Delft University of Technology

FACULTY OF CIVIL ENGINEERING AND GEOSCIENCES
DEPARTMENT OF HYDRAULIC ENGINEERING

Academic year 2007–2008

ARMOUR LAYER STABILITY ON A BERMED SLOPE BREAKWATER

OSCAR P.J. DIJKSTRA

A thesis submitted in partial fulfillment of the requirements
for the degree of
MASTER OF SCIENCE IN HYDRAULIC ENGINEERING

Preface

The final section of the Masters study program at Delft University of Technology consists of an individual thesis on a subject related to the student's specialization. This report is an overview of the results of an investigation on the thesis subject: *Stability of breakwater armour layers on a bermed slope under wave attack*. The main research objective is to develop insight on the influence of the governing parameters of a berm on the stability of breakwater armour layers.

This study, existing of a research of the literature and small-scale experiments, is performed in close collaboration with the engineering department of Van Oord. The experiments were executed in the Laboratory of Fluid Mechanics at Delft University of Technology. I would like to thank both departments for offering the facilities and financial support needed to accomplish this study.

During this period a graduation committee, consisting of the following persons, supervised the thesis work, for which I am very grateful;

prof.dr.ir. M.J.F. Stive	Delft University of Technology
ir. H.J. Verhagen	Delft University of Technology
dr.ir. W.S.J. Uijttewaal	Delft University of Technology
ir. G.B.H. Spaan	Van Oord
dr.ir. M.R.A. van Gent	WL|Delft Hydraulics

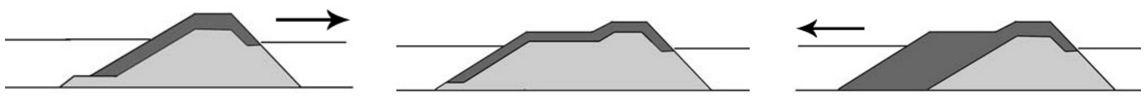
Oscar Dijkstra
Delft, February 2008

Summary

In coastal regions where the land is sensitive to erosion, but also for harbours and ports, numerous types of defensive structures have been developed to protect it from the effects of incident waves.

Most common are uniform sloped breakwaters, build-up of a rubble mound core and a two-layered armour layer, Figure 1a. An other type is a 'berm breakwater', Figure 1c. The berm reduces the impact and run-up of the incident waves and the slope may deform to a S-curved profile. Therefore it requires smaller armour stones then necessary for uniform sloped breakwaters. For both types of breakwaters design formulae or design guidelines are available.

More recently breakwaters have been constructed which combine the stability characteristics of a conventional two-layered uniform sloped breakwater with the wave energy dispersive character of a berm breakwater. This type is referred to as a 'breakwater with a bermed slope' or a 'bermed sloped breakwater', Figure 1b.



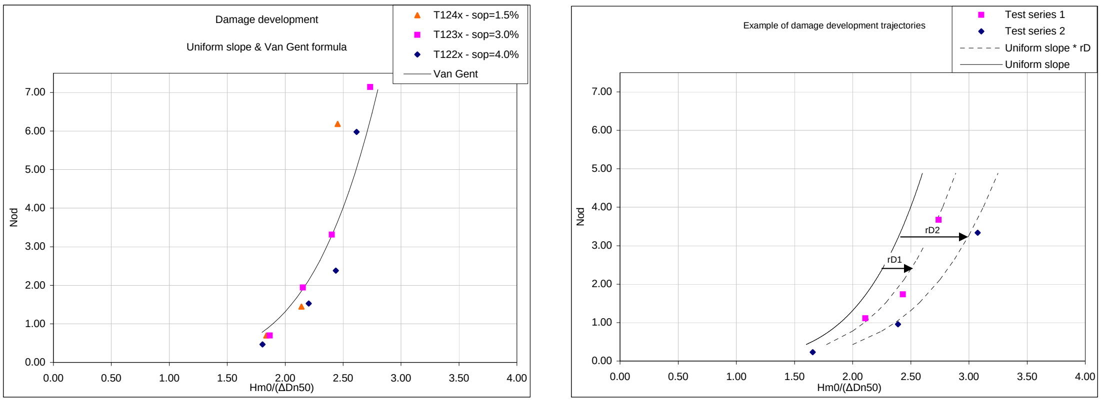
(a) (b) (c)

Figure 1: Cross-section breakwaters

The hydraulic load due to waves on a bermed slope are not similar to the hydraulic load on an uniform slope. The stability of the armour layer of a bermed sloped breakwater can therefore not be predicted with conventional design formulae developed for uniform sloped breakwater. Furthermore, the design guidelines developed for berm breakwaters are also not applicable as they evaluated damage through the assessment of the reshaping profile. Therefore design of bermed sloped breakwaters is based on experience without fully understanding the influence of the governing parameters of a berm, on the stability of breakwater armour layers. Consequently this doesn't always result in the best possible design.

In order to develop more insight in the development of damage on bermed sloped breakwaters, small-scale experiments have been performed in which the two most important parameters related to a berm were tested. These governing parameters were the relative berm length and the relative water level with a range of respectively $0.00 < B/L_{m-1,0} < 0.35$ and $-0.8 < R_c/H_{m0} < 0.7$.

In order to validate the recorded damage, assessment by means of the number of displaced stones, with the predicted damage by the stability formulae small-scale experiments were performed on an uniform sloped breakwater. Three stability formulae were compared; Hudson (1953, 1959), Van der Meer (1988) and Van Gent *et al.* (2003). All predicted the development of damage rather well. However, as the influence of the wave period on the development of damage was unclear, it was chosen to use the formula of Van Gent *et al.* (2003) to express the damage on an uniform slope, Figure 2a.



(a) Stability formula of Van Gent

(b) Stability increase factor

Figure 2

On the development of damage on a bermed sloped breakwater the following conclusions were made:

1. The development of damage on a bermed slope has a similar, but more stable, trajectory as predicted for an uniform slope with the stability formula of Van Gent. The increase in stability can be indicated with a constant factor (r_D), Figure 2b.

2. The damage level parameter for 'start of damage' and 'failure' on a bermed slope are independent of the governing parameters and correspond to the values of an uniform sloped breakwater.

For all test series the increase in stability factors and the governing parameters were determined for the damage level 'start of damage'. Unfortunately damage was mainly created on the lower slope and berm and hardly any on the upper slope. Therefore further analysis only focused on the lower slope and berm.

All test series showed an increase in stability with regard to an uniform slope as predicted by the formula of Van Gent. For an interpretation of the influence of the governing parameters on the stability the test results are plotted in Figure 3. Conceivable trendlines are drawn for constant values of R_c/H_{m0} and B/L_{m0}.

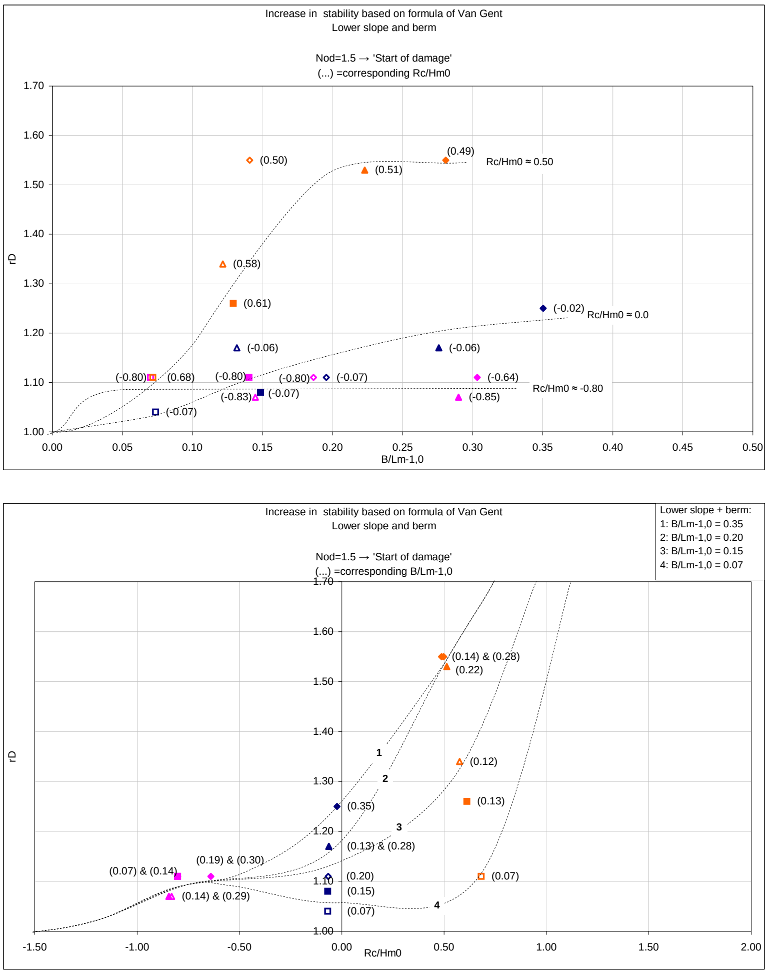


Figure 3: Influence of the governing parameters with conceivable trendlines

On the influence of the governing parameters the following conclusions were made:

1. When regarding constant values of R_c/H_{m0} the test results show, for initial values of the relative berm length, an increase in stability as $B/L_{m-1,0}$ increases. As the relative berm length gets increasingly larger, the increase in stability indicates a horizontal limit.

2. The range for which the relative berm length has a positive contribution on the increase in stability is strongly related to the relative water level. For $R_c/H_{m0} < 0$ this range is small but it widens quickly as the water level approaches the berm level. As the water level on the berm increases, it gradually becomes smaller again.

A research by WL|Delft Hydraulics (Vermeer (1986)), which performed similar tests but for $R_c/H_{m0} > 0.9$, showed different results. The development of the increase in stability for constant values of the relative berm length showed a peak at $B/L_o = 0.15$, Figure A.1. This peak seemed to flatten out as the water level on the berm became smaller. This process could possible link the findings of this study to the findings of Vermeer (1986), however, more research has to be done to confirm this hypothesis.

Finally the two design principles, which in practice are used as indication of the increase in stability, were validated with the results of this study, Figure 4. The first principle applies stability formulae for uniform slopes on the average slope of a bermed profile. The second principle adopts the characteristics of low-crested structures on the bermed profile.

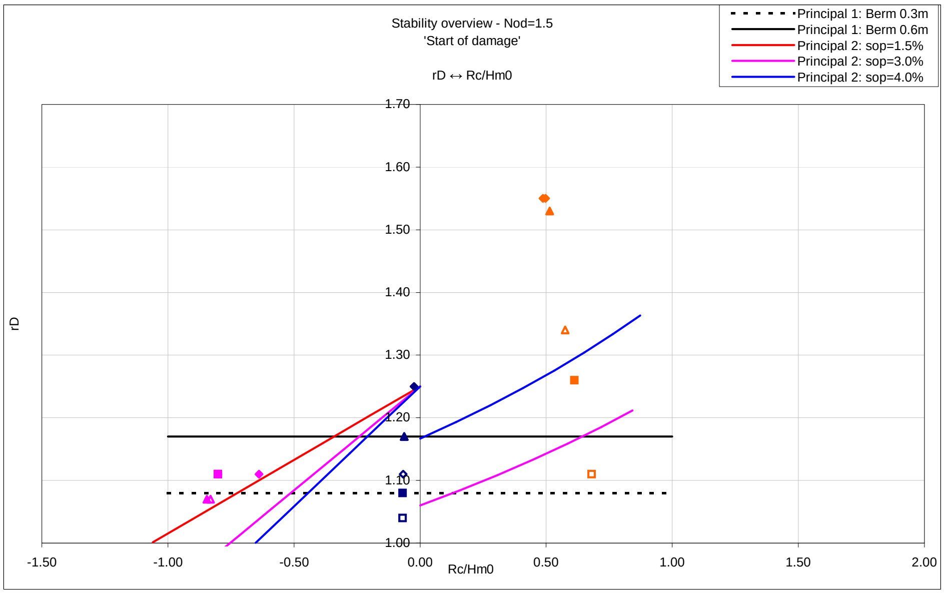


Figure 4

The correlation between the predicted increase in stability and the results of the test series was very low. Apparently the complexity of the processes related to the (in)stability of armour layers on a bermed slope can not be overcome by means of the design principles. This is most probably caused by the influence of the return current, which has large impact on the stability, is not accounted for. Therefore both principles are not well suited to predict the increase in stability of armour layers on bermed slopes.

List of Symbols

A_e	Erosion area on rock profile	(m^2)
B	Berm width	(m)
D	Height of armour stone	(m)
D_n	Nominal block diameter, $D_n = (M/\rho_r)^{1/3}$	(m)
D_{n50}	Median nominal diameter, $D_{n50} = (M_{50}/\rho_r)^{1/3}$	(m)
D_{n85}	85% value of sieve curve	(m)
D_{n15}	15% value of sieve curve	(m)
d	Structure height relative to bed level	(m)
g	Gravitational acceleration	(m/s^2)
H	Wave height, from trough to crest	(m)
$H_{2\%}$	Wave height exceeded by 2% of waves	(m)
H_{m0}	Significant wave height calculated from the spectrum, $H_{m0} = 4\sqrt{m_0}$	(m)
H_s	Significant wave height	(m)
h	Water depth; water depth at structure toe	(m)
i	Hydraulic gradient of (phreatic) water level	(-)
K_D	Stability coefficient, Hudson formula	(-)
k	Wave number, $k = 2\pi/L$	(-)
L	Wavelength, in the direction of propagation	(m)
L_o	Deep-water wavelength, $L_o = gT^2/2\pi$	(m)
L_{om}	Deep-water wavelength of mean period, T_m	(m)
L_{op}	Deep-water wavelength of peak period, T_p	(m)
L_m , L_p	Wavelength at structure toe, based on T_m and T_p	(m)
$L_{m-1,0}$	Deep-water wavelength of spectral wave period, $T_{m-1,0}$	(m)
M	Mass of an armour unit	(kg)
M_{50}	Mass of particle for which 50% of the granular material is lighter	(kg)
N	Number of waves over the duration test	(-)
N_d	Number of armour units displaced in area considered	(-)
N_{od}	Number of displaced units per width D_n across armour face	(-)
N_s	Stability number, $N_s = H_s/(\Delta D_{n50})$	(-)

p	Porosity of the rock	(-)
r_D	Increase in stability factor	(-)
r_D	Reduction factor of armour stones size	(-)
S_d	Non-dimensional damage, $S_d = A_e/D_{n50}^2$	(-)
s	Wave steepness, $s = H/L$	(-)
s_o	Wave steepness, defined as $H_s/L_o = 2\pi H_s/(gT_m^2)$	(-)
s_{om}	Wave steepness for mean period wave, $s_{om} = 2\pi H_s/(gT_m^2)$	(-)
s_{op}	Wave steepness for peak period wave, $s_{op} = 2\pi H_s/(gT_p^2)$	(-)
s_p	Wave steepness at toe for peak period wave, $s_p = H_s/L_p$	(-)
$s_{m-1.0}$	Wave steepness for mean energy period, $s_{m-1.0} = 2\pi H_{m0}/(gT_{m-1.0}^2)$	(-)
T	Wave period	(s)
T_m	Mean wave period	(s)
$T_{m-1,0}$	Mean energy wave period or spectral wave period,	(s)
T_p	Spectral peak period, inverse of peak frequency	(s)
α	Structure slope angle	(°)
Δ	Relative buoyant density of material, ie for rock $\Delta = (\rho_r - \rho_w)/\rho_w$	(-)
ξ	Surf similarity parameter or Iribarren number, $\xi = \tan\alpha/\sqrt{s_o}$	(-)
ξ_{cr}	Critical surf similarity parameter, Van der Meer formula	(-)
ξ_m	Surf similarity parameter or Iribarren number for mean wave period Tm	(-)
ρ	Mass density	(kg/m^3)
ρ_r	Mass density of rock	(kg/m^3)
ρ_w	Mass density of water	(kg/m^3)
ω	Angular frequency of waves, $\omega = 2\pi/T$	(1/s)

Contents

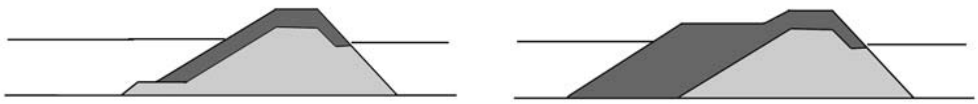
Chapter 1

Introduction

1.1 General

In coastal regions where the land is sensitive to erosion, but also for harbours and ports, numerous types of defensive structures have been developed to protect it from the effects of incident waves and longshore drifts. One of these defensive structures is a breakwater.

A wide range of different types of breakwaters exist, but most common is the rubble mound type. In essence a rubble mound breakwater exists out of a rubble core protected on the outside with a heavier layer, the armour layer. Generally this layer is constructed from heavier stones but specially designed concrete elements are also possible.



(a) Uniform slope (b) Bermed slope

Figure 1.1: Cross-section breakwater

Great advantage of this type of breakwater is that they are relatively simple and therefore relatively cheap to build. Also repair works are straight foreword and due to settlements of the armour layer the construction has a self repairing capability. A drawback is the quadratic increases of material as the depth increases.

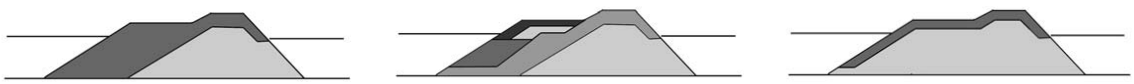
In general the seaward side of a breakwater is uniform sloped, Figure 1.1a. Construction of this type of breakwater is straight foreword and therefore often the most economic solution. However, in some circumstances it can be more economic to construct a bermed slope instead of a uniform slope, Figure 1.1b. The berm reduces the impact and run-up of the incident waves and therefore requires smaller armour stone then necessary for uniform sloped breakwaters.

1.2 Stability formulae

On the stability of breakwater armour layers lot of research has been done which concentrate on the stability of stones on a slope. In this a distinguish can be made between stability due to currents, waves or a combination of the two. The impact of the incident waves is most often normative for the design and therefore this study will only concentrate on this type of hydraulic loading.

The complexity of the process underlying the (in)stability of individual stones created by the water movement of breaking waves on a slope make it very difficult to express the interaction of forces into a theoretical model. Therefore a number of empirical formulae have been developed, all based on results of small-scale experiments. In these formulae stability of individual stones are often defined by a ratio of the unit size (weight/length scale) and the wave height.

Greater part of these formulae describe the stability of stones on an uniform non-overtopped slope, but also for other types of structures like low-crested, submerged and reef breakwaters formulae have been developed. In practice these structures are often designed with a rubble mound core protected by a two-layer armour layer which is required to be almost statically stable for design wave conditions.



(a) Berm breakwater, reshaping (b) Berm breakwater, Icelandic (c) Breakwater with bermed slope

Figure 1.2: Bermed sloped breakwater

On breakwaters with a bermed slope research has mainly been focused on 'berm breakwaters' resulting in several design guidelines presented by PIANC (2003). This type of breakwater offer great flexibility as design can be 'supply based' and not necessarily 'demand based'. In other words, optimum use can be made of the in the surrounding area available stone.

Two types of berm breakwaters are most common; 1. Berm breakwater with a homogenous berm, Figure 1.2a. The slopes of this type will reshape for design wave conditions to a statically stable or a dynamically stable profile. 2. Multi-layer berm breakwater, also known as an 'Icelandic breakwater', Figure 1.2b. This is a non-reshaping statically stable berm breakwaters which allows a better use of the quarry stone material then the homogenous berm breakwater.

An other type of breakwater combines the stability characteristics of a conventional two-layered uniform sloped breakwater with the wave energy dispersive character of a berm breakwater. This type is referred to as a 'breakwater with a bermed slope' or 'bermed sloped breakwater', Figure 1.2c.

The stability of the armour layer of a bermed sloped breakwater can not be predicted with conventional design formulae for uniform sloped breakwater as the hydraulic loads are not similar. The design guidelines developed for berm breakwaters are also not applicable as they evaluated damage through the assessment of the reshaping profile. Therefore design of bermed sloped breakwaters is based on experience without understanding the influence of the governing parameters of a berm, on the stability of breakwater armour layers. Consequently this doesn't always result in the best possible design.

1.3 Problem description

Over the past years exceptional land reclamation projects have been executed to extend the shore line of Dubai. To protect these projects from erosion, breakwaters have been constructed. The building materials for these breakwaters were supplied by several quarries in the region.

From the client severe requirements on the maximum amount of overtopping and maximum crest level were imposed to insure good conditions for buildings along the shore line. These requirements in combination with the available stones from the quarries, made certain location unsuitable for uniform sloped breakwaters as very gentle slopes were required to reduce the run-up. In order to find a solution the application of berm breakwaters and a bermed sloped breakwaters was investigated.

The requirement on the stability of the breakwater armour layer was expressed with a damage level parameter of S_d = 2-3, corresponding to 'start of damage'. To meet this requirement only statically stable structures were possible and therefore reshaping berm breakwaters were ruled out. Breakwaters with a bermed slope - the hybrid form between a uniform slope breakwater and a berm breakwater - offered the solution.

As mentioned no design formulae or guidelines are available for this type of breakwater. Design is therefore based on experience and design principles. To verify if a proposed design meets the requirements small-scale test are performed and if necessary adjustments are made.

Disadvantage of this method is that structures are designed without fully understanding the physics behind the process of breaking waves on a bermed slope. Therefore its influence on the stability of the armour layer can possibly be estimated incorrect which can lead to a failing design. As a result more expenses are made on design and testing than necessary.

1.4 Problem definition

At present projects are developed for which the imposed requirements, like a low crest level and minimal overtopping, create the need to design statically stable non-reshaping breakwaters with a bermed slope, Figure 1.2c. This type of structure is preferred as it limits the run-up of incident waves if the berm is designed at the same level as the water level by which the requirement can be met.

However, no design formulae or guidelines are available for this type of breakwater and therefore design is based on experience without understanding the influence of the governing parameters related to a berm, on the stability of breakwater armour layers. Consequently this doesn't always result in the best possible design.

Therefore a better understanding of these governing parameters is needed to develop design guidelines, and eventually a design formula, to predict the stability of breakwaters armour layers on a bermed slope.

1.5 Research objectives

In a first step this study will investigate former stability researches and determine whether or not they can be related to the stability of armour layers on a bermed slope. In addition it will concentrate on creating more insight on the development of damage on a bermed slope and the physics underlying this process.

With these results the influence of the governing parameters related to a berm, on the stability of breakwater armour layers will be determined. Finally the correlation between the design principles and the new insights will be determined and if possible improved.

1.6 Outline

This report will begin in Chapter 2 with a study on stability researches which can be related to stability of bermed slopes. After this Chapter 3 will give an outline of the model set-up and the performed test series. The wave conditions and the properties of the tested breakwaters will be discussed as well as the methods which are applied to measure the wave conditions and the damage.

The results of the test series will be analysed in Chapter 4. A distinction is made between the damage development on an uniform sloped breakwater and a bermed sloped breakwater.

The results will be used to determine the influence of the governing parameters on the stability. These will be compared with the findings of a similar research. Two design principles will be discussed which in practice are used to obtain an indication of the influence of a berm on the stability.

Finally the conclusions of this study accompanied with some recommendations for further research are presented in Chapter 5.

Chapter 2

Literature study of the theory

2.1 Wave properties

In this study the stability of armour stone is related to the impact of the incident waves. The wave conditions are principally described with the following governing parameters; the incident wave height, the wave period and the water depth.

The wave height, H (m), is defined as the difference between the maximum and minimum elevations of the sea-surface (peak to trough wave height) over the duration of the wave. This duration is called the period, T (s), in the time domain and the wavelength, L (m), in the spatial domain, Figure 2.1.

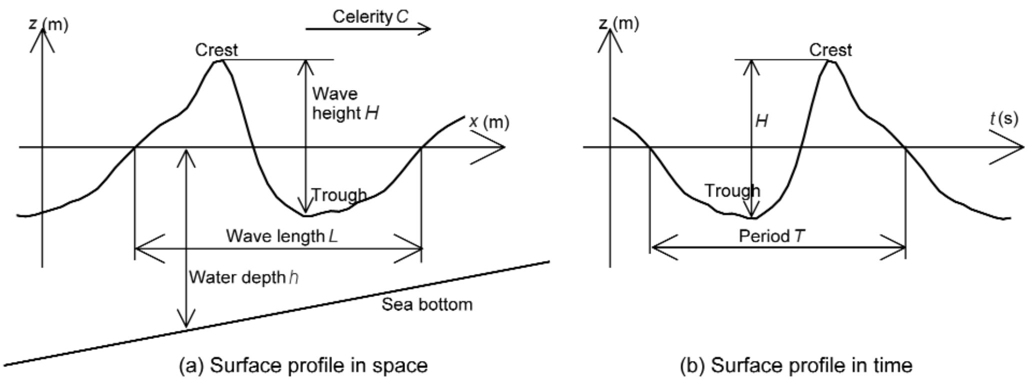


Figure 2.1: Definition sketch for individual wave parameters

For an individual wave one can use the angular frequency $\omega = 2\pi/T$ (rad/s) to describe the temporal periodicity, and the wave number, $k = 2\pi/L$ (rad/m), to describe the spatial periodicity. The parameters describing spatial periodicity (k or L) are related to the parameters describing temporal periodicity (T or ω), together with the water depth, h (m), by the dispersion relation, which for the case of linear (small amplitude) wave theory is given by Equation 2.1.

$$\omega^2 = gktanh(kh) \tag{2.1}$$

When the water depth and wave period are known, the determination of the wavelength requires the resolution of the implicit Equation 2.1. For deep-water or short wave approximation, corresponding to a large value of kh, this wavelength is approach by $L_o = gT^2/(2\pi)$.

