has been attempted to adjust the wave height at the wave board in such a way, that the wave conditions at the toe were (more or less) the same. This is the second comparison and also for these cases, the resulting spectra and damage levels of the armour layer have been compared.

4.2.1 Experiments with similar wave conditions at the wave board

In these tests, the wave conditions that were created at the wave board were the same in the situations with a mild and a steep foreshore. These were irregular waves, with a standard Jonswap-spectrum. In detail, the waves had the following properties:

Parameter	Value	Meaning
T_p	1.31s	spectral peak period
H_b	0.110m	wave height at the wave board
H_0	0.118m	(theoretical) deep water wave height
s_0	0.044	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

Table 4-1: Wave properties

The indication “theoretical” means that these values have been calculated back to genuine deep-water wave conditions.

In Table 4-2, the results of the tests are given. The damage is in terms of N_{od}, i.e. the normalised number of stones removed from their original position, defined as:

$$N_{od} = \frac{N_S d_{n50}}{B}$$

in which:

N_S	number of stones removed from their original position	[-]
d_{n50}	nominal stone diameter	[m]
B	width of the measured section (i.e. width of the flume)	[m]

The test numbers are clustered: numbers T010xx were on the mild foreshore, numbers T020xx were on the steep foreshore. Furthermore, the incoming wave heights at the toe as the were found from the decomposition are indicated for each test (denoted as $H_{m0,t}$, i.e. the H_{m0}-value at the toe). Also, the values $H_{m0,u}$ are given. These are the H_{m0}-values as they were calculated from the undecomposed spectra (see previous paragraph). Finally, the percentual difference between the average damage levels of the tests on the mild and steep foreshores are given.

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01010	18.6	8.58	9.10	T02010	28.6	9.31	10.01	+44.9
T01011	19.4	8.57	9.10	T02011	27.4	9.35	10.25	
T01012	19.8	8.46	8.89					

Table 4-2: Comparison for mild and steep foreshores, equal wave conditions at the wave board

Some things can clearly be seen from this table:

- The damage levels are significantly larger at the steep foreshore
- The incoming wave heights (calculated from decomposition) are higher at the steep foreshore
- The higher waves seem to be confirmed by the undecomposed data.

In order to compare these data a little bit further, in Figure 4-1 the spectra of the incoming waves, as calculated in the decomposition, are given. In the legend, the numbers correspond with the test numbers above. The S in front of them indicates "spectrum", the S behind them indicates shallow water (i.e. at the toe). (The N and the .asc can be neglected.) Figure 4-2 indicates the same but now for the undecomposed wave data at the toe (the U2 in front of the test number indicates this).

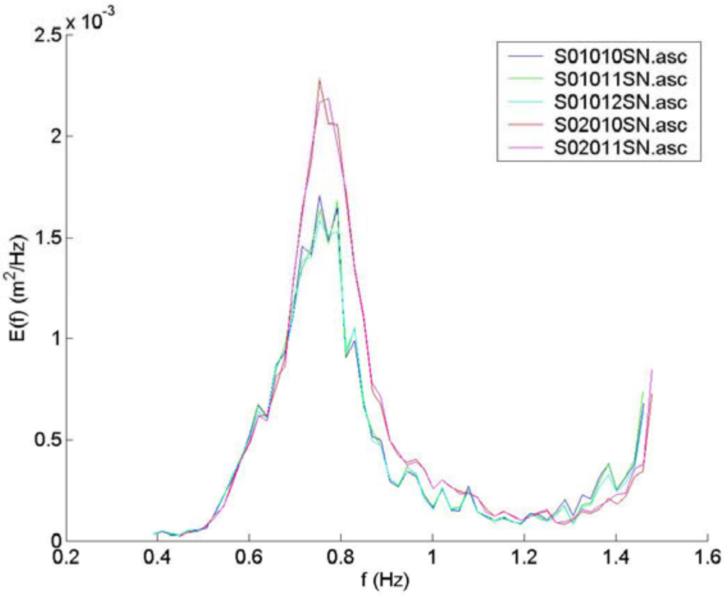


Figure 4-1: Incoming wave spectra at the toe

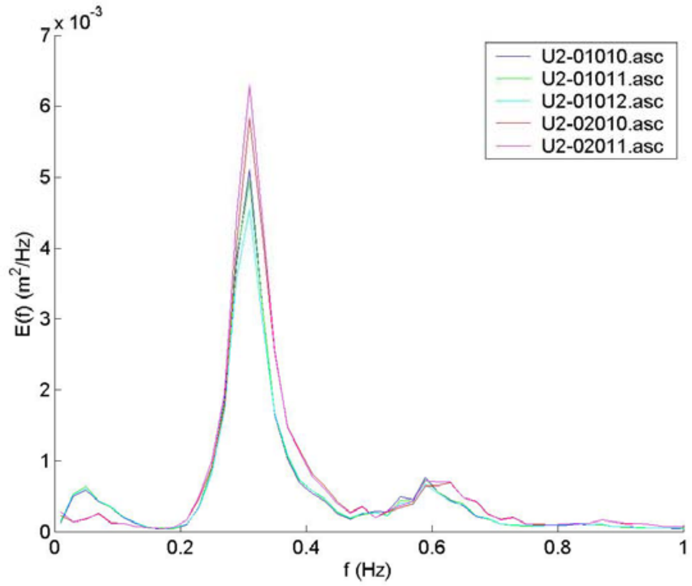


Figure 4-2: Undecomposed wave spectra at the toe

It can clearly be seen from both figures, that in the steep foreshore case, more energy has been preserved in comparison with the mild foreshore case. This is in line with the higher wave heights that have been found in the calculation.

It has to be noted that in the right tail of Figure 4-1, the spectral energy density is rising again. This is not a physical effect, but is introduced artificially by the decomposition method and it should be neglected.

As has been said before, the rest of the tests have been analysed the same way. Their results are similar (see Appendix F). Conclusion for this part of the experiment:

At a steep foreshore, damage levels may be significantly higher due to greater incoming wave heights at the breakwater by equal offshore wave conditions.

4.2.2 Experiments with similar wave heights at the toe

Basically these experiments are the same as the experiments in the same paragraph. The mild foreshore experiments are the same as in the previous paragraph, but now it has been tried to adjust the wave heights at the wave board in the steep foreshore case in such a way, that the wave conditions at the toe were the same as in the mild foreshore case. This has been accomplished by keeping the peak period the same, but lowering the wave height. This resulted in the following wave program for the steep foreshore case:

Parameter	Value	Meaning
T_p	1.31s	spectral peak period
H_b	0.096m	wave height at the wave board
H_0	0.104m	(theoretical) deep water wave height
s_0	0.039	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

Table 4-3: Adjusted wave program at the steep foreshore

In Table 4-4, the experiments are compared again the same way as in the previous paragraph. The numbering of the experiments may seem a bit out of line with the previous paragraph, but the T02070-series (steep foreshore) belongs to the T01010-series (mild foreshore).

Mild foreshore				Steep foreshore				Difference Average N_{od}
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01010	18.6	8.58	9.10	T02070	21.2	8.31	10.01	+11.3
T01011	19.4	8.57	9.10	T02071	21.7	8.43	10.25	
T01012	19.8	8.46	8.89					

Table 4-4: Comparison for mild and steep foreshores, equal wave conditions at the toe

In this table, an important effect can be seen:

- The incoming wave heights at the toe were about the same for the mild and steep foreshore
- Although these wave conditions were about the same, the damage level on the steep foreshore was an average more that 11% higher

Also for these tests, the spectra have been compared again, in order to gain better insight in the wave conditions at the toe. In Figure 4-3 and Figure 4-4 below, the spectra are plotted, the same way as in the previous paragraph. In Figure 4-3, the rising tail at the right side of the spectrum should be neglected again, as this is an artificial effect form the decomposition method used.

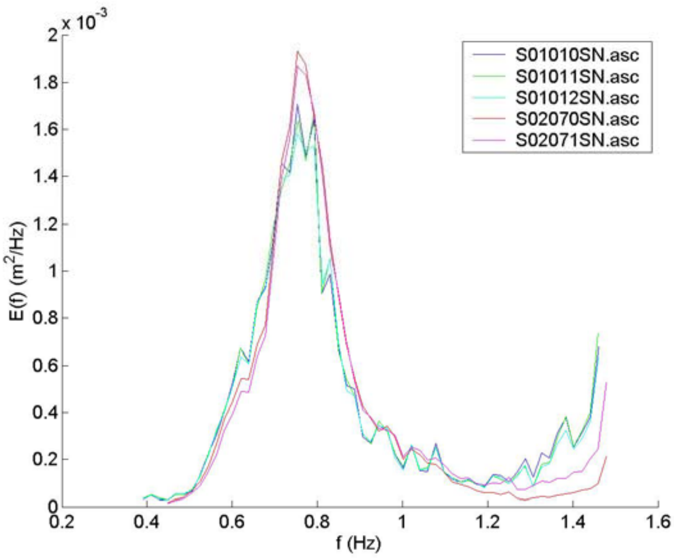


Figure 4-3: Incoming wave spectra at the toe

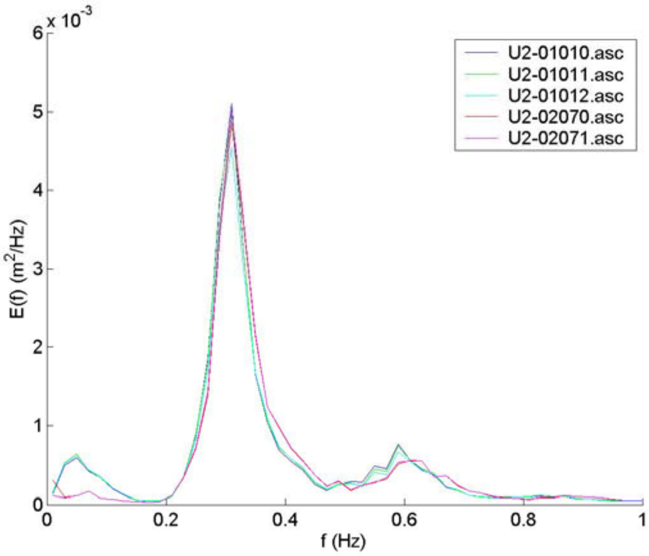


Figure 4-4: Undecomposed wave spectra at the toe

In Figure 4-3 it can be seen that the incoming spectra are almost the same for the waves on the steep foreshore and on the mild foreshore. This is confirmed by Figure 4-4: in the central part of the spectra, hardly any difference can be seen any more between the different lines. So we may assume that the wave conditions at the toe were really about the same for the steep and mild foreshore cases.

In Appendix F, the same analysis has been performed again for the other sets of tests. The results were similar for most cases. Only for the case with the longest waves, there was no difference in the comparison with similar waves at the toe. This may be explained by that the waves in these tests were relatively long in comparison to the horizontal length of the slope. Therefore, the deformation of the waves in that particular series may not have taken place good enough.

This leads to a very important conclusion:

Even if the waves at the toe of the structure are about the same, the damage level in case of a breakwater on a steep foreshore is higher compared to the damage level of a breakwater on a mild foreshore.

At the same time, this conclusion means that the wave spectrum alone, or a wave height derived from it, is not good enough a load parameter. Apparently, there is some other property of the waves that influences the stability of the armour layer.

4.2.3 Other wave descriptors

So, if the spectra don't give enough information, what does? It is important to realise, that the spectrum is more or less a summary of the waves that passed through a measuring location. In the summarising process, information is taken out and lost. In the case of the spectra, the wave heights and periods are calculated via a Fourier-transformation, but the information on, for example, phases, wave shapes and the internal water movement is thrown away. This also means that once a spectrum has been calculated, it is not possible to transform it back to the real sea state.

In order to find another descriptor for the waves, we would have to look at such parameters as wave shapes or phases. In this research, an additional analysis to the wave shapes, in particular the steepness of the wave front, has been performed, using some new descriptors and another wave height descriptor, an indicator for the peakedness of the waves, has been tried.

4.2.3.1 Steepness of the wave front

In the shoaling process, waves become more and more non-linear as the water depth decreases. Strictly speaking, the linear wave theory is not valid any more in these areas, but for most applications, the results are good enough to apply this theory somewhat outside its strict boundaries. However, in the case of a breakwater on a steep foreshore, the non-linearities may become more important. The wave troughs become long and shallow, the wave crests become higher and shorter. The quick rising of the water level as the wave tops approach may have influence on the water level. Therefore, a parameter must be found in order to describe this quick rising and if this parameter can be found, it may give an indication of the magnitude of non-linearity of the wave.

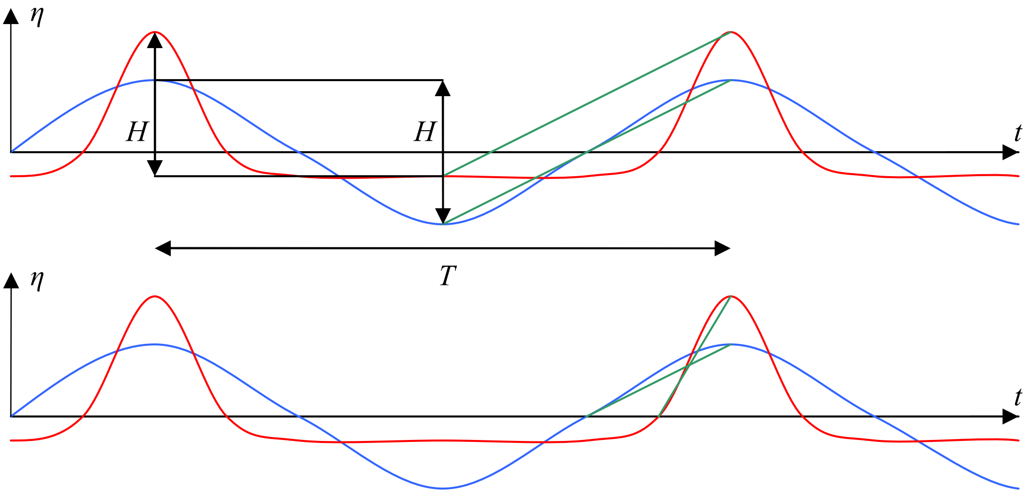


Figure 4-5: Principle of the wave front steepness

In figure 5.18 two fictive time registrations of waves are drawn. One of the waves has a more-or-less sine-shape, the other has short and high peaks and a long, shallow trough. In the upper picture it can be seen, that both waves have the same length and the same wave height. If one would try to evaluate a descriptor like $\frac{H}{T}$ (like the green lines drawn in the right half of the upper half of the figure), one would not find any difference between the waves, so this would not be a useful definition for such a descriptor.

In the lower half of the figure however, lines are drawn from the mean water level to the wave crests. Now, it can be seen that the inclination of the line is quite different. So, by “cutting off” the registration at the water level, a descriptor could be found.

