Stability of Gravel on Mild Slopes in Breaking Waves

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Preface

This MSc thesis presents the work performed during my graduation project, which is part of the Master programme of Hydraulic Engineering from the Faculty of Civil Engineering and Geosciences at the Delft University of Technology.

I would like to thank my entire committee for their enthusiasm and support throughout my thesis project. Daan Jumelet for the interesting conversations and constructive advice, Coen Kuiper for giving answers to all my questions and keeping me on track, Marcel Stive for this views and thoughts on the matter, Marcel Zijlema for his help with the start of the numerical experiment and my unofficial committee member Robert McCall for providing knowledge about XBeach-G throughout his busy schedule.

From DEME, I want to thank, Jonas Maertens and Arjan Mol for their advice during crucial points of my thesis project.

Finally, I am indebted to my family, friends and especially my boyfriend Jelmer Knuivers, who supported me throughout my study and always keeps believing in me.

Marieke Wit
Delft, November 2015
Summary

A recurrent challenge in the hydraulic engineering is to establish the required stone diameter for a stone protection on mild slopes, since the stability of gravel on a mild slope is not yet sufficiently covered by existing design methods. The objective of this study is to investigate the relation between the loads and the stability of stones on a mild slope in order to be able to design a protection with more accuracy. The method used to investigate this relation is the numerical model XBeach-G, which is generally used to predict the morphological changes of gravel beaches during extreme conditions, and is used for both statically stable and dynamically stable structures.

For statically stable structures, defined as structures for which no, or minor, damage is allowed during extreme conditions, generally the Van der Meer (1988) formulae, designed for steep slopes (steeper than 1:6), are used to calculate the required stone size. Although these formulae can be applied to slopes in between 1:1.5 and 1:6, lack of accurate design methods for mild slopes led to applying the same Van der Meer formulae for slopes milder than 1:6.

In this thesis it is found that the accepted damage level (S=2), denoted as the start of damage, can become larger for milder slopes, because more stones need to be moved around the water level to obtain the same level of erosion as for steep slopes (Van der Meer, 1988). How large this increase of the damage level may be, depends on the desired safety level which can be set by the client.

Although a higher damage level for mild slopes already results into smaller stone sizes, for the combination of mild slopes with small stone sizes the damage level automatically becomes very large (order of 1000 or higher), but with relative low values for the eroded area. Therefore another description of damage is proposed to be implemented for this type of structures, such as the eroded depth (d_e) or S_{norm} which is the damage level normalized over the length over which damage occurs.

Numerical results in this thesis show that dynamically stable structures can also be used as protection for mild slopes, which may be even more economical, because some damage is allowed to these structures. For dynamically stable structures an equilibrium profile is formed during extreme conditions, after which only minor changes occur. This equilibrium profile cannot change significantly, therefore strict requirements concerning the eroded area are needed, in order to guarantee the protection of the under-layer.

For dynamically stable structures the eroded area depends on the difference between the initial slope and the equilibrium slope of the sediment. When the equilibrium profile is steeper than the initial profile, sediment is transported upward creating a crest profile and the reversed process creates a submerged berm profile. The larger the difference between the equilibrium profile and the initial profile, the larger the eroded area. The equilibrium profile is described by the response angle of the equilibrium profile and depends on the stone size and probably to a small extent to the wave conditions. More research is needed to investigate the influence of wave conditions on the equilibrium profile.

When comparing the numerical results of the statically stable structures with the Van der Meer formula for plunging waves (1988), significant differences are found, as is illustrated in Figure 1. The Van der Meer formula for plunging waves is valid till a slope of 1:6 (area 2 and 3 in Figure 1), which
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coincides partly with the conducted numerical experiment. However, most part is outside this region and in order to compare these results, the Van der Meer formula is extrapolated outside its validity region (area 1). All numerical results have a higher stability number than the (extrapolated) points of Van der Meer formula, which means that lower stone diameters are found to be stable even within the applicable range of Van der Meer.