Based on the above the relation between the wave height and the wavelength, defined as the wave steepness, s_o (-), is given with Equation 2.2.

$$s_o = H/L_o = \frac{2\pi}{g}\frac{H}{T^2} \tag{2.2}$$

Breakwaters are commonly not designed with respect to one individual wave - a so-called maximum wave - but are based on the characteristic values of sea-states. Therefore the incident wave height is usually given as the significant wave height. This can be based of a time domain analysis ($H_{1/3}$) or a spectral analysis (H_{m0}). The wave period can be given as either the mean period, T_m, the mean energy period, $T_{m-1,0}$, or the peak period, T_p.

2.2 Waves on slopes

To describe wave action on a slope, and some of its effects, a useful parameter is the surf similarity or breaker parameter, ξ (-), also known as the Iribarren number, Equation 2.3. In Battjes (1974) this surf similarity parameter is used to describe the shape of waves breaking on a beach or structure, Figure 2.2.

$$\xi = tan\alpha/\sqrt{s_o} \tag{2.3}$$

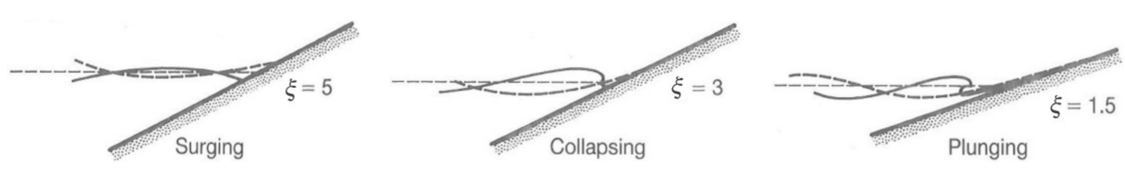


Figure 2.2: Breaker types as a function of the surf similarity parameter

The surf similarity plays a roll on the reflection of a wave by a structure and is therefore related to the stability of stones on a slope. Energy that is not reflected or transmitted, is absorbed on the slope and this absorption is always at the expense of the protection. A low ξ means little reflection given a certain wave creating more absorption which is an unfavourable loading situation for an armour layer. However, the energy absorption per m^2 is lower and the final result is therefore more favourable.

2.3 Stability concepts

2.3.1 General

The principle of conventional design methods is to prevent the initial movement of rock by defining threshold conditions. If there is an imbalance between the hydraulic loading and stabilizing forces it will lead to instability of the armour layer. Some of the parameters used to evaluate this hydraulic stability of rock structures consist of combinations of hydraulic (loading) parameters and material (resistance) parameters.

2.3.2 Wave impact

In the case of wave impact on a sloping structure an important parameter is the stability number, N_s (-). This gives a relationship between the armour properties and the wave conditions, Equation 2.4:

$$N_s = \frac{H}{\Delta D} \tag{2.4}$$

where H (m) is the wave height usually given as a significant wave height H_s or H_{m0}, Δ (-) is relative buoyant density, described by Equation 2.5 and D (m) is the characteristic size or diameter of the armour stone.

$$\Delta = \frac{\rho_r - \rho_w}{\rho_w} \tag{2.5}$$

Normally the diameter used for armour stone is the median nominal diameter, D_{n50} (m), defined as the median equivalent cube size, Equation 2.6:

$$D_{n50} = (M_{50}/\rho_r)^{1/3} \tag{2.6}$$

where ρ_r is the mass density of the rock (kg/m^3), ρ_w is the mass density of water (kg/m^3) and M_{50} is the median of the mass distribution.

By substituting the median nominal diameter and the significant wave height, the stability parameter, $H/(\Delta D)$ or stability number, N_s (-), takes the form:

$$N_s = \frac{H_{m0}}{\Delta D_{n50}} \tag{2.7}$$

2.3.3 Response of the structure

This study deals with non-reshaping statically stable breakwater with a berm, thus the armour layer is required to be almost statically stable for design wave conditions. This is similar to the requirements for a conventional rubble mound breakwater. Damage can be described with two types of damage parameters:

1. Damage to the armour layer can be given as the number of displaced stones, N_d (-), related to a certain area, eg. the entire layer or part of it. In order to quantify the damage independent of the width of the area in consideration the non-dimensional damage level parameter N_{od} is used. This is defined as the number of displaced rocks per width D_{50} across the armour face, Equation 2.8.

$$N_{od} = \frac{\text{number of displaced rocks out of armour layer}}{\text{width of tested section}/D_{n50}} \tag{2.8}$$

2. Another possibility is to describe the damage by the erosion area of the cross-section. When this erosion area, A_e (m^2) is related to the stone size, a non-dimensional damage level S_d can be determined which is independent of the slope angle, length and height of the structure, Equation 2.9. This damage level parameter can physical be described by the number of cubic stones with a side of D_{n50} eroded within a D_{n50} wide strip of the structure, Figure 2.3.

$$S_d = \frac{A_e}{D_{n50}^2} \tag{2.9}$$

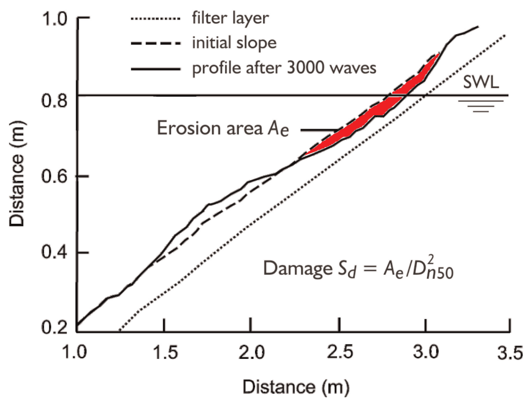


Figure 2.3: Damage level parameter S_d (-) based on erosion area A_e (m^2)

2.4 Empirical theories for uniform slopes

2.4.1 General

Over the last 70 years many empirical methods have been developed to predicted the stability of stones on a slope under wave attack. First research was done by Iribarren (1938) which considered the equilibrium of forces acting on a block place on a slope. After him Hudson (1953, 1959) systematically investigated the stability of rubble slopes for over 10 years and proposed an expression as the best fit for the complete set of experiments. The model tests were based on regular waves on non-overtopped rock structures with a permeable core. Although regular waves were applied his expression is still widely used up to today.

Due to the introduction of wave generators that can generate irregular waves according to a certain predefined spectrum several researchers attempted to overcome shortcomings of the Hudson approach.

In 1988 Van der Meer presented a reliable formulae to predict the stability of stones on an uniform slope with a crest level above the maximum run-up level. These formulae were based on a large amount of model tests, of which the majority were performed with relatively deep water at the toe. These stability formulae are more complex than the Hudson formula, but - as a great advantage - do include the effect of storm duration, wave period, structure's permeability and a clearly defined damage level.

In practice coastal structures are not always built in deep water. Therefore, it was desired to extend the range of application of the stability formulae such that they can be applied not only for deep water but also for shallow water. In more recent works Van Gent *et al.* (2003) formulae are derived to overcome this problem.

2.4.2 Hudson

Hudson (1953, 1959) developed a stability formula based on model tests with regular waves on non-overtopped rock structures with a permeable core. It gives the relationship between the median weight of armour stone, W_{50} (N), the wave height at the toe of the structure, H (m), and the various relevant structural parameters. This stability formula, widely known as the Hudson formula, is presented here in SI units instead of the original units and related notation, Equation 2.10.

$$W_{50} = \frac{\rho g H^3}{K_D \Delta^3 cot\alpha} \tag{2.10}$$

where K_D is the stability coefficient (-), ρ_r is the apparent rock density (kg/m^3), Δ is the relative buoyant density of the stone (-) and α is the slope angle (-).

The main advantage of the Hudson formula is its simplicity. However its use is limited as the storm duration and a description of the damage level are not included. Furthermore the tests were only performed with regular waves on permeable structures.

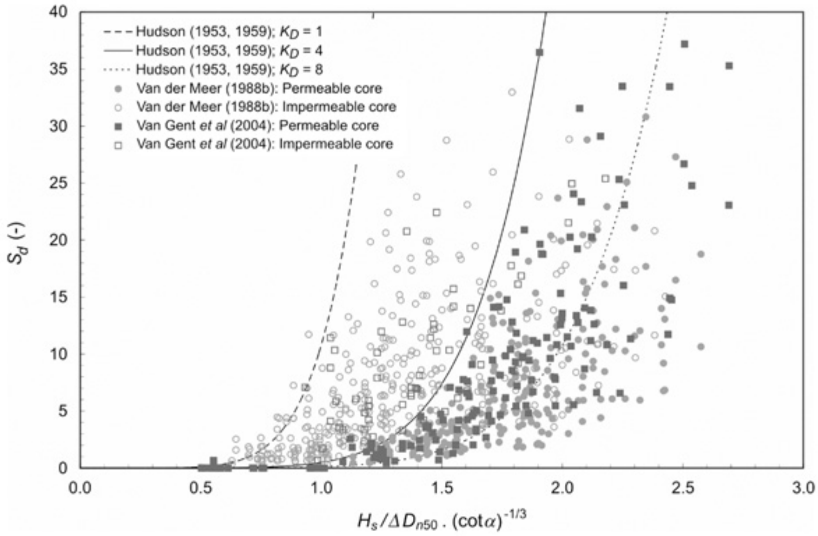


Figure 2.4: Accuracy of stability formula for three KD-values

For practical application some of the problems that may arise due to these limitations can be overcome by using one of the various specific values of the stability (or damage) coefficient, K_D; this particularly applies to permeability of the structure and the wave spectrum.

The original Hudson formula can be rewritten in terms of the stability parameter, $N_s = H_s/(\Delta D_{n50})$. The relationship between this stability number, the structure slope and the stability coefficient K_D is given in Equation 2.11:

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D cot\alpha)^{1/3}}{1.27} \tag{2.11}$$

It is also possible to include Equation 2.11 the damage level parameter S_d. Van der Meer (1988) proposed to use Equation 2.12 as the expression for the stability number, N_s.

$$\frac{H_s}{\Delta D_{n50}} = 0.7\,(K_D cot\alpha)^{1/3}\, S_d^{0.15} \tag{2.12}$$

Figure 2.4 shows all data gathered by Van der Meer (1988) and Van Gent *et al.* (2003) compared with (the re-written) Equation 2.12 for three K_D-values. Three curves are shown: for $K_D = 1$, $K_D = 4$ and $K_D = 8$. For structures with a permeable core it can be concluded that Equation 2.12 with $K_D = 8$ can be used to describe the main trend through the data, however, for design purposes $K_D = 4$ is used.

The simplicity of the Hudson formula manifests itself in a large amount of scatter. Apparently the diversity of the parameters included in the formula are not sufficient to comprehend the complexity of the stability process.

2.4.3 Van der Meer

The formulae of Van der Meer (1988) are based on the work by Thompson & Shuttler (1975). Large amount of tests were performed with relative deep water at the toe. These formulae make a distinction between 'plunging' and 'surging' conditions:

For plunging conditions, ($\xi_m < \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{H_{2\%}}{H_s}\right)^{-1} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5} \tag{2.13}$$

For surging conditions, ($\xi_m \geq \xi_{cr}$):

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{H_{2\%}}{H_s}\right)^{-1} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{cot\alpha} \cdot \xi_m^P \tag{2.14}$$

where H_s (m) is the significant height of the incident waves at the toe, P (-) is a parameter that takes the influence of the permeability of the structure into account (Figure 2.5), N (-) is the number of incident waves, Δ (-) is the relative buoyant density, ξ_m (-) is the surf-similarity parameter using the mean wave period T_m (s) from the time-domain analysis, and α (rad) is the structure slope.

The transition from plunging conditions ($\xi_m < \xi_{cr}$) to surging condition ($\xi_m \geq \xi_{cr}$) can be calculated with Equation 2.15, using the critical breaker parameter ξ_{cr}:

$$\xi_{cr} = \left(\frac{c_{pl}}{c_s} P^{0.31} \sqrt{tan\alpha}\right)^{\frac{1}{(P+0.5)}} \tag{2.15}$$

The surf-similarity parameter, ξ_m (-), can be calculated with Equation 2.3 for which the deep-water wavelength is replaced by the mean wave period, T_m (s).

In Van der Meer (1988) the ratio $H_{2\%}/H_s$ for relatively deep-water wave conditions at the toe can be replaced by the factor 1.4. For slopes more gentle than $cot\alpha > 4$, Van der Meer recommended using Equation 2.13, irrespective of the surf-similarity.

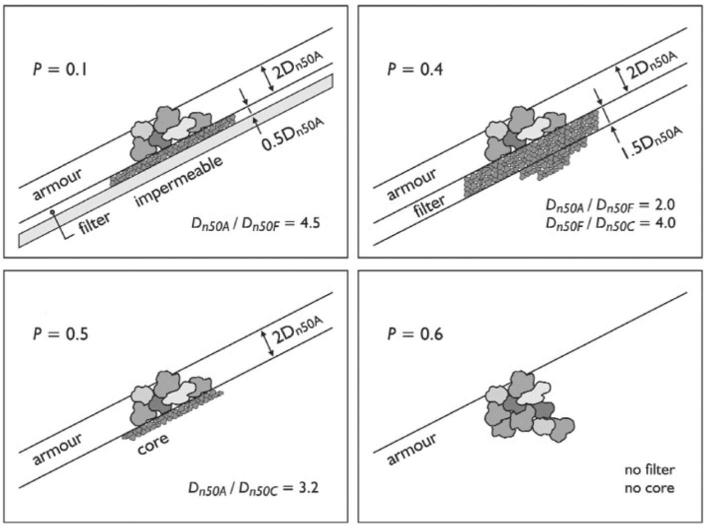


Figure 2.5: Notional permeability factor P for the formulae by Van der Meer (1988)

2.4.4 Van Gent

In Van Gent (1999, 2001) it is shown that wave run-up and wave overtopping can be described best with the spectral wave period $T_{m-1,0}$. From the results by Smith *et al.* (2002) and Van Gent *et al.* (2003) it was determined that this is also the case for stability. This is consistent with earlier work on the influence of wave energy spectra on processes on coastal structures.

To extend the field of application - including shallow foreshores - of the stability formulae by Van der Meer (1988) this spectral wave period is used instead of the mean wave period from the time domain analysis to take the influence of the spectral shape into account. Therefore the coefficients are re-calibrated to $c_{pl} = 8.4$ and $c_s = 1.3$ and exceedence levels of 5% are adapted.

Beside these modifications Van Gent *et al.* (2003) concluded that the influence of the wave period is small compared to the amount of scatter in the data due to other reasons. Also the influence of the ratio $H_{2\%}/H_s$ is considerd small. Therefore a more simple stability formula is presented in which the influence of the wave period and $H_{2\%}/H_s$ have been omitted, Equation 2.16:

$$\frac{H_s}{\Delta D_{n50}} = 1.75cot\alpha^{0.5}\left(1 + D_{n50-core}/D_{n50}\right)\left(\frac{S}{\sqrt{N}}\right)^{1/5} \tag{2.16}$$

The influence of the permeability of the structure is incorporated by using the ratio between the diameter of the core and the diameter of the armour material: $D_{n50-core}/D_{n50}$. The influence of filters located in between the core and armour layer is not accounted for in this ratio. Furthermore, the influence of the number of waves and the parameter $H_s/\Delta D_{n50}$ are the same as found by Van der Meer (1988).

2.5 Empirical theories for special structures

2.5.1 Low-crested structures

The formulae in Section 2.4 were derived for non-overtopped slopes. When slopes are overtopped, there is a certain wave transmission. This means that not all energy will be dissipated on the slope and thus the stability of the armour stone will increase. This type of structures, also called low-crested structures, Figure 2.6, can be subdivided in:

1. emergent structures with crest level above still water level: $R_c > 0$
2. submerged structures with crest level below still water level: $R_c < 0$.

Emergent structures

A part of the wave energy can pass over the breakwater reducing the impact of the incident waves on the front slope. In contrast to non-overtopped structures - where waves mainly affect the stability of the front slope - the stability of crest and rear slope are also affected for low-crested structures. The stability of the armour stone on these segments is therefore more critical for overtopped structure than for non-overtopped structures.

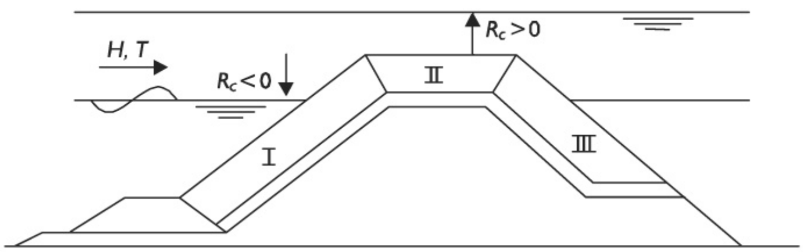


Figure 2.6: Low-crested structure

The stability of the armour stone on the front slope of a low-crested emergent structure can be related to the stability of a non-overtopped structure. This can be achieved by first calculating the nominal diameter, D_{n50} (m), required for the armour stone with one of the design formulae presented in Section 2.4 and then applying a reduction factor, r_D (-), on this nominal diameter. In Van der Meer (1990) a relation for this reduction factor is derived, Equation 2.17:

$$r_D = \left(1.25 - 4.8\frac{R_c}{H_s}\sqrt{\frac{s_{op}}{2\pi}}\right)^{-1} \tag{2.17}$$

where R_c (m) is the crest freeboard, and s_{op} (-) the wave steepness in deep water, based on the peak wave period, T_p (s). The minimum value of the reduction factor is 0.8 and the maximum is 1. This formula gives an estimate for the reduction of the required stone diameter on the front slope. For the crest and the rear side a similar size of material or larger material may be required.

Submerged structures

Large part of the wave energy is distributed onto the crest and the rear slope of the structure and less on the seaward slope. Damage is therefore also shifted onto the crest. For that reason Van der Meer (1990) suggests to exclude the slope angle as being a governing parameter for stability, Equation 2.18:

$$\frac{H_s}{\Delta D_{n50}} = -7\ln\left(\frac{1}{2.1+0.1S}\frac{h_c}{h}\right)\sqrt[3]{s_p} \tag{2.18}$$

where s_p (-) is the local wave steepness based on the peak wave period, h (m) is the water level measured for the toe of the construction and h_c (m) is the height of the structure.

Note: The relation for emergent and submerged structures have been established separately and should be used with caution. Further research is needed in order to establish an overall relation for the stability of low-crested structures.

2.5.2 Bermed sloped breakwaters

In a research by WL|Delft Hydraulics (Vermeer (1986)) the stability of armour stone on a uniform slope was compared to the stability of the armour stone on a bermed slope for similar wave conditions. A distinction was made between the berm material (lower slope and berm) and the upper slope. The development of damage was determined for various series of tests in which the wave height was increased for each test and the ratio between the berm length and the wavelength, or relative berm length, B/L_{om}, was kept constant.

An edit version of the results of this research is presented in Figure 2.7. The increase in stability is given for a damage level parameter of S_d = 2-3. The lower slope and the upper slope are respectively 1:6 and 1:3.

From Figure 2.7 it can be concluded that for the tested cross-section the maximal increase in stability of the complete front slope is achieved for a relative water level in the range of $1.0 < R_c/H_s < 1.3$. Furthermore it can be concluded that the relationship between the

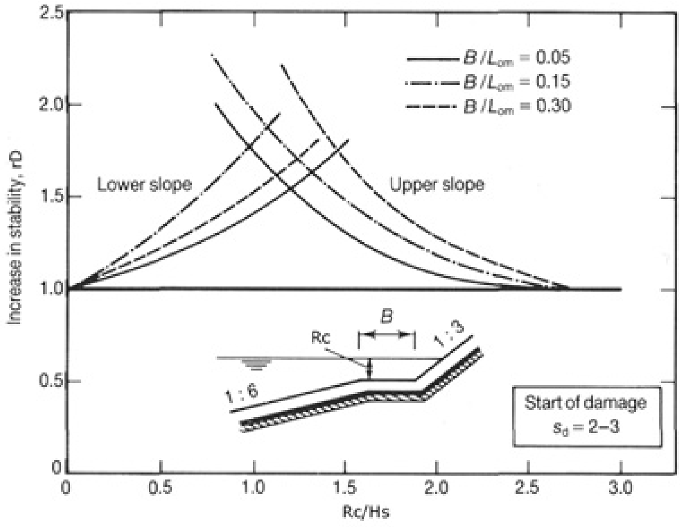


Figure 2.7: Stability increase factors, r_D, for breakwater armour layers with a bermed slope

increase in stability, r_D, and the relative berm length, B/L_{om}, develops differently for the lower slope then for the upper slope. On the upper slope it develops linear but for the lower slope a peak in r_D is noticeable at $B/L_{om} = 0.15$.

The original data are presented in Appendix A; Figure A.1 and Figure A.2 describe the increase in stability for the berm material and Figure A.3 describes the increase in stability of the upper slope.

With this new insight comments can be made on Figure 2.7. First of all the correspondence between the data and the plotted curves is disputable. They agree with the main trend but one must be aware of the large amount of scatter and the limited amount of data. Furthermore the relative water levels are limited to a range of approximately $0.7 < R_c/H_s < 2.0$ and therefore large part of Figure 2.7 is based on extrapolation.

2.6 Conclusion

A great amount of knowledge is available on the complex process underlying stability of breakwater armour layers. This is especially the case for uniform sloped and low-crested breakwater. Not much research has been done on the stability of armour layers for berm sloped breakwaters.

The only research found on this topic is the research by WL|Delft Hydraulics, (Vermeer (1986)). Its objectives seem to correspond to the objectives of this study. However, in practice a berm level is located near the still water level during design wave conditions, $R_c/H_{m0} = 0$, as it results in an optimal reduction of overtopping. This is an important design criterion and thus the results of Vermeer (1986) are not directly applicable. Therefore it is necessary to preform new experiments to investigated the influence of a berm for water levels around the berm level.

Chapter 3

Model set-up

This chapter gives an overview of the model set-up. Successively the wave conditions, the test facilities, the configuration of the breakwater and the properties of the stones will be discussed. Finally the testing programme is presented.

3.1 Wave conditions

To investigate the development of damage for a defined breakwater configuration and a constant water level, test series were performed with an increasing wave height. It was not possible to keep both the wave steepness (H/L) and the relative berm length (B/L) constant as both are related to the wavelength but only the wave steepness to the wave height. It was chosen to keep the wave steepness a constant as the change of breaker type resulting from a changing wave steepness, Battjes (1974), could possibly influence the damage mechanism.

As mentioned in Section 2.6 the berm level of a breakwater is in most cases designed for a relative water level of $R_c/H_{m0} = 0$. This level is optimal for the reduction of overtopping. This level is not necessarily the same for the increase in stability as concluded in Vermeer (1986). Because overtopping is an important design issue, the influence of relative water level is investigated for water levels around the berm level; R_c/H_{m0} = -0.5, 0.0 and 0.5.

The extent in which a wave is influenced by a bermed slope depends on the ratio between the berm length and the wavelength indicated by the relative berm length or B/L. To test a wide range of B/L test series have been executed for three different wave steepnesses and two different berm length.

The chosen wave steepnesses, s_{op} = 1.5, 3.0 and 4.0, were based on in practice occurring design situations. For berms this is roughly B/L_o = 0.2 to 0.25; however, this research is interested in a wider range up to approximately $B/L_o = 0.4$

To restrict other processes of influencing the stability - beside berm length - the properties of the breakwater and the wave spectrum were kept the same. Also overtopping was prevented and the water condition at the toe corresponded to 'deep water'.

3.2 Facility

The 2D physical model tests were carried out in the long sediment flume of the Laboratory of Fluid Mechanics at the Delft University of Technology. The 42 meters long flume with a flat bottom profile has a width of 80cm and a maximal depth of 90cm. The facility is equipped with a wave board, Figure B.1a, generating irregular/random waves up to a significant wave height of approximately $H_{m0} = 13$cm. Wave energy spectra can be prescribed by using standard or non-standard spectral shapes.

The wave generator is equipped with an active reflection compensation system to minimise re-reflections on the wave board. This means that the motion of the wave board compensates for the reflected waves preventing them to re-reflect towards the model and thus avoiding any disturbance on the measurements. Hereby undesirable long-periodic resonance waves in the flume are also avoided.

Wave data was acquired with three analogue wave gauges, Figure B.1b, from which computer facilities for data acquisition and data processing, Figure B.2a, made the computation of relevant wave characteristics possible.

3.3 Scale effects

This study is not a model research of a prototype breakwater so one can not speak of scale effects as such. However, this research simulates damage on full scale breakwaters so an accurate simulation of the water pressures inside the breakwater is necessary. Extra attention must therefore be given to the flow through the porous which is determined in Section 3.5.2.