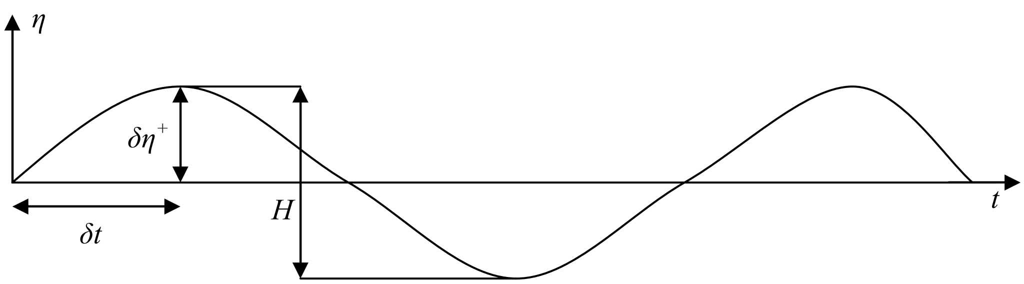


Figure 4-6: Definition of the wave front steepness above mean water level

In figure 5.19 above, the definitions of the descriptor are given.

In this picture, η denotes the instantaneous water level, $\delta\eta^+$ the elevation of the wave crest above the mean water level, δt the time lag between the moment of the zero-up-crossing and the moment of passage of the crest. H and t are, as usual, the wave height and the time.

The descriptor would thus have a mathematical shape like: $\frac{\delta\eta^+}{\delta t}$. This can be averaged over all waves. In order to shorten the writing, a new parameter is introduced here to denote this, so:

$$R_1 = \overline{\frac{\delta\eta^+}{\delta t}}$$

But by doing so, very low but steep waves would have relatively a very large influence on the descriptor. This can be solved by a weight averaging, using the wave height, so:

$$R_2 = \frac{\overline{H_i \frac{\delta\eta_i^+}{\delta t_i}}}{\overline{H_i}} = \frac{\sum_i \left(H_i \frac{\delta\eta_i^+}{\delta t_i} \right)}{\sum_i H_i}$$

If $R_1 > R_2$, this indicates the presence of relatively many steep, but low waves, whereas $R_1 < R_2$ indicates relatively more high and steep waves.

From linear wave theory, it is further more known, that the wave energy E is proportional to the square of the wave height: $E \sim H^2$. So, maybe it makes sense not to weigh the top steepness with the wave height, but with the square of the wave height, so:

$$R_3 = \frac{\overline{H_i^2 \frac{\delta\eta_i^+}{\delta t_i}}}{\overline{H_i^2}} = \frac{\sum_i \left(H_i^2 \frac{\delta\eta_i^+}{\delta t_i} \right)}{\sum_i H_i^2}$$

This introduces a real focussing of the descriptor on the highest waves, so on the waves with the highest energy.

Another possibility is to look at a type of root-mean-square value. This can be denoted as:

$$R_4 = \sqrt{\frac{\overline{H_i^2 \left(\frac{\delta\eta_i^+}{\delta t_i} \right)^2}}{\overline{H_i^2}}} = \sqrt{\frac{\sum_i \left(H_i^2 \left(\frac{\delta\eta_i^+}{\delta t_i} \right)^2 \right)}{\sum_i H_i^2}}$$

This value focuses on the waves that are both high and steep. This makes sense, as from the measurements observations it could be seen that the steep waves have far more impact than the mild ones.

It has to be noted that the values of R_1-R_4 are not dimensionless. They have the dimension of velocity [m/s]. They can be made dimensionless by dividing by the average wave celerity; this causes the velocity scale to disappear from the equation. In order to do this, a "frozen movement" like it is used in turbulence research is assumed (Taylor hypothesis). Thus, assuming this, something like an average angle of steepness of the wave front is defined. For shortness of notation, this can be denoted by:

$$Q_1 = \frac{R_1}{\bar{c}}$$

$$Q_2 = \frac{R_2}{\bar{c}}$$

$$Q_3 = \frac{R_3}{\bar{c}}$$

$$Q_4 = \frac{R_4}{\bar{c}}$$

4.2.3.2 Peakedness

DRAKE and CALANTONI (2001) used a special equation to find the skewness of accelerations in waves. A similar form is proposed here, applied to the wave heights, in order to find a description for the peakedness of the waves. This descriptor is defined as:

$$H_{spike} = \overline{H^3} \Big/ \overline{H^2}$$

Higher values of H_{spike} indicate that relatively more high peaks occur in the wave registrations. Note that also this form is not dimensionless: it has the dimension of length [m].

4.2.4 Riprap: application of the steepness descriptor to the wave data

The descriptor for the wave steepness has been applied to the decomposed data first. However, this didn't give usable results. This is clearly a consequence of the invalid application of the wave decomposition method. A close look at the wave data that were calculated by the decomposition revealed that these waves had a rather sinusoid shape. The visual observations contradict this. Therefore, this possibility has not been explored any further.

Once again, it has been tried to apply the descriptor to the undecomposed data of the wave height meter closest to the breakwater, as being representative for the wave load on the breakwater. The $R_{1..4}$- and $Q_{1..4}$-values for a the tests that were treated in the previous paragraphs are printed in Table 4-5 on the next page. The complete table for all tests can be found in Appendix F.

The application at the undecomposed data automatically means that the assumption of "frozen waves" can't possibly be true. Nevertheless, as will be shown, the results are interesting.

Testnr.	R_1	R_2	R_3	R_4	Q_1	Q_2	Q_3	Q_4
	[m/s]	[m/s]	[m/s]	[m/s]	[-]	[-]	[-]	[-]
T01010	0.214	0.274	0.323	0.372	0.183	0.234	0.275	0.316
T01011	0.215	0.274	0.323	0.373	0.183	0.233	0.275	0.317
T01012	0.214	0.271	0.318	0.366	0.182	0.231	0.271	0.312
T02010	0.294	0.419	0.529	0.650	0.251	0.357	0.451	0.554
T02011	0.296	0.423	0.536	0.657	0.252	0.360	0.456	0.560
T02070	0.240	0.343	0.442	0.559	0.205	0.292	0.377	0.476
T02071	0.235	0.339	0.436	0.540	0.200	0.288	0.372	0.460

Table 4-5: Comparison of the wave "steepnesses"

The Q_2-and Q_3-values show an interesting pattern. Therefore, in Figure 4-7 they are printed for all tests, against their peak periods.

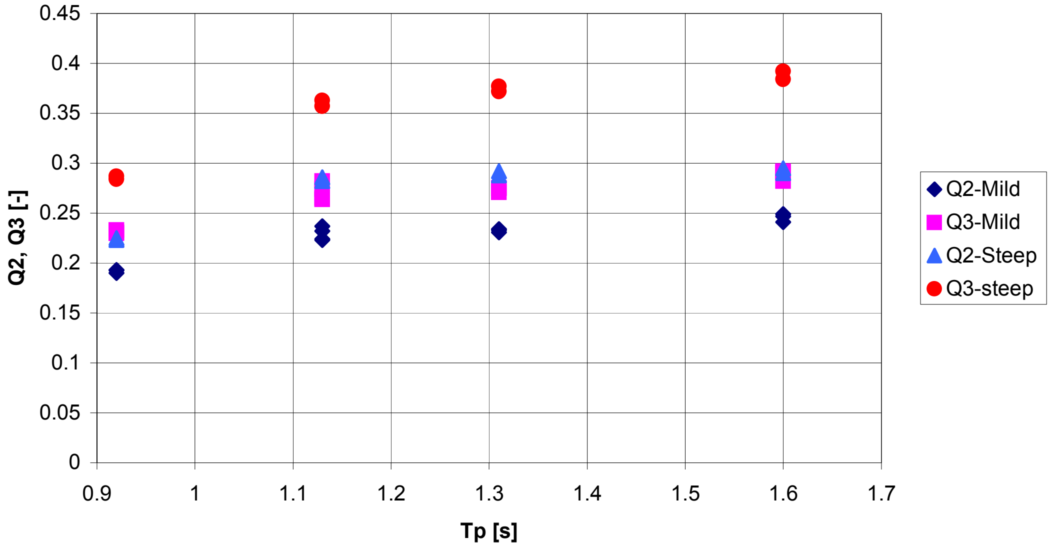


Figure 4-7: Q2- and Q3-values

It can be seen in this figure, that these values are more or less lying on a horizontal line. For the mild foreshore, the Q_2-values are all in a range between 0.223 and 0.249. For the steep foreshore, they are all between 0.282 and 0.295. The Q_3-values show a similar pattern: for the steep foreshore, they are all in a range between 0.264 and 0.292 and for the steep foreshore, they are between 0.357 and 0.392. Only at the shortest wave period, the values diverge somewhat. This may be caused by the fact that the wave program was rather short and by the unusual high steepness of these waves. Yet, the trend remains the same as for the other wave periods. If the values at this wave period are neglected, the bands of Q_2 and Q_3 are even narrower.

The pattern of the values of Q_2 and Q_3 is not followed by the waves that had a different wave height at the toe (see tests T02010 and T02011 in the table), indicating that it is necessary to do this analysis with equal spectra (or wave heights derived from them).

This leads us to the conclusion that if the wave heights at the toe are equal, the Q_2- and Q_3-values are constant for a given bottom steepness. Wave fronts are apparently steeper in shallow water on a steep foreshore. This may cause the higher damage levels to the breakwater. If non-linear decomposition methods become available, it could be very interesting to redo this analysis for the decomposed waves.

4.2.5 Riprap: application of the peakedness descriptor to the wave data

The parameter of the peakedness of the waves has also been applied to the undecomposed data from the wave gauge closest to the breakwater. The results for the tests described in the previous paragraphs are in Table 4-6, for all tests they are plotted against their peak period in Figure 4-8.

Testnumber	$\overline{H^3} / \overline{H^2}$ [m]
T01010	0.0875
T01011	0.0878
T01012	0.0857
T02010	0.1023
T02011	0.1046
T02070	0.0935
T02071	0.0939

Table 4-6: Results for the peakedness parameter

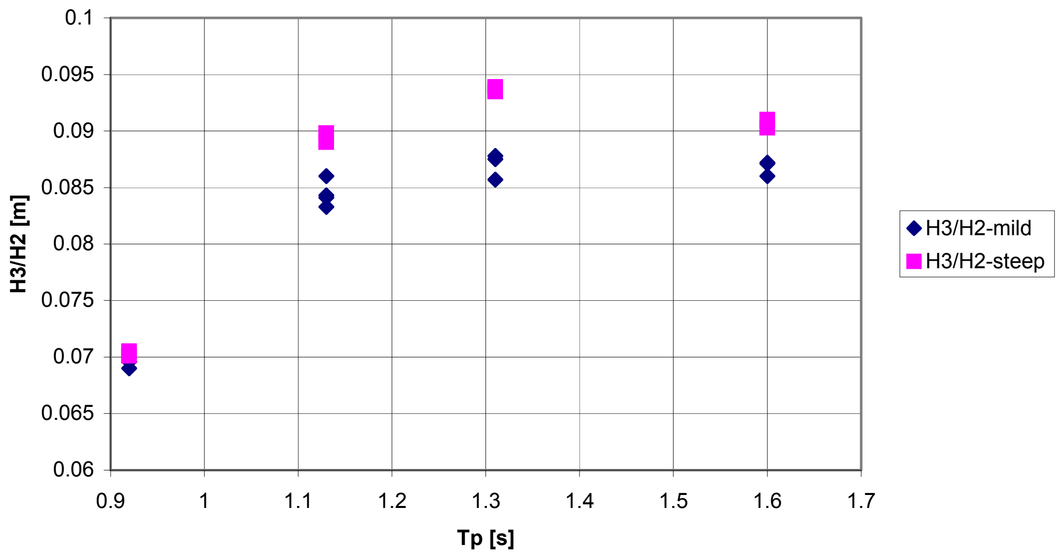


Figure 4-8: Hspike-values

The figure shows that for the steep foreshore, the H_{spike}-parameter is a little higher for most test series. This indicates that in the steep foreshore tests, the chance that higher waves occur is larger. As the highest waves account for the largest part of the damage, this may explain the higher damage levels at the breakwater.

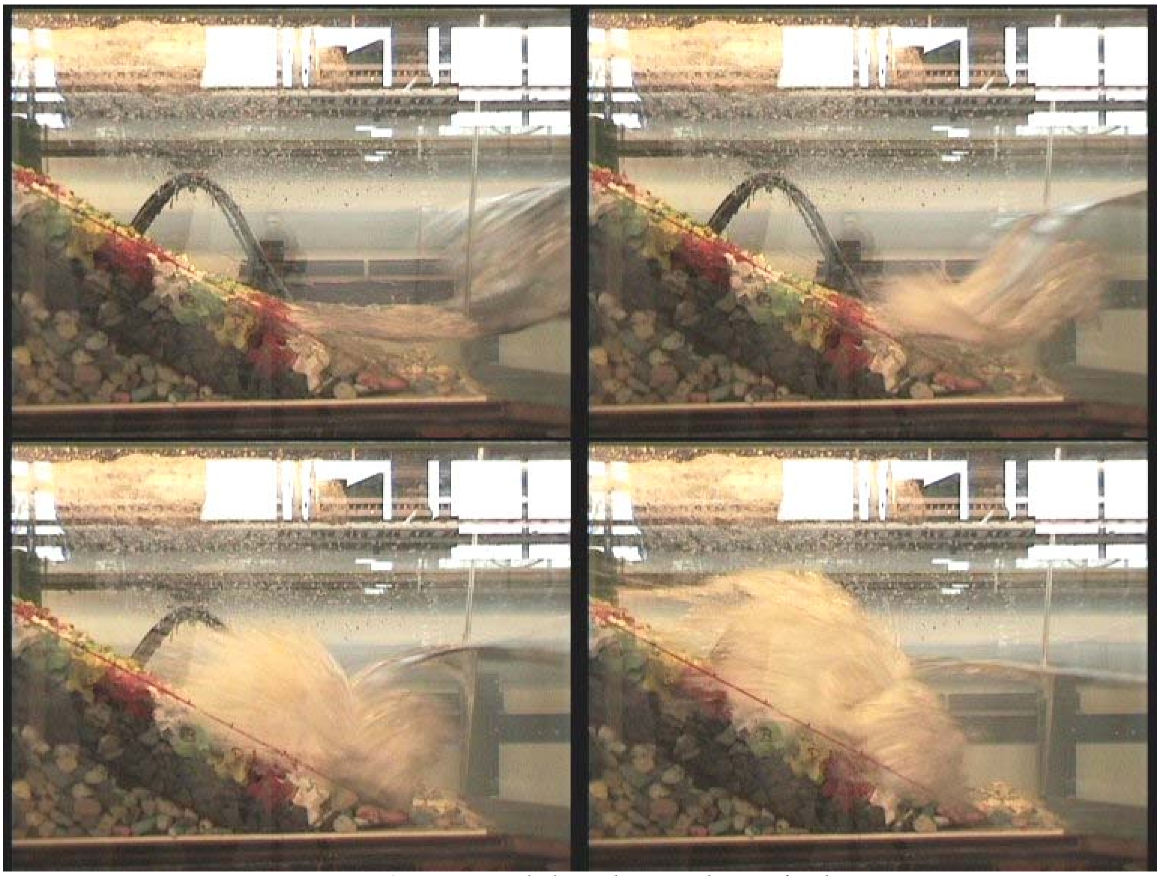
Solely for the series with the shortest wave period, this difference is less pronounced, so for this situation, another explanation would have to be found. Once again, the difference may be explained by the underdeveloped spectra and the steep waves in these series.

4.3 ANALYSIS OF THE TEST RESULTS FOR XBLOC UNITS

The tests with the Xbloc-armour were executed in a somewhat different way. Here, the wave height was low in the beginning and increased somewhat in every test. The wave height was increased till the armour failed or to the limit that the wave flume could handle.