**Comparison of results for homogeneous, statically stable structures**

Two possible reasons may explain these differences. The first concerns the Van der Meer formula for homogeneous (and permeable) structures is only validated for slopes of 1:2, shown with the black stars in Figure 1. Therefore the validity of homogenous structures is limited to steep slopes and most likely not applicable to milder slopes. Besides, the permeability of the structure influences the stability of stones on mild slopes more than stones on steeper slopes, because for mild slopes the effective area is larger, resulting in a more stable structure. This relation is also embedded in the Van der Meer formula for surging waves (1988). The second explanation is concerning the sensitivity of XBeach-G in which the Nielsen (2002) transport formula is used to predict the amount of damage. This formula uses two calibration factors which are rather sensitive to the calculated damage. Which of these reasons explains the differences best is to be examined in further research.

XBeach-G proves that numerical models provide an accurate way of determining the required stone size for gravel protections on mild slopes in breaking waves. This offers a solution to the challenges faced during the design process for both statically and dynamically stable stone protections on mild slopes.

Figure 1: XBeach-G results and the Van der Meer formula (1988) for plunging breakers (statically stable structures)
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<th>Sym.</th>
<th>Definition</th>
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<tr>
<td>$A$</td>
<td>Eroded area</td>
<td>$[m^2]$</td>
<td>$\alpha$</td>
<td>Slope angle</td>
<td>$[^\circ]$</td>
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<tr>
<td>$c_f$</td>
<td>Dimensionless friction factor</td>
<td>$[-]$</td>
<td>$\beta$</td>
<td>Angle of the bed</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$C_H$</td>
<td>Hazen empirical coefficient</td>
<td>$[-]$</td>
<td>$\Delta$</td>
<td>Relative density</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$D$</td>
<td>Stone diameter</td>
<td>$[m]$</td>
<td>$\zeta$</td>
<td>Free surface elevation above an arbitrary horizontal plane</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$D_{10}$</td>
<td>Particle size for which 10% of the soil is finer</td>
<td>$[m]$</td>
<td>$\phi$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
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<tr>
<td>$D_{50}$</td>
<td>Median stone size</td>
<td>$[m]$</td>
<td>$\xi_1$</td>
<td>1. Elevation of the bed</td>
<td>$[m]$</td>
</tr>
<tr>
<td>$D_{o50}$</td>
<td>Median nominal stone diameter</td>
<td>$[m]$</td>
<td>$\xi_2$</td>
<td>2. Iribarren number/breaker parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$f_s$</td>
<td>Friction factor</td>
<td>$[-]$</td>
<td>$g$</td>
<td>Acceleration of gravity</td>
<td>$[m/s^2]$</td>
</tr>
<tr>
<td>$h$</td>
<td>Total water depth</td>
<td>$[m]$</td>
<td>$\xi_p$</td>
<td>Iribarren number related to the peak period</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$h_{gw}$</td>
<td>Height of the groundwater surface above the bottom of the aquifer</td>
<td>$[m]$</td>
<td>$\xi_m$</td>
<td>Iribarren number related to the mean period</td>
<td>$[-]$</td>
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<tr>
<td>$H$</td>
<td>Wave height</td>
<td>$[m]$</td>
<td>$\rho_s$</td>
<td>Density of sediment</td>
<td>$[kg/m^3]$</td>
</tr>
<tr>
<td>$\bar{H}$</td>
<td>Hydraulic head, depth averaged</td>
<td>$[m]$</td>
<td>$\rho_w$</td>
<td>Density of water</td>
<td>$[kg/m^3]$</td>
</tr>
<tr>
<td>$H_{2%}$</td>
<td>Wave height exceeded by 2% of all waves</td>
<td>$[m]$</td>
<td>$\tau_b$</td>
<td>Bed shear stress</td>
<td>$[N/m^2]$</td>
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<td>Significant wave height</td>
<td>$[m]$</td>
<td>$\tau_c$</td>
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<tr>
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<td>$\nu$</td>
<td>Kinematic velocity</td>
<td>$[m^2/s]$</td>
</tr>
<tr>
<td>$L_0$</td>
<td>Deep-water wave length</td>
<td>$[m]$</td>
<td>$\varphi$</td>
<td>Phase lag angle</td>
<td>$[^\circ]$</td>
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<tr>
<td>$M$</td>
<td>Mass of the stone</td>
<td>$[kg]$</td>
<td>$\phi$</td>
<td>1. Angle of response of the sediment</td>
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</tr>
<tr>
<td>$n$</td>
<td>Porosity</td>
<td>$[-]$</td>
<td></td>
<td>2. Natural angle of response</td>
<td>$[^\circ]$</td>
</tr>
<tr>
<td>$N$</td>
<td>Number of waves</td>
<td>$[-]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
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</tr>
<tr>
<td>$P$</td>
<td>Permeability of the structure</td>
<td>$[-]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$\bar{q}$</td>
<td>Depth-average dynamic pressure</td>
<td>$[Pa]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
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</tr>
<tr>
<td>$q_s$</td>
<td>Volumetric sediment rate</td>
<td>$[m^3/s]$</td>
<td>$\psi_c$</td>
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<tr>
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<td>1. Damage level in the Van der Meer formula 2. Surface water-groundwater exchange flow</td>
<td>$[-]$</td>
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<tr>
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<tr>
<td>$T_{m-1.0}$</td>
<td>Offshore spectral period</td>
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<tr>
<td>$T_p$</td>
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<td>$\psi_c$</td>
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<td>Depth-average cross-shore velocity</td>
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<td>$\psi_c$</td>
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<tr>
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</tr>
<tr>
<td>$u_{gw}$</td>
<td>Depth average horizontal groundwater velocity</td>
<td>$[m/s]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$u_c$</td>
<td>Critical velocity</td>
<td>$[m/s]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$u_{c*}$</td>
<td>Critical shear velocity</td>
<td>$[m/s]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$v_h$</td>
<td>Horizontal viscosity</td>
<td>$[m^2/s]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$w_{gw,s}$</td>
<td>Vertical groundwater velocity at the groundwater surface</td>
<td>$[m/s]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$W_{50}$</td>
<td>Median weight of the stones</td>
<td>$[kg]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
<td>$[-]$</td>
</tr>
<tr>
<td>$x$</td>
<td>Spatial, horizontal coordinate</td>
<td>$[m]$</td>
<td>$\psi_c$</td>
<td>Shields parameter</td>
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</tr>
</tbody>
</table>
1 Introduction