3.4 Cross-section

In this study three cross-sections were tested for which only the length of the berm was varied. It was determined that berm lengths of $B = 0.3$m and $B = 0.6$m would give an optimal spread of the relative berm length over the range of $0.0 < B/L_o < 0.4$. A third cross-section with a 0.0m berm, thus an uniform slope, was tested to validate stability formulae for uniform sloped breakwaters.

The criterion of no overtopping was translated in a minimal distance between the maximal water level and the crest. For bermed slopes with a rough armour layer like rubble this should be equal to minimal two times the maximal significant wave height, 2x $H_{m0,max}$. The deep water criterion was met in the same way. The distance between the toe and the minimal water level should be at least 3x $H_{m0,max}$.

Based on the maximum performance of the wave board and the minimal criterion for deep water the maximum level of the berm was determined at d = 39cm. With the variation in water level of one time H_{m0} and the criterion for overtopping the crest level became d = 78cm. The height of the flume was limited to d = 90cm so this left 12cm of space. To secure the deep water criterion the berm level was increased and constructed at d = 51cm and therefore the crest level was constructed at d = 90cm.

As no overtopping would take place, the stability of the inner slope would not be effected by the incident waves. Therefore only the seaward slope was constructed. To insure the same permeability of a complete breakwater and not to influence the internal reflection wave, a wire mesh tail board was constructed, Figure B.2b.

The breakwater was constructed with a slope angle of 1:2. In practice this slope angle is often used, but more gentle angles like 1:3 would also have been possible. The distance between the wave board and the toe of the breakwater was 22m. In Appendix C an overview of the model set-up and the three cross-sections is given.

3.5 Materials

3.5.1 Armour

It was estimated with the stability formula by Van der Meer (1988) that for armour stone with a nominal diameter of D_{n50} = 2.5cm the hydraulic loading would exceed the stabilizing forces creating damage on the armour layer. Stones with a sieve size of 25mm to 40mm were used to derive the desired nominal diameter.

The density of the armour rock was determined by weighing a stone and measuring its volume with a measure cylinder. This was done for 27 individually stones and also for three batches existing of approximately 50 stones. A graphic interpretation of the results is given in Figure 3.1. The results of the individual stones had a large scatter probably due to the inaccurate measurement of the volume. The results of the three batches were quite uniform. The rock density is therefore determined over the average of the three batches, $\rho_r = 2582 kg/m^3$.

$$D_n = (M/\rho_r)^{-1/3} \tag{3.1}$$

For randomly picked stones the mass, M (kg), was determined. The dimensions of the equivalent cubes, D_n (m), known as nominal diameter were calculated with Equation 3.1. For assessment of the size distribution the cumulative curve is presented in Figure 3.2.

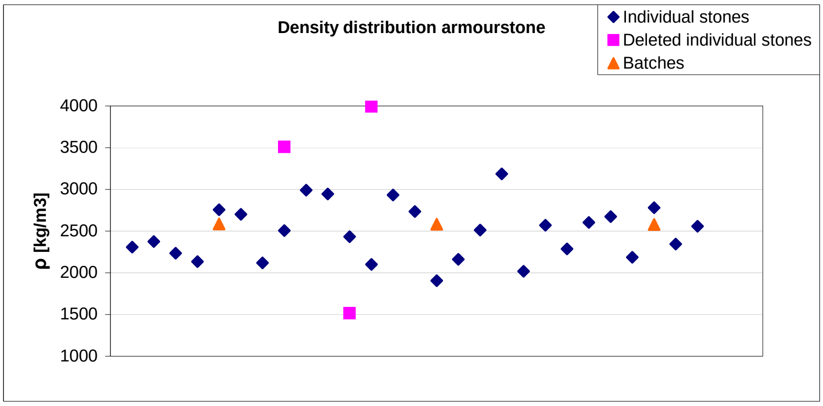


Figure 3.1: Density armour stone

The overall steepness of the curve is an indication of the uniformity of the mass, generally called the grading width or gradation. For armour stone euronorm EN13383-1 (2002) defines requirements for the distribution of the mass. However, these requirements can not be applied directly as they are defined for prototypes. Therefore a more general quantitative indication for the uniformity of armour grading is used stating that armour stone gradings should be narrow. This imposes restrictions on the ratio between D_{n15} and D_{n85}.

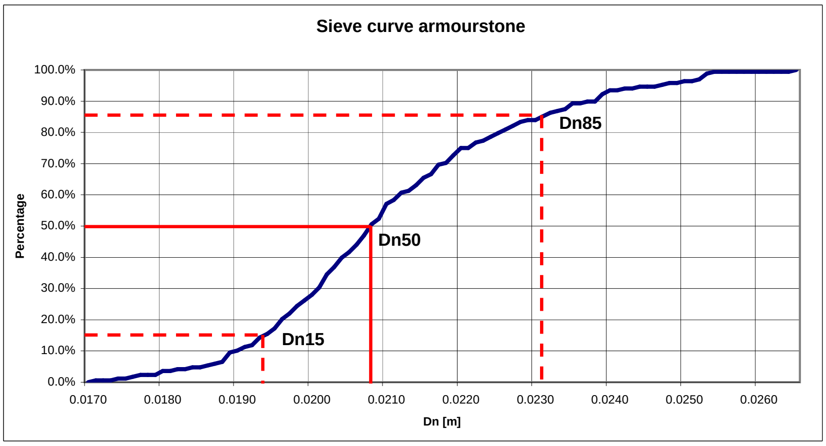


Figure 3.2: Sieve curve armour stone

From Figure 3.2 the median nominal diameter and the gradation were determined. The gradation was $D_{n85}/D_{n15} = 1.19$ which is classified as very narrow. The median nominal diameter was $D_{n50} = 2.07$cm which is smaller than the prescribed size. However, the stones were nevertheless used for the armour layer as it, if necessary, could be compensated with a lower significant wave height.

The shape of individually placed armour stone is of importance as it affects the layer thickness, packing density and the stability. According to euronorm EN13383-1 (2002) the length-to-thickness ratio greater than 3 for heavy gradings must be smaller than 5%. A percentage of 9% was determined but by removing large part of these stones the requirements were met.

The porosity, p (-), of the armour layer - indicating the percentage of pores - was calculated by filling a known volume with stones and determining the amount of water needed to fill it up. The influence of the sides was minimized by using a sufficiently large volume. From an average of seven tests a porosity was determined of $p = 0.45$, Appendix D. This is larger than normally the case. However, it can be explained by the narrow grading and angular shape of the armour stones.

The thickness of the armour layer is equal to 2x D_{n50} which corresponds to $d = 4.2$cm.

3.5.2 Core

The permeability of the core material influences the stability of an armour layer. The *Shore protection manual*, CERC (1984), prescribes for breakwaters a geo-technical filter rule which gives the ratio between the nominal diameters of the armour material, D_{n50a}, and the material of the underlayer, D_{n50u}, Equation 3.2. This prevents transport of fine material through the upper layers, but respects the permeable character of a breakwater.

$$D_{n50a}/D_{n50u} = 2.2 - 2.5 \tag{3.2}$$

This filter rule doesn't hold for model research as linear geometric scaling - which follows from Froude scaling - leads to larger viscous forces in the core, corresponding to a drop of the Reynolds number. The related increase in flow resistance reduces the in- and out-flow of water through the core. This causes larger up-rush and down-rush velocities resulting in a decrease of the armour layer stability.

To by-pass this problem similarity between the length scale of the flow fields of the prototype and the model is required, Burcharth *et al.* (1999). In other words; scaling is done correctly if the hydraulic gradients in prototype and model are the same.

As mentioned before, this is not a model research of a prototype breakwater so no scaling factor was used. In order to estimate the influence of 'scale effects' on the stability of the armour layer a scale factor of $n = 40$ was chosen and therefore the terms 'model' and 'prototype' are used.

The nominal diameter of the stones for the model armour layer was D_{n50}= 2.07cm. With the geo-technical scaling rule for the prototype and the Burcharth *et al.* (1999) scaling rules for

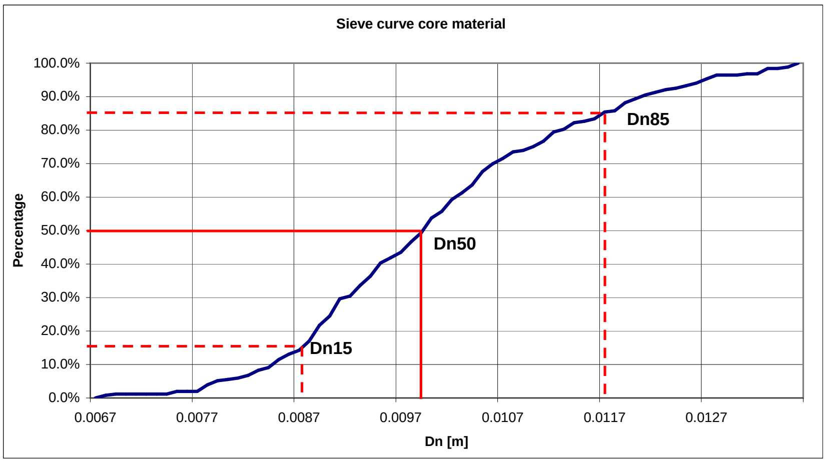


Figure 3.3: Sieve curve core material

Location	$D_{n50-model}$ [cm]	$D_{n50-proto}$ [cm]
Armour	2.07	84
Filter	1.48	38
Core	1.05	17

Table 3.1: Scaling according to Burcharth and geo-technical filter rule with $n = 40$

the relation between model and prototype, this resulted in a nominal diameter for the model of $D_{n50} = 1.48$cm for the filter and $D_{n50} = 1.05$cm for the core material, Table 3.1.

In this study no distinction is made between the nominal diameter for the filter layer and the core material. This has two reasons. First of all the outcome of the scaling method depends on the applied scaling factor. Secondly it is in practical sense difficult to accurately sieve a predefined nominal diameter. Therefore the filter is left out and only a core is used. The ratio between the armour stone and the core material still apply to the geo-technical filter rule.

The core material was derived from a batch with a sieve size of 11mm to 16mm. In the same way as for the armour stone the median nominal diameter and the gradation were determined resulting in $D_{n50} = 0.99$cm and $D_{n85}/D_{n15} = 1.33$, Figure 3.3.

3.6 Measurements

3.6.1 Wave measurements

The spectrum of the waves used for all test series was a JONSWAP spectrum with a peak enhancement factor of $\gamma = 3.3$. This is a standard spectrum for fetch-limited sea-states, ie growing sea, as used in practice. More detail on this spectrum can be found in Appendix E.

The measurement of the actual wave spectrum of the incident wave was done with three wave gauges placed in front of the toe of the breakwater at a distance of 21m from the wave board. The spectrum is expressed in terms of spectral moments, Equation 3.3. The surface elevations have been corrected for reflected waves using the theory described in Zelt & Skjelbreia (1992).

$$m_n = \int_{f_{min}}^{f_{max}} E(f)\, \mathrm{d}f \tag{3.3}$$

From spectral analysis several representative wave parameters like the significant wave height H_{m0} and the mean energy period $T_{m-1,0}$ can be computed, Equation 3.4:

$$H_{m0} = 4\sqrt{m_0} \tag{3.4a}$$

$$T_{m-1,0} = m_{-1}/m_0 \tag{3.4b}$$

For a JONSWAP spectrum the ratio between the peak period T_p and the mean energy period $T_{m-1,0}$ is determined by Dingemans (1987), Equation 3.5:

$$T_p = 1.107 T_{m-1,0} \tag{3.5}$$

3.6.2 Damage recording

The development of damage of the armour layer was determined for several series of tests. A test series was build up of a number of tests in which the wave height was gradually increased up to failure of the armour layer. For each test, containing of 3000 waves, damage was recorded after 1000 and 3000 waves. Damage was not repaired during a test series, in this way the cumulative damage during the test series was determined.

Damage was determined by comparing the condition of the slope prior to the test with the condition of the slope after a test. The number of displaced stones was quantified using photographs taken from fixed camera position above the flume. Successive prints were compared to identify armour displacement, Appendix F. Displacement of an armour unit is defined as a displacement over more than one unit dimension. To make it easier to count the number of displaced stones the armour layers was placed in bands of different colours with a width of approximately 5xD_{n50}. The position of the colour bands for the different cross-sections is presented in Appendix G.

The recorded damage, presented as the number of displaced stones per colour band (N_d), was made independent of the width of the flume resulting in the non-dimensional damage level parameter N_{od}, Equation 2.8. In literature the non-dimensional damage level parameter S_d

is often used to indicate damage, Equation 2.9. The relation used in this study between the two damage level parameters is defined by Equation 3.6:

$$N_{od} = \frac{S_d(1-p)}{1.1131} \tag{3.6}$$

This formula is derived from the following equations in which p is the porosity, A_e is the eroded area and A_{rock} is the average area of rock.

$$A_e = N_{od} \frac{A_{rock}}{(1-p)} \tag{3.7a}$$

$$A_{rock} = 0.25\pi \left(\frac{D_{n50}}{0.84} \right)^2 = 1.1131 D_{n50}^2 \tag{3.7b}$$

Furthermore, recordings of the tests have been made to analysis of the water movement created by the incident waves on the breakwater.

3.7 Test programme

The test programme was set-up according to the wave conditions described in Section 3.1. The wave conditions are specified only with a wave steepness as the development of damage was uncertain. The significant wave height and the wave peak period were determined during testing with trial test series.

The tested cross-sections are given in Appendix Figure C.2. For all test series a standard JONSWAP spectrum ($\gamma = 3.3$) was used with a storm duration expressed by the number of waves of $N = 3000$. In Table 3.2 the target values for the wave conditions at the toe of the construction are presented.

Cross-section, 0.6m berm			Cross-section, 0.3m berm			Cross-section, no berm		
Test series	h [m]	s_{op} [-]	Test series	h [m]	s_{op} [-]	Test series	h [m]	s_{op} [-]
T-01	0.51	0.040	T-10	0.51	0.004	T-19	0.51	0.040
T-02	0.51	0.030	T-11	0.51	0.030	T-20	0.51	0.030
T-03	0.51	0.015	T-12	0.51	0.015	T-21	0.51	0.015
T-04	0.57	0.040	T-13	0.57	0.040			
T-05	0.57	0.030	T-14	0.57	0.030			
T-06	0.57	0.015	T-15	0.57	0.015			
T-07	0.45	0.040	T-16	0.45	0.040			
T-08	0.45	0.030	T-17	0.45	0.030			
T-09	0.45	0.015	T-18	0.45	0.015			

Table 3.2: Test programme with target values for the wave conditions

Note: The numbering of the test series in the Appendix H and Appendix L do not correspond to the numbering of Table 3.2.

Chapter 4

Analysis of test results

4.1 Introduction

In this chapter the results of the test series will be discussed and analysed. To start of a validation of the data of the uniform sloped breakwater will be performed after which the test series executed on an uniform slope will be analysed. The measured damage and the predicted damage by stability formulae will be compared to determine correlation between the two.

Subsequently the damage development on the bermed slope will be analysed. The results of this study will then be compared to the findings of a research by WL|Delft (Vermeer (1986)). Finally two design principles will be discussed which in practice are used to obtain an indication of the influence of a berm on the stability.

4.2 Data validation

To fulfill the required testing programme 24 test series were executed each build up of several tests. For each test - existing of 3000 waves - the damage and the moments of the wave spectra were recored after 1000 and 3000 waves.

During the testing period it was discovered that the steering files for the wave generator for test series T101x, T102x and T109x were incorrect. These test series were rejected and the first two were replaced by test series T103x and the third by test series T112x. The rejected test series were not used in further analysis.

In this study the development of damage of the armour layer was determined for an increasing wave height. The main property of the armour layer is to protect the core material; if this core material is exposed the armour layer has failed. If during a test the core material became visible - or was likely to become visible for a slight increase of the wave height - a test series was ended. Depending on the extend of the damage it was determined if this last test would be useful for further analysis.

A test was rejected if the core material was visible. All armour stones at a certain location would have already been displaced and the recorded damage level would be incorrect. If the core material was not, or only sightly visible, the test was accepted. The total amount of damage could therefore differ per test series. For some test series this total amount of damage on the armour layer could be described as 'failure', as for others, this could be described as 'intermediate damage'.

The distinction between damage after 1000 and 3000 waves proved to be unnecessary as this research is interested in the damage development of a test series and not so much of an individual test. Therefore the average value of the spectral moments and the cumulated damage were taken. This processed data is presented in Appendix H.

Two exceptions were made. The significant wave height generated over the first 1000 waves of the first test in test series T113x was to low. To increase this value the steering file was adjusted for the second part. For a correct interpretation of the test results only the spectral moments of test T1131-3000 were used. Furthermore, the last test in test series T114x was used for further analysis despite the fact that the core material was clearly visible. Without this test the test series would exist of only two tests which would be to little to be useful for further analysis. In order not to make the complete series redundant this last test was be used as an indicator.

4.3 Uniform slope

4.3.1 Test results

The results of the test series on an uniform slope correspond in Appendix H to the test series T122x, T123x and T124x. The damage development of each test series is plotted in Figure 4.1 with on the horizontal axis the stability parameter $H_{mo}/\Delta D_{n50}$ and on the vertical axis the damage parameter N_{od}.

For each of the three test series the wave height H_{m0} was increased up to the point where the core material was slightly visible. This is just beyond the point which in literature is defined as 'failure'. For uniform slopes this corresponds to a design value of the damage parameter of $S_d = 8$. In Figure 4.1 this damage level is indicated with the upper dotted line corresponding to the damage parameter of $N_{od} = 5$.

A relation between the two damage parameter is given by Equation 3.6. With a porosity of p = 0.45 this can be approach by $N_{od} = 0.5S_d$. For the test series the damage parameter

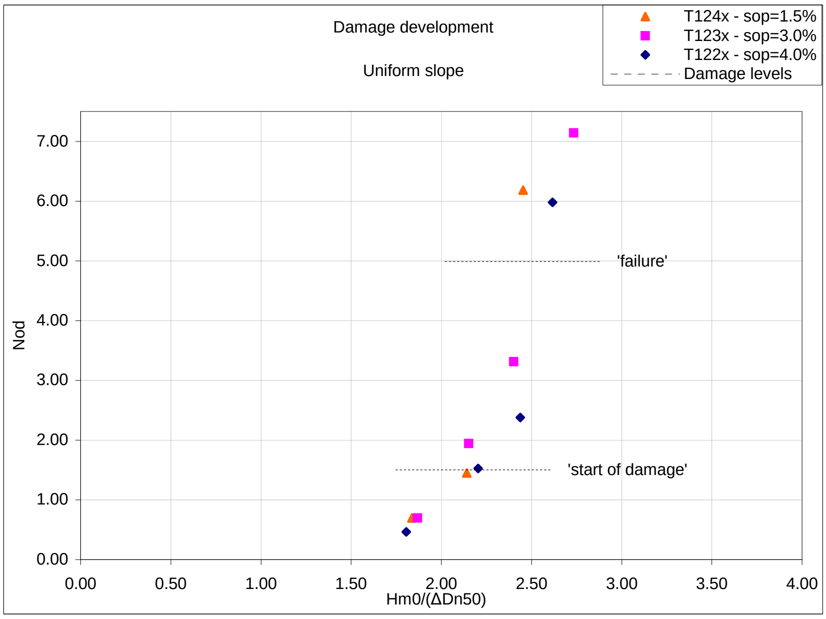


Figure 4.1: Damage development on uniform slope

for failure then becomes $S_d = 10$ which is larger than the design value. This difference can be explained by the conservative approach of design value which always account for scatter and uncertainties. So the given relation between the two damage parameters is believed to be correct.

For design purposes a value of the damage level parameter of $S_d = 3$ can be used. With Equation 3.6 this corresponds to a damage parameter of $N_{od} = 1.5$. This damage level, defined as 'start of damage', is indicated in Figure 4.1 with the lower dotted line.

4.3.2 Stability formulae

For the prediction of the stability of armour stone on uniform slopes several design formulae have been developed. The three stability formulae described in Chapter 2.4 are Hudson (1953, 1959), Van der Meer (1988) and Van Gent *et al.* (2003). Each formula has its distinct approach and therefore different damage could be predicted for similar wave conditions. The results of the test series executed on an uniform slope are compared with the design formulae to validate the correctness of the execution method for determining damage. Regardless of which design formulae is used the results should be in line with the predicted damage. Large or unexpected deviations could indicate an incorrect execution method. The results are presented in Appendix I.1.

Note: The following interpretations are based on only one execution for three different wave steepnesses. As there is no repetition the influence of scatter can not be seen.

Hudson

The Hudson formula is a rather simple equation including only few parameter. Several values of K_D are derived making it applicable for a wide range of armour units and configurations. In Figure 2.4 it can be seen that for a large amount of tests on structures with a permeable core the main trend is best described with $K_D = 8$. Consequence of including only few parameters - therefore neglecting the influence of certain parameters - is a large amount of scatter. The more conservative value $K_D = 4$ is therefore used for design purposes. Both value of K_D are plotted in Figure I.1a. It can be seen that the main trend is well described with $K_D = 8$.

Van der Meer

In comparison to the Hudson formula the formula of Van der Meer is quite progressive. It includes a large amount of parameter increasing its range of application. The effect of wave steepness is included such that stability decreases for an decreasing wave steepness resulting in a different prediction of the stability for each test series. For the two higher values of the wave steepnesses - T122x and T123x - the stability is predicted rather well although the last test in test series T122x deviates quite a bit. For the lowest value of the wave steepness - T124x - the prediction is not correct. This could indicate that the formula is less applicable for lower waves steepnesses.

Van Gent

An other progressive formula is presented by Van Gent *et al.* (2003). This stability formula also includes many different parameter but unlike the formula by Van der Meer it excludes the influence of the wave period. It was concluded that the influence of the wave period is small compared to the amount of scatter in the data due to other reasons. Therefore the same stability is predicted for the three different wave steepnesses. In Figure I.1.c the graph it can be seen that the predicted stability complies well to the main trend.

4.3.3 Conclusion

The predicted stability of the considered design formulae all correspond relatively well to the results. Therefore it is believed that the method of determining damage is correct and the relation between the two damage parameters S_d and N_{od} given by Equation 3.6 is valid. The damage level parameter $S_d = 3$ used to express 'start of damage' on uniform sloped breakwaters can therefore be converted into $N_{od} = 1.5$.

Based on only this data set the main trend is best described by the Hudson formula, however the individual contribution of the different parameters is not clear with the single coefficient

K_D. A misjudgment of the coefficient could lead to an incorrect prediction making this formula unfavoured.

The formulae of Van der Meer and Van Gent also give a good prediction of the damage in which the individual contribution of the different parameters is clear. However the influence of the wave period - based only on these test results - is unclear. Including the wave period is therefore questionable as it insinuates an increase in accuracy which is not present. The formula of Van Gent is therefore favourable.

4.4 Bermed slope

4.4.1 Test results

The results of the test series on a bermed slope correspond in Appendix H to test series T103x up to T121x. In Table H.1 two different damage area's can be indicated; one round the transition point between the lower slope and the berm and one on the upper slope. Consequently damage in Table H.2 is determined separately for these two area's.

4.4.2 Physical processes analysis

Damage between the test series vary significantly for a similar stability parameter $H_{mo}/\Delta D_{n50}$. Apparently the mechanism which creates damage depends on the interaction between the berm level, berm length and the wave conditions. To understand this interaction an analysis is made of these mechanisms based on video footage, the damage development shown in Appendix J and the location of the damage shown in Table H.1.

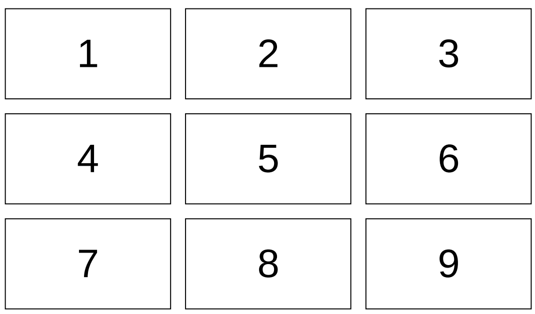


Figure 4.2: Numbers referring to the graphs in Appendix J

For an easier reference to the graphs in Appendix J they will be numbered according to Figure 4.2.

Water level below berm level

Despite a lower water level then the berm level the waves overtop the berm level. The wave energy is distributed not only on the lower slope but also on the berm. This redistribution decreases the wave attack on the lower slope and increases it on the berm.

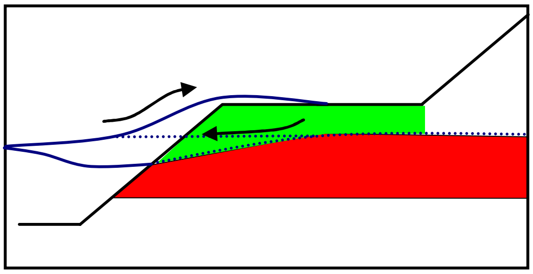


Figure 4.3: Return current for water level lower then berm level

As a wave retreats water transported on to the berm creates a return current. Because the still water level is below the berm level, the porosity of the berm is great enough to transport all this water through the structure. No water is redirected over the surface area, Figure 4.3. As the velocity of this return current is low, no damage is created. Also, the return current doesn't influence the incident waves. Damage is therefore only created by the incident waves illustrated in Figure 4.4.