On the mild foreshore, it turned out that it was not possible to do any damage to the armour layer. On the steep foreshore, the same wave programmes at the wave board were used as in the mild foreshore.

From the visual observations, it was clear that the impact of the waves on the breakwater was different. In the pictures below, some examples of waves on the steep foreshore are shown. The time lag between the pictures is $^{1}/_{12}$s.



Picture 4-1: Wave impact by large plunges on the steep foreshore

The impact of this wave caused heavy rocking of the units in a large area. The type of impact as in the pictures above did not occur on the mild foreshore. Also, due to this impact, rocking of the units was much more prevalent.

As the armour layer did not fail at the mild foreshore, it is difficult to further compare the damage levels at the breakwater.

4.3.1 Influence of the slope of the breakwater face

It is important for armour units, and interlocking units in special, to have some ability to rearrange themselves along the breakwater face. This way, they can take over the function if a unit is removed from the profile.

It the tests that were performed on the breakwater with Xbloc units, the slope of the breakwater was 1:2, instead of the usual 1:1.5 or 3:4. Theoretically, this should give the units more stability, as the gravity component perpendicular to the slope increases; see Figure 4-9. In this figure the following forces are drawn:

F_G gravity force (equal for both cases)
$F_{//}$ slope parallel force
$F_\perp$ slope perpendicular force

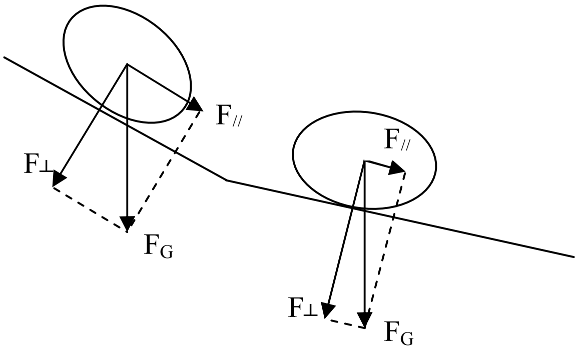


Figure 4-9: Influence of the steepness of the breakwater face on the forces on an armour unit of arbitrary shape

The increased perpendicular force helps to keep the armour units at their place. However, at a milder slope, the gravity component along the breakwater face becomes less. This means, that breakwater units settle more difficultly if a unit is removed below them. This effect has been observed: the Xbloc units settled difficultly.

This contradiction makes it difficult to predict what the influence is of the breakwater face slope on the stability of the armour units and further research in this field is strongly recommended.

5 PRACTICAL AND DESIGN CONSIDERATIONS

In the research objective it was said that a quantitative magnitude of impact of the foreshore steepness would be tried to be found. In this chapter, this analysis will be made.

5.1 GENERAL

Only a limited amount of tests has been performed. For riprap, this was on two different foreshores, with four different wave periods and one deepwater wave height. This limits the applicability of any formulation; for reliable design formulae, many more tests need to be performed. Nevertheless, some trends will be discussed here with an order-of-magnitude quantification. These values are indicative and should not be used yet for design of real life structures.

In this analysis, only the two middle wave steepnesses will be taken into account (i.e. s_0=0.044 and s_0=0.058). This is done, because waves at the lowest steepness (s_0=0.030) may have been too long in comparison with the length of the "shore" in the steep foreshore case. This may give unreliable results. Also, the case with the highest steepness (s_0=0.086) has been taken out of this comparison. This is because this steepness is out of any practical range (for example, Van der Meer is only valid up to s_0=0.06) and because the spectra were ill-developed and may therefore give inaccurate results.

5.2 DESIGN OF COASTAL STRUCTURES ON A STEEP FORESHORE

This research has proved, that the steepness of the foreshore is a really important parameter. It severely influences the stability of a breakwater armour layer. Therefore, design of structures in areas with steep foreshore needs to be done very carefully. As long as reliable design formulae are not available, extensive model testing of to-be-built breakwaters is indispensable. As has been shown in this report, severe rocking of concrete armour units may occur. Therefore, rocking needs to be monitored very carefully during model tests.

5.3 DESIGN FORMULAE

In the previous chapter, it has been argued that the steepness of the wave front above the water, expressed in the Q_2- or Q_3-parameter, may indicate the influence of the bottom steepness. However, it is not yet possible to calculate such a parameter using the wave theories that are known at present. Therefore, one step will be taken back now: just the measurements will be considered and a empirical parameter will be defined, which indicates the influence of the foreshore.

The most well-known formula (or in fact set of formulae) in breakwater design is the Van der Meer formula. Although in this research it proved difficult to compare the results with the Van der Meer formula, it is still possible to say something about a foreshore correction factor. Next to that, something will be said for the Hudson formula. Although this formula is less revealing than the Van der Meer formula, it is still frequently used in engineering and therefore a correction factor for the Hudson formula makes sense.

5.3.1 Application to the Van der Meer formula

The starting point is the Van der Meer formula. For the case of plunging (as defined by Van der Meer) it looks like:

$$\frac{H_s}{\Delta D_n} = 6.2 P^{0.18} \left(S / N \right)^{0.2} \; 1 / \sqrt{\xi_{m0}}$$

and for surging waves it is:

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

All the tests that were performed were valid in the plunging range, so the surging formula will be disregarded now. All the parameters on the right hand side of the plunging formula, except S, will now be lumped in one factor, Ψ, giving:

$$\frac{H_s}{\Delta D_n} = \Psi S^{0.2}$$

Now, a correction factor for the foreshore steepness is introduced: γ. This correction factor is to be applied to the right hand side of the formula, so:

$$\frac{H_s}{\Delta D_n} = \gamma \Psi S^{0.2}$$

The correction factor can take two shapes, depending on the parameters used. If the breakwater is calculated as if it were on a mild foreshore, with the offshore conditions known, it has been shown in this research that the damage can be more than 64% higher. If we use this for S (so use $1.65 \cdot S$), the right hand side of the formula is increased by a factor 1.105. As Ψ and the left hand side of the formula do not change, this can only be rebalanced by using γ=0.90. This is the first shape of the correction parameter, and as it has to be applied to the Van der Meer formula; it will be denoted as γ_M. So:

γ_M=0.90 if only offshore conditions are known and the breakwater is calculated as if it were on a mild foreshore, for usage in the Van der Meer-formula

Another shape is the (more theoretical) situation with the same nearshore wave heights in the steep and shallow foreshore cases. Here, the damage was more than 34% more in the steep foreshore case. On the basis on a same type of analysis as above, it can be shown that the correction factor, denoted as γ_{MM}, has to have a value of 0.94. So:

γ_{MM}=0.94 if the nearshore wave heights are known

5.3.2 Application to the Hudson formula

The Hudson formula does not have a damage term. Nevertheless, something can be said here as well, by modifying the K_D-value by a correction factor γ. For that, we have to rewrite the Hudson formula first.

The starting point is:

$$M \geq \frac{\rho_s H^3}{\Delta^3 K_D \cot\alpha}$$

Substituting:

$$M = \rho_s D_n^3$$

This can be evaluated to:

$$\left(\frac{H_s}{\Delta D_n}\right)^3 = \frac{K_D}{\cot\alpha} \text{ or } \frac{H_s}{\Delta D_n} = \sqrt[3]{\frac{K_D}{\cot\alpha}}$$

From the Van der Meer formulae, the value $\frac{H_s}{\Delta D_n}$ has been reduced. This will be substituted here. As the correction factor has to be applied to the K_D, this will be written:

$$\frac{H_s}{\Delta D_n} = \sqrt[3]{\frac{\gamma K_D}{\cot\alpha}}$$

Analogous to the Van der Meer-formula, the correction factor can take two shapes. The first one is again the case with the same offshore conditions and the breakwater calculated as if it were on a mild foreshore. From the Van der Meer-formula, it could be seen that the value of $\frac{H_s}{\Delta D_n}$ had to be reduced with a factor γ_M=0.9. In the case for the Hudson-formula, the correction factor stands within a cube root, so in order to achieve the same reduction, we have to use the third power of the correction factor of the Van der Meer-correction factor. This number will be referred to as γ_H, where the index *H* indicates the Hudson formula. So:

γ_H=0.73 if only offshore conditions are known and the breakwater is calculated as if it were on a mild foreshore, for usage in the Hudson-formula

Similarly, we can derive a value for the case with the same wave conditions at the toe. Thus we derive a correction factor γ_{HH}. So:

γ_{HH}=0.83 if the nearshore wave conditions are known

5.3.3 Summary

In summary, the Van der Meer- and Hudson-formulae can be altered with a foreshore correction factor. These have to be applied as shown on the next page.

Van der Meer:

$$\frac{H_s}{\Delta D_n} = 6.2\gamma_M P^{0.18} \left(S/N \right)^{0.2} \; 1/\sqrt{\xi_{m0}} \quad \text{(plunging)}$$

Hudson:

$$M \geq \frac{\rho_s H^3}{\Delta^3 \gamma_H K_D \cot\alpha}$$

In both formulae, γ_M and γ_H may be replaced by γ_{MM} or γ_{HH} respectively, depending on which wave conditions are known. Their values are:

Parameter	Value	Applicability	Formula
γ_M	0.90	Only offshore conditions known	Van der Meer (plunging case only)
γ_{MM}	0.94	Nearshore conditions known	
γ_H	0.73	Only offshore conditions known	Hudson
γ_{HH}	0.83	Nearshore conditions known	

Table 5-1: Resume of correction factors

For mild foreshore situations, all γ-values become equal to unity.

The definition of steep and mild is relative. In the experiments that were performed in this research, 1:8 was clearly steep, whereas 1:30 was mild. The transition between the two will be on slopes in between. It is to be expected that this will coincide with the transition of the type of wave breaking on the foreshore, expressed by the Iribarren-parameter:

$$\xi = \tan\beta / \sqrt{s}$$

If the breaking on the foreshore is of the plunging type (ξ>0.5), a steep foreshore-correction will probably be necessary. This also means that the transition point is not fixed, but depends on the wave steepness.

Beyond the transition point, there are two ways the correction factor may develop. The first possibility is that it doesn't change. This is the case if the correction factor is only dependent on the breaking type on the foreshore. Waves are either plunging or spilling (although in irregular waves, a transition zone will be present).

The other possibility is that the dimensionless steepness of the waves (i.e. the Q_2- and Q_3-values) keeps becoming larger as the foreshore keeps becoming steeper. If this is the main influence that causes the difference in damage, the correction factor will become smaller as the foreshore becomes steeper. The lower limit of this will be reached at the point where the foreshore is so steep that the waves on the foreshore are surging ($\xi \approx 3$).

In future investigations, the γ-values can be elaborated further. A dependency on the wave period can be expected, as these experiments have shown, but in order to do a reliable analysis on this, much more experiments are necessary.

6 CONCLUSIONS AND RECOMMENDATIONS

This research has shown that the steepness of the bottom does matter when designing coastal structures like breakwaters or sea defences. Due to a steep sea bottom, average wave heights become higher compared to the wave heights at a mild foreshore. This is due to the fact, that on a steep slope, waves can shoal to larger wave heights before they break. The larger wave heights consequently cause larger damage to the breakwaters. This report has shown that in the tested circumstances, the damage to the breakwater was up to 64% higher on the 1:8-foreshore compared to the 1:30-foreshore, with equal deep-water wave conditions.

Furthermore, this report has shown that even if the wave conditions at the toe of the breakwater (instead of at deep water) are almost similar in the steep and the mild foreshore case, the damage to the breakwater can still be about 30% higher on the steep foreshore. In these cases, the wave spectra of the incoming waves close to the toe were about equal. As the waves at the toe cause the damage to the breakwater, it would up till now be expected that if the spectra there are equal, the damage is comparable. As this was not the case, this leads to the conclusion that the wave spectrum does not give all the information that is needed to design a reliable breakwater.

As the decomposition method of the waves in the zone close to the breakwater may not be valid, also an analysis has been performed of the "raw" wave data close to the toe. This means that incoming and reflected waves were not separated form each other. This analysis also didn't show large differences between the conditions at the steep and the shallow foreshore, meaning that, based on the spectra, the wave conditions were really about the same.

A possible explanation of this extended damage may be the non-linearity of the waves close to the toe. As waves shoal, they more and more loose their nice sine-shape as the water becomes shallow, leading to short, but high wave crests and long, shallow wave troughs. As the peaks cause the impacts on the breakwater, it makes sense to look at the steepness of the front of the peaks above the still water level. As the waves are a time registration, the steepness that is calculated was in fact a rising velocity of the water level. For all waves in a test, this steepness of the wave front was calculated and this was averaged in several ways. This can be made dimensionless by dividing this result by the wave celerity.

As the actual wave celerity was not measured, the linear wave theory was used to derive a theoretical average wave celerity, based on the peak period.

An attempt to calculate the steepness of the incoming waves close to the toe did not work. This clearly showed that the decomposition method for incoming and reflected waves does not work close to the toe of the breakwater: the output data of the waves could not be used. So again, the "second-best" method of an assessment of the raw wave data was performed. This clearly showed an influence of the bottom steepness: the values were more or less constant for all steep foreshore tests constant at a different value for all mild foreshore tests. Although this result has to be treated with care, as they are applied to data where incoming and reflected waves are not separated, it may be that the wave steepness is dependent on the bottom steepness. If it is this steepness that influences the damage to the breakwater, than this is a possible explanation for the extended damage to the breakwater in the steep foreshore tests. Observations also showed that the steepness of the waves was higher in the steep foreshore tests. As the wave steepness is information that is not revealed by just using the wave spectrum as an input parameter, this explains why this difference cannot be explained by using the spectrum only.

As for the tests on the Xblocs, the difference was also clear. At the mild foreshore, it was not possible to do damage to the breakwater, given the circumstances and the possible wave heights. On the steep foreshore, the armour failed several times. This can be explained by the higher average wave height and the higher maximum wave height, that is possible in this case. But even before failure, rocking is much more prevalent in the case of the steep foreshore. As could be seen in observations, this can be explained by a kind of water hammer effect by the plunges of the waves. Waves that just don't break develop a high steepness of the front as they approach the breakwater, wave that do break are of the plunging-type and develop a jet-like water movement, which can hit the breakwater with much force on a limited area, which may cause high water pressures in this area. This jet can either hit the armour, causing rocking, or it can hit the toe as well. As the toe was completely fixed, damage could not be done to it, but this demonstrates that research to the stability of the toe is also necessary in future research.

In case of the mild foreshore, these jets do not occur and the wave steepness is much lower. The wave movement at the breakwater is much smoother, causing only minor rocking to the Xblocs.

6.1 CONCLUSIONS

In summary, the conclusions of this research project are:

- The steepness of the bottom slope does have an important influence on the stability of armour units on a breakwater
- On steep foreshores, damage levels may be significantly higher in comparison with mild foreshores, for equal offshore wave conditions
- With almost similar wave conditions near the toe, damage levels at the breakwater are higher at the steep foreshore
- From the raw spectra, the waves on the steep foreshore showed on average steeper wave fronts
- The waves on the steep foreshore were also more peaked
- As a preliminary result, from the present investigation, it can be said that on an 1:8-slope, the value of $\frac{H_s}{\Delta D_n}$ has to be reduced with a steep foreshore coefficient γ_M=0.90. If the Hudson-formula is being used, this means a reduction of K_D with a coefficient γ_H=0.73. These values are valid if the wave heights are calculated as if the foreshore bottom were mild.
- If wave heights at the toe are known, the values of $\frac{H_s}{\Delta D_n}$ reduce by γ_{MM}=0.95 and the value of K_D by γ_{HH}=0.85.