Recent design and construction works show that the stability of stones on a mild slope is not yet sufficiently covered by existing design methods. The relation between the loads and the stability of stones on a gentle slope are investigated in this master thesis project. An introduction is given in this chapter, starting with the background information which shows the origin of the problem. Next, the thesis objective and the research questions are presented.

1.1 Background information

The primary function of a protection of stones on a mild slopes, is to protect the slope against erosion. Mild slopes that need to be protected are for instance foreshores in estuaries and lakes. The foreshore, which is important for the stability of dikes, has to withstand a combination of loads from waves and currents, depending on the geographical location of the dike and the depth of the foreshore. These loads can cause foreshore erosion thereby jeopardizing the stability of the dike itself. This protection may be designed as a statically or dynamically stable structure.

De Vries & van de Wiel, a contractor operating throughout the Netherlands, encounters a problem concerning the design of the required grave size for a foreshore protection. De Vries & van de Wiel is part of the DEME group, which is an abbreviation for Dredging, Environmental and Marine Engineering. Recently, De Vries & van de Wiel was assigned to install a stone protection on the sandy foreshores of the Eastern Scheldt. The contract included very strict requirements on the loads and the design formulae to be used. Furthermore the contract required a statically stable structure. All together, these requirements resulted in a very large stone protection, which may not be the most cost efficient form of protection of the foreshore.

1.2 Problem

The objective of this study is to investigate the relation between the loads and the stability of stones on a gentle slope in order to be able to design a protection for a mild slope with more accuracy which can lower the construction costs significantly. The problem statement is formulated as follows:

"The stability of stones on mild slopes cannot be calculated with sufficient detail, using extended existing design methods, led to over dimensioning and therefore high investment costs. In order to lower these costs it is required to establish a method which can determine the stability of gravel on a mild slope in breaking waves". This leads to the following research questions:

1. What method (scale model tests, numerical model or extension of existing empirical design methods) can be used to establish an accurate way to determine the stability of gravel on a mild slope in breaking waves (in order to lower the investment costs)?
2. What is a suitable model to design a stone protections for statically stable profiles and shallow mild foreshores?
3. Which (different) parameters are of importance for the design of a stone protection on mild slopes, by comparing XBeach-G with current literature?
4. What is the location of the occurring damage and is it possible to optimise stone size on the foreshore?
5. Is a statically stable structure for these type of applications an economical solution?
1.3 Scope

Prior to describing the approach and methodology used to answer the research questions, it is important to define the scope of this study. Mild slopes in shallow water naturally occur along estuaries and lakes, for instance along the Western Scheldt, along coasts, in large lakes and also for pipeline protection structures for at landing locations of such a pipeline. This study focuses on conditions which occur in Dutch estuaries and lakes.