This situation can be compared to low-crested breakwaters, Section 2.5.1. Damage is predicted with an arbitrary chosen formula for the stability of uniform sloped breakwaters, Section 4.3, multiplied with an additional reduction factor to include overtopping.

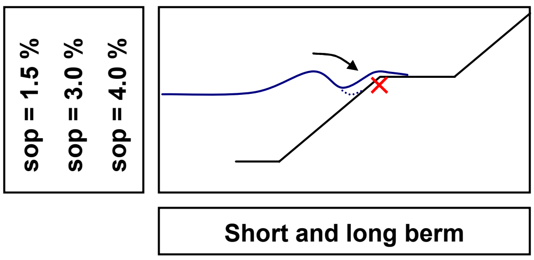


Figure 4.4: Dominating damage mechanism for a water level lower then the berm level

Graph 1, 4 and 7:

The test results for the six test series are quite similar. The trajectories of the damage development are alike and there is no difference in damage between the 0.3m and 0.6m berm.

Water level on berm level

The amount of water transported on the berm is significant. This influences the character of the return current. As a wave retreats the water level will drop locally, in reaction, the water table in the breakwater will drop too. In Figure 4.5 an example is given for two different berm lengths. For both structures the area in which the water table in the breakwater is lower than the still water level - indicated with the green area - is identical.

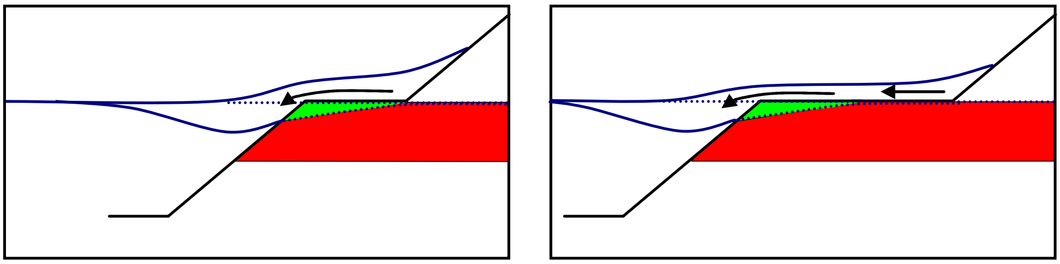


Figure 4.5: Return current for different berm length

As this area is limited, so is the amount of water which can be transported through the breakwater. A return current over the surface area of the breakwater is therefore created. This effects the stability of the armour stone located on transition area between the lower slope and berm. At this point the stones do not interlock very well and are easily displaced by the drag force of the return current.

The hydraulic gradient of the water on the berm, i (-), acts as a driving force on the return current over the surface area. It therefore determines its velocity and also the 'draining time' of the berm.

Dependent on the frequency of incident waves all water on the berm can retreat completely or partly remain on the berm. In the first case the incident waves will not be influenced by the return current and the armour layer on the berm will be exposed. In the second case the still retreating water will collide with the incident waves somewhere on the berm.

With this understanding of the character of the return current, an analysis of the dominating damage mechanisms for the different test series can be made. An overview is given in Figure 4.6.

Graph 2:
The incident waves 'glide' over the berm and on impacted do not create much turbulency and therefore also not much damage. In test series T114x damage is created as the water retreats. The large amount of water on the berm, in combination with a large hydraulic

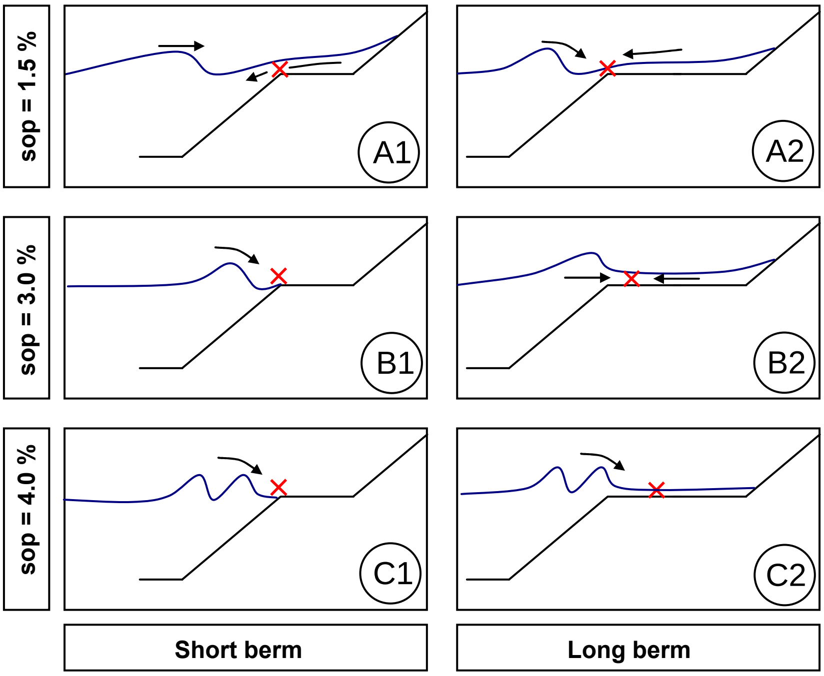


Figure 4.6: Dominating damage mechanism for a water level on the berm level

gradient, tributes to a high velocity of the return current. The drag force of this current removes stones from the transition area. For test series T105x the velocity of the return current is lower and therefore less damage is created. However, the incident waves collides with the return current at the transition point. This creates lots of turbulency and stones are neverhteless displaced.

If a closer look is taken it can be assumed that test series T114x is less stable than test series T105x. However, this is not clear as only the high values of $H_{m0}/(\Delta D_{n50})$ differ. To confirm this tests have to be done with a smaller interval. If the assumption is true the return current apparently creates slightly more damage than the colliding water bodies on the transition point.

Graph 5:
In test series T115x the water retreats quick enough in order not to influence the incident waves. However, the return current isn't the dominating damage mechanism. The impacted of the incident waves create most damage. In test series T104x the same can be seen for test series T105x. The return current also collides with the incident waves but now higher up the berm. This creates less damage as the stones on the berm are more stable.

It can be concluded that although the damage development is the same for both test series, they are a result of different damage mechanisms.

Graph 8:
In test series T113x the return current doesn't influence the incident waves. Damage is only created by the inpact of the incident waves on the transition point. In test series T103x the long berm prevents a large hydraulic gradient of developing. This, in combination with a high frequency of incident waves, creates a set-up on the berm. Due to the higher water level, the waves break at a higher part of the berm where the stones are more stable. Therefore less damage is created in regard to test series T113x. As no significant return current is developed there is no collision with the incident waves.

The influence of the set-up on the 0.6m berm is visible in a more stable front slope. For the 0.3m berm the complete return current can retreat the waves break on the exposed transition point resulting in a less stable front slope.

Water level above berm level

The layer of water on the berm acts as a protective cover for the armour stone on the lower slope and berm. They are therefore less exposed to the incident waves. However, the waves dissipate less energy on the berm and therefore create more damage on the upper slope. In Appendix J the influence of relative berm length can be seen. For the lower slope and berm, as well as the upper slope, stability increases for an increasing relative berm length.

The effect of the relative berm length on the dominating damage mechanisms is analysed for the different test series. An overview is given in Figure 4.7.

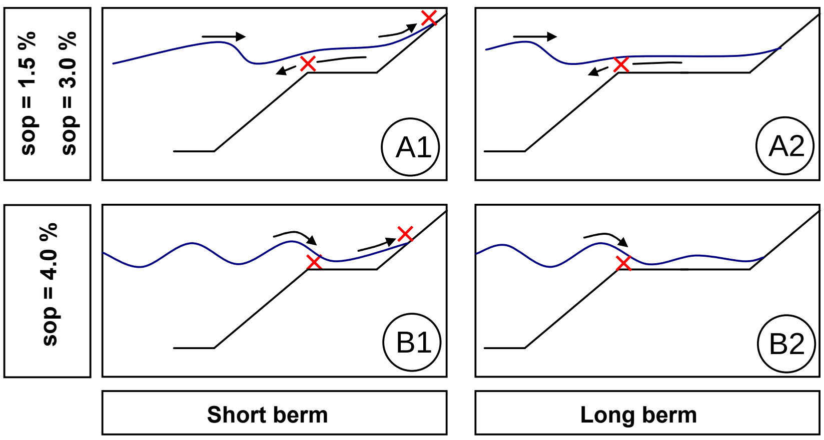


Figure 4.7: Dominating damage mechanism for a water level above the berm level

Graph 3 and 6:
The incident waves 'glide' over the lower slope and don't create much damage on the transition area between the lower slope and berm. Damage on this point is mainly created by the return

current. Its intensity - depending on the hydraulic gradient - is not the same for the different berm length. Although the amount of water transported on the berm for both lengths the same, the longer berm can spread the water over a larger area resulting in a smaller hydraulic gradient than on the short berm. The velocity of the return current is therefore smaller resulting in a larger stability.

Graph 9:
The short wave period prevents the development of a return current with a high velocity. Damage is therefore only created by the incident waves. This explains why both test series have a similar damage development.

Conclusion

In all test series the stability of the stones on the transition area between the lower slope and the berm was most critical. Two damage mechanisms are indicated; first of all the impact of the incident waves and secondly the drag force of the return current over the surface area.

The extent of each mechanism depends on the water level. For a water level below the berm level, damage created by the impact of the incident waves is dominant over the return current. For a water level equal to the berm level there is a balance between the two. For a water level larger then the berm level the return current is dominant, damage due to the impact of the incident waves is of minor importance.

4.4.3 Quantitative analysis

For a better understanding on the influence of the governing parameters related to a berm, on the stability of breakwater armour layers a quantitative analysis is made of the tables in Appendix H and the graphs in Appendix J.

First an analysis will be made on the development of the damage and the related damage level parameters. Next the results of the individual test series will be compared to determine the influence of the governing parameters. Finally the findings of this study will be compared with the results of the research by Vermeer (1986).

Damage level

To determine the damage level of the test series executed on a bermed slope, the graphs from Appendix J are used in combination with the pictures of the actual damage.

For all test series it was determined that 'failure' of the armour layer, corresponding to a visible under layer, was reached for a damage parameter larger than $N_{od} = 5.0$. Apparently the development of damage is independent of the damage mechanisms. This damage parameter corresponds to the criterion for armour layers on an uniform slopes, Section 4.3.1. By analogy, the damage parameter for 'start of damage' for bermed slopes is the same as for uniform slopes and therefore equal to $N_{od} = 1.5$.

Increase stability

It is generally assumed that bermed slopes, in regard to an uniform slope, have a positive effect on the stability. To validate this assumption the stability parameter of a bermed slope can be compared to the stability parameter of an uniform slope for a similar damage parameter. The increase in stability is expressed by r_D, Equation 4.1.

$$r_D = \frac{H_{mo}/\Delta D_{n50}\text{-berm}}{H_{mo}/\Delta D_{n50}\text{-uniform}} \tag{4.1}$$

To determine the increase in stability in regard to an uniform slope, the results of the test series executed on a bermed slope are compared to the predicted stability of an uniform slope. Based on Section 4.3.3 this prediction is made with the stability formula of Van Gent.

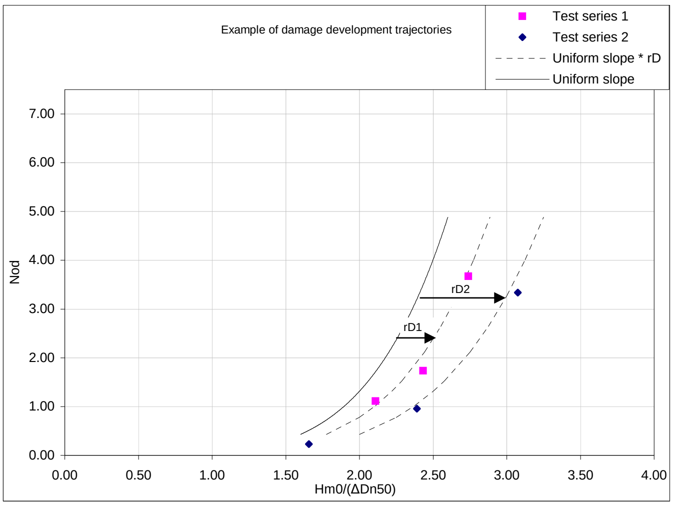


Figure 4.8: Example of damage development trajectories

In Appendix K the formula of Van Gent *et al.* (2003) is plotted in with the test results of the test series executed on a bermed slope. Not all graphs of Appendix J are included as they show no damage. Determining the increase in stability for these graphs is therefore unnecessary as it would be infinite.

From the graphs in Appendix K it becomes clear that the damage development on the bermed slope have a similar - but more stable - trajectory as the damage predicted by the Van Gent formula. An example is given in Figure 4.8. Nearly all test series are described correctly by multiplying the Van Gent formula with the stability increase factor r_D. For each test series the value of this factors is determined, Table 4.1.

Test series	r_D (LS+B)	r_D (US)	Test series	r_D (LS+B)	r_D (US)
T103x	1.25	-	T113x	1.11	-
T104x	1.17	-	T114x	1.04	-
T105x	1.08	-	T115x	1.17	-
T106x	1.55	2.25	T116x	1.34	1.65
T107x	1.53	2.10	T117x	1.11	1.43
T108x	1.26	2.20	T118x	1.55	1.67
T110x	1.07	-	T119x	1.11	-
T111x	1.11	-	T120x	1.11	-
T112x	1.11	-	T121x	1.07	-

Table 4.1: Average increase in stability in regard to an uniform slope (lower slope and berm, upper slope)

Three test series are not describe in a proper manner:
T114x: The trend of the test results of test series T114x the seems to be steeper than the trendline indicates. This is based on the third test of test series but is was previously mentioned that this data point is uncertain and would only be used as indicator.
T117x: The test results of test series T117x seem to deviate more than the other test series but most probably this can be attributed to scatter.
T121x: For test series T121x it seems that the short berm is more stable than the long berm. But based on video analysis it is concluded that this is not possible as the berm length was to long to has any influence. Also the wave conditions have been check but no significant differences can be pointed out. Therefore scatter is most probable be the cause and the same r_D is used for T121x as for test series T110x.

It can be concluded that - in general - the test results have a large correlation to the predicted damage on an uniform slope by means of the formula of Van Gent *et al.* (2003). For an individual test series the increase in stability, over the complete range of $H_{mo}/(\Delta D_{n50})$, can be indicated with a constant value of r_D. Consequently r_D is independent of the considered damage level.

Governing parameters

For design purposes the damage level 'start of damage' is of most interest. In contrast to the higher damage level, the stability parameter can be determined without extrapolation the test results. As previously concluded the same damage parameter for 'start of damage' can be used for bermed sloped as used for uniform slope. This corresponds to a damage parameter of $N_{od} = 1.5$.

For all the test series presented in Appendix K, the values of $H_{m0}/(\Delta D_{n50})$ are determined for a damage parameter of $N_{od} = 1.5$. The corresponding values of R_c/H_{m0}, $B/L_{m-1,0}$ and r_D are plotted in Figure L.1. In the arrangement of the graphs a distinction is made for the different water levels. The first three present the increase in stability for the lower slope and berm for all three water levels. The fourth presents the same for the the upper slope but only for the highest water level as damage for the lower water levels was negligible.

Unfortunately damage was mainly created on the lower slope and berm and hardly any on the upper slope. Therefore further analysis will only focused on the lower slope and berm.

Conceivable trendlines

For a better insight on the influence of the governing parameters the test results of the lower slope and berm are merged in Figure L.2. The upper graph shows the influence of the relative berm length on increase in stability and the lower shows the influence of the relative water level on the increase in stability.

If the increase in stability in determined for constant values of these governing parameters, certain trends can be seen. To emphasize these trends conceivable line are plotted.

The trajectories of the lines only give an indication on the development of r_D and are open to question as they are based on a small amount of data. As mentioned scatter can easily give a distorted view. Especially the trajectories in for $R_c/H_{m0} > 0.6$ are uncertain as only no data is available.

Conclusions on influence of governing parameters

From the graphs in Appendix L several conclusions can be made on the influence of the governing parameters of a berm on the stability of breakwater armour layers:

1. For the tested range of the governing parameters of a berm, a berm has a positive effect on the stability.

2.When regarding constant values of R_c/H_{m0} the data shows for initial values of the relative berm length an increase in stability as $B/L_{m-1,0}$ increases. As the relative water level gets increasingly larger the results indicate a horizontal limit.

3. In general the stability increases as the relative water level increases. However, for the lower values of the relative berm length a decrease is noticeable for a relative water level of $-0.5 < R_c/H_{m0} < 0.5$. This can possibly be explained by the intensity of the return current

which drags the instable stones on the transition point between the lower slope and the berm down resulting in a large amount of damage.

4. For water levels in the range of $-1.0 < R_c/H_{m0} < -0.5$ the length of the berm has hardly any influence of the increase in stability. Damage is only created by the impact of the incident waves. The load of the incident wave is effected by the amount of overtopping. Apparently a minimal berm length is enough to induce overtopping and therefore the maximal increase in stability is reached for small values of $B/L_{m-1,0}$.

5. As the water level approaches the berm level, $-0.5 < R_c/H_{m0} < 0.0$, the influence of the berm becomes evident. The increase in stability enhances over a wide range of the relative berm length, $0.0 < B/L_{m-1,0} < 0.35$. As the water level exceeds the berm level, a strong increases of r_D is noticeable. However, the range of the relative berm length in which this increase in stability is enhanced becomes smaller, $0.0 < B/L_{m-1,0} < 0.20$.

4.5 Comparison with Vermeer (1986)

In Section 2.5.2 a research by WL|Delft (Vermeer (1986), Appendix A) is discussed which also investigated the influence of the governing parameters of a berm on the stability of breakwater armour layers. However, it was not directly applicable as it focused on larger values of the relative water level and on a more gentle lower slope than desired for this study.

The test results of Vermeer (1986) show for constant values of the relative water level an increase in stability in the range $0.0 < B/L_o < 0.15$, Figure A.1. As the relative berm length gets larger, the results show a decrease in stability. The results of this study do not show this down fall but indicate a more horizontal limit for r_D. Apparently the influence of the relative water level and/or the slope angle is such that it is not possible to use the results of Vermeer (1986) as presented in Figure 2.7.

A well-founded explanation for the peak, created by the decrease in stability for larger values of the relative berm length, is not given by Vermeer (1986). It is only remarked that it could be created by interference of the waves. Interesting to see is that the peak tends to flatten out as the water level on the berm decreases. If this is caused by a decrease of the interference, it would be possible that the peak eventually develops into a horizontal limit. This would connect the findings of the two researches creating the possibility to develop a consistent theory.

4.6 Design principles

For breakwaters with a bermed slope no design formulae or design guideline are available, therefore design is mainly based on experience gained from previous projects. However, for an indication of the increase in stability due to a berm two different design principles are used. Both are based on stability formulae which are developed for different types of structures.

Principle 1

The first design principle incorporates the positive influence of a bermed slope on the stability in the assumption that the stability of a bermed slope is expected to be approximately equal to the stability of its average slope, Figure 4.9. This principle doesn't make a distinction between the different damage area's, the influence of the water level or the active zone of the waves.

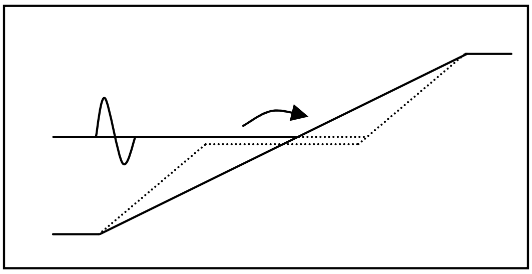


Figure 4.9: Average slope of a bermed sloped breakwater

In this study to types of bermed slopes are investigated; a 0.3m berm and a 0.6m berm. For both cross-sections the average slope is determined as shown in Figure 4.9 after which the stability number is determined with the formula of Van Gent *et al.* (2003). The increase in stability is determined in regard to a uniform 1:2 slope, Table 4.2.

Cross-section	$cot\alpha$	r_D
No berm	2.00	1.00
0.3m berm	2.33	1.08
0.6m berm	2.67	1.17

Table 4.2: Average slope of cross-sections with the increase in stability factor

Principle 2

The second design principle uses formulae for low-crested structures, Section 2.5.1, to determine the increase in stability for the lower slope and berm. It assumes that part of the wave energy is distributed over the berm which therefore reduces the impact of the incident waves on the lower slope. A distinction is made between emerged and submerged structures, Figure 4.10.

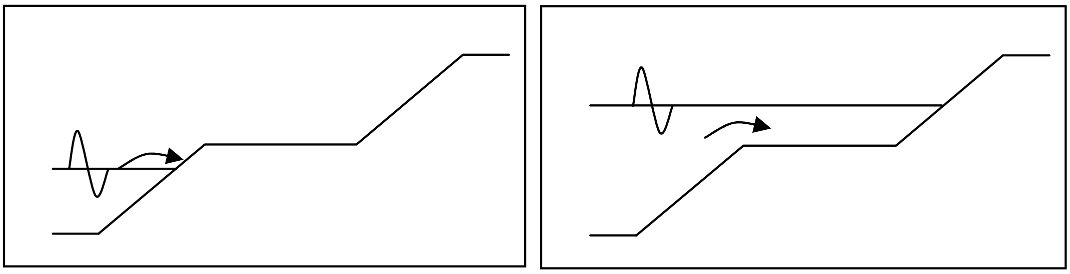


Figure 4.10: Principle of low-crested structure for bermed slopes

For emerged structures the reduction factor for the stone size is determined with Equation 2.17. This is the inverse of the increase in stability factor, r_D. For submerged structures the stability number was determined with Equation 2.18. To determine the increase in stability factor, r_D, it is compared to the stability number of an uniform 1:2 slope determined with the formula of Van Gent *et al.* (2003).

Both formulae are related to the wave steepness. However, it was determined that the influence of the wave period is not clear, Section 4.3.3. Therefore the formula by Van Gent *et al.* (2003) was used to predict the stability on uniform slopes and not the formulae by Van der Meer (1988).

The increase in stability of the low-crested structures is determined in regard to the stability of an uniform slope. Therefore a wave period independent stability formula is combined with a wave period depended stability formula. This has its effects on the determination of the increase in stability factor.

Correlation

The results of both design principles, together with the results of this study, are plotted in Figure 4.11. The correlation between the two can be determined:

Principle 1: The independents of this design principle to the relative water level manifests itself by a horizontal line. This results in a very poor correlation between the actual damage and the predicted damage. As long as the relative berm length is not incorporated in the design principle it is not suitable to use for the prediction of the increase in stability.

Principle 2: The increase in stability predicted with this principle for both emerged as submerged structures is not much better. It does predict the trend of the influence of the relative water level correct but the overall correlation is still poor. The abrupt transition between the two types of structures is caused as two different formulae are used. Furthermore, the

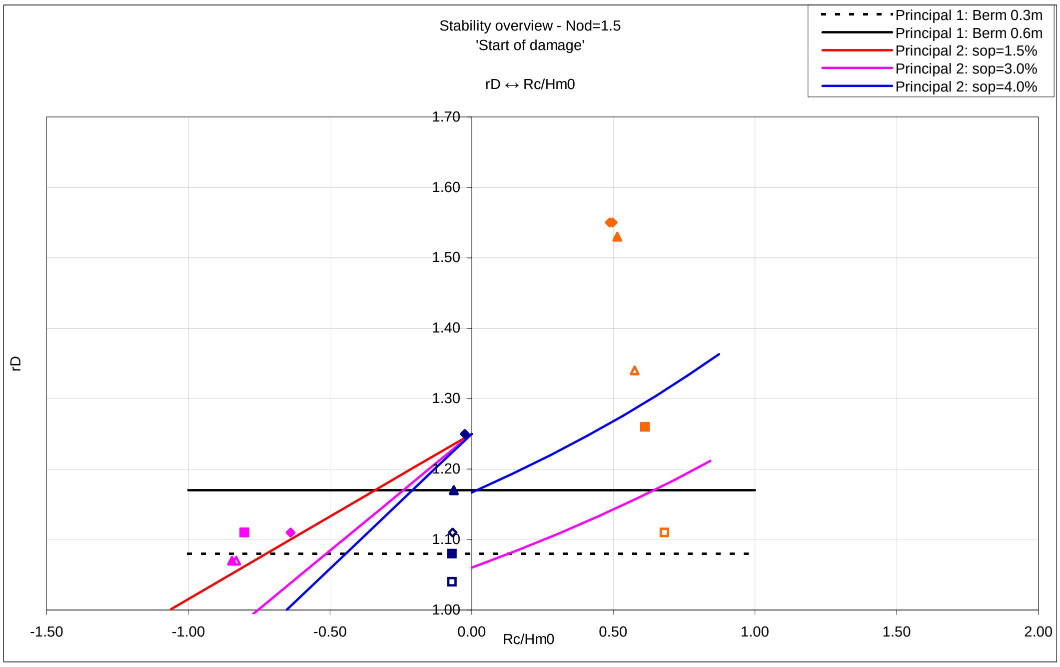


Figure 4.11: Predicted stability increase by the design principles

prediction of the increase in stability for the submerged structures is distorted as result of the above-mentioned influence of the wave period.