- Concrete armour units show more rocking due to the different impact of the waves on the breakwater.
- From visual observation, it is clear that the toe suffers a more direct attack on the steep foreshore due to plunging breakers.

6.2 Research recommendations

As the number of measurements performed is limited, it is recommended that more tests should be performed it order to obtain more reliable results and better parameters that show the influence of the foreshore. The following parameters need special attention and should be varied:

- The steepness of the foreshore $\tan\beta$
- The deep water wave height H_{m0}
- The wave period T_p
- The water depth at the toe, h_t
- This research should be done for both rock armour layers and interlocking armour layers.

A possible explanation of the differences of damage levels may be a different internal water movement for several types of waves. Earlier research showed that a bottom protection under a constriction may suffer more damage as the constriction is more abrupt, although local velocities are the same. A similar process may take place in the case of a steep foreshore, and this may also influence the stability of a breakwater armour layer. Further investigation in this direction, including the internal water movement, therefore could give interesting results.

The exact mechanism that forces stones to leave the breakwater profile is poorly understood. If this would be known better, this could reveal why a different type of impact causes different damage. Also a time-dependent description of the energy flux in waves for both linear and non-linear waves could add to the understanding of this process.

As the toe was completely fixed in these experiments, it could not suffer damage. From the visual observation it became clear that the toe may be attacked heavily by plunging breakers if the foreshore is steep. Toe stability formulae are scarce anyway and further research to the stability of toes, especially for the case of steep foreshores, is therefore strongly recommended.

The influence of the steepness of the breakwater face on the stability of the breakwater armour is unclear for interlocking units. On one hand, the increased gravity-component on a mild slope increases the stability of the units, but on the other hand, the decreased slope-parallel weight component decreases the possibility of the units to settle. A research to the influence of the steepness of the breakwater face to interlocking units may therefore be very interesting.

Decomposition methods for non-linear waves or breaking waves are not available. The development of such calculation methods could greatly improve measurement results in the area close to the breakwater.

6.3 Practical and design recommendations

If designing a breakwater or sea defence in an area where steep foreshores are prevalent, very special attention should be paid to the areas where the foreshore is steep. As long as real design guidelines are not available, extensive model research is recommended for these areas and especially rocking should be carefully monitored.

In summary, the recommendations are:

- Much more research should be performed to the influence of the foreshore steepness on the stability of armour layers.
- The stability of the toe should also be researched in more detail.
- The influence of the steepness of the breakwater face on the stability of interlocking armour units should be researched.
- A calculating method for the decomposition of non-linear or breaking waves could boost shallow water model research.
- As long as reliable calculating methods are not available, special attention should be paid to breakwaters and sea defences in areas with steep foreshores.

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NOMENCLATURE

Armour layer	Outer protective layer of a breakwater
Armour stability	Situation where the strength of the armour layer is larger than the load
Armour unit	Specially designed unit (usually concrete) to be used in armour layers
Breakwater	Structure protecting a harbour entrance from severe wave action
Coastline	Border between sea and land
Core	Inner part of a breakwater
Crown wall	Wall on top of a breakwater (L-shaped or an asymmetric T upside-down)
Damage	State where the breakwater has lost some armour units or where some of the have broken, but where breakwater is still able to perform its function
Downcrossing	The crossing of a horizontal level in a downward sense
Failure	State where the damage is so large that the breakwater cannot perform its normal function any more
Filter	Permeable layer(s) separating rougher from finer material in order to prevent washing out of the finer material
Foreshore	Bottom in the nearshore zone
Interlocking	Cooperation between armour units against washing-out
Nearshore	Close to the shore, on shallow water
Offshore	Far away from the shore, on deep water
Peak frequency	The frequency at which the spectral density is the highest
Riprap	Loose stones or rocks
Rocking	Movement of armour units without being removed permanently from their initial position
Sea defence	Structure protecting the land behind it against the sea
Toe	Structure supporting the armour layer
Wave	Vertically oscillating water movement between two zero-downcrossings
Wave breaking	Collapsing of waves due to the water becoming too shallow or the wave too steep
Wave flume	Installation where hydraulic research can be executed
Wave gauge	Equipment recording water levels as a function of time
Wave height	Maximum difference between the highest and lowest water level in a wave
Wave period	Time lag between two zero-downcrossings
Wave spectrum	Diagram showing the distribution of wave energy over the frequencies
Wavelength	Horizontal length between two zero-downcrossings
Zero-downcrossing	Downcrossing through the still water level

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APPENDIX A: WAVE PROGRAMS

In this Appendix, the wave programs that were used are described.

A1: MAIN TESTS

Each test that was performed has an unique five-digit number. This number recurs in all files that are connected with the test. A test number is always preceded by the letter “T”, for “Test”. For example: in “T01031”, the test number is 01031.

As a rule of thumb, test numbers that start with 00 indicate the preparatory tests, numbers that start with 01 indicate the measurements on the mild foreshore and the numbers that start with 02 indicate the steep foreshore. If the third digit is 0 or 1, they indicate the rip-rap measurements; if it is 9, it indicates the Xbloc measurements.

The third and fourth digit together give an indication of the wave program used. If they are the same, then the same wave program (at the wave board) was used). The last number is an ordinal number.

For example: “T02010” indicates a test on the steep foreshore with a riprap armour layer. It was performed under the same conditions as “T02011”. Their mild foreshore counterparts are “T01010” to “T01012”.

For the steep foreshore tests with and adapted wave program (in order to obtain equal conditions at the toe), the numbers in the third and fourth digit are different and which case belongs to which can be found in the table.

T_p [s]	Mild foreshore	Equal waves at the board		Equal waves at the toe	
		Steep foreshore	$H_{m0,b}$ [cm]	Steep foreshore	$H_{m0,b}$ [cm]
1.60	01000, 01001, 01002	02000	11.0	02090, 02091	9.6
1.31	01010, 01011, 01012	02010, 02011	11.0	02070, 02071	9.6
1.13	01020, 01021, *(1022*)*, 01023	02020	11.1	02080, 02081	9.8
0.92	01030, 01031, 01032	02030	11.3	02110, 02111	9.4

(Test 01022 used a slightly different wave program and was therefore redone as test 01023.)*

A2: OTHER TESTS

All (other) tests can be indicated by a wave program number. All wave test numbers are printed in the tables below. The last two numbers in all programs indicate the water depth at the wave board in cm.

In the first table below, the basic wave programs are printed. The letter in the middle gives an indication of the deepwater wave steepness.

Program		M06B66	M08B66	M10B66	M12B66	M06C66	M07C66	M08C66	M10C66	M12C66	M14C66
T_p	[s]	1.39	1.6	1.79	1.96	1.13	1.22	1.31	1.46	1.6	1.73
$H_{m0,b}$	[m]	0.082	0.110	0.139	0.169	0.083	0.096	0.110	0.137	0.165	0.194
h_b	[m]	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66
$H_{m0,0}$	[m]	0.089	0.120	0.152	0.183	0.087	0.102	0.118	0.149	0.181	0.212
s_0	[-]	0.030	0.030	0.030	0.030	0.044	0.044	0.044	0.045	0.045	0.045

Program		M05D66	M06D66	M07D66	M08D66	M09D66	M10D66	M11D66	M08E66	M08F66
T	[s]	0.89	0.98	1.06	1.13	1.2	1.27	1.33	1.01	0.92
$H_{m0,b}$	[m]	0.071	0.084	0.098	0.111	0.124	0.137	0.151	0.112	0.113
h_b	[m]	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66	0.66
$H_{m0,0}$	[m]	0.071	0.086	0.101	0.116	0.131	0.147	0.162	0.114	0.114
s_0	[-]	0.057	0.057	0.057	0.058	0.058	0.058	0.059	0.072	0.086

In the next table, the wave programs at a modified depth are printed.

Programma		M06B72	M08B72	M10B72	M12B72	M06C72	M08C72	M06B74	M08B74	M10B74	M12B74	M13B74
T	[s]	1.39	1.6	1.79	1.96	1.13	1.31	1.39	1.6	1.79	1.96	2.04
$H_{m0,b}$	[m]	0.082	0.110	0.138	0.168	0.084	0.110	0.082	0.110	0.138	0.168	0.183
h_b	[m]	0.72	0.72	0.72	0.72	0.72	0.72	0.74	0.74	0.74	0.74	0.74
$H_{m0,0}$	[m]	0.089	0.120	0.151	0.183	0.086	0.117	0.088	0.120	0.151	0.183	0.198
s_0	[-]	0.029	0.030	0.030	0.030	0.043	0.044	0.029	0.030	0.030	0.030	0.030

In the last table, the modified wave programs are printed. Here, the middle letter does not indicate the steepness any more, but the correspondence of wave periods with the wave periods in the first table.

Programma		M07BX66	M07CX66	M075CX66	M07DX/Y66	M065FX66	M07FX66
T	[s]	1.6	1.31	1.31	1.13	0.92	0.92
$H_{m0,b}$	[m]	0.096	0.096	0.105	0.098	0.094	0.099
h_b	[m]	0.66	0.66	0.66	0.66	0.66	0.66
$H_{m0,0}$	[m]	0.105	0.104	0.112	0.102	0.095	0.100
s_0	[-]	0.026	0.039	0.042	0.051	0.072	0.076

APPENDIX B: MEASUREMENT RESULTS

The table below shows the tests, with the numbers, the wave programs that were used and the damage levels that occurred. The meaning of the numbers is explained first, the tables follow on the following pages.

In the leftmost column, the test numbers are printed, in the next column, the wave program that was used in the test (in which "x2" means the double program was run). The details on these wave programs can be found in the previous Appendix.

The centre columns indicate the number of stones that were removed from each layer, sorted by colour. The leftmost is the lowest band of stones (i.e. just above the toe). The abbreviations above the columns indicate the colours:

Wh	White (in fact: light grey)
Rd	Red
Bl	Blue
Pk	Pink
Ye	Yellow
Gr	Green
Rd^+	Red "plus"(red layer in the upper section of the armour)
Ye^+	Yellow "plus" (idem)
Gr^+	Green "plus" (idem)
Bk	Black (i.e. stones from the filter layer)

If a field is left empty, this means that in that particular test no stones from that bans were removed.

In the last two columns, the total number of stones removed from their original band are added and normalised.

The other symbols have their usual meanings:

d_{n50}	nominal stone diameter	[m]
B	width of the flume	[m]
$\tan\alpha$	steepness of the breakwater face	[-]
P	notional permeability	[-]
N	number of waves	[-]
N_S	(total) number of stones moved	[-]
N_{od}	normalised number of stones moved	[-]

d_{n50}	0.0157	[m]	$\tan\alpha$	0.50	
B	0.8	[m]	Δ	1.78	
			P	0.6	
			N	1000	(T01040/41/42: 2000)

Testnumber	Program	Wh	Rd	Bl	Pk	Ye	Gr	Rd+	Ye+	Gr+	Bk	N_S	N_{od}
T00010	M05D66	4	15	15	8	4						46	0.90
T00011	M06D66	6	24	51	41	23	1					146	2.87
T00012	M07D66	8	56	100	74	43	6					287	5.63
T00013	M08D66	15	73	138	123	113	35	20				517	10.15
T00014	M09D66	18	125	192	185	209	150	23				902	17.70
T00015	M10D66	20	238	224	211	285	261	57				1296	25.43
T00016	M11D66	31	324	344	225	298	324	55	30			1631	32.01
T00020	M05D66	2	19	23	13	1						58	1.14
T00021	M06D66	6	35	59	37	10	1					148	2.90
T00022	M07D66	10	55	108	82	59	4					318	6.24
T00023	M08D66	13	86	158	147	123	40	1				568	11.15
T00024	M09D66	16	116	191	192	186	127	11				839	16.47
T00025	M10D66	24	223	224	227	286	211	42	21	7		1265	24.83
T00026	M11D66	36	273	257	258	425	303	54	29	13		1648	32.34
T00100	M05D66	9	26	43	9							87	1.71
T00101	M06D66	14	49	96	82	8						249	4.89
T00102	M07D66	26	80	164	183	82	6					541	10.62
T00103	M08D66	33	123	249	346	266	75					1092	21.43
T00104	M09D66	39	148	295	415	391	284	9				1581	31.03
T00110	M05D66	1	18	38	7							64	1.26
T00111	M06D66	9	51	83	51	9						203	3.98
T00112	M07D66	15	94	190	207	101	7					614	12.05
T00113	M08D66	22	115	238	311	231	126					1043	20.47
T00114	M09D66	30	156	286	386	364	254	3			1	1480	29.05
T00120	M05D66	5	36	31	5							77	1.51
T00121	M06D66	9	70	81	39	1						200	3.93
T00122	M07D66	18	131	164	162	60	3					538	10.56
T00123	M08D66	30	196	237	303	231	78					1075	21.10
T00124	M09D66	37	228	296	410	387	256	3				1617	31.73
T00130	M08D66	10	145	214	260	210	30					869	17.05
T00131	M08D66	10	138	200	262	162	19					791	15.52
T00132	M08D66	12	155	240	255	163	6					831	16.31
T00140	M08B66	18	175	357	261	339	258	6				1414	27.75
T00141	M08B66	7	176	258	371	337	275	1				1425	27.97
T00142	M08B66	14	163	251	371	364	292	10				1465	28.75
T00150	M08C66	15	171	239	323	271	122					1141	22.39
T00151	M08C66	21	207	221	274	214	74					1011	19.84
T00152	M08C66	10	215	229	329	268	113					1164	22.84
T00160	M08E66	14	162	160	165	37	1					539	10.58

Testnumber	Program	Wh	Rd	Bl	Pk	Ye	Gr	Rd+	Ye+	Gr+	Bk	N_S	N_{od}
T01000	M08B66	29	203	270	367	327	267	18	1			1482	29.08
T01001	M08B66	19	225	271	356	303	243					1417	27.81
T01002	M08B66	24	202	278	349	306	222	5				1386	27.20
T01010	M08C66	21	184	222	287	190	43	1				948	18.60
T01011	M08C66	19	144	240	303	217	67					990	19.43
T01012	M08C66	21	207	221	274	214	74					1011	19.84
T01020	M08D66	26	96	164	232	124	11					653	12.82
T01021	M08D66	20	79	172	220	152	21					664	13.03
T01022	M08D66L*	20	82	174	251	156	34					717	14.07
T01023	M08D66	61	89	156	233	134	21					694	13.62
T01030	M08F66	26	63	89	85	21						284	5.57
T01031	M08F66	13	71	125	78	8						295	5.79
T01032	M08F66	16	67	91	87	16						277	5.44
T01040	M08D66L	21	116	208	340	258	126	3				1072	21.04
T01041	M08D66x2	24	113	230	325	260	102	2				1056	20.72
T01042	M08D66L	20	100	209	328	265	82	1				1005	19.72
T02000	M08B66	27	276	237	316	324	394	127	85			1786	35.05
T02010	M08C66	44	150	214	311	308	327	99	2			1455	28.55
T02011	M08C66	27	155	217	309	317	312	57				1394	27.36
T02020	M08D66	30	130	200	274	283	181	4				1102	21.63
T02030	M08F66	12	87	171	220	126	16					632	12.40
T02050	M07C66	12	103	187	275	245	150	1				973	19.10
T02051	M07C66	8	114	175	279	246	121	4				947	18.58
T02060	M075CX66	31	138	215	316	293	287	30	1			1311	25.73
T02070	M07CX66	26	115	176	289	259	211	6				1082	21.23
T02071	M07CX66	19	104	175	290	268	226	21	3			1106	21.71
T02080	M07DX66	35	92	185	258	224	119	2				915	17.96
T02081	M07DY66	21	107	178	235	220	126	3				890	17.47
T02090	M07BX66	42	155	225	300	293	297	71	7			1390	27.28
T02091	M07BX66	47	192	250	301	277	305	77	15	2		1466	28.77
T02100	M07FX66	12	65	120	173	85	3					458	8.99
T02110	M065FX66	4	38	124	171	58						395	7.75
T02111	M065FX66	8	43	100	143	65						359	7.05

* This was a double-length program. Half of this program was used.