Different types of protection are possible, this thesis focuses on gravel protection only. It is very common that protection layers of a slope need to be statically stable, which means that only minor damage is allowed during extreme events. For mild slopes it should be determined whether it is desirable to have a statically stable structure or to have a dynamically stable structure in which some profile development is accepted. For mild slopes more damage is acceptable because the moving gravel will not exit the profile, how much movement is allowed without severe damage shall be investigated.

Mild slopes generally are part of a dike profile with steeper parts, for example in the Western Scheldt river where the mild slope is connected by the toe of the dike and at the other side with a slope to the navigation channel, as is illustrated in Figure 2. These profiles differ a lot from place to place and can influence the dynamics of the water on the mild slopes. Because these different factors can influence the relation between the loads and stability of stones it is chosen to only investigate a uniform slope without any other parts of the dike or channels. The water line intersects the uniform slope. In this way the effects can be studied which are of importance to mild slopes only.

A changing water level can also occur, caused by the tide, or during storm events, which can cause damage at different levels along the profile. In this study, we focus on the peak of the storm, which means that the water level is kept unchanged during an extreme event in order to study the stability of gravel in one region in which damage occurs. The effect of changing water level is not part of this study. Currents caused by the tide, changing water levels or other processes are also not part of this study.

1.4 Report structure

Chapter 2 provides an overview of relevant literature. The used methodology is expounded in Chapter 3, including the description of the conducted numerical experiment. Chapter 4 presents the results obtained in the numerical experiment. The main topic of discussion in chapter 5 is the comparison of the numerical results with the results obtained by using the current design methods. Final conclusions and recommendations are found in Chapter 6.
2 Theoretical framework

To create a connection with the existing literature, a theoretical framework is written. First, a review of literature is given. Second, it is shown that in order to design a protection for a mild slope in breaking waves, the existing design formulae and literature cannot be used. Third, from the existing literature it can be concluded which parameters are essential to take into account for the stability of stones on mild slopes.

2.1 Review of the literature

A comprehensive literature review was conducted, consisting of different theories for statically stable structure as well as for dynamically stable structures. In current literature, different formulae were established to describe the movement of stones in fluid motions. The formulae are based on the empirical relation between the hydraulic loads, for instance currents and waves, and the resistance of the loose material. Currents, waves and other water conditions act on the loose material through shear stresses and lift forces. The resistance of the loose material are gravity and friction forces and possibly some cohesion, depending on the material size.

In this literature review, different stability approaches are described. First for the statically stable structures for a) uniform flows ,b) oscillatory flows and c) breaking waves. Second, different methods for dynamically stable structures are elaborated.

2.1.1 Statically stable structures

One of the stability concepts is determining the stability using a critical velocity, Izbash (1935) used this approach. The approach is based on a force balance, flow and resistance forces, for individual stones. When a certain critical velocity is reached, the stone is displaced. Stability is only depended on the velocity near the stone, however, where this velocity should be determined is unclear (Schiereck & Verhagen, 2012). The following formula was found by using experiments.

\[ u_c = 1.2 \sqrt{2 \Delta g D} \]  

\[ u_c \] = Critical velocity [m/s]  
\[ \Delta \] = Relative density [-]  
\[ g \] = Acceleration of gravity [m/s²]  
\[ D \] = Diameter of the stones [m]

This approach can be used in non-uniform flows for big stones in relatively shallow water, on the other hand it is not applicable to small grains.