Both design principles don't give reliable prediction of the stability increase for any range of the relative water level. Apparently the complexity of the processes related to the (in)stability of armour layers on a bermed slope can not be overcome by means of the design principles. This is most probably caused as the influence of the return current, which has large impact on the stability, is not accounted for. Therefore both principles are not well suited to predict the increase in stability of armour layers on bermed slopes.

Chapter 5

Conclusions and recommendations

As a result of this study insight is gained on the influence of the governing parameters of a berm, on the stability of breakwater armour layers. This chapter will emphasizes the most important conclusions and in addition give some recommendations for further research.

5.1 Conclusions

To determine the influence of the governing parameters of a berm, on the stability of breakwater amour layers, test series were performed on bermed sloped breakwaters for which the development of damage was determined. Analysis of this data revealed great similarities to the damage development of the test series which were performed on similar breakwater but without a berm; an uniform sloped breakwater.

First of all it was concluded that the damage parameter corresponding 'failure' of the armour layer, was approximately the same for bermed sloped breakwaters as for uniform sloped breakwaters. By analogy, the damage parameter for 'start of damage' for bermed slopes was the same as for uniform slopes. The corresponding value of the damage parameter was $N_{od} = 1.5$ or $S_d = 3$.

Secondly it was determined that damage on a bermed slope could be related to damage on an uniform slope by the stability increase factor, r_D. This factor relates the stability parameter of a bermed slope to the stability parameter of an uniform slope for a similar damage level, Equation 5.1:

$$r_D = \frac{H_{mo}/\Delta D_{n50}\text{-berm}}{H_{mo}/\Delta D_{n50}\text{-uniform}} \tag{5.1}$$

It became clear that, for the tested range of the governing parameters of a berm, a berm had a positive effect on the stability. The damage development of an individual test series could be predicted correctly by multiplying the predicted damage on an uniform slope - in this study determined with Van Gent *et al.* (2003) - with a constant value of r_D. An example is given in Figure 5.1. The values of these constants are presented in Table 4.1.

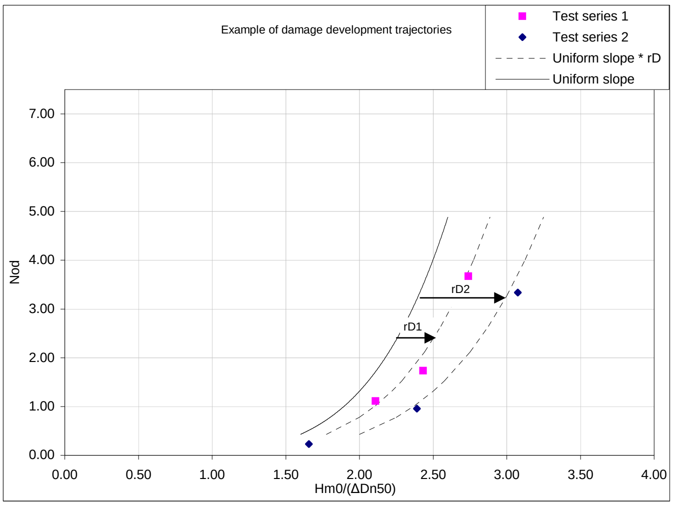


Figure 5.1: Example of damage development trajectories

The correlation between the increase in stability factors and the governing parameters, $B/L_{m-1,0}$ and R_c/H_{m0}, for the damage level 'start of damage' were determined, Appendix L. Unfortunately damage was mainly created on the lower slope and berm and hardly any on the upper slope. Therefore further analysis only focused on the lower slope and berm.

For constant values of R_c/H_{m0} the data showed for initial values of the relative berm length an increase in stability as $B/L_{m-1,0}$ increased. As the relative berm length got increasingly larger the results indicated a horizontal limit, Figure 5.2.
The range in which the relative berm length had a positive contribution on the increase in stability was strongly related to the relative water level. For $R_c/H_{m0} < 0$ this range was small but it widened quickly as the water level approached the berm level. As the water level on the berm increased, the range became gradually smaller again.

A research by WL|Delft Hydraulics (Vermeer (1986)), which performed similar tests but for $R_c/H_{m0} > 0.9$, showed different results. The development of the increase in stability for constant values of the relative berm length showed a peak at $B/L_o = 0.15$, Figure A.1.

A well-founded explanation for the peak, created by the decrease in stability for larger values of the relative berm length, is not given by Vermeer (1986). It is only remarked that it could be created by interference of the waves. Interesting to see is that the peak tends to flatten out as the water level on the berm decreases. If this is caused by a decrease of the interference, it would be possible that the peak eventually develops into a horizontal limit. This would

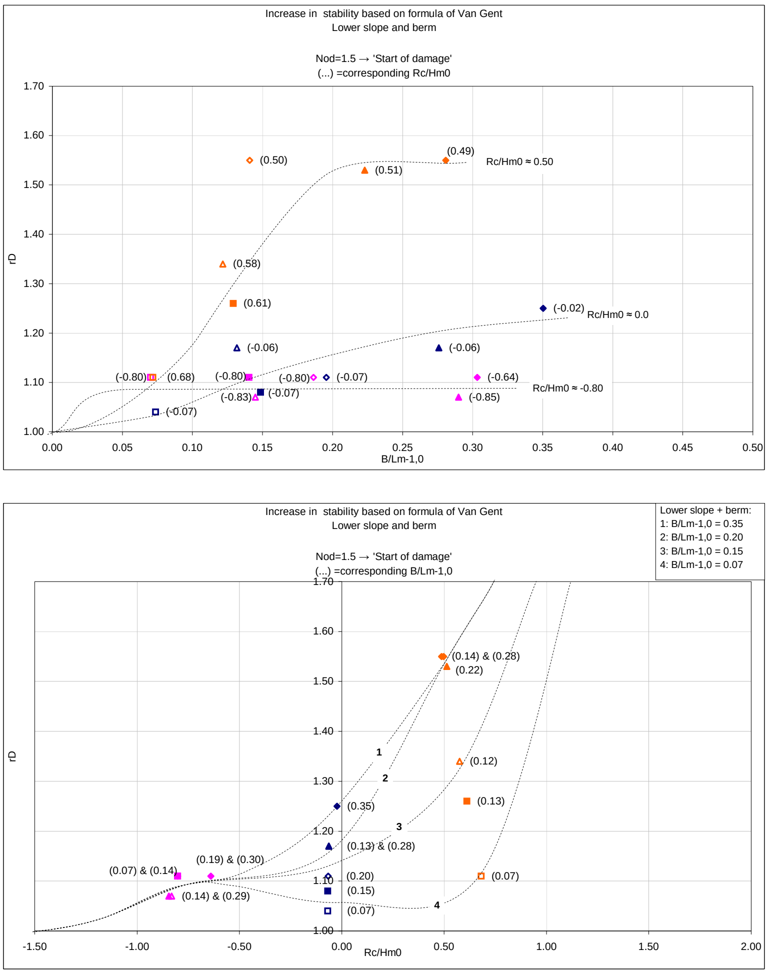


Figure 5.2: Influence of the governing parameters with conceivable trendlines

connect the findings of the two researches creating the possibility to develop a consistent theory.

Because it wasn't possible to analyse the stability of the upper slope as result of a lack of data, it wasn't possible to determine the relative water level for which a maximum increase in stability would be reached for the complete front slope. However, as no damage developed on the upper slope it indicated that for $R_c/H_{m0} < 0.5$ the stability of the lower slope and berm are normative to the complete front slope. This applies with the findings of Vermeer (1986) which determined that the maximal increase in stability is reached for a relative water level of approximately $R_c/H_{m0} = 1.0$.

Finally two design principles, which in practice are used as indication of the increase in stability, were validated with the results of this study. The first principle applies stability formulae for uniform slopes on the average slope of a bermed profile, Figure 4.9. The second principle adoptes the characteristics of low-crested structures on the bermed profile, Figure 4.10.

For both principles the correlation between the predicted stability increase and the results of the test series is very low, Figure 4.11. Apparently the complexity of the processes related to the (in)stability of armour layers on a bermed slope can not be overcome by means of these principles. This is most probably caused as the influence of the return current, which has large impact on the stability, is not accounted for. Therefore both principles are not well suited to predict the increase in stability of armour layers on bermed slopes.

Only in situations where the influence of the return current is low, i.e. extremely long berms or relatively high berm level, the second principle could give good results. But for situations where the return current can not be disregarded it is better to use the tests results this study, Appendix L, to get in indication of the increase in stability. If larger values of R_c/H_{m0} are regarded, the results of Vermeer (1986) can be used, Appendix A.

Last of all it should be emphasized that for design situation one must be very reserved when adopting the findings of this study as well as the findings of Vermeer (1986). Both researches are based on a small amount of test series and certain interpretations could be challenged.

5.2 Recommendation

To get a better understanding of the influence of a berm on the stability of breakwater armour layers under wave attack more research to this topic is required. In the following four recommendation will be given for further research;

1. The results of this study gives a first insight in the influence of a berm on the stability of breakwater armour layers. A similar research, but for a different angle of the lower slope and a the different range of R_c/H_{m0}, was performed by WL|Delft Hydraulics (Vermeer (1986)).

The finding of both studies are based on a relatively small amount of tests. The relations between the governing parameters of a berm and the increase in stability are therefore not always very clear. It is therefore recommended to obtain more test data to derive more reliable and less ambiguous relations.

2. When comparing the finding of both studies on the influence of the relative berm length on the increase in stability a difference can be seen. For initial values of the relative berm length both studies show an increase in stability as the relative berm length increases. As it gets increasingly larger the results of this study indicate a horizontal limit, as for the results of the study by Vermeer (1986) a decrease in stability can be seen for $B/L_o > 0.15$.

An interesting additional research would be to investigate the cause of this drop in stability and determine if it could also occur for the range of R_c/H_{m0} as tested in this study.

3. The armour layer of the complete front slope was build-up out of one type of armour stone; no distinction was made between the lower slope, berm and upper slope. From the test results it became evident that the armour on the upper slope remained relatively stable for the range of the tested governing parameters. Therefore it wasn't possible to analyse the influence of these parameters on the stability of the upper slope.

A second interesting additional research would be to preform test series in which the stability of the armour stone on the upper slope becomes critical. This is possible by using a smaller size of the armour stone or by completely fixating the berm and/or lower slope and increasing the wave load.

4. It is mentioned in the introduction that due to the complexity of the process underlying the instability of individual stones created by the water movement of breaking waves on a slope, it is very difficult to express the interaction of forces into a theoretical model.

To simplify this process somewhat, a distinction can be made between the two most important damage mechanisms on a bermed slope. The first is the impact of the incident waves and the second is the drag force of the return current over the surface area of the berm.

The first damage mechanism remains difficult to express in a theoretic model as the influence of the air encapsuled in the wave isn't fully understood. However, it should be possible to describe the characteristics of the return current with a theoretical model.

A third interesting additional research would be to investigate if such models are available. If so, an effort can be made to adapt it to breakwaters with a bermed slope and correlate the predicted velocities of a return current with the damage development of this study and Vermeer (1986).

Appendix

Appendix A

Test results Vermeer (1986)

The authentic graphs as presented in Vermeer (1986) are included in this Appendix. Not all symbols correspond to the ones used in this study, thus:

- $f_s = r_D$
- $d_B/H_{si} = R_c/H_s$

The pink areas indicates the test results from this study.

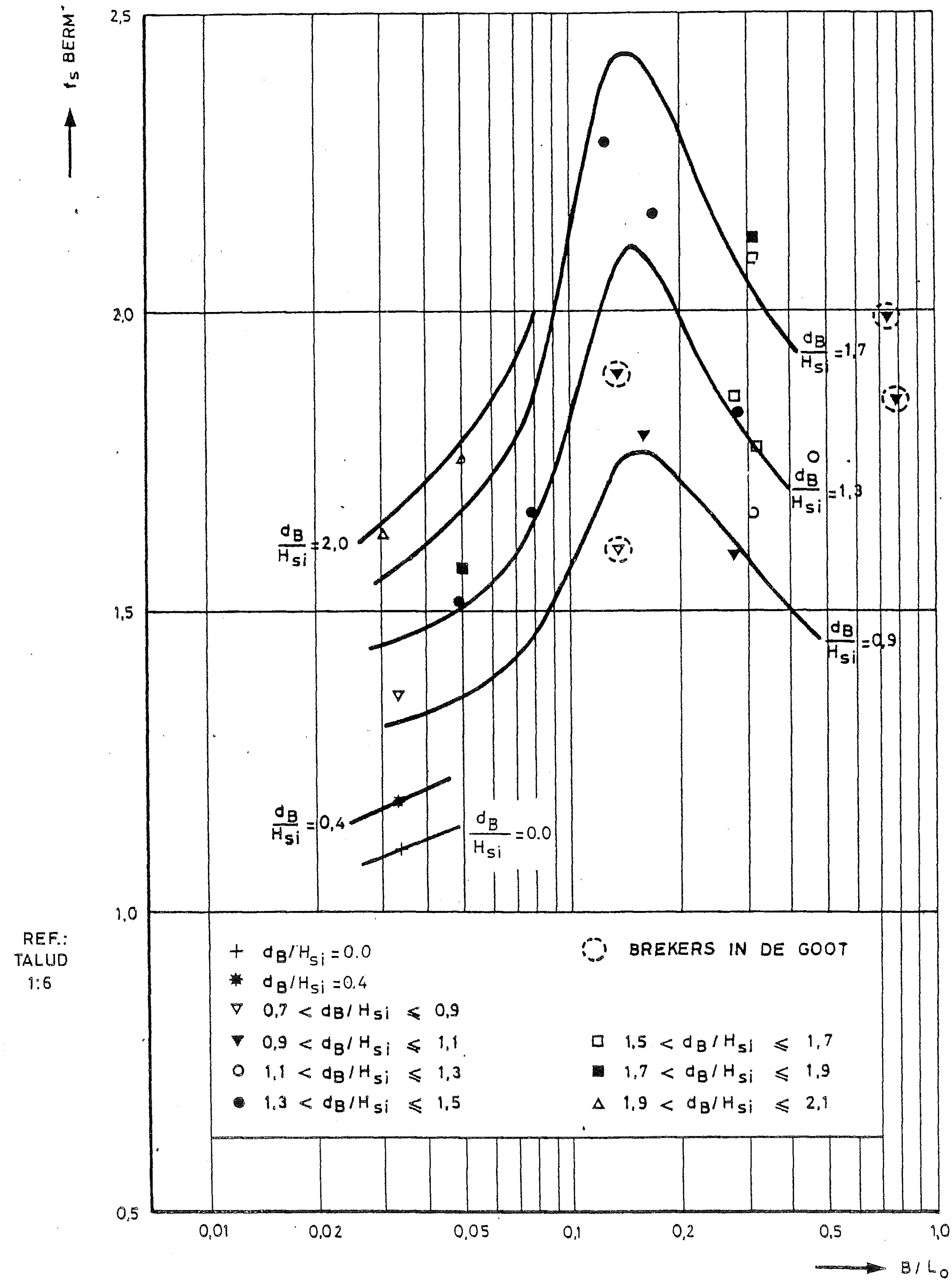


Figure A.1: Development of the increase in stability of the berm material for $N_{od} = 1.67$.

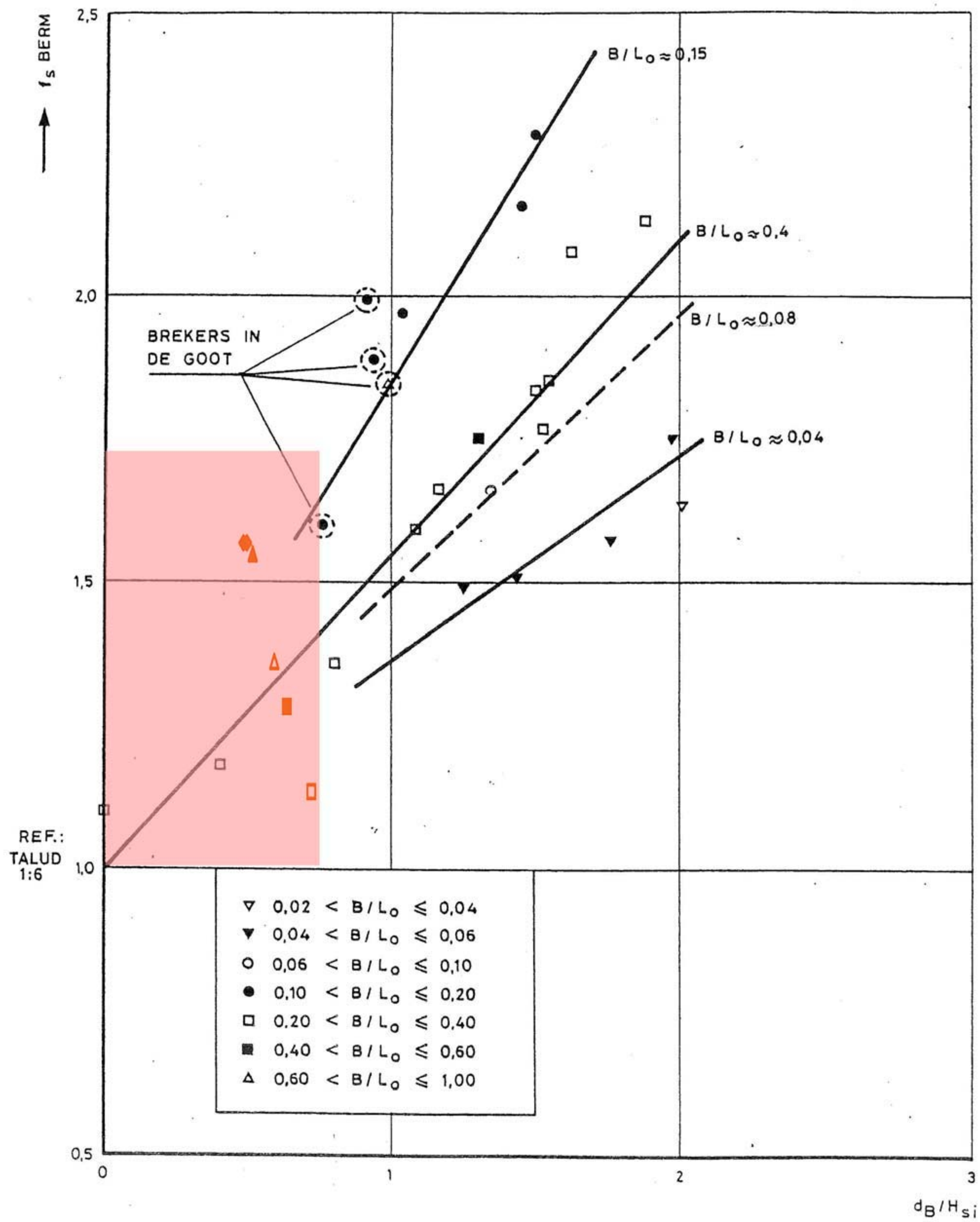


Figure A.2: Development of the increase in stability of the berm material for $N_{od} = 1.67$.

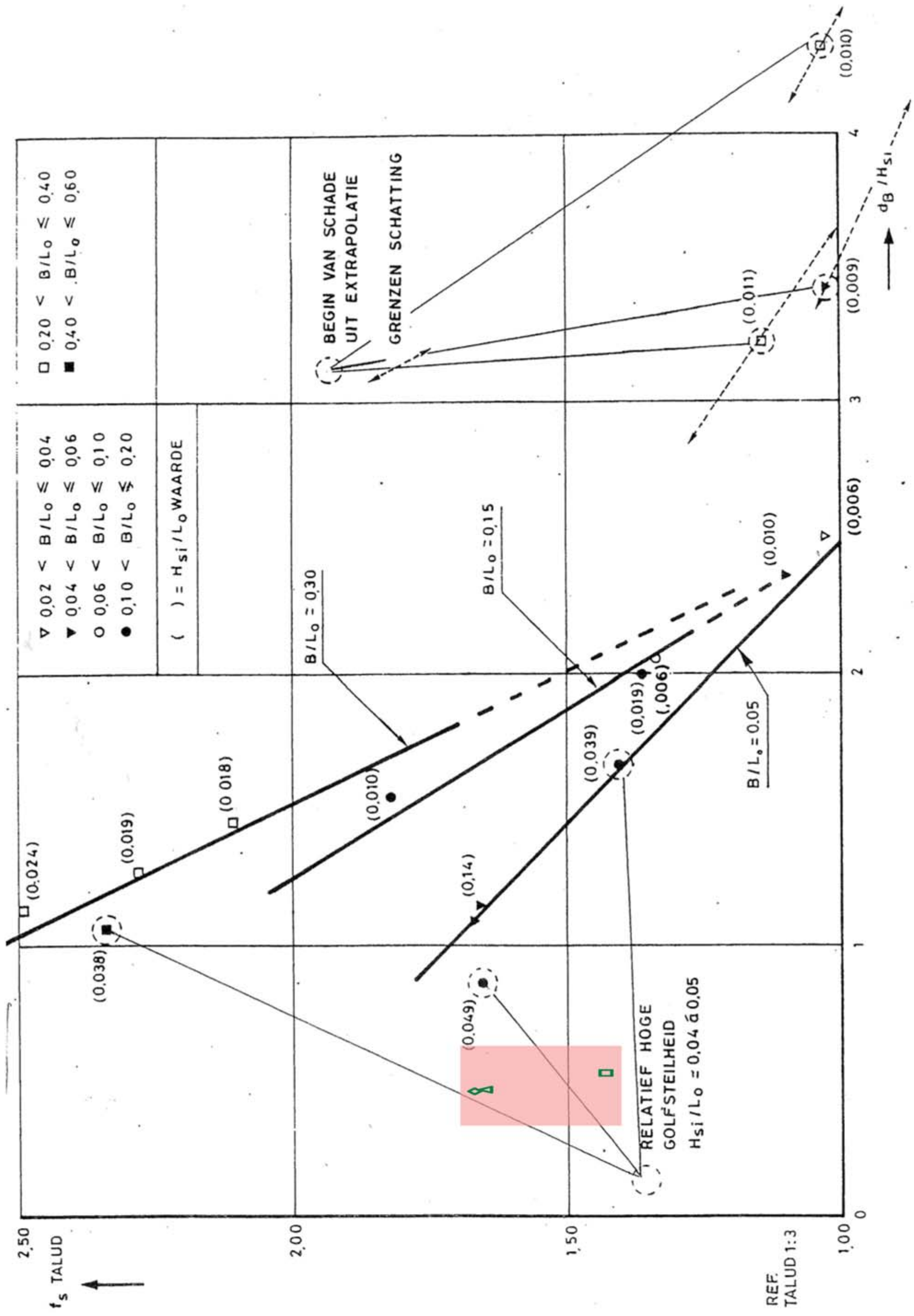
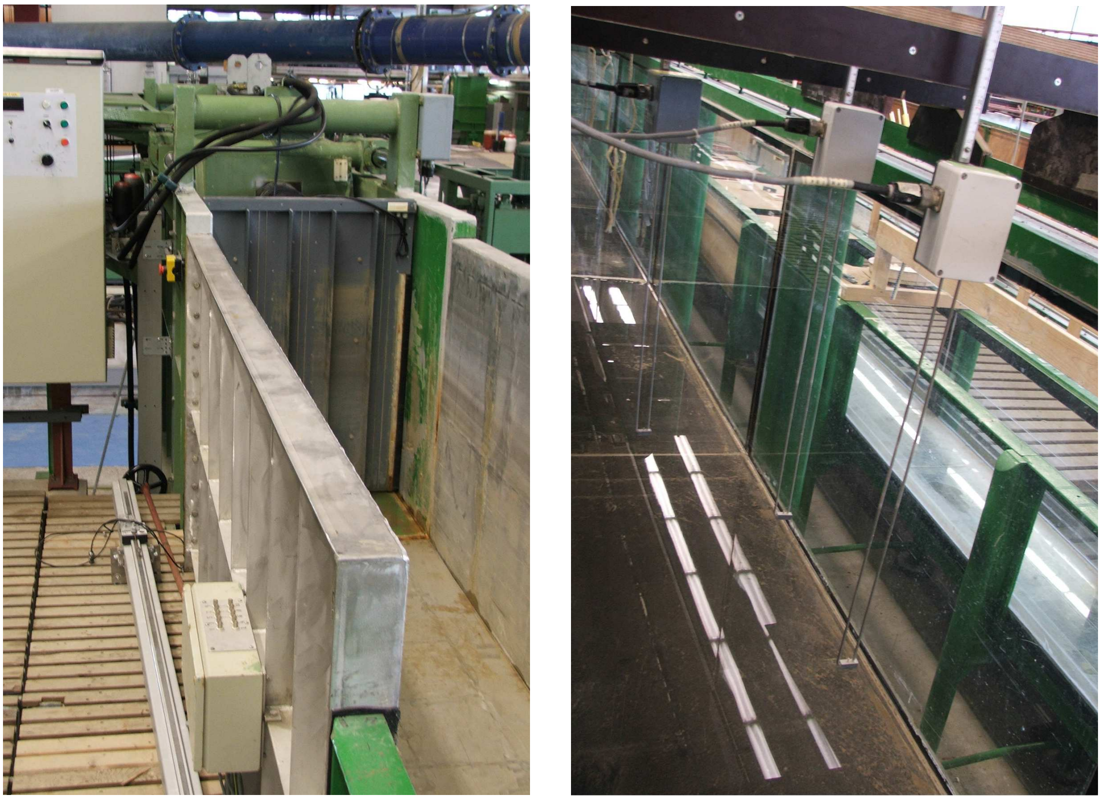


Figure A.3: Development of the increase in stability of the upper slope for $N_{od} = 1.11$.