APPENDIX C: FORMULAE

This Appendix contains a number of well-known derivations, which were not in the original text, as well as the Goda-formulae for wave height calculation.

C1: CALCULATION OF STONE DENSITY AND NOMINAL DIAMETER

If the stones are weighed dry and under water, their density can easily be calculated.

V_s	volume of a stone	[cm^3]
ρ_s	density of the stone	[g/cm^3]
ρ_w	density of water	[g/cm^3]
m_D	dry mass of the stone	[g]
m_U	underwater mass of the stone	[g]

By definition, the following equations are valid:

$$m_D = \rho_s V_s$$

$$m_U = (\rho_s - \rho_w) V_s$$

From this, the volume of the stone can be solved:

$$V_s = \frac{m_D - m_U}{\rho_w}$$

Then follows:

$$\rho_s = \rho_w \frac{m_D}{m_D - m_U}$$

As by definition:

$$D_n = \sqrt[3]{m_D / \rho_s}$$

This yields:

$$D_n = \sqrt[3]{\frac{(m_D - m_U)}{\rho_w}}$$

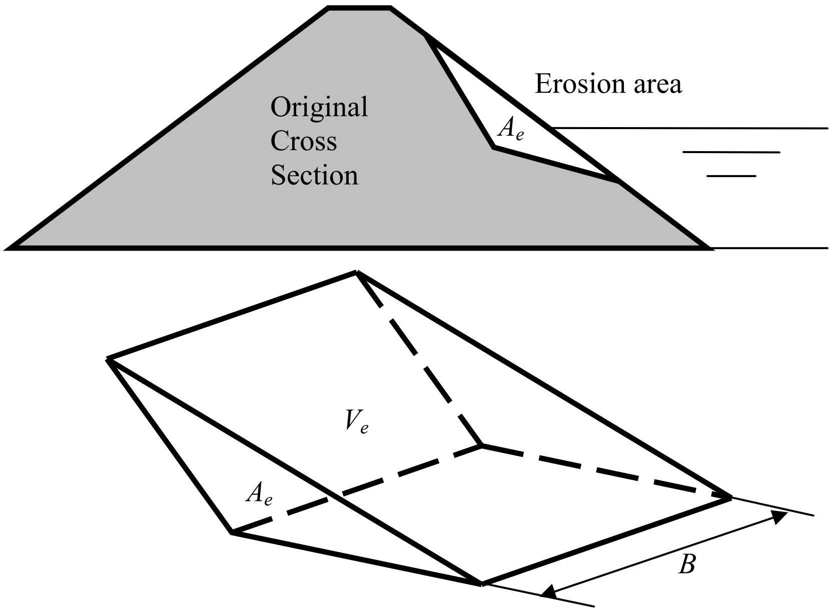
C2: DAMAGE DEFINITIONS

Van der Meer defines the damage number as:

$$S \equiv \frac{A_e}{d_{n50}^2}$$

in which:

S	damage number	[-]
A_e	erosion area	[m^2]
d_{n50}	nominal stone diameter	[m]



So in order to calculate the damage according to Van der Meer, the erosion area in the breakwater cross section has to be determined. In this research, a different definition of damage was used: all the stones that were removed from their original coloured band were counted. This amount, *N*, can be normalised to the “number of displaced units” by:

$$N_{od} = \frac{N_S d_{n50}}{B}$$

with:

N_{od}	number of displaced units	[-]
N_S	number of stones that were removed from their original band	[-]
B	width of the measured section	[m]

Theoretically from the above, the volume of the erosion area V can be derived as: $V_s = ND_{n50}^3$ and with $N_S = \frac{N_{od} B}{D_{n50}}$ this yields:

$$V_s = N_{od} B d_{n50}$$

From Van der Meer, a similar expression can be derived: $V_e = A_e B$ and with $A_e D_{n50}^2$, this yields:

$$V_e = SBd_{n50}$$

Despite their similarity, these expressions are not the same. In the definition of *S*, a stone that is removed from its original position, but that remains in the erosion area, is not counted. In the definition of N_{od}, this stone is counted. This makes it hard to calculate *S* from N_{od} or vice versa.

Furthermore, these formulations don’t take account of porosity. In the Van der Meer-definition, the A_e is the real area of the erosion cross section, so the volume is the real volume of the erosion area. In the definition of N_{od}, the volume is the volume of removed stones. If one wants to calculate between *S* and N_{od}, one needs data about the porosity, *n*. Theoretically, the following conversion is possible:

$$V_e = A_e B = SBD_{n50}^2$$
$$V_s = (1-n)V_e = (1-n)SBD_{n50}^2$$
$$N = \frac{V_s}{D_{n50}^3} = \frac{(1-n)SB}{D_{n50}}$$
$$N_{od} = \frac{ND_{n50}}{B} = (1-n)S$$

But as can be seen from the test results, this correlation is really bad. Therefore, the results of the experiments cannot well be compared to the Van der Meer-formulae.

C3: CALCULATION OF THE SHOALING COEFFICIENT

By definition,

$$K_s \equiv \frac{H}{H_0}$$

in which

K_s	shoaling coefficient	[-]
H	wave height	[m]
H_0	deep water wave height	[m]

The linear wave theory (see e.g. BATTJES, 1997) shows that:

$$\frac{H}{H_0} = \sqrt{\frac{c_{g_0}}{c_g}} = \sqrt{\frac{n_0 c_0}{nc}}$$

in which:

c_g	wave group speed	[m/s]
c	wave celerity	[m/s]
n	coefficient, defined by: $n = \frac{1}{2} + \frac{kh}{\sinh 2kh}$	[-]
k	wave number, $k \equiv \frac{2\pi}{L}$	[-]
sub *0*	deep water	

For deep water, n=½ so n_0=½. Further, the following relation is known:

$$c = c_0 \tanh kh$$

Now, the shoaling coefficient can be re-evaluated to:

$$K_s = \sqrt{\frac{1}{n \tanh kh}}$$

So if the water depth is known, only the wave length has to be evaluated to calculate the shoaling coefficient. This can be done through:

$$L = L_0 \tanh \frac{2\pi h}{L}$$

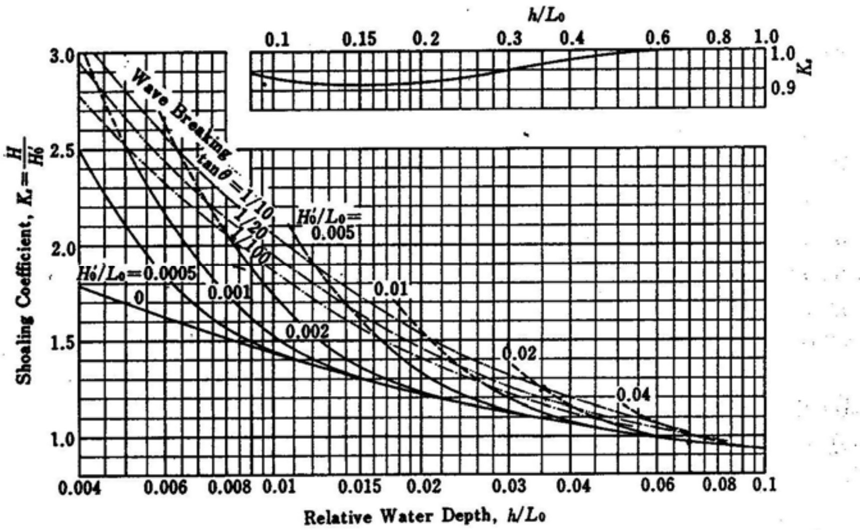
in which L_0, the deep water wave length, is given by $L_0 = \frac{gT^2}{2\pi}$, with:

T	wave period	[s]
g	gravity acceleration	[m/s^2]

C4: WAVE HEIGHT FORMULAE BY GODA

Goda developed a large set of formulae for wave calculation, based on extensive measurements. Some of the will be dealt with here.

He used amongst others the diagram prepared by Shuto, which includes shoaling and breaking for different steepnesses of the sea bottom. The diagram is in the figure below and is taken directly from Goda's book.



With this diagram, it is possible to estimate waves in the nearshore zone. (Unfortunately, the reference Goda gives to Shuto is a book in Japanese only, so more details of the derivation or the application cannot be given here.)

Goda uses this diagram to derive a limiting wave height for regular waves. For calculating the depth at which waves break, Goda developed the following relation:

$$\frac{H_b}{L_0} = A\left[1 - \exp\left(1.5\frac{\pi h}{L_0}(1 + 15\tan^{4/3}\tan\theta)\right)\right]$$

In this relation, A is an empirical factor, for which Goda recommends A=0.17 for regular waves and $0.12 \le A \le 0.18$ for irregular waves.

Using these results, Goda applies an assumption for breaking of waves above the limiting wave height and he ends with the following formulae for the calculation of wave heights:

$$H_{\max} = \begin{cases} 1.8H_{1/3} & : h/L_0 \ge 0.2 \\ \min\{(\beta_0^* H_0' + \beta_1^* H_0'), \beta_{\max}^* H_0', 1.8H_{1/3}\} & : h/L_0 < 0.2 \end{cases}$$

$$H_{1/3} = \begin{cases} K_s H_0' & : h/L_0 \ge 0.2 \\ \min\{(\beta_0 H_0' + \beta_1 h, \beta_{\max} H_0', K_s H_0'\} & : h/L_0 < 0.2 \end{cases}$$

$$\begin{cases} \beta_0 = 0.028(H_0'/L_0)^{-0.38}\exp(20\tan^{1.5}\theta) \\ \beta_1 = 0.52\exp(4.2\tan\theta) \\ \beta_{\max} = \max\{0.92, 0.32(H_0'/L_0)^{-0.29}\exp(2.4\tan\theta)\} \end{cases}$$

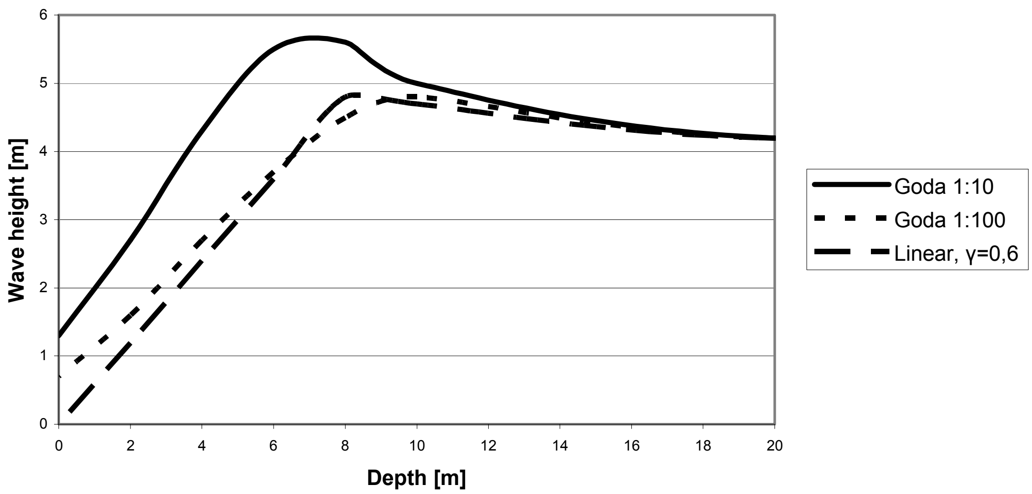
$$\begin{cases} \beta_0^* = 0.052(H_0'/L_0)^{-0.38}\exp(20\tan^{1.5}\theta) \\ \beta_1^* = 0.63\exp(3.8\tan\theta) \\ \beta_{\max}^* = \max\{1.65, 0.53(H_0'/L_0)^{-0.29}\exp(2.4\tan\theta)\} \end{cases}$$

In all of these formulae:

H_{max}	maximum wave height (i.e. $H_{1/250}$]	[m]
$H_{1/3}$	the average of the 1/3-highest waves	[m]
$H_{1/250}$	the average of the 1/250-highest waves	[m]
h	water depth	[m]
L_0	deep water wave length	[m]
θ	bottom slope	[-]
K_s	shoaling coefficient from linear wave theory	[-]
H_0'	equivalent deep water significant wave height	[m]
β_0, β_1, β_{max}, β_0^*, β_1^*, β_{max}^*	coefficients	[-]

(Note: in the rest of this report, β has been used for the bottom slope. On this page, θ has been used instead, like it was used by Goda originally. This has been retained in order to avoid confusion with the β-coefficients.)

In the following graph, an example of a shoaling calculation is compared for linear wave theory, with γ=0.6 and with the Goda-formula for both tanθ=1:10 and tanθ=1:100. In this calculation It can clearly be seen that Goda diverges from the linear theory towards shallower water. Furthermore, it can be seen that the linear wave theory still gives a reasonable result for mild foreshores, but that the results for steep foreshores are significantly different. Note that in the Goda-calculation, the wave height does not reduce to zero for zero water depth, as Goda takes wave set-up into account. Of course, set-up can be calculated for linear wave theory as well, using the radiation stress theory (see BATTJES (1997)).



For more on the Goda-formulae, see GODA (2000) or for example D'ANGREMOND.

APPENDIX D: SCALING LAWS

In model research, many scaling laws exist, depending on the goal of the research. In hydraulic engineering, the Froude- and the Reynolds-scaling laws are the most important ones. For a more in-depth review of scaling laws, see HUGHES.

Froude scaling criterion

Perhaps the most utilised scaling law in hydraulic engineering is the Froude scaling law. This criterion requires the Froude numbers in prototype and scale model to be the same:

$$Fr_m = Fr_p \Leftrightarrow \left(\frac{U}{\sqrt{gL}}\right)_m = \left(\frac{U}{\sqrt{gL}}\right)_p$$

With:

Fr	Froude number	[-]
U	velocity	[m/s]
g	gravity acceleration	[m/s^2]
L	length	[m]
sub m	model	
sub p	prototype	

As the gravity is always the same in the model as in the prototype (except for tests in centrifuges, like they are sometimes performed in geotechnical engineering), this can be taken from the equation, which after some rewriting, yields:

$$\sqrt{\frac{L_m}{L_p}} = \frac{U_m}{U_p}$$

Defining:

N	scaling factor	[-]
sub L	length	
sub U	velocity	
sub T	time,	

one can see that:

$$\sqrt{N_L} = N_U$$

As U=L/T, this becomes:

$$\sqrt{N_L} = N_L / N_T \Leftrightarrow N_T = \sqrt{N_L}$$

This tells us that the time scale has to scale with the square root of the scaling factor.