Instead of the critical velocity, Shields (1936) used the critical shear stress acting on the grains on a horizontal bed. By conducting experiments Shields obtained a diagram in which the initial motion is drawn in relation with the Shields parameter and the particle Reynolds-number. The Shields parameter is only justified for a quasi-steady flow in limited water depths, and is defined as follows:

\[ \psi_c = \frac{\tau_c}{(\rho_s - \rho_w)gD} \]  

\[ \psi_c \] = Shields parameter [-]  
\[ \tau_c \] = Critical bed shear stress [N/m²]  
\[ \rho_s \] = Density of sediment [kg/m³]  
\[ \rho_w \] = Density of water [kg/m³]
Experiments were carried out by Rance and Warren (1968) in order to predict the threshold of movement of shingle in an oscillatory flow. The results were presented in different diagrams in which different dimensionless parameters were plotted in order to find the best description of the threshold of movement. However, the large number of variables made the analysis extremely difficult and therefore did not lead to a formula. Schiereck et al. (1994) fitted a formula to the obtained data, valid for a horizontal bottom:

$$D_{50} = \frac{2.56 \times \hat{u}_b^{2.5}}{\sqrt{T_p} \times (\Delta g)^{1.5}}$$

(3)

$$\hat{u}_b = \text{Maximum orbital velocity [m/s]}$$

$$D_{50} = \text{Median stone size [m]}$$

$$T_p = \text{Peak wave period [s]}$$

Sleath (1978) has done measurements of the quantities of sediment moving in an oscillatory flow, analogous to the Shields approach in which he gives the relation between the shear stress and the stone weight. He analysed results obtained by others, together with his own measurements, and combined it to a modified line in de Shields curve for waves (Schiereck & Verhagen, 2012).

The next step in stability formulae is to add a slope, which entails breaking of waves. Starting with the diagram proposed by Sleath combined with correction concerning wave breaking and the slope, Iribarren (1938) proposed a proportionality as mentioned in Schiereck and Verhagen (2012):

$$M \propto \frac{\rho_s H^3}{\Delta^3 (\tan \phi \cos \alpha \pm \sin \alpha)^3}$$

(4)

$$M = \text{Mass of the stone [kg]}$$

$$H = \text{Wave height [m]}$$

$$\phi = \text{Natural angle of response [°]}$$

$$\alpha = \text{Slope angle [°]}$$

To find the constant in this proportionality, Hudson (1953) carried out many experiments. These experiments were conducted for structures with a porous core, which means that the Hudson formula is not applicable for dikes and foreshores with a sand or clay core.

Van der Meer (1988), however, derived stability formulae for different types of structures also for structures with an impermeable core. Other effects such as wave period, storm duration and damage level were also incorporated into two formulae, one for plunging breakers and one for surging breakers:

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi^{-0.5} \quad \text{(plunging breakers)}$$

(5)

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi \sqrt{\cot \alpha} \quad \text{(surging breakers)}$$

(6)
Chapter 2 - Theoretical Framework

\[ D_{50} = \text{Median nominal stone diameter [m]} \]
\[ H_s = \text{Significant wave height [m]} \]
\[ P = \text{Permeability of the structure [-]} \]
\[ S = \text{Damage level [-]} \]
\[ N = \text{Number of waves [-]} \]
\[ \alpha = \text{Slope angle [°]} \]

These formulae are used to calculate a statically stable stone protection, for \( H/\Delta d_{n50} < 4 \) and is valid for slopes in the range 1:1.5 – 1:6. For dynamically stable structures, higher wave height parameters (\( H/\Delta d_{n50} < 3 \)), van der Meer determined an equilibrium profile under wave action.

After defining empirical relations, different attempts were made in order to find the physical background of the stability of gravel on a mild slope, for instance by Schiereck and Fontijn (1996) who wanted to create a connection of a simple physical description of the involved phenomena with experimental results. The experiments carried out by Sistermans (1993) and Ye (1996), were conducted on mild slopes with a slope of 1:10 and 1:25, in combination with a statically stable protection. The results are shown in Figure 3. It turned out that their theoretical description is reasonable in a qualitative way, but is not accurate enough to use for design practice. Despite this fact, it turned out that the results of the experiment were consistent with the Van der Meer formulae for the stability of stones on steep slopes (Schiereck & Fontijn, 1996). This means that from the statically stable point of view a first estimate can be made with the Van der Meer formulae applied to mild slopes.