Appendix B

Pictures facilities



(a) Wave board (b) Wave gauges

Figure B.1

(a) Computer facilities

(b) Tail board

Figure B.2

(a) Side view cross-section

(b) Side view cross-section

Figure B.3

Appendix C

Model set-up

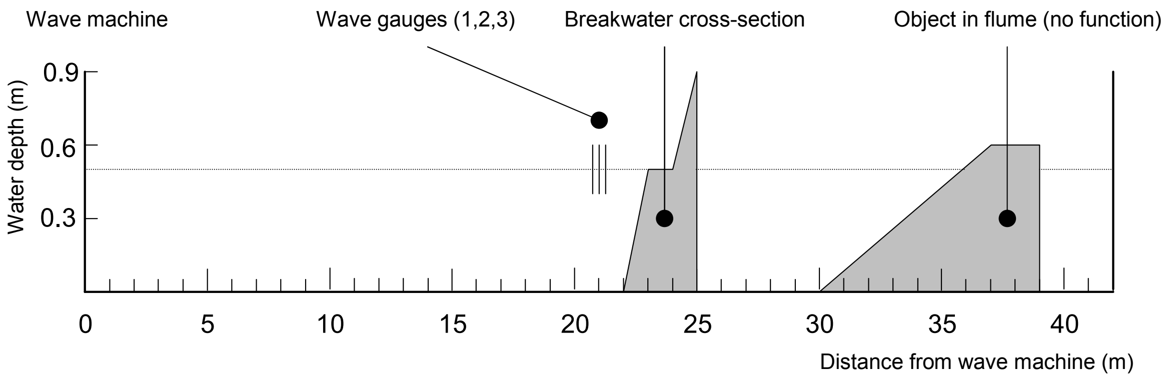
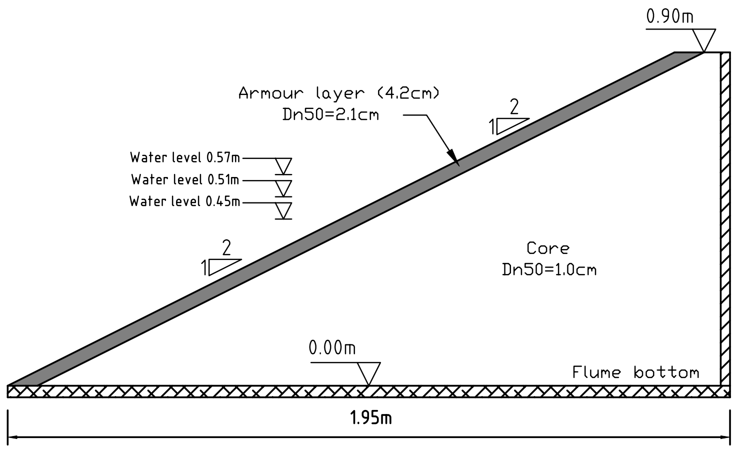
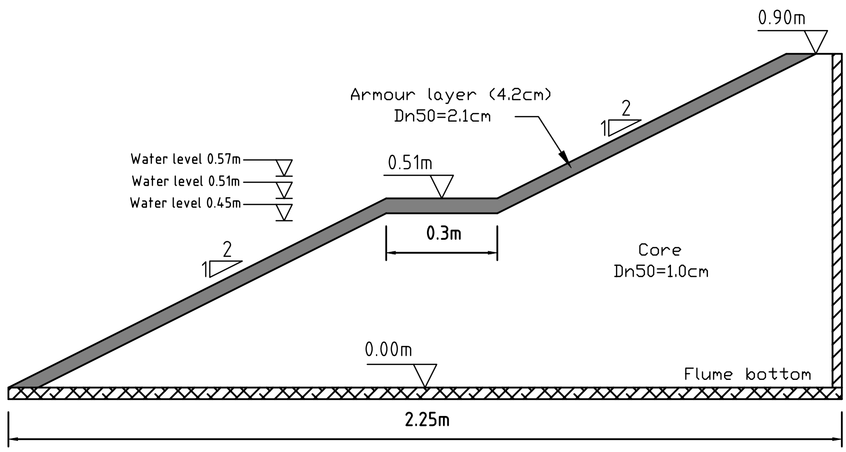


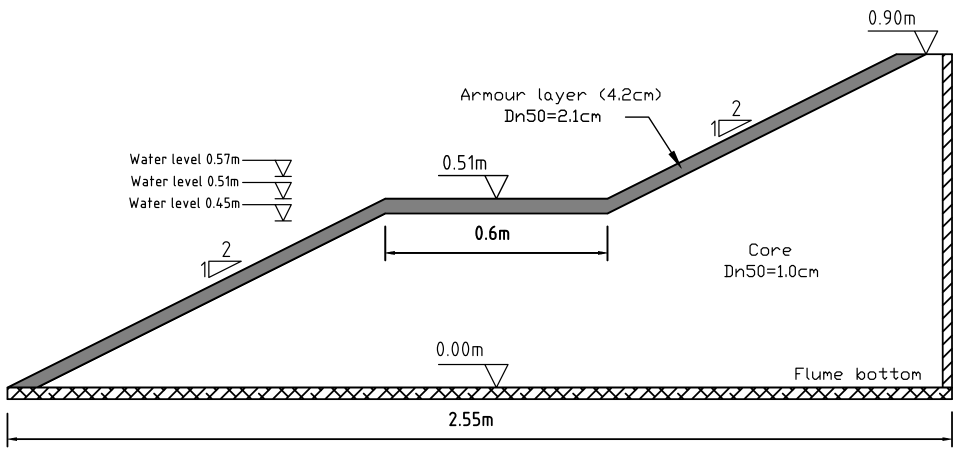
Figure C.1: Model set-up



(a) Cross-section, no berm



(b) Cross-section, 0.3m berm



(c) Cross-section, 0.6m berm

Figure C.2: Cross-sections

Appendix D

Porisity armour stone

Test	Volume water [dm^3]	Volume total [dm^3]	p [-]
1	3.9	8.5	0.45
2	4.4	10.0	0.44
3	4.6	10.0	0.46
4	4.5	10.0	0.45
5	4.6	10.0	0.46
6	4.6	10.0	0.46
7	4.3	10.0	0.43
		Average	p=0.45

Table D.1: Porosity of armour layer stones

Appendix E

Wave spectra

The type of waves covered in this MSc thesis are gravity waves propagating at the surface of a water body which are locally generated by the action of the wind at the free surface. This type of sea-state is called a wind-sea and are characterised by short periods (2 s to 10 s typically) and provide an irregular aspect of the sea surface.

A sea-state can be described with a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency, this is called a wave spectrum. Two of the most widely used spectra are those described by Pierson & Moskowitz (1964) and the JONSWAP spectrum (Hasselmann *et al.* (1973)), shown in Figure E.1. These spectra are formulated using a power function with respect to the frequency containing several scaling parameters and constants.

The main difference between the two is that the Pierson and Moskowitz (PM) spectrum represents a fully developed sea in deep waters. This is only the case if the fetch is very long or the winds are weak. Mostly this is not the case. In addition to the PM spectrum the Joint North Sea WAve Project (JONSWAP) studied fetch-limited sea-states, ie growing sea, resulting in the JONSWAP spectrum.

$$E(f) = \alpha g^2 (2\pi)^{-4} f^{-5} exp\left[-\frac{5}{4}\left(\frac{f}{f_p}\right)^{-4}\right] \gamma^{exp\left(-\frac{(f/f_p-1)^2}{2\sigma^2}\right)} \tag{E.1}$$

with:

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

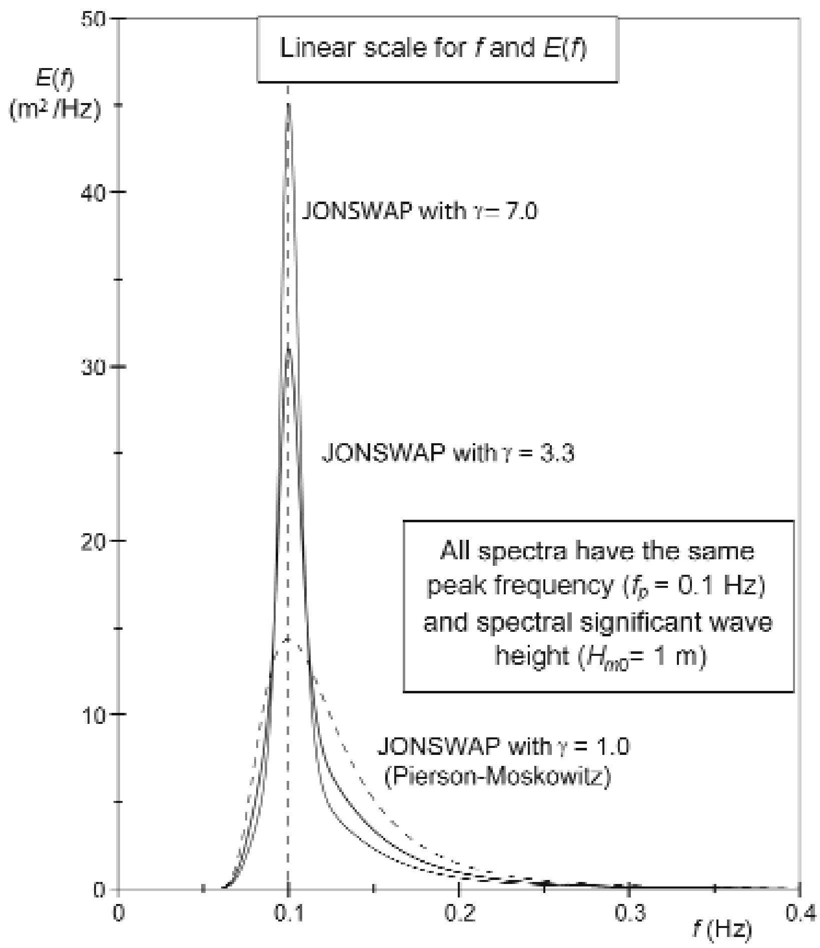
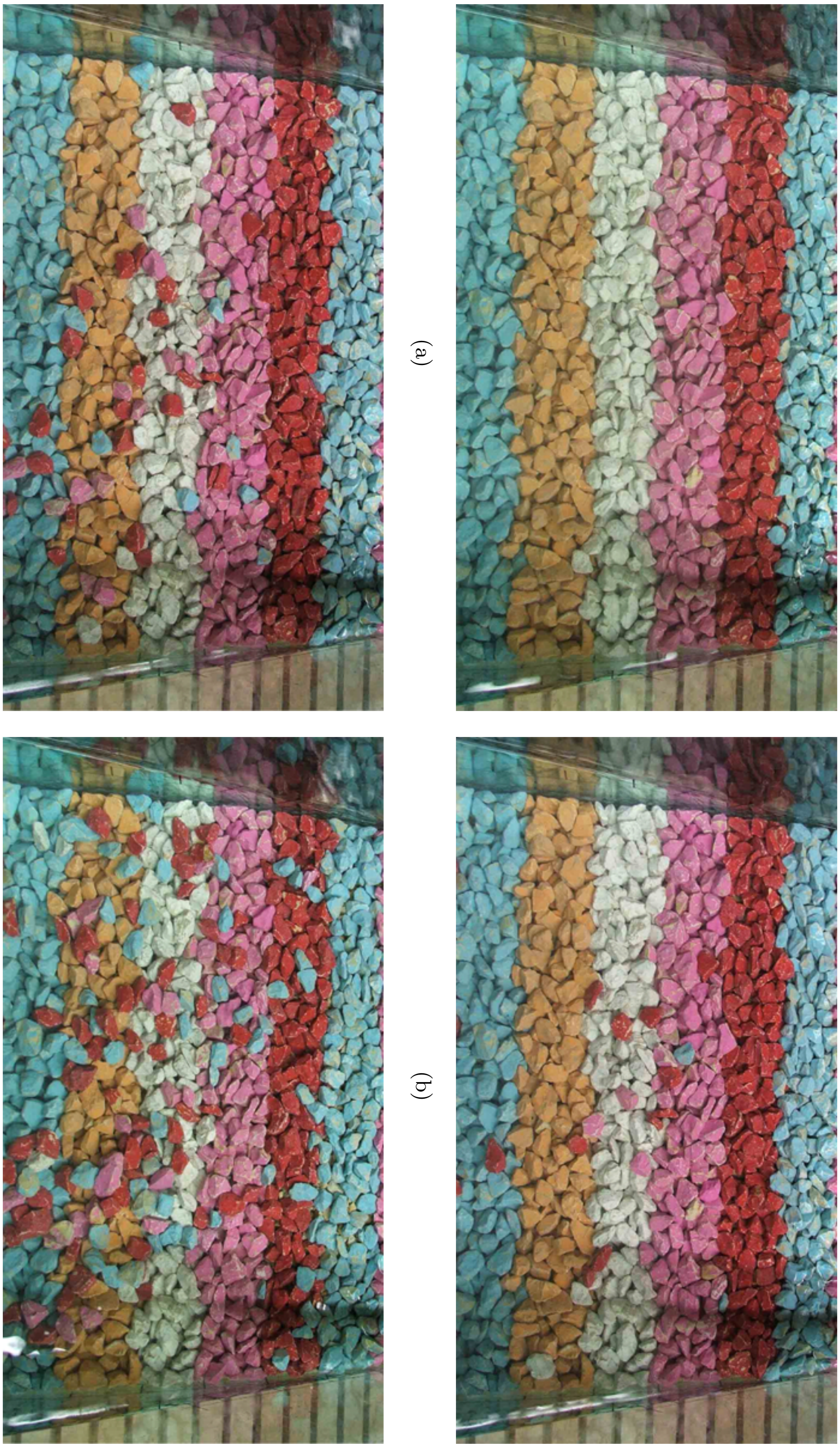


Figure E.1: Pierson&Moskowitz and JONSWAP spectrum

Appendix F

Damage recording pictures



(a) (b) (c) (d)

Figure F.1: Example of pictures of a test series for the determination of damage

Appendix G

Location of colour bands

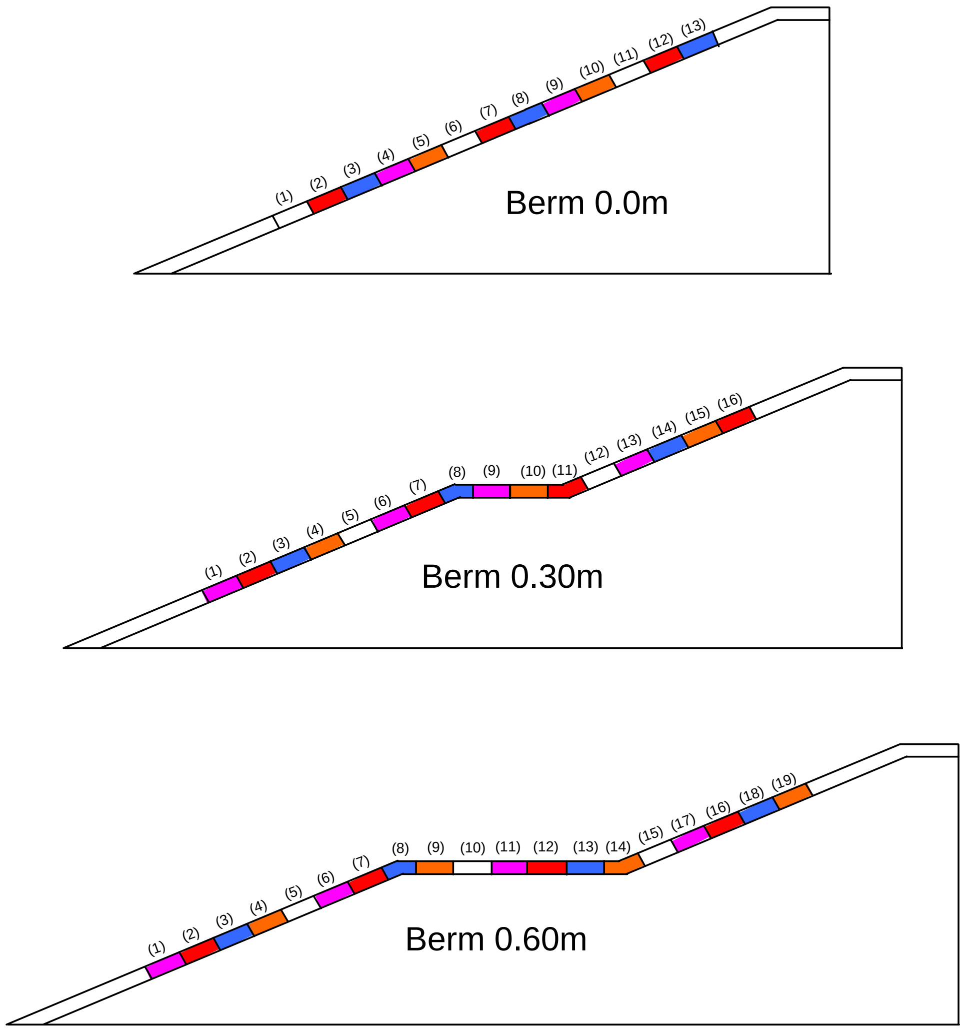


Figure G.1: Location and numbering of the colour bands

Appendix H

Processed test results

- Table H.1: Cumulative damage N_d
- Table H.2: Wave characteristics and damage
- Table H.3: Increase stability factor r_D relative to Van Gent for $N_{od} = 1.5$

Test series		B [m]	Pi (1)	Re (2)	Bl (3)	Or (4)	Wh (5)	Pi (6)	Re (7)	Bl (8)	Or (9)	Wh (10)	Pi (11)	Re (12)	Bl (13)	Or (14)	Wh (15)	Pi (16)	Re (17)	Bl (18)	Or (19)
T103x	1	0.6	-	-	-	-	-	-	4	1	2	1	1	-	-	-	-	-	-	-	-
	2		-	-	-	-	1	2	20	5	6	1	1	1	-	-	-	-	-	-	-
	3		-	-	-	-	3	12	57	33	8	6	4	5	1	-	-	-	-	-	-
T104x	1	0.6	-	1	-	-	1	1	9	2	1	5	-	1	-	-	-	-	-	-	-
	2		-	1	-	1	3	5	26	9	4	13	4	1	3	2	-	-	-	-	-
	3		-	1	1	1	6	12	51	25	5	15	9	6	7	4	-	-	-	-	-
T105x	1	0.6	-	-	-	-	-	1	1	1	1	1	-	-	-	-	-	-	-	-	-
	2		-	-	-	1	-	9	22	10	2	3	-	-	1	-	-	-	-	-	-
	3		-	-	-	1	4	25	70	44	5	7	3	5	2	-	1	1	-	-	-
T106x	1	0.6	-	-	-	-	-	1	1	-	-	-	-	-	-	-	-	-	-	-	-
	2		-	-	-	-	-	3	10	6	-	1	-	-	-	-	-	2	-	-	-
	3		-	-	-	-	-	4	20	26	1	1	-	-	-	-	1	5	-	-	-
	4		-	-	-	2	1	9	25	32	2	2	1	5	3	-	3	10	-	-	-
T107x	1	0.6	-	-	-	-	-	-	7	2	1	-	-	-	-	-	-	1	-	-	-
	2		-	-	-	-	-	5	24	17	3	-	-	-	1	-	1	6	2	-	-
	3		1	-	-	-	1	8	36	31	6	-	1	2	1	-	4	11	5	-	-
T108x	1	0.6	-	-	-	-	-	2	4	7	-	-	-	-	-	-	-	1	-	-	-
	2		-	-	-	-	-	4	18	16	1	1	-	-	-	-	-	3	-	-	-
	3		-	-	-	-	-	4	31	33	4	1	1	-	-	1	-	4	-	-	-
	4		-	-	1	-	1	7	49	59	27	1	1	1	-	1	2	6	-	-	-
T110x	1	0.6	-	-	-	-	2	4	2	-	-	-	-	-	-	-	-	-	-	-	-
	2		-	-	-	1	11	20	6	1	1	1	-	-	-	-	-	-	-	-	-
	3		-	1	1	3	16	47	25	2	1	2	-	-	-	-	-	-	-	-	-
T111x	1	0.6	-	-	-	-	2	5	1	1	-	-	-	-	-	-	-	-	-	-	-
	2		-	-	1	1	6	16	7	1	-	-	-	-	-	-	-	-	-	-	-
	3		2	1	2	10	9	36	28	4	2	-	-	1	-	-	-	-	-	-	-
	4		2	1	3	20	16	55	52	11	6	2	3	7	1	-	-	-	-	-	-
T112x	1	0.6	-	-	-	-	-	2	8	-	-	-	-	-	-	-	-	-	-	-	-
	2		-	-	-	1	-	4	19	3	1	-	-	-	-	-	-	-	-	-	-
	3		-	-	1	2	4	17	42	5	1	-	-	-	-	-	-	-	-	-	-

Test series		B [m]	Pi (1)	Re (2)	Bl (3)	Or (4)	Wh (5)	Pi (6)	Re (7)	Bl (8)	Pi (9)	Or (10)	Re (11)	Wh (12)	Pi (13)	Bl (14)	Or (15)	Re (16)
T113x	1	0.3	-	-	-	-	1	1	17	12	9	3	-	-	-	-	-	-
	2		-	-	-	-	2	1	26	25	10	3	-	-	-	-	-	-
	3		-	-	-	-	4	4	56	59	14	5	-	-	-	-	-	-
T114x	1	0.3	-	-	-	-	-	1	3	-	4	-	-	-	-	-	-	-
	2		-	-	-	-	-	2	10	7	9	-	-	-	-	-	-	-
	3		-	-	-	1	6	42	89	109	27	8	-	-	-	-	-	-
T115x	1	0.3	-	-	-	-	-	1	7	3	3	4	-	1	-	-	-	-
	2		-	-	-	-	1	3	12	5	3	8	1	1	-	-	-	-
	3		-	-	-	-	1	16	26	13	5	8	2	2	-	-	-	-
T116x	1	0.3	-	-	-	-	-	-	1	3	-	-	-	2	4	-	-	-
	2		-	-	-	-	-	-	3	3	-	2	-	4	5	-	-	-
	3		-	-	-	2	-	3	24	36	4	3	-	7	19	4	-	-
	4		-	-	1	2	-	10	35	68	11	4	-	10	26	6	-	-
T117x	1	0.3	-	-	-	-	-	1	13	21	1	-	-	5	6	1	-	-
	2		-	-	-	-	-	1	23	35	2	-	-	7	9	2	-	-
	3		-	-	-	-	1	11	61	99	19	-	-	18	18	4	-	-
T118x	1	0.3	-	-	-	-	-	-	2	2	-	-	-	3	2	-	-	-
	2		-	-	-	-	1	1	7	7	1	-	-	6	5	-	-	-
	3		-	-	1	3	2	2	16	24	1	2	-	12	23	4	-	-
	4		-	-	1	3	3	14	31	57	6	3	-	20	41	17	1	-
T119x	1	0.3	-	2	-	3	5	20	19	2	4	2	-	1	-	-	-	-
	2		1	2	1	3	7	39	57	16	7	2	-	1	-	-	-	-
	3		1	2	1	3	14	68	82	24	7	3	2	1	-	-	-	-
T120x	1	0.3	-	-	-	-	4	12	3	1	-	-	-	-	-	-	-	-
	2		-	-	-	-	5	20	8	1	-	-	-	-	-	-	-	-
	3		-	-	-	-	9	36	34	8	-	-	-	-	-	-	-	-
T121x	1	0.3	-	-	-	2	7	12	5	-	-	-	-	-	-	-	-	-
	2		-	-	-	2	9	42	30	2	-	-	-	-	-	-	-	-
	3		-	-	-	2	16	52	93	7	-	1	-	-	-	-	-	-

Test series		B [m]	Wh (1)	Re (2)	Bl (3)	Pi (4)	Or (5)	Wh (6)	Re (7)	Bl (8)	Pi (9)	Or (10)	Wh (11)	Re (12)	Bl (13)
T122x	1	0.0	-	-	-	-	-	-	6	11	1	-	-	-	-
	2		-	-	-	-	-	-	24	29	6	-	-	-	-
	3		-	-	-	-	-	1	29	44	17	1	-	-	-
	4		-	-	-	-	-	3	40	100	73	15	-	-	-
T123x	1	0.0	-	-	-	-	-	-	7	13	7	-	-	-	-
	2		-	-	-	-	1	5	16	34	19	-	-	-	-
	3		-	-	-	-	4	6	25	53	38	2	-	-	-
	4		-	-	-	-	4	7	39	90	87	49	-	-	-
T124x	1	0.0	-	-	-	-	-	5	8	13	1	-	-	-	-
	2		-	-	-	1	2	7	16	20	10	-	-	-	-
	3		-	-	1	3	4	17	45	84	58	27	-	-	-