Reynolds scaling criterion

Another very important scaling law is the Reynolds scaling criterion, stating that the Reynolds numbers in prototype and model should be the same:

$$\mathrm{Re}_m = \mathrm{Re}_p \Leftrightarrow \left(\frac{\rho LU}{\mu}\right)_m = \left(\frac{\rho LU}{\mu}\right)_p$$

where:

ρ	specific density	[kg/m^3]
μ	dynamic viscosity	[kg/(m·s)]

Strictly speaking, density and viscosity in a fresh water model are slightly different from their values in saline seawater, but here, this effect will be neglected, so they can be taken from the equation. This yields:

$$(LU)_m = (LU)_p \Leftrightarrow N_L = {}^{1}\!/\!_{N_U}$$

And with again $U=L/T$, this becomes:

$$N_T = N_L^2$$

Contradiction

If the Reynolds criterion is compared to the Froude criterion, it is evident that a problem arises:

$$\begin{cases} N_T = \sqrt{N_L} & \text{(Froude)} \\ N_T = N_L^2 & \text{(Reynolds)} \end{cases}$$

The time scale has to scale to both the square root of the length scale and squared to the length scale. Often, this problem is solved by reducing the Reynolds criterion to requiring that in the model, the Reynolds number should be that large that viscous effects can be neglected, so the flow should be turbulent. Depending on the application, this number ranges from about (1 to 3)·10^4. However, this indicates that one has to be well aware of the Reynolds numbers in the tests.

APPENDIX E: LITERATURE REVIEW

In this chapter, the existing literature on wave theory and breakwater design is reviewed. There will be a focus on steep foreshores

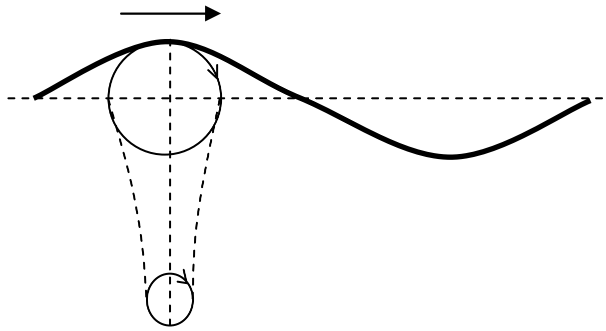
E1 WAVE PROPERTIES

In the previous chapter, it has been addressed already, that wave properties change if the waves move over a sloping bottom. These changes may be different at different values of the bottom inclination, so first, what is known about waves on slopes will be reviewed.

E1.1 No-breakwater situation

Before considering breakwaters, it may be very useful to look at what happens to waves in absence of a breakwater.

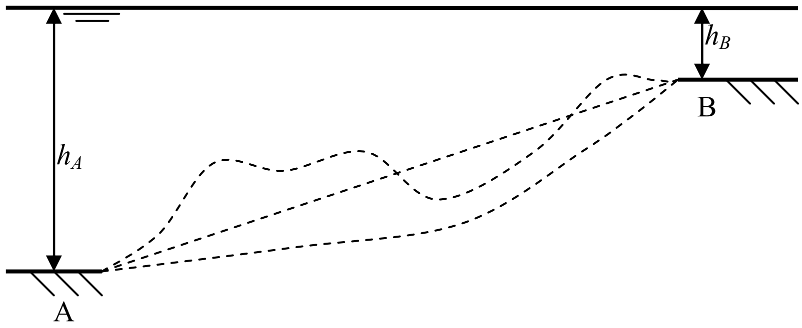
The most easy-to-use wave theory is the linear wave theory; see e.g. BATTJES (1997). This theory assumes small wave heights and sinusoidal waves. For waves far offshore, these conditions can be met; in this case, the waves behave as so-called "ideal" waves. They are internally characterised by a rotating water movement, the orbital movement. In deep water, the "water particles" describe an ideal circle, as in drawn in the figure below. The orbitals along which the water particles move, become smaller at greater depths. If the water is deep enough, the movement eventually fades out completely.



Orbital velocity

As the waves propagate into shallower water, they start to "feel" the bottom, as there is still some orbital movement close to the bottom. As the water cannot move through the bottom, the movement becomes flattened: the circles become ellipses. This also influences the movement of the water higher up, so eventually, the movement within the complete wave alters. As a result, wave heights, wavelengths, wave celerities and wave steepnesses will change; only the wave periods stay the same. As long as the water is deep enough, the linear theory is still able to describe this altered movement well.

The linear wave theory, although easy to use, does not take the bottom steepness into account. It only takes the water depth into account, regardless of the bottom slope, like in **Error! Reference source not found.**. Here, the depth is defined at locations A and B, but what lies in between them, is not defined, so each depth profile between A and B (three examples are drawn as dashed lines) may be possible (as long as waves don't break). Also, the horizontal and vertical scales may differ.



Different depth profiles

The alteration in the wave height is, in the linear wave theory, calculated using a shoaling coefficient K_s:

$$K_s \equiv \frac{H}{H_0}$$

in which:

K_s	shoaling coefficient	[-]
H	wave height	[m]
H_0	deep water wave height	[m]

More details on the calculation of the shoaling coefficient can be found in Appendix C. Although this formulation necessitates an iteration, the wave height at a certain depth can always be calculated if the wave period, the local water depth and the deep-water wave height are known. Notice that only the depth "counts", so what happens between the deep water and the desired location (including the slope between them) is not taken into account in this method.

E1.2 Wave breaking

Wave breaking occurs when the orbital speed of water "particles" becomes so large in the wave crests (the orbital speed being horizontal there), that they leave the wave profile. This can be caused by either the water becoming too shallow, or by the wave becoming too steep. MICHE (1944, quoted in D'ANGREMOND, 2001) derived some theoretical limits:

$$H/L < 0.14 \text{ and } H/h < 0.78 .$$

These limits apply for individual or regular waves. Very often, the maximum wave height is denoted as $H_{br} = \gamma_{br} h_{br}$, with

H_{br}	maximum wave height at the breaking point	[m]
γ_{br}	breaker parameter	[-]
h_{br}	water depth at the breaking point	[m]

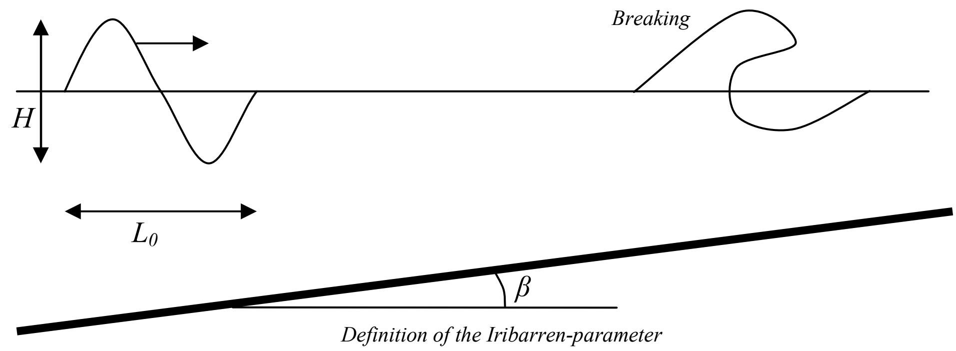
Irregular waves behave somewhat different from regular waves. For irregular waves, the breaker parameter often is applied on H_{m0} or H_{sig}, and is usually taken in the range 0.5-0.6.

The linear wave theory itself does not yield any information about the depth at which the waves break. In theory, they rise to infinite wave height for zero water depth, which is in reality of course impossible. This necessitates the usage of a parameter like γ_{br}. As will be shown later, linear wave theory describes the water motion quite well from deep water up to almost the breaking point. Beyond this, another formulation is necessary.

E1.2.1 Types of breakers

It is known from many studies (e.g. BATTJES (1997), also quoted in SCHIERECK, 2001) that the inclination of the sea bottom influences the way the waves break. This can be seen in the Iribarren-parameter:

$$\xi = \tan\alpha / \sqrt{s}$$



Definition of the Iribarren-parameter

in which:

β	sea bottom slope	[-]
H	wave height	[m]
L_0	deep water wave length	[m]

Although the limits are not sharp, roughly the following values can be indicated:

ξ>3	(i.e. very steep slopes or long waves)	surging waves
0.5< ξ<3	(i.e. steep slopes)	plunging waves
ξ<0.5	(i.e. mild or very mild slopes or short waves)	spilling waves

Some scientists discern a transitional class between surging and plunging: collapsing waves. This occurs for a ξ-value about equal to three.

For this situation, it is very clear that a major influence is present due to the steepness of the foreshore. Breaking occurs as an ultimate result of shoaling, i.e. the fact that the water becomes shallower. Shoaling starts already at relatively deep water. Deep-water conditions are valid as long as the water depth h is larger than ½*L. To give a numerical example: in a moderate wave with period T=7s, the limit is at 38m of water depth. If the depth is under this limit, shoaling will start. At first, the wave height will decrease somewhat (the lower limit is about 91% of the original wave height), which is a very gradual effect. After the wave height minimum, the wave height will start to rise and the shoaling becomes more and more pronounced as the waves approach the breaking point.

Now, it seems very unlikely that the essentially different behaviour at the breaking point develops exactly at the breaking point; waves are changing their properties before the breaking point already. This means that the wave properties should be different already at some distance before the breaking point, which could then also have an influence on the stability of a possible breakwater.

As will be shown later in this appendix, in breakwater design formulae, the Iribarren-parameter is used in quite an odd manner, if at all. This completely neglects the effect of (the steepness of) the foreshore.

There are no formulae that incorporate some term for the Iribarren-parameter on the foreshore or the steepness of it directly. Although a foreshore Iribarren-parameter doesn't say anything yet about the influence of its effect on a structure, it has already become clear that the waves could behave different on a mild or a steep slope. Therefore it is very interesting to investigate the influence of the Iribarren-parameter on the foreshore, which will be denoted as ξ_F, so:

$$\xi_F = \frac{\tan\beta}{\sqrt{H/L_0}}$$

E1.3 Wave height estimation

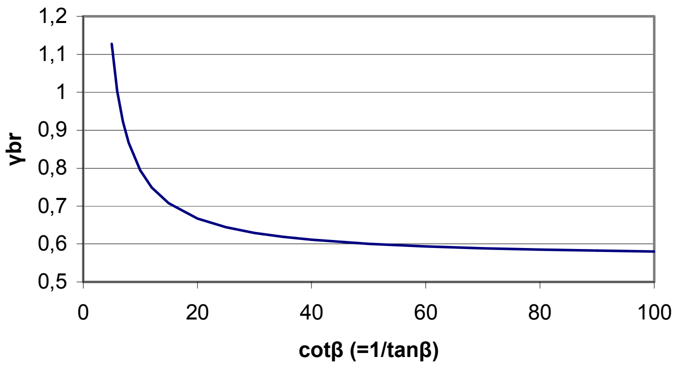
Another thing that is known from hydraulics is that waves on a steep slope tend to break later than waves on a mild slope, i.e. waves on a steep slope can grow higher before breaking than waves on a mild slope. This can be explained by the fact that the breaking of waves takes some time. If the slope is very mild, the waves have enough time to adapt to the new depth, so they are able to break. If the slope is steep, the change is far more abrupt, the waves cannot break "in time", and so they shoal up higher before breaking. Some formulations for breaking wave heights will be treated briefly in the next sections.

E1.3.1 Model of Kamphuis

KAMPHUIS describes an easy-to-use parameter for the breaker parameter, which includes an influence from the bottom steepness. He states:

$\gamma_{br} = 0.56e^{3,5\tan\beta}$.

If this parameter is plotted, it becomes clear immediately that the bottom slope has a strong influence if the slope is steep (i.e. the values of cotβ get small): see the figure below.



Breaking criterion according to Kamphuis

For cotβ-values in the range 30 and more, the influence of the bottom slope in the breaker parameter is weak; the lower limit is γ_{br}=0.56 and this increases to γ_{br}=0.62 for cotβ=30. As cotβ drops below 30, γ_{br} increases significantly, which may explain the larger load on structures exposed to waves on a steep foreshore. Unfortunately, it is not clear whether this breaker parameter has to be applied on regular or on irregular waves, and if on irregular waves, on which wave height.

Although this formula gives a nice first estimation of the breaking point, it still doesn't say anything special about what happens before the breaking point, as the linear wave theory is still applied here, neither does it say anything about the wave height distribution.

E1.3.2 Model of Battjes and Groenendijk

Once waves start breaking, they may come off the Rayleigh distribution, which is used to describe wave heights. This is caused by the fact that the highest waves break first, while the lower waves still propagate unbroken for some distance. Therefore the Rayleigh distribution is valid only if the water is deep enough ($h >\sim 3H_s$). Many authors have tried to establish a new distribution function.

One of the models is the model described by BATTJES and GROENENDIJK (2000). They describe a composite Weibull wave height distribution.

The problem of this model, as pointed out by the authors themselves, is that the resulting probability density function is not continuous, which is physically not realistic. Furthermore, this model uses difficult-to-interpret and difficult to calculate-shape-parameters, which makes it hard to use.

E1.3.3 Model of Mendez et al.

MENDEZ et al. (2004) try to overcome the shortcomings of the model of Battjes and Groenendijk, by introducing a new probability density function for the wave height distribution. They derive a dimensionless wave height and a distribution for it, which is dependent on a shape parameter. The model yields a method to calculate a $1/q$ wave height, i.e. the average of the $1/q$-highest waves, but this solution cannot be solved analytically. Mendez offers some numerically calculated values of a polynomial fitting, but these values unfortunately cannot be used in this research.

E1.3.4 Model of Allsopet al.

Also ALLSOP et al.(1998) et al. recognise that wave heights may change if waves break, depending on the bottom slope. They derived a set of simple formulae, but these formulae need some parameters, that cannot be derived analytically. Numerically, they derived a number of these parameters, for three different foreshore steepnesses, with which it is possible to calculate $H_{1/3}$ or H_{max}.

E1.3.5 Model of Tajima and Madsen

TAJIMA and MADSEN (2002) use a somewhat different method. They define a so-called equivalent linear wave, which retains the energy flux of the non-linear wave. After transformation, the linear theory can be applied for this equivalent linear wave. Tajima and Madsen use the Watanabe-breaking criterion, which uses, amongst other things, the bottom steepness as an input parameter. In the breaker zone, they use a dissipation model. After the inverse transformation, the wave properties of the real waves can be derived again. This method appears to yield a good result for regular waves. They also make the extension to irregular waves with some assumptions, but assume that the wave height distribution is still of the Rayleigh-type. This may be a doubtful assumption, although measurements seem to support their method.

E1.3.6 The Goda formulae

Another set of well-known formulae has been proposed by GODA (2000). These formulae calculate wave heights due to shoaling for both the significant waves as well as the, what Goda calls, maximum wave height (which is the $H_{1/250}$, i.e. the 1 in 250-wave), taking different values for the bottom slope as well as the wave set-up into account.

The values for the wave heights calculated from the Goda formulae are equal to the linear wave theory at locations where $h \geq 0.2L_0$. In shallower water, they diverge and Goda generally gives higher wave heights.