![Figure 3: Results of experiments carried out on a slope of 1:10 and 1:25 (Schiereck & Fontijn, 1996)](image)

2.1.2 Dynamically stable structures

Dynamically stable structure are structures in which reshaping of the material under severe wave attack occurs until an equilibrium profile is reached. Dynamically stable structures which are used to protect coasts or slopes are for instance berm breakwaters and gravel beaches. A lot of research is conducted about the equilibrium profile, Van Huijm and Pilarczyk (1982) described this profile with length and height parameters together with the angles obtained from model testing.
Van der Meer (1988) conducted even more experiments for different slopes, not only uniform slopes and expressed the characteristics of the equilibrium in the following way:

![Schematized equilibrium profile of dynamically stable slope of 1:5 (Van der Meer, 1988)](image)

The profile is described by the following parameters, which are relative to the still waterline:

- \( l_r \) = Runup length
- \( h_c \) = Crest height
- \( l_c \) = Crest length
- \( h_s \) = Step height
- \( \beta, \gamma \) = Angles

The curve between the step and the crest is described by a power function, which can be found in Van der Meer (1992). The submerged step is described by the two angles. Van der Meer (1992) stated that this profile is more or less independent of its location in relation to the initial profile. In order to find the location of this profile, the mass balance has to be fulfilled by shifting the profile along the still water line.

All relations of the parameters which describe the equilibrium profile were put together into a computer model called BREAKWAT, which can be used for design purposes. It can be used for any arbitrary slope, which means that it is also applicable to mild slopes (Van der Meer, 1992).

### 2.2 Design method used in literature

When designing a statically stable protection layer for a foreshore with a mild slope, a recurrent challenge is to establish the required stone size. A lot of different methods can be used to solve similar problems, none of which is applicable to foreshores with a mild slope or shallow waters.
For the stability of stones on slopes, generally the Van der Meer (1988) formulae or the Hudson (1953) formula is used. However, these formulae are all only valid for steep slopes ranging from 1:1.5 to 1:6 which means that they are not applicable to very mild slopes. In Figure 3 a comparison is given of the data gathered from experiments on mild slopes and the (extrapolated) Van der Meer formula for plunging waves. It can be clearly seen that the stability calculated with the Van der Meer formula is higher than the experimental data, which could mean that the calculated stone size is over dimensioned which leads to higher total costs for the structure.

Instead of categorizing mild foreshores as normal slopes, a more suitable categorization might be to categorize them as a type of beach. This can be done in two different ways, namely as a dynamically stable beach profile or as a statically stable protection of stones on a beach. Concerning the statically stable protection, research is conducted by Sistermans (1993), Ye (1996) and Schiereck & Fontijn (1996) which is described in 2.1.1. On the other hand a dynamically stable structure can be used as a protection, for instance a gravel beach. Gravel beaches naturally occur along the coastline of England and on other high-latitudes and form a slope in the order of 1:10 when there are being exposed to extreme events. Dynamic structures are considered because the slopes and the size of gravel are very similar to the mild slopes which need to be protected.

From the dynamically stable point of view, different methods can be used in order to calculated the stability after extreme conditions. For example, numerical models, like XBeach and Durosta, which can calculate the morphological response of sandy beaches and the empirical relation of Van der Meer (1988) for dynamically stable structures, which calculates an equilibrium profile (see 2.1.2). However, these methods are difficult to use for the intermediate area in between dynamically and statically stable structures. For the named numerical models this is because the intermediate area does not consist of sand. The problem for the empirical model of Van der Meer (1988) is that the calculation of an equilibrium profile differs too much from the statically stable calculation which makes it difficult to connect those concepts. This means that a different model should be used, preferable with a dynamic character in which also statically stable structures can be calculated. Recently, a numerical model was developed which can predict the morphological response of storm events on gravel beaches, called XBeach-G. More details about this model is given in Chapter 3.1.

The literature study shows that the static stability of stones on mild slopes is not yet sufficient covered by the existing design methods. The literature study does give parameters which are important for the stability of stones on mild slopes, which are described in the next section.