Table H.1: Cumulative damage N_d

Test series		B	R_c	h	H_{mo}	$T_{m-1,0}$	$s_{m-1,0}$	s_{op}	$H_{mo}/(\Delta D_{n50})$	R_c/H_{mo}	$B/L_{m-1,0}$	Lower slope + berm N_d	Lower slope + berm N_{od}	Upper slope N_d	Upper slope N_{od}
		[m]	[m]	[m]	[m]	[s]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
T103x	1	0.6	-0.002	0.515	0.0542	0.94	0.0392	0.0320	1.66	-0.037	0.43	9	0.23	0	0.00
	2				0.0783	1.01	0.0495	0.0404	2.39	-0.026	0.38	37	0.96	0	0.00
	3				0.1007	1.15	0.0490	0.0400	3.07	-0.020	0.29	129	3.34	0	0.00
T104x	1	0.6	-0.005	0.512	0.0650	1.03	0.0396	0.0323	1.98	-0.077	0.37	21	0.54	0	0.00
	2				0.0825	1.20	0.0364	0.0297	2.52	-0.061	0.26	72	1.86	0	0.00
	3				0.0950	1.36	0.0328	0.0268	2.90	-0.053	0.21	143	3.70	0	0.00
T105x	1	0.6	-0.005	0.512	0.0523	1.34	0.0187	0.0152	1.60	-0.096	0.21	5	0.13	0	0.00
	2				0.0705	1.59	0.0179	0.0146	2.15	-0.071	0.15	48	1.24	0	0.00
	3				0.0887	1.80	0.0176	0.0144	2.71	-0.056	0.12	166	4.30	2	0.05
T106x	1	0.6	0.051	0.568	0.0582	0.88	0.0480	0.0392	1.78	0.876	0.50	2	0.05	0	0.00
	2				0.0857	1.05	0.0495	0.0404	2.62	0.595	0.35	20	0.52	2	0.05
	3				0.1005	1.15	0.0490	0.0400	3.07	0.507	0.29	52	1.35	6	0.16
	4				0.1142	1.22	0.0492	0.0401	3.49	0.447	0.26	82	2.12	13	0.34
T107x	1	0.6	0.053	0.570	0.0815	1.13	0.0407	0.0332	2.49	0.650	0.30	10	0.26	1	0.03
	2				0.0997	1.34	0.0355	0.0290	3.05	0.531	0.21	50	1.29	9	0.23
	3				0.1132	1.37	0.0389	0.0317	3.46	0.468	0.21	87	2.25	20	0.52
T108x	1	0.6	0.052	0.569	0.0696	1.53	0.0191	0.0156	2.13	0.747	0.16	13	0.34	1	0.03
	2				0.0792	1.70	0.0176	0.0144	2.42	0.656	0.13	40	1.04	3	0.08
	3				0.0897	1.80	0.0178	0.0145	2.74	0.580	0.12	75	1.94	4	0.10
	4				0.0998	1.85	0.0187	0.0153	3.05	0.521	0.11	148	3.83	8	0.21
T110x	1	0.6	-0.061	0.456	0.0566	1.01	0.0353	0.0288	1.73	-1.078	0.37	8	0.21	0	0.00
	2				0.0701	1.14	0.0348	0.0284	2.14	-0.871	0.30	41	1.06	0	0.00
	3				0.0791	1.21	0.0345	0.0282	2.42	-0.771	0.26	98	2.54	0	0.00
T111x	1	0.6	-0.060	0.457	0.0491	1.36	0.0170	0.0139	1.50	-1.223	0.21	9	0.23	0	0.00
	2				0.0657	1.52	0.0183	0.0149	2.01	-0.914	0.17	32	0.83	0	0.00
	3				0.0802	1.70	0.0177	0.0145	2.45	-0.748	0.13	95	2.46	0	0.00
	4				0.0961	1.90	0.0171	0.0139	2.93	-0.624	0.11	179	4.63	0	0.00
T112x	1	0.6	-0.060	0.457	0.0536	0.86	0.0459	0.0375	1.64	-1.119	0.51	10	0.26	0	0.00
	2				0.0716	0.98	0.0480	0.0391	2.19	-0.838	0.40	28	0.72	0	0.00
	3				0.0808	1.03	0.0485	0.0395	2.47	-0.743	0.36	72	1.86	0	0.00

Test series		B	R_c	h	H_{mo}	$T_{m-1,0}$	$s_{m-1,0}$	s_{op}	$H_{mo}/(\Delta D_{n50})$	R_c/H_{mo}	$B/L_{m-1,0}$	Lower slope + berm N_d	Lower slope + berm N_{od}	Upper slope N_d	Upper slope N_{od}
		[m]	[m]	[m]	[m]	[s]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
T113x	1	0.3	-0.005	0.512	0.0691	0.95	0.0487	0.0397	2.11	-0.072	0.21	43	1.11	0	0.00
	2				0.0797	1.02	0.0491	0.0400	2.43	-0.063	0.18	67	1.73	0	0.00
	3				0.0897	1.09	0.0486	0.0397	2.74	-0.056	0.16	142	3.67	0	0.00
T114x	1	0.3	-0.005	0.512	0.0549	1.40	0.0179	0.0146	1.68	-0.091	0.10	8	0.21	0	0.00
	2				0.0708	1.59	0.0179	0.0146	2.16	-0.071	0.08	28	0.72	0	0.00
	3				0.0856	1.75	0.0178	0.0145	2.61	-0.058	0.06	282	7.30	0	0.00
T115x	1	0.3	-0.005	0.512	0.0601	1.05	0.0347	0.0283	1.83	-0.083	0.17	18	0.47	1	0.03
	2				0.0735	1.17	0.0344	0.0281	2.25	-0.068	0.14	33	0.85	1	0.03
	3				0.0859	1.26	0.0348	0.0284	2.62	-0.058	0.12	71	1.84	2	0.05
T116x	1	0.3	0.052	0.569	0.0688	1.13	0.0343	0.0280	2.10	0.755	0.15	4	0.10	6	0.16
	2				0.0801	1.14	0.0397	0.0324	2.45	0.649	0.15	8	0.21	9	0.23
	3				0.0929	1.47	0.0277	0.0226	2.84	0.559	0.09	72	1.86	30	0.78
	4				0.1070	1.38	0.0360	0.0294	3.27	0.486	0.10	131	3.39	42	1.09
T117x	1	0.3	0.051	0.568	0.0708	1.28	0.0275	0.0225	2.16	0.721	0.12	36	0.93	12	0.31
	2				0.0805	1.71	0.0176	0.0143	2.46	0.634	0.07	61	1.58	18	0.47
	3				0.0896	1.77	0.0183	0.0149	2.74	0.569	0.06	191	4.94	40	1.04
T118x	1	0.3	0.052	0.569	0.0746	1.00	0.0482	0.0393	2.28	0.697	0.19	4	0.10	5	0.13
	2				0.0907	1.09	0.0492	0.0401	2.77	0.574	0.16	17	0.44	11	0.28
	3				0.1061	1.17	0.0497	0.0406	3.24	0.490	0.14	51	1.32	39	1.01
	4				0.1192	1.24	0.0494	0.0403	3.64	0.436	0.12	118	3.05	79	2.04
T119x	1	0.3	-0.060	0.457	0.0761	1.65	0.0180	0.0147	2.33	-0.788	0.07	57	1.47	1	0.03
	2				0.0853	1.76	0.0176	0.0143	2.60	-0.703	0.06	135	3.49	1	0.03
	3				0.0951	1.90	0.0170	0.0138	2.91	-0.631	0.05	207	5.36	1	0.03
T120x	1	0.3	-0.060	0.457	0.0595	0.94	0.0432	0.0353	1.82	-1.008	0.22	20	0.52	0	0.00
	2				0.0690	0.97	0.0471	0.0385	2.11	-0.869	0.20	34	0.88	0	0.00
	3				0.0798	1.02	0.0492	0.0401	2.44	-0.752	0.18	87	2.25	0	0.00
T121x	1	0.3	-0.060	0.457	0.0650	1.10	0.0345	0.0281	1.98	-0.923	0.16	26	0.67	0	0.00
	2				0.0730	1.16	0.0348	0.0284	2.23	-0.822	0.14	85	2.20	0	0.00
	3				0.0823	1.22	0.0353	0.0288	2.51	-0.729	0.13	171	4.42	0	0.00

Test series		B	R_c	h	H_{mo}	$T_{m-1,0}$	$s_{m-1,0}$	s_{op}	$H_{mo}/(\Delta D_{n50})$	R_c/H_{mo}	$B/L_{m-1,0}$	N_d	N_{od}
		[m]	[m]	[m]	[m]	[s]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
T122x	1	-	-	0.512	0.0591	0.89	0.0479	0.0391	1.80	-	-	18	0.47
	2				0.0722	0.97	0.0488	0.0398	2.20	-	-	59	1.53
	3				0.0798	1.03	0.0482	0.0393	2.44	-	-	92	2.38
	4	-			0.0857	1.07	0.0477	0.0389	2.62	-	-	231	5.98
T123x	1	-	-	0.512	0.0612	1.06	0.0351	0.0286	1.87	-	-	27	0.70
	2				0.0705	1.14	0.0346	0.0282	2.15	-	-	75	1.94
	3				0.0787	1.24	0.0327	0.0267	2.40	-	-	128	3.31
	4				0.0895	1.26	0.0359	0.0293	2.73	-	-	276	7.14
T124x	1	-	-	0.512	0.0602	1.47	0.0179	0.0146	1.84	-	-	27	0.70
	2				0.0701	1.55	0.0188	0.0153	2.14	-	-	56	1.45
	3				0.0803	1.67	0.0185	0.0151	2.45	-	-	239	6.18

Table H.2: Wave characteristics and damage

Test series	Lower slope				Upper slope			
	r_D	$H_{m0}/(\Delta D_{n50})$	R_c/H_{mo}	$B/L_{m-1,0}$	r_D	$H_{m0}/(\Delta D_{n50})$	R_c/H_{mo}	$B/L_{m-1,0}$
T103x	1.25	2.58	-0.024	0.35	-	-	-	-
T104x	1.17	2.41	-0.063	0.28	-	-	-	-
T105x	1.08	2.22	-0.069	0.15	-	-	-	-
T106x	1.55	3.19	0.488	0.28	-	-	-	-
T107x	1.53	3.15	0.513	0.22	-	-	-	-
T108x	1.26	2.60	0.612	0.13	-	-	-	-
T110x	1.07	2.20	-0.845	0.29	-	-	-	-
T111x	1.11	2.29	-0.801	0.14	-	-	-	-
T112x	1.11	2.87	-0.639	0.30	-	-	-	-
T113x	1.11	2.29	-0.067	0.20	-	-	-	-
T114x	1.08	2.22	-0.069	0.07	-	-	-	-
T115x	1.17	2.41	-0.063	0.13	-	-	-	-
T116x	1.34	2.76	0.575	0.12	1.65	3.40	0.467	0.10
T117x	1.11	2.29	0.681	0.07	1.43	2.95	0.529	0.06
T118x	1.55	3.19	0.497	0.14	1.67	3.44	0.462	0.13
T119x	1.11	2.29	-0.801	0.07	-	-	-	-
T120x	1.11	2.29	-0.801	0.19	-	-	-	-
T121x	1.03	2.20	-0.831	0.14	-	-	-	-

Table H.3: Increase stability factor r_D relative to Van Gent for $N_{od} = 1.5$

Appendix I

Correlation stability formulae for uniform slopes

- Figure I.1: Damage development on uniform slope compared with design formulae.

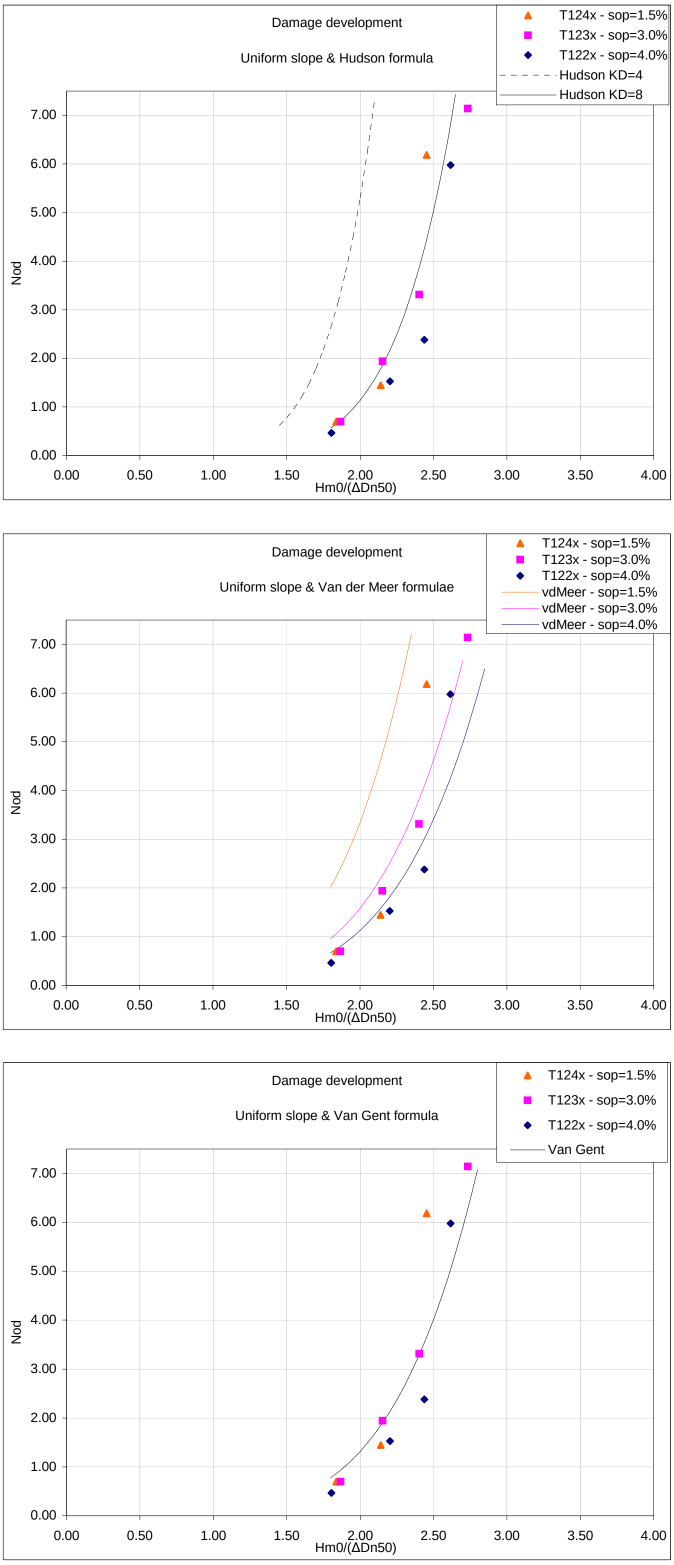


Figure I.1: Damage development on uniform slope compared with design formulae

Appendix J

Damage development on bermed slope

- Figure J.1: Damage development for lower slope and berm
- Figure J.2: Damage development for upper slope

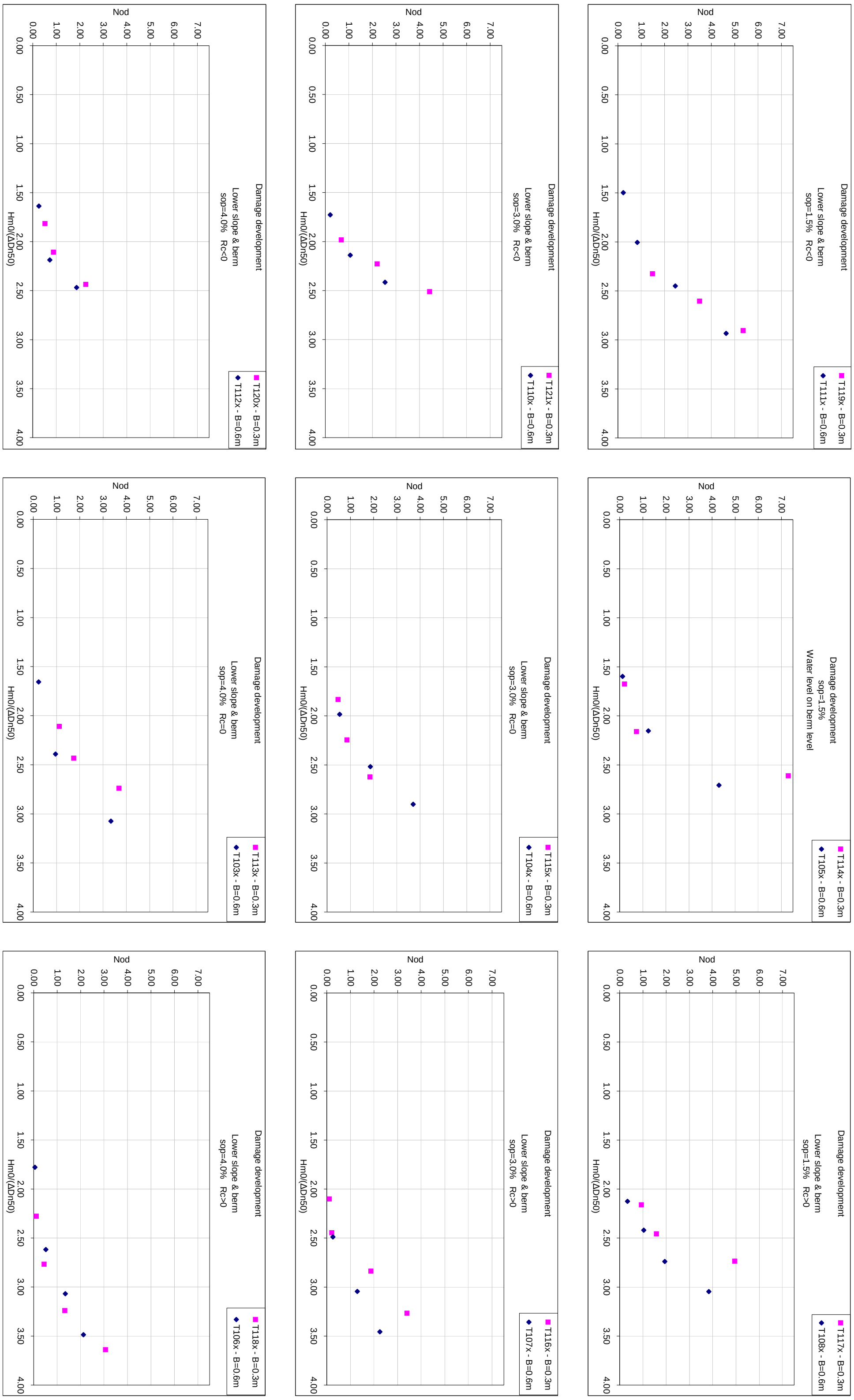


Figure J.1: Damage development for lower slope and berm

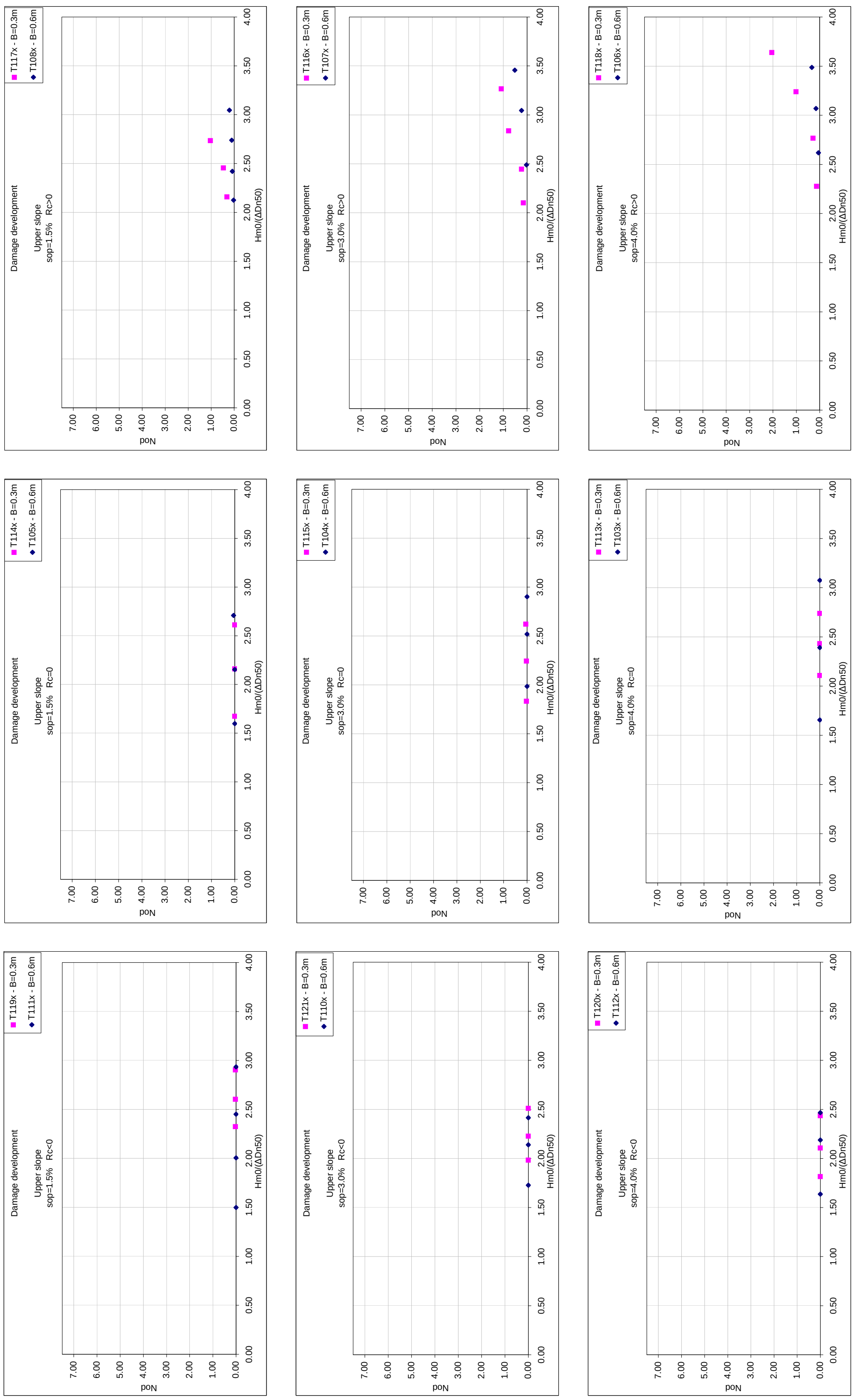


Figure J.2: Damage development for upper slope

Appendix K

Stability increase bermed slope

The increase in stability - expressed by r_D - is related to formula described by Van Gent *et al.* (2003). This formula doesn't include the influents of the wave period and therefore makes no distinction between wave steepness. The values of r_D for each test series can be found in Table 4.1.