The disadvantage of the method of Goda, however, is that it has not one clear formula, but consists of several formulae, yielding a number of coefficients, in which it is very hard to discern what the influence is of one specific parameter. (A résumé of these formulae is printed in this report in Appendix C.) So, although the formulae look sophisticated and prove to work well in many situations, it is unfortunately hard to interpret the results.

E1.3.7 Model of Muttray and Oumeraci

MUTTRAY and OUMERACI (2000) did a review on a number of wave prediction formulae and tested them in an experiment. They found out that for shoaling, the value of H_{m0} can be well estimated using linear wave theory. For regular waves and for the maximum (i.e. $^1/_{250}$-) wave, a non-linear approximation gives the best result.

For calculating the breaking point, they found that an estimation for the breaking point as used by Goda works best. For H_{m0}, a scaling factor A equal to 0.10 should be used (see Appendix C). For the maximum (or $^1/_{250}$-) wave, A=0.15 should be used, while for regular waves A=0.17 can be used.

After breaking, H_{m0} is roughly equal to half the water depth; the maximum wave height is roughly equal to the maximum critical wave height. For regular waves, the wave height is in the range of 0.6-1.0 times the critical wave height.

Muttray and Oumeraci indicate that significant scatter occurs, especially of the H_{m0}-values, where the standard deviation may be up to 20%.

E1.4 Resumé of wave height formulae

From the theories above, it becomes clear that the shoaling of waves, propagating into shallower water, can be calculated very well using linear wave theory until some distance before the breaking point. After this, the formulae diverge somewhat, also dependent on which parameter is needed. So, if these wave heights, or better the spectra, are known, it should be possible to safely design a breakwater.

As seen from the Goda-formulae, the influence of the bottom steepness is not important as long as the water is deep enough. The linear wave theory can safely be used here. As the water grows shallower, another formulation may be necessary.

E2 ARMOUR STABILITY FORMULAE

For the design of armour stones on a breakwater, there are two major formulae: the Hudson-formula and the Van der Meer-formula, the latter in fact being a set of two formulae.

E2.1 Hudson-formula

The Hudson-formula is a relatively easy formula and therefore popular, although its application is restricted. In its basic form, it reads:

$$W \geq \frac{\rho_s g H^3}{\Delta^3 K_D \cot\alpha} \text{ or } M \geq \frac{\rho_s H^3}{\Delta^3 K_D \cot\alpha}$$

In this formula, the meanings of the parameters are:

W	necessary stone weight	[N]
M	stone mass	[kg]
ρ_s	density of stone	[kg/m^3]
Δ	relative underwater density $\equiv \frac{\rho_s - \rho_w}{\rho_w}$	[-]
K_D	breakwater coefficient	[-]
α	slope angle of the breakwater	[-]
g	acceleration of gravity	[m/s^2]

In this, K_D is a constant that incorporates all effects that are not in the formula, like the armour type. This constant has a wide range of values. Furthermore, the definition of H varies. Originally, this was the H_s, so the significant wave height, but later, the Shore Protection Manual and its successor, the Coastal Engineering Manual, altered this to $H_{1/10}$, so the average of the 1/10-th highest waves, being 27% higher than H_s. (For more information concerning the application and validity ranges of the Hudson-formula, see USACE, 1984 and 2002)

Apart from the parameters mentioned above, the Hudson formula does not take any other parameter into account. As a result, it is not useful for the present research as the bottom slope is not present in the formula.

E2.2 Van der Meer-formula

After extensive research, Van der Meer introduced a new design formula for breakwaters, which is widely accepted and used in hydraulic engineering. This formula takes more parameters into account, like the damage level. It also discerns for plunging and surging breakers on the breakwater, in the form of an Iribarren-parameter on the breakwater. The formula (or in fact: the two formulae) reads:

$$\frac{H_s}{\Delta d_{n50}} = 6.2P^{0.18}\left(S/\sqrt{N}\right)^{0.2} 1/\sqrt{\xi_{\alpha 0}} \qquad \text{for plunging breakers } (\xi_{\alpha 0}<\xi_{\alpha,crit})$$

$$\frac{H_s}{\Delta d_{n50}} = 1.0P^{-0.13}\left(S/\sqrt{N}\right)^{0.2} \sqrt{\cot\alpha}\cdot\xi_{\alpha 0}^{P} \qquad \text{for surging breakers } (\xi_{\alpha 0}<\xi_{\alpha,crit})$$

The critical value $\xi_{\alpha,crit}$ can easily be found:

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

with:

H_s	significant wave height $\equiv H_{1/3}$	[m]
d_{n50}	nominal diameter $\equiv \sqrt[3]{\frac{M_{50}}{\rho_s}}$	[m]
M_{50}	median stone mass	[kg]
P	notional permeability	[-]
S	damage number $\equiv \frac{A_e}{d_{n50}^2}$	[-]
A_e	cross section of the erosion area	[m^2]
N	number of waves	[-]
$\xi_{\alpha 0}$	Iribarren-parameter using deep water steepness and breakwater slope $\equiv \frac{\tan\alpha}{\sqrt{H_0/L_0}}$	[-]

So, in these formulae, the wave steepness is taken into account, but it is applied in a rather remarkable manner: it is incorporated through the Iribarren-parameter ξ, which normally uses the steepness of the slope and the wave steepness. However, in the Van der Meer-formula, the Iribarren-parameter is calculated by using the deep water wave steepness and the inclination of the slope of the breakwater. Van der Meer could do so, as he performed his tests with relatively deep water, but as the water gets shallower and shoaling or even breaking is more present, this is a very questionable assumption, as the steepness of the waves at the toe has no clear relation with the offshore steepness anymore. The deep-water assumption also implies that the bottom slope is not an important issue in this method. So, although the Van der Meer-formula reveals more details, there is still no expression for the influence of the foreshore wave steepness.

The formulae above have been modified for different stones and for special armour units. For more information on this, the reader is referred to CUR, 1994.

E2.3 Van Gent et al. and Smith et al.

VAN GENT et al. (2003) pointed out that the Van der Meer formula is based for the largest part on measurements in deep water. Therefore they performed a model investigation to the effect of shallow water. They propose to use a different definition for the wave period: $T_{m-1,0}$. This wave period is defined by $T_{m-1,0} = \frac{m_{-1}}{m_0}$. In SMITH et al. (2002) it was shown that using this period gives a better fit of the Van der Meer-formula to their experiments and with a smaller standard deviation.

They also propose a new formula, which excludes the wave period and the notional permeability, but introduces a new parameter for the permeability, by using the diameter of the core material, neglecting the filter. The formula reads:

$$\frac{S}{\sqrt{N}} = \left(0.57 \frac{H_s}{\Delta D_{n50}} \sqrt{\tan\alpha} \frac{1}{1 + D_{n50-core}/D_{n50}} \right)^5$$

All parameters in this formula are the same as in the Van der Meer-formula, but with the addition of:

$D_{n50-core}$ nominal diameter of the core material [m]

The ratio $D_{n50-core}/D_{n50}$ replaces the notional permeability of Van der Meer.

This formula appeared to yield values that were as accurate as the standard Van der Meer-formula. However, this relation was tested for values of $D_{n50-core}/D_{n50}$ up to 0.3, so this formulation cannot be used yet for e.g. homogeneous breakwaters.

E1.4 Wave forces on stones

HALD and BURCHARTH (2000) tried to establish an alternative stability formula for stones on breakwaters, based on the forces that act on a stone. They did a model research and investigated both the magnitude force and direction of the force. They used these results to create both a Hudson-like and a Van der Meer-like formula. (For details, the reader is referred to the publication.) Their experiment did not take the influence of the change in waves due to a bottom slope into account, but could be useful as a starting point for further investigations.

APPENDIX F: EXPERIMENT EVALUATION

F1 ANALYSIS OF THE TEST RESULTS FOR RIPRAP

In this Appendix, the results of the analysis will be given in more detail. The analysis is the same as in paragraph 4.2, only the results will be given here.

F1.1 Results for waves with T_p=1.60s

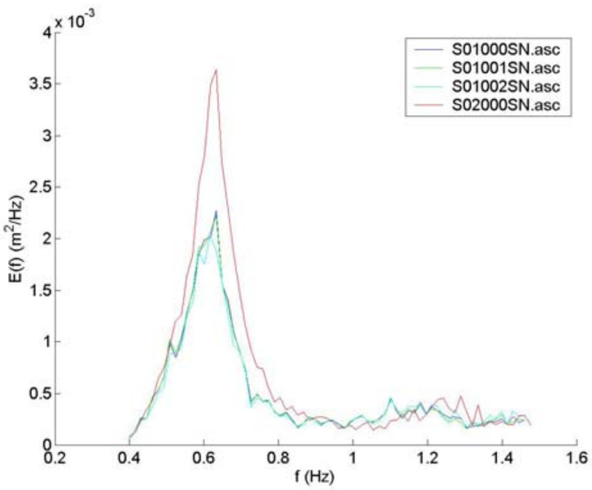
Initially, the following waves were used:

Parameter	Value	Meaning
T_p	1.60s	spectral peak period
H_b	0.110m	wave height at the wave board
H_0	0.121m	(theoretical) deep water wave height
s_0	0.030	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

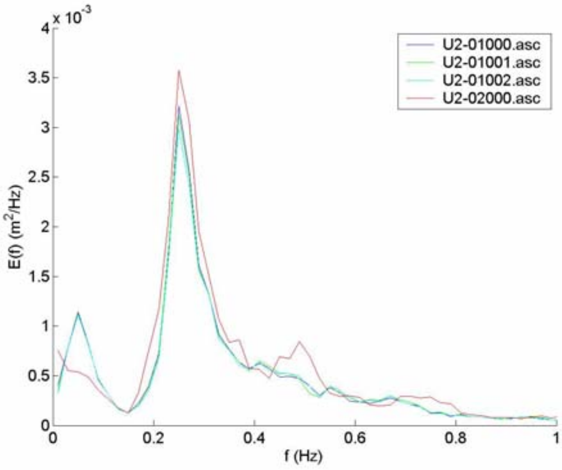
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01000	29.1	9.36	8.27	T02000	35.1	10.80	9.09	+25.0
T01001	27.8	9.35	8.32					
T01002	27.2	9.24	8.20					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the wave board, T_p=1.60s



Comparison of undecomposed spectra with the same wave heights at the wave board, T_p=1.60s

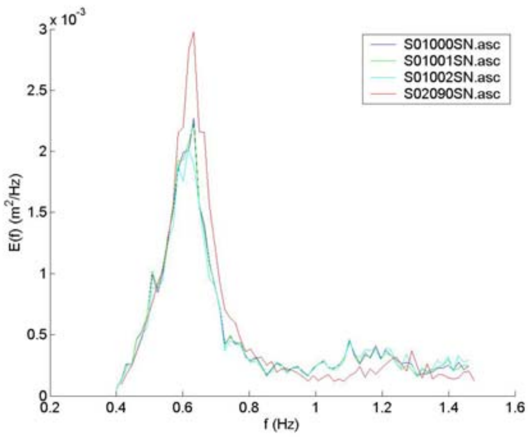
The second comparison is with the same wave heights at the toe. Here, the wave program at the steep foreshore was adapted as follows:

Parameter	Value	Meaning
T_p	1.60s	spectral peak period
H_b	0.096m	wave height at the wave board
H_0	0.105m	(theoretical) deep water wave height
s_0	0.026	(theoretical) deep water steepness, using H_0 and *Tp*
N	1000	number of waves

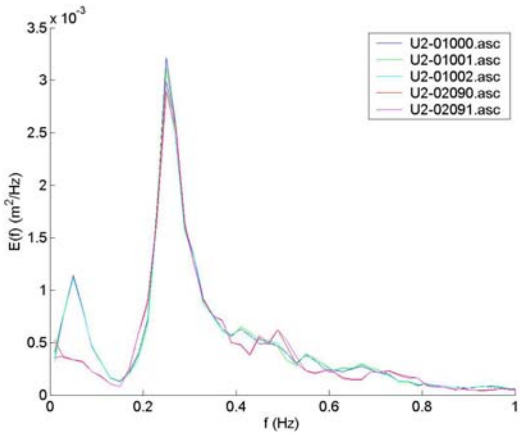
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01000	29.1	9.36	9.24	T02090	27.3	9.68	8.73	+0.0
T01001	27.8	9.35	9.22	T02091	28.8	9.71	8.79	
T01002	27.2	9.24	9.10					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the toe, T_p=1.60s



Comparison of undecomposed spectra with the same wave heights at the toe, T_p=1.60s

F1.2 Results for waves with T_p=1.31s

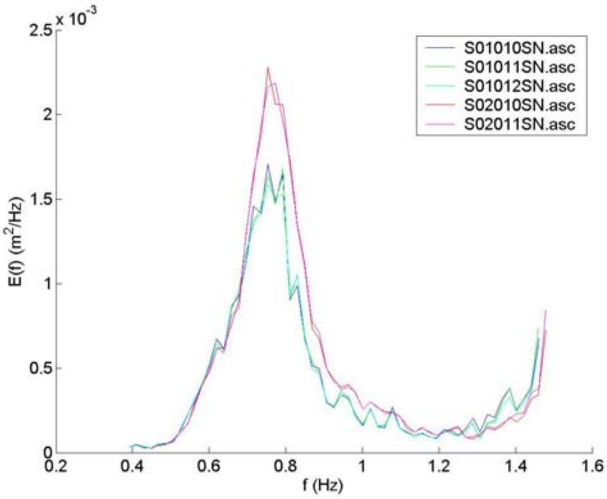
Initially, the following waves were used:

Parameter	Value	Meaning
T_p	1.31s	spectral peak period
H_b	0.110m	wave height at the wave board
H_0	0.118m	(theoretical) deep water wave height
s_0	0.044	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

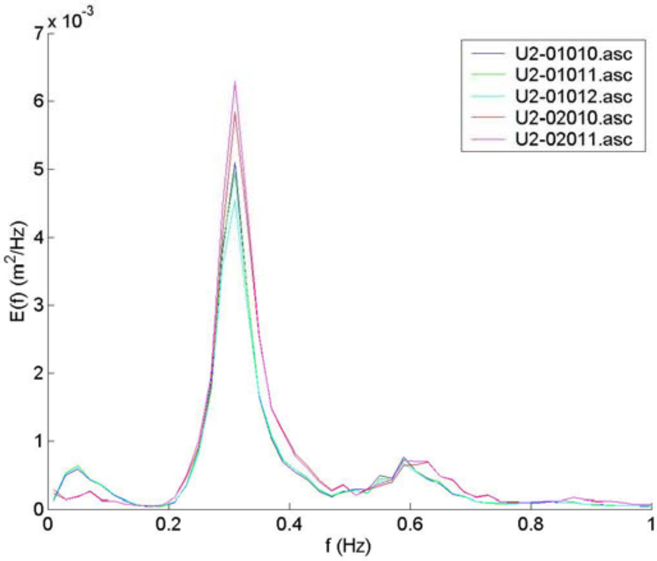
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01010	18.6	8.58	9.10	T02010	28.6	9.31	10.01	+44.9
T01011	19.4	8.57	9.10	T02011	27.4	9.35	10.25	
T01012	19.8	8.46	8.89					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the wave board, T_p=1.31s