### 2.3 Governing parameters

The governing parameters are divided into two categories which describe the stability of stones in breaking waves, namely hydraulic parameters and structural parameters. The hydraulic parameters are caused by the water motion and can be categorized as loads. Structural parameters concern the parameters which can be set by an engineer who will design the structure or protection and can be categorized as the strength. The stability of the stones on a mild slope depends on the relation between the load and strength, as described in the literature review.
2.3.1 Hydraulic parameters
The loads are composed of forces caused by the orbital motion in combination with turbulent velocities (Schiereck & Fontijn, 1996). The parameters which describe these processes are described in the following order: wave height, wave period, water depth and storm duration.

**Wave height**
Wave conditions in deep water can be characterized with two parameters, one of which is the wave height, \( H \), which is related to the wave forces acting on a slope. Waves approaching a mild slope change due to the decreasing water depth, wavelengths decrease, wave heights increase and the steepness of the wave increases. Eventually waves will break when they become too steep or when the ratio of the wave height with the water depth becomes too large. The wave height influences the location of breaking, which influences the stability of stones, this is further explained in 2.3.4. Because of the changing wave height it is important to use one expression for the wave height which is easy to determine.

In order to find such an expression it is convenient to use the deep water wave height. Before a wave approaches the coast it can be described by a Rayleigh distribution which is valid for a random sea state in deep water. With this distribution the wave height exceedance can be calculated using the significant wave height, \( H_s \). This expression of the wave height is very common to use and therefore also be used in this study.

Another way of expressing the wave height can be with for example \( H_{2\%} \), which is the wave height which is exceeded by only 2% of the waves. If it becomes clear that only the highest waves influence the stability of gravel on a mild slope, it may be better to use the \( H_{2\%} \) expression instead of the \( H_s \). Furthermore in shallow water conditions, the wave heights distribution deviates from a Rayleigh distribution. This deviation causes a change in the shape of the spectrum. Because of this, Van der Meer (1988) concluded that the stability of stones in breaking waves is better described by the \( H_{2\%} \). Whether the \( H_{2\%} \) is also better to use during the numerical experiment is explained in section 3.4.2.

**Wave period**
The other parameter which characterize wave conditions in deep water is the wave period, \( T \), which determines the wave length. Longer waves contain more energy and result in a different breaking pattern which influences the stability of gravel on a slope.

Several wave period measures exists. The most used are the spectral wave period measures \( T_p \), being the peak wave period and the \( T_{m-1,0} \), being the spectral mean wave period. The peak period can be derived from the spectral shape of random sea state, it is equal to the period at which the maximum value of the energy density spectrum is found.

**Water depth**
The water depth influences the location of breaking. The solitary wave theory states that this limit of breaking due to shallow water is \( H_s/h \approx 0.4 - 0.5 \). However section 2.3.4 shows that this limit is not constant and changes for different slopes and breaker types. It is key to know at which location the wave break, because at this location probably the largest wave loads occur. Consequently, it may be possible to optimize the required stone size at other location with lower load.
**Storm duration**

Normally in a storm first the water level rises for a certain amount of time, then at the peak of the storm the water level remains constant at when the storm is passing by at the same time the water level decreases, as illustrated in Figure 5. It was already said in the scope of this thesis that the water level is kept constant which means that only the peak of the storm is taken into account.

![Figure 5: Water level change during a storm (Ministerie van Verkeer en Waterstaat, 2007)](image)

2.3.2 Structural parameters

The structural parameters determine the strength of the protection and are mainly depend on the chosen material. The relevant properties of such material are described. The following parameters are described: stone diameter, stone shape, grading, mass density, slope angle, hydraulic conductivity and the under-layer.

**Stone diameter**

The word stone is chosen, because for a protection on a mild slope the stones can have a size which can be considered to be in-between small rocks and gravel. The stone diameter can be described in two different ways, namely by the median stone size, \( D_{50} \), and by the nominal diameter, \( D_{n50} \). The median diameter is the sieve diameter of which 50% of the stones are larger than \( D_{50} \). For bigger stones it is more common to use the nominal diameter which depends on the weight of the stone and is defined in the following way:

\[
D_{n50} = \sqrt[3]{\frac{W_{50}}{\rho_s}} 
\]

Where:
- \( W_{50} \) = Median weight of the stones [kg]
- \( \rho_s \) = Mass density of the stone [kg/m³]

Different names are given for different sizes of stone diameter, a classification is made called the Wentworth scale. It defines sand when