- Figure K.1: Stability increase lower slope and berm for $R_c < 0$
- Figure K.2: Stability increase lower slope and berm for $R_c = 0$
- Figure K.3: Stability increase lower slope and berm for $R_c > 0$
- Figure K.4: Stability increase upper slope for $R_c > 0$

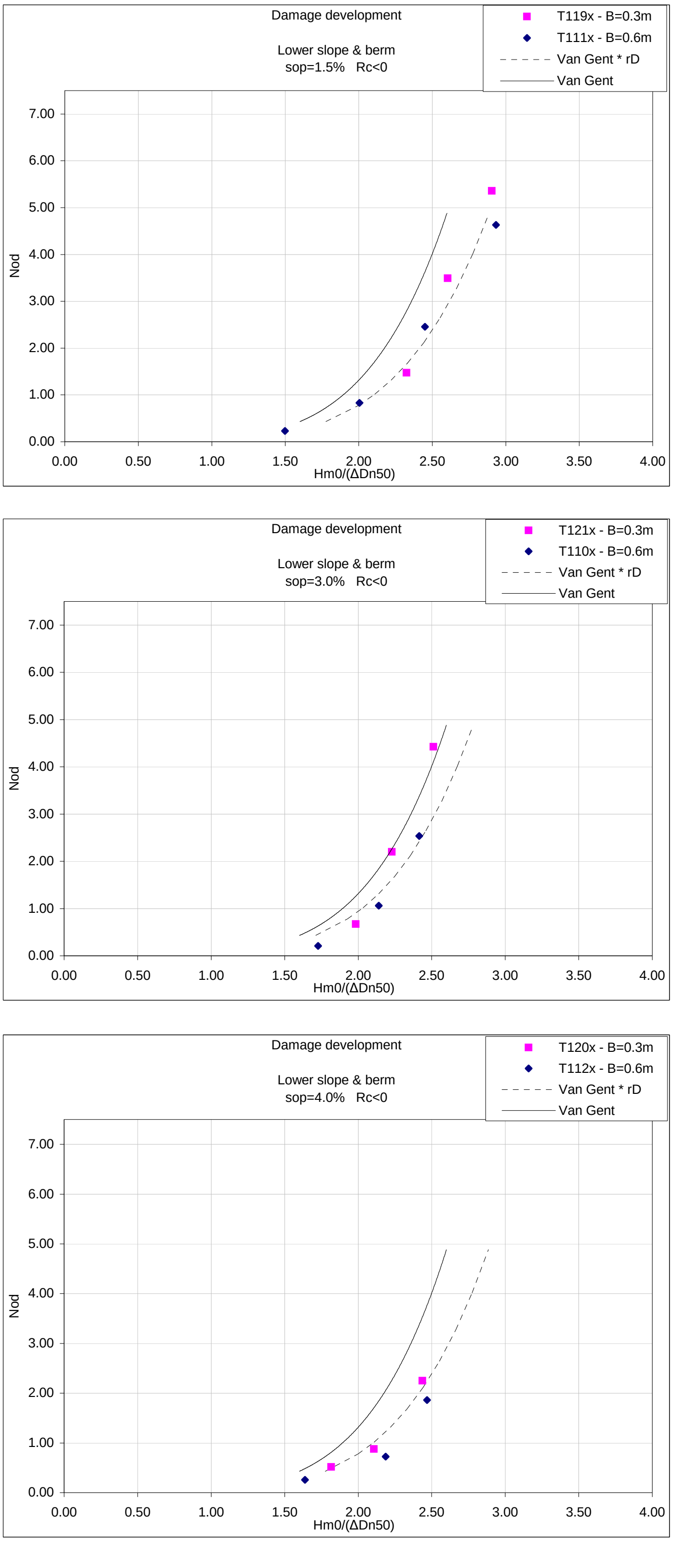
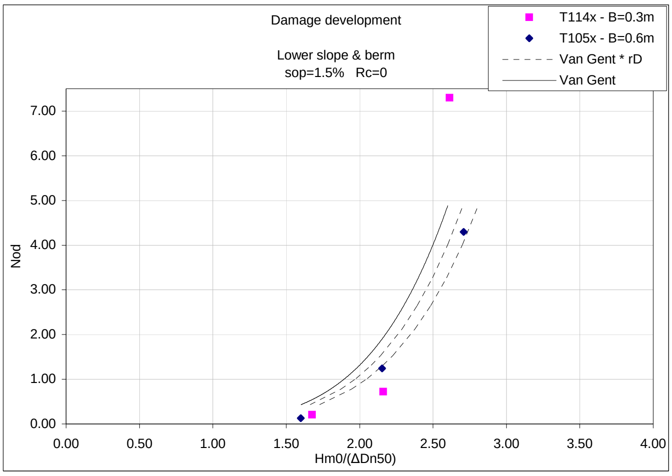
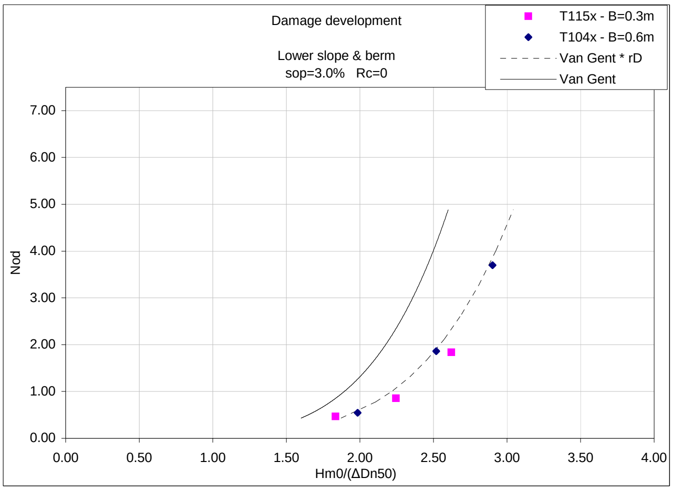


Figure K.1: Stability increase lower slope and berm for $R_c < 0$





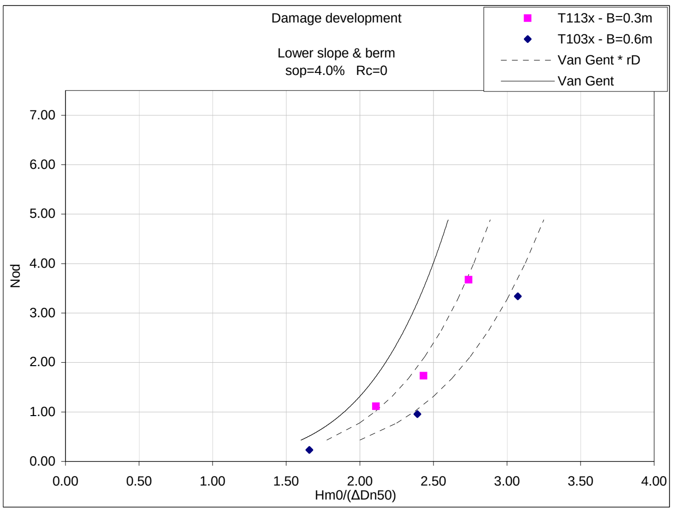


Figure K.2: Stability increase lower slope and berm for $R_c = 0$

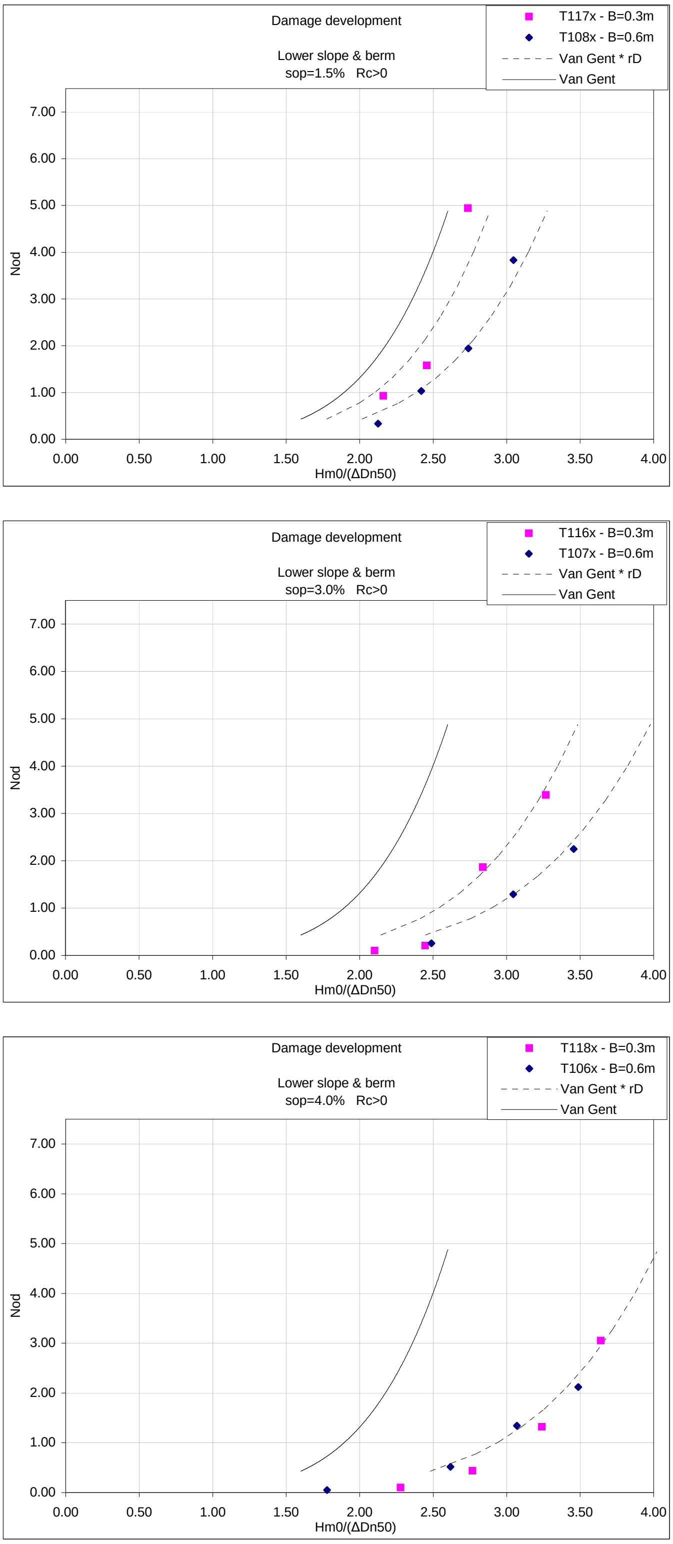


Figure K.3: Stability increase lower slope and berm for $R_c > 0$

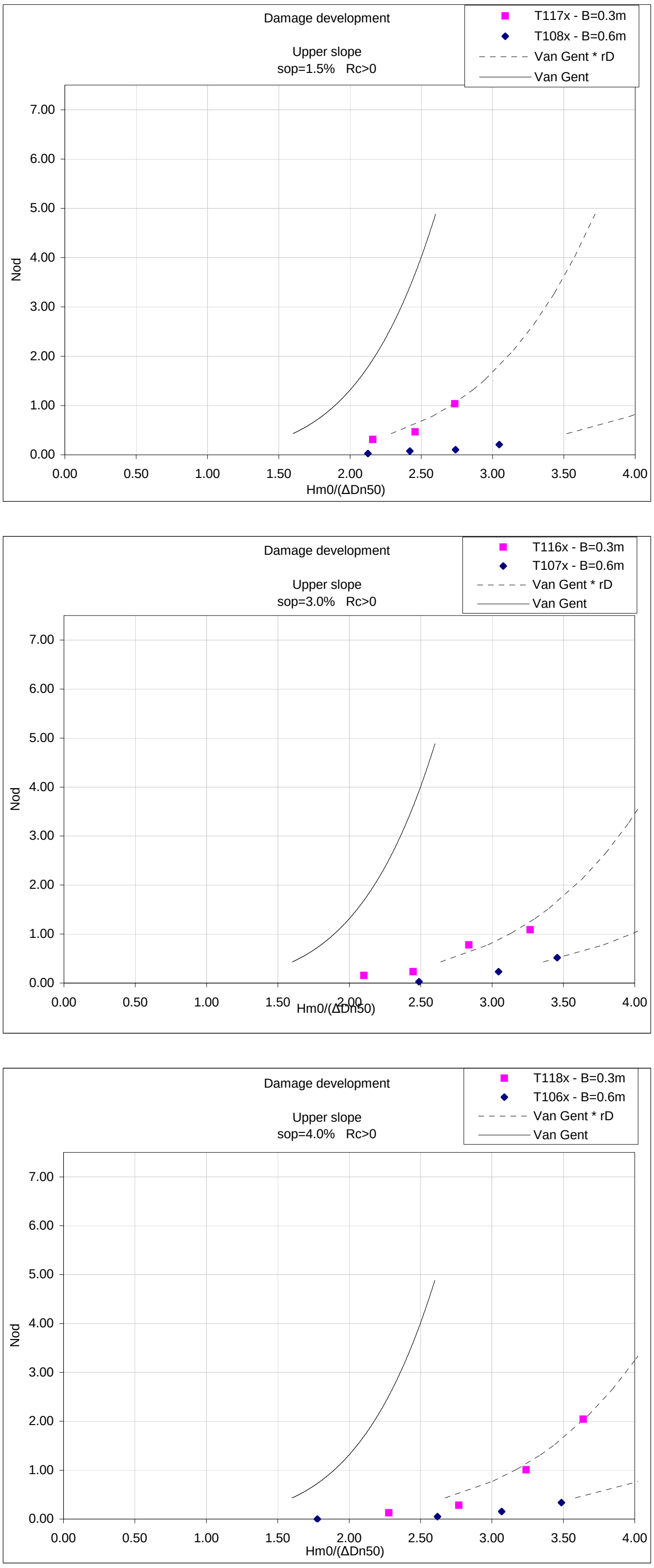


Figure K.4: Stability increase upper slope for $R_c > 0$

Appendix L

Influence governing parameters

- Figure L.1: Influence of the relative berm length on r_D
- Figure L.2: Influence of governing parameters on increase in stability.

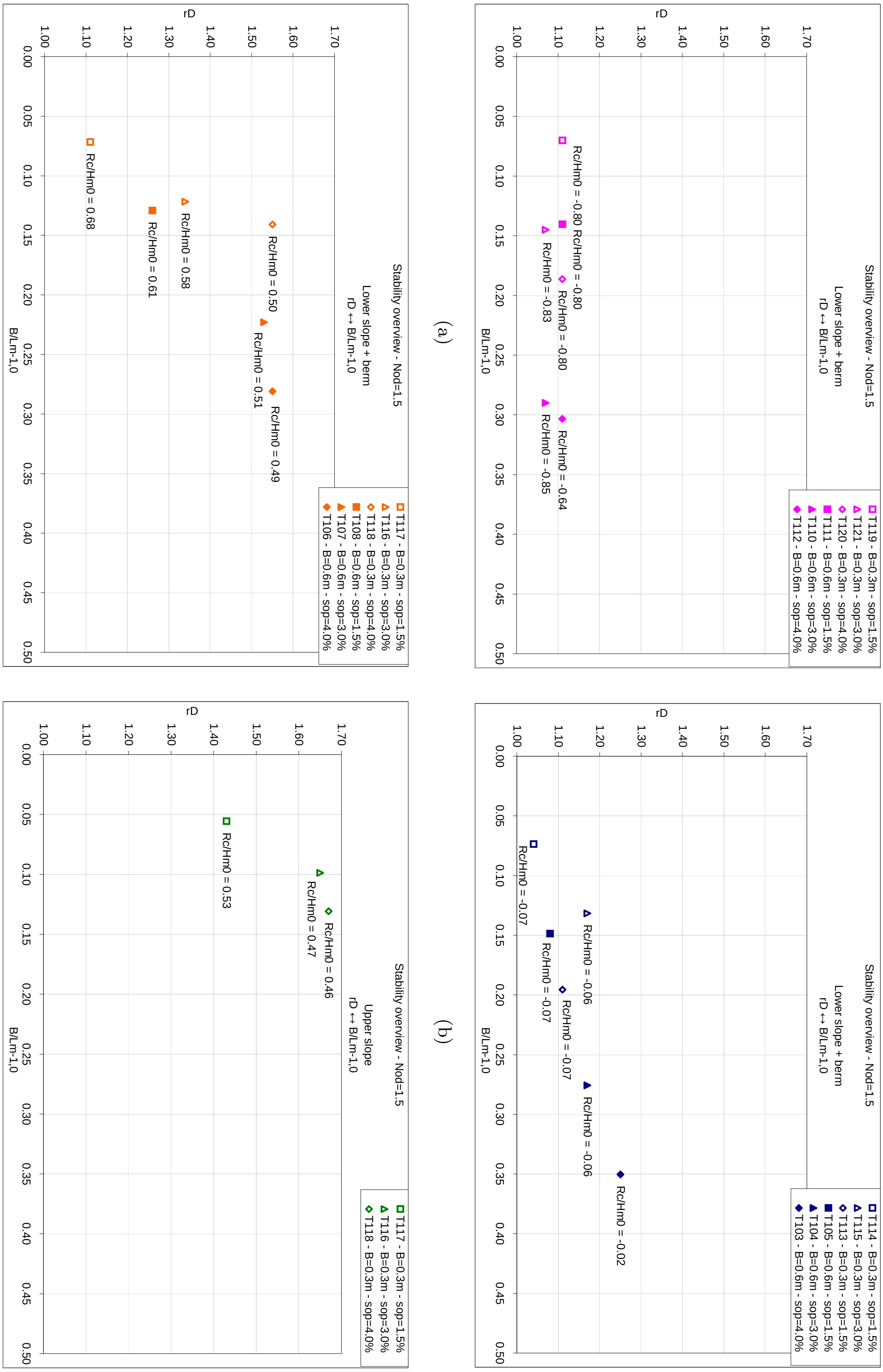
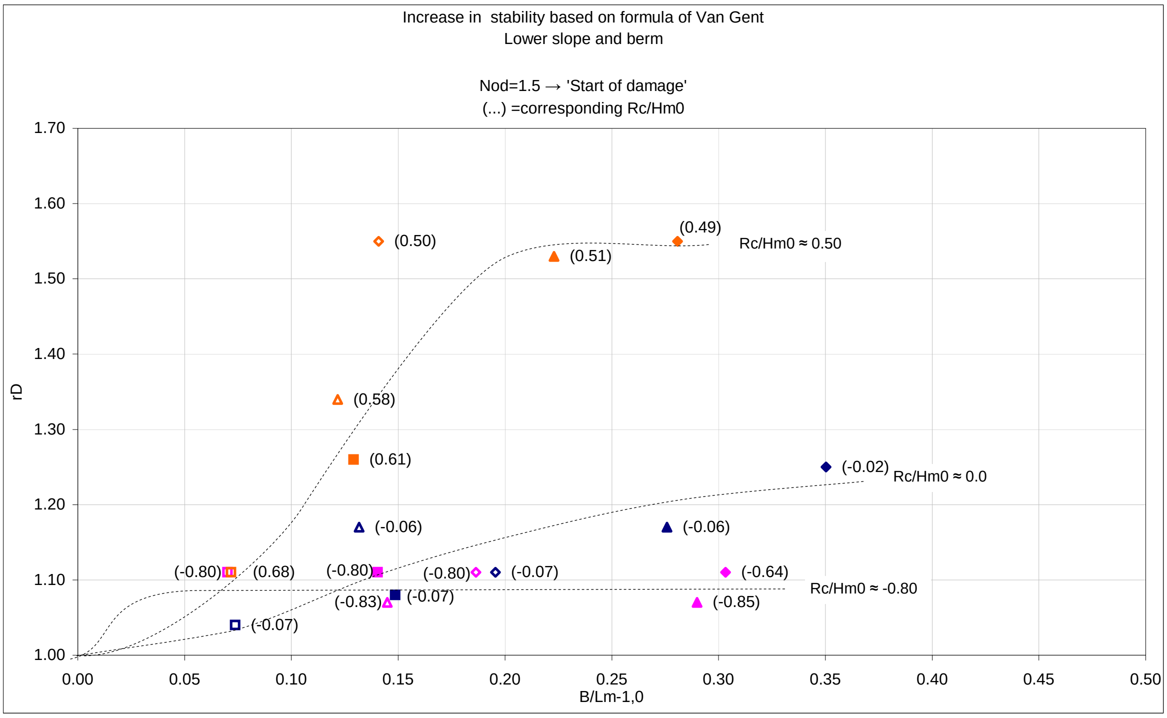
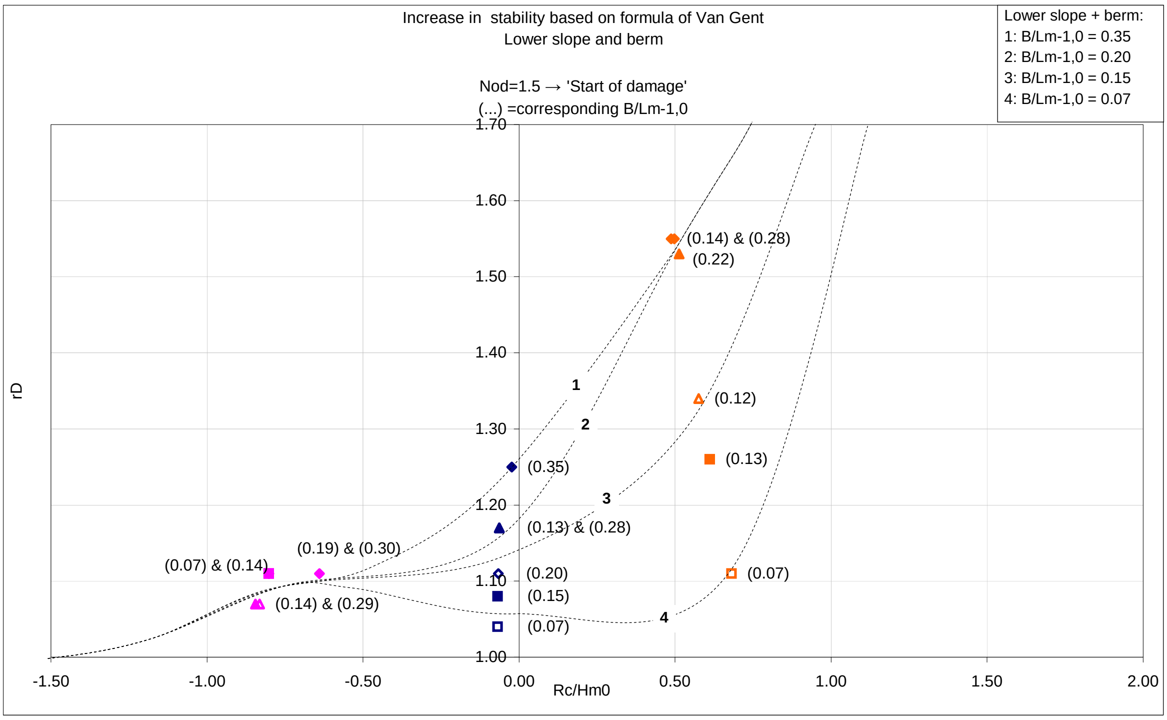


Figure L.1: Influence of the relative berm length on r_D per tested water level



(a)



(b)

Figure L.2: Influence of governing parameters on increase in stability

Bibliography

J. A. Battjes (1974). *Computation of set-up, longshore currents, run-up and overtopping due to wind generated waves.* Report 74-2, Comm on Hydraulics, Dept of Civil Engg, Univ of Technology, Delft.

H. F. Burcharth, Z. Liu & P. Troch (1999). *Scaling of Core Material in Rubble Mound Breakwaters Model Test.* Proceeding COPEDEC V.

CERC (1984). *Shore protection manual.* 4th edn. Coastal Engineering Research Center, US Army Corps of Engineers, Vicksburg, MS.

CUR/CIRIA (2007). *The Rock Manual. The use of rock in hydraulic engineering (2nd edition).* C683, CIRIA, London.

M. Dingemans (1987). *Verification of numerical wave propagation models with laboratory measurements, HISWA verification in the directional wave basin.* Technical Report H228, Part 1B, Appendices A-G. Delft Hydraulics, Delft.

EN13383-1 (2002). *Armourstone. Specification.* Euronorm.

K. Hasselmann, T. P. Barnett, E. Bouws, H. Carlson, D. E. Cartwright, K. Enke, J. A. Ewing, H. Gienapp, D. E. Hasselmann, P. Kruseman, A. Meerburg, P. Müller, D. J. Olbers, K. Richter, W. Sell & H. Walden (1973). *Measurements of wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP).* Deutschen Hydrographischen Zeitschrift, Reihe A , volume 8, no 12.

R. Y. Hudson (1953). *Wave forces on breakwaters.* Trans Am Soc Civ Engrs, volume 118, pp 653-674.

R. Y. Hudson (1959). *Laboratory investigations of rubble mound breakwaters.* J Waterways Harbors Div & Am Soc Civ Engrs, volume 85, no WW3, Paper no 2171, pp 93-121.

C. R. Iribarren (1938). *Una formula para el calculo de los diques de escollera.* M. Bermejillo-Pasajes, Madrid, Spain.

PIANC (2003). *State-of-the-art of designing and constructing berm breakwaters.* Report of MarCom WG40, PIANC, Brussels.

W. J. Pierson & L. Moskowitz (1964). *A proposed spectral form for fully developed wind seas based on the similarity law of S.A. Kitaigorodskii.* J Geophys Res, volume 69, no 24.

G. M. Smith, I. Wallast & M. R. A. van Gent (2002). *Rock slope stability with shallow foreshores.* ASCE, Proc 3th int coastal structures conf, 2002, Portland, pp 1524-1536.

D. M. Thompson & R. M. Shuttler (1975). *Riprap design for wind wave attack. A laboratory study in random waves.* Report EX 707, Hydraulics Research, Wallingford.

J. W. van der Meer (1988). *Rock slopes and gravel beaches under wave attack.* PhD thesis, Delft University of Technology, Delft. Also WL|Delft Hydraulics publication no 396.

J. W. van der Meer (1990). *Low-crested and reef breakwaters.* Report no H986, WL|Delft Hydraulics, Delft.

M. R. A. van Gent (1999). *Physical model investigations on coastal structures with shallow foreshores; 2D model tests with single and double-peaked wave energy spectra.* Report no H3608, WL|Delft Hydraulics, Delft.

M. R. A. van Gent (2001). *Wave run-up on dikes with shallow foreshores.* ASCE, Journal of Waterways, Port, Coastal and Ocean Engineering, volume 127, no 5, pp 254-262.

M. R. A. van Gent, A. J. Smale & C. Kuiper (2003). *Stability of rock slopes with shallow foreshores.* ASCE, Proc 4th int coastal structures conf, 2003, Portland.

A. C. M. Vermeer (1986). *Stability of rubble mound berms and toe constructions.* Report on literature survey and model investigation [in Dutch]. Report no M2006, WL|Delft Hydraulics, Delft.

J. A. Zelt & J. E. Skjelbreia (1992). *Estimating Incident and Reflected Wave Fields Using an Arbitrary Number of Wave Gauges.* Proceedings of the 23rd International Conference on Coastal Engineering (ICCE), Venice.