Comparison of undecomposed spectra with the same wave heights at the wave board, T_p=1.31s

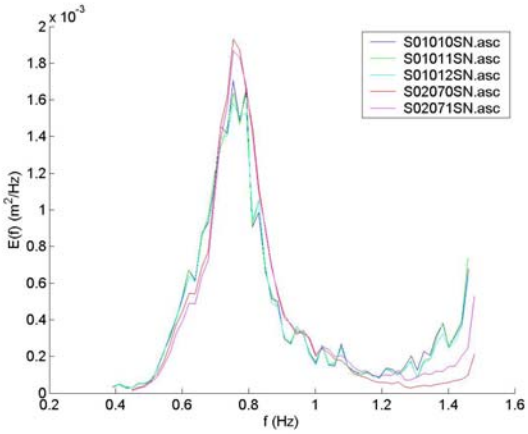
The second comparison is with the same wave heights at the toe. Here, the wave program at the steep foreshore was adapted as follows:

Parameter	Value	Meaning
T_p	1.31s	spectral peak period
H_b	0.096m	wave height at the wave board
H_0	0.104m	(theoretical) deep water wave height
s_0	0.039	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

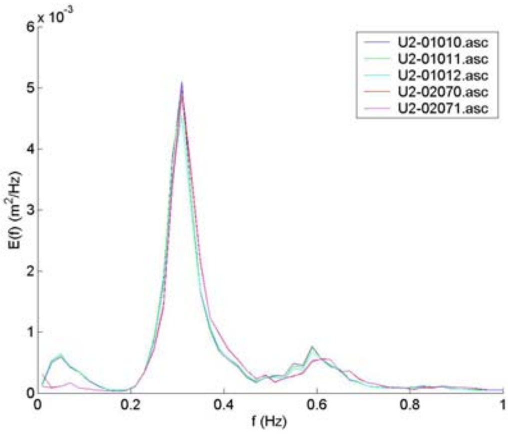
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01010	18.6	8.58	9.10	T02070	21.2	8.31	9.01	+11.3
T01011	19.4	8.57	9.10	T02071	21.7	8.43	9.15	
T01012	19.8	8.46	8.89					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the toe, T_p=1.31s



Comparison of undecomposed spectra with the same wave heights at the toe, T_p=1.31s

F1.3 Results for waves with T_p=1.13s

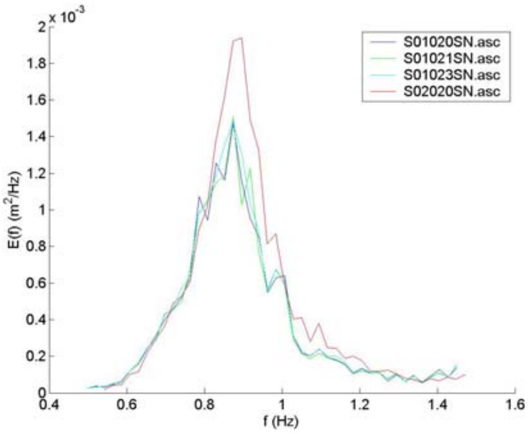
Initially, the following waves were used:

Parameter	Value	Meaning
T_p	1.13s	spectral peak period
H_b	0.111m	wave height at the wave board
H_0	0.116m	(theoretical) deep water wave height
s_0	0.058	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

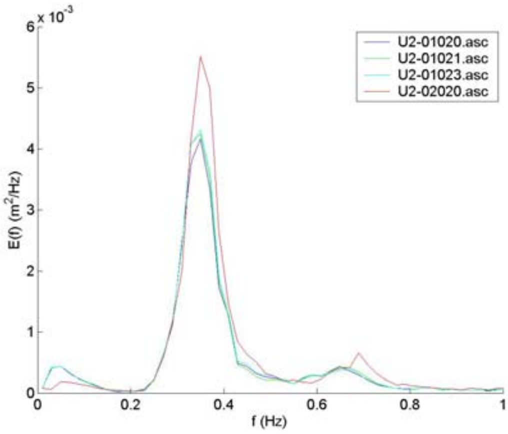
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01020	12.8	7.67	8.90	T02020	21.6	8.65	9.91	+64.4
T01021	13.0	7.66	8.96					
T01023	13.6	7.79	9.06					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the wave board, T_p=1.13s



Comparison of undecomposed spectra with the same wave heights at the wave board, T_p=1.13s

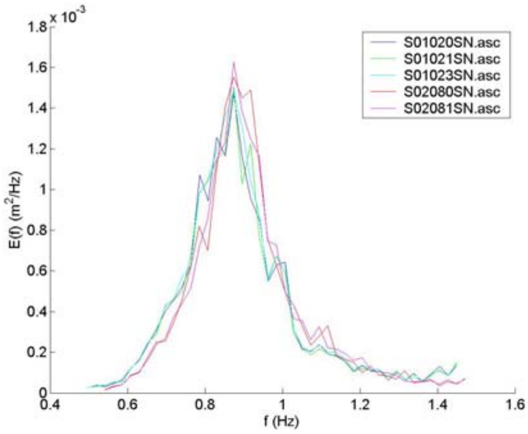
The second comparison is with the same wave heights at the toe. Here, the wave program at the steep foreshore was adapted as follows:

Parameter	Value	Meaning
T_p	1.13s	spectral peak period
H_b	0.098m	wave height at the wave board
H_0	0.102m	(theoretical) deep water wave height
s_0	0.051	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

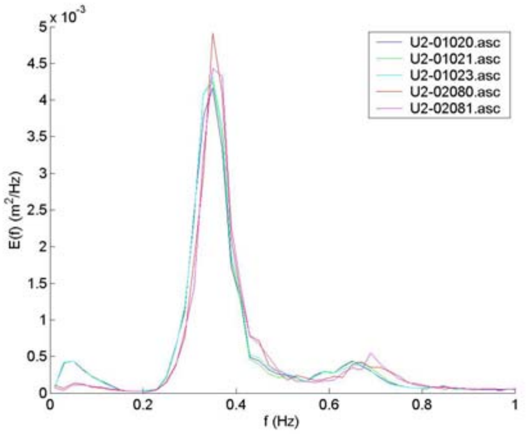
This gave the following results:

Mild foreshore				Steep foreshore				Difference
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Average N_{od}
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01020	12.8	7.67	8.90	T02080	18.0	7.74	9.03	+35.2
T01021	13.0	7.66	8.96	T02081	17.5	7.70	9.02	
T01022	13.6	7.79	9.06					

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the toe, T_p=1.13s



Comparison of undecomposed spectra with the same wave heights at the toe, T_p=1.13s

F1.4 Results for waves with T_p=0.92s

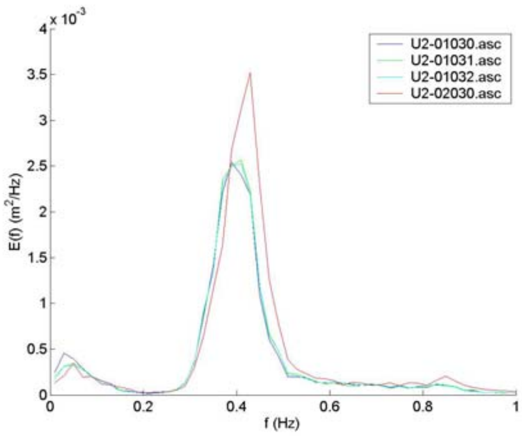
Initially, the following waves were used:

Parameter	Value	Meaning
T_p	0.92s	spectral peak period
H_b	0.113m	wave height at the wave board
H_0	0.114m	(theoretical) deep water wave height
s_0	0.086	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

This gave the following results:

Mild foreshore				Steep foreshore				Difference Average N_{od}
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01030	5.6	6.48	7.37	T02030	12.4	7.33	8.27	+121.5
T01031	5.8	6.46	7.49					
T01032	5.4	6.54	7.46					

In this case, the spectrum for the incoming waves had a very strange shape. This has been caused by the fact that the wave program was too short. Also, there was a limitation from the decomposition method, which cut off the spectrum. Therefore, these spectra will not be used here. The spectra for the decomposed waves have been established however, and printed below:



Comparison of undecomposed spectra with the same wave heights at the wave board, T_p=0.92s

The second comparison is with the same wave heights at the toe. Here, the wave program at the steep foreshore was adapted as follows:

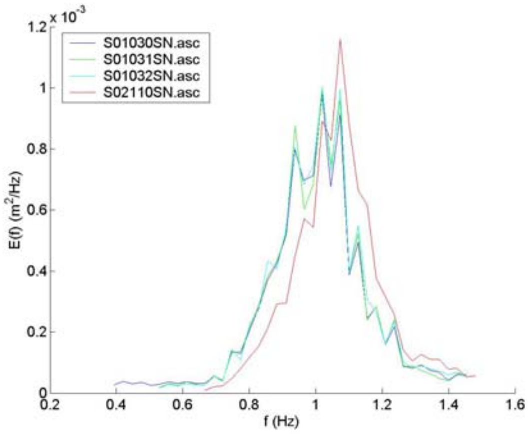
Parameter	Value	Meaning
T_p	0.92s	spectral peak period
H_b	0.094m	wave height at the wave board
H_0	0.095m	(theoretical) deep water wave height
s_0	0.072	(theoretical) deep water steepness, using H_0 and Tp
N	1000	number of waves

This gave the following results:

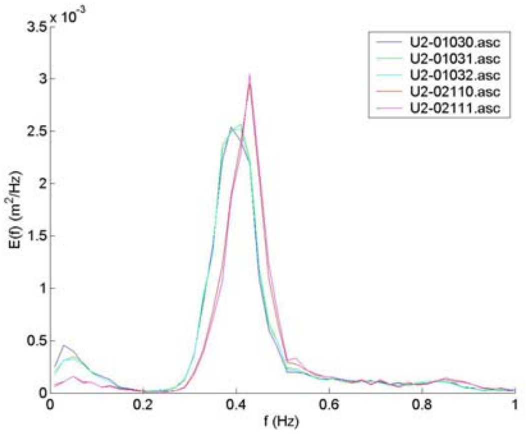
Mild foreshore				Steep foreshore				Difference Average N_{od}
Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	Testnr.	N_{od}	$H_{m0,t}$	$H_{m0,u}$	
[-]	[-]	[cm]	[cm]	[-]	[-]	[cm]	[cm]	[%]
T01030	5.6	6.48	7.37	T02110	7.8	6.49	7.33	+32.1
T01031	5.8	6.46	7.49	T02111	7.0		7.34	
T01032	5.4	6.54	7.46					

(The calculation of the value $H_{m0,t}$ at test T02111 gave an error in the calculation and has therefore been omitted here.)

The following spectra were established:



Comparison of incoming spectra with the same wave heights at the toe, T_p=0.92s



Comparison of undecomposed spectra with the same wave heights at the toe, T_p=1.13s

F1.5 Résumé of the results with equal wave conditions at the wave board

The fact that the damage is higher on the steep foreshore did not come as a surprise. It is known from the Goda-formulae, that the average wave height close to the breaking point is higher in the case of a steep foreshore, compared to the mild foreshore. If the wave height is higher, one would expect a higher damage level. The measurements described above confirm this idea. This means that if a breakwater designer faces a steep foreshore at the breakwater or sea defence he is designing, he should pay special attention to the area where the foreshore is steep.

These results clearly show that a bottom slope does matter. It is demonstrated that damage levels may be significantly higher.

F1.6 Résumé of the results with equal wave conditions at the toe

For all the wave conditions that were considered above, the spectra had more or less the same shapes and numerical values for each set of measurements. Yet, the damage levels were sometimes dozens of percents higher in the steep foreshore case.

It makes sense to assume that the wave conditions at the toe determine the damage level of the breakwater. So the conclusion is that the spectra at the toe alone don't give enough information to calculate the damage to an armour layer.

F2 OTHER WAVE DESCRIPTORS

In this section, the wave steepness-descriptors, as defined in paragraph 4.2.3 are printed.

F2.1 Wave front-steepness-values

Below are the R- and Q-values for all relevant tests. The values that are in grey are the values that were used in Figure 4-7.

Testnr.	R_1	R_2	R_3	R_4	Q_1	Q_2	Q_3	Q_4
T01000	0.227	0.296	0.350	0.412	0.189	0.247	0.291	0.343
T01001	0.229	0.299	0.351	0.412	0.191	0.249	0.292	0.344
T01002	0.224	0.289	0.338	0.395	0.187	0.241	0.282	0.329
T02000	0.312	0.460	0.590	0.723	0.260	0.384	0.491	0.603
T02090	0.231	0.354	0.471	0.604	0.192	0.295	0.392	0.503
T02091	0.228	0.348	0.461	0.580	0.190	0.290	0.384	0.483
T01010	0.214	0.274	0.323	0.372	0.183	0.234	0.275	0.316
T01011	0.215	0.274	0.323	0.373	0.183	0.233	0.275	0.317
T01012	0.214	0.271	0.318	0.366	0.182	0.231	0.271	0.312
T02010	0.294	0.419	0.529	0.650	0.251	0.357	0.451	0.554
T02011	0.296	0.423	0.536	0.657	0.252	0.360	0.456	0.560
T02070	0.240	0.343	0.442	0.559	0.205	0.292	0.377	0.476
T02071	0.235	0.339	0.436	0.540	0.200	0.288	0.372	0.460
T02050	0.233	0.333	0.424	0.514	0.201	0.287	0.365	0.443
T02051	0.232	0.323	0.408	0.494	0.200	0.278	0.351	0.425
T01020	0.201	0.257	0.304	0.349	0.176	0.224	0.265	0.304
T01021	0.208	0.266	0.314	0.360	0.181	0.232	0.274	0.314
T01022	0.211	0.272	0.324	0.369	0.184	0.237	0.282	0.322
T01023	0.200	0.256	0.303	0.348	0.175	0.223	0.264	0.304
T02020	0.271	0.375	0.468	0.559	0.236	0.327	0.408	0.487
T02080	0.234	0.328	0.417	0.500	0.204	0.286	0.363	0.436
T02081	0.231	0.323	0.409	0.494	0.201	0.282	0.357	0.430
T01030	0.163	0.211	0.255	0.297	0.150	0.193	0.233	0.272
T01031	0.165	0.211	0.253	0.290	0.151	0.193	0.231	0.265
T01032	0.161	0.208	0.252	0.291	0.148	0.190	0.230	0.266
T02110	0.180	0.246	0.314	0.388	0.165	0.225	0.287	0.354
T02111	0.180	0.244	0.310	0.384	0.165	0.223	0.284	0.351

F2.2 Peakedness-values

Below are the relevant values for H_{spike}. The values in grey are the values that were used in Figure 4-8.

Testnumber	$\overline{H^3} / \overline{H^2}$
T01000	0.0872
T01001	0.0871
T01002	0.0860
T02000	0.1011
T02090	0.0903
T02091	0.0910
T01010	0.0875
T01011	0.0878
T01012	0.0857
T02010	0.1023
T02011	0.1046
T02070	0.0935
T02071	0.0939
T01020	0.0833
T01021	0.0841
T01022	0.0860
T01023	0.0843
T02020	0.0965
T02080	0.0898
T02081	0.0890
T01030	0.0690
T01031	0.0696
T01032	0.0696
T02030	0.0786
T02110	0.0705
T02111	0.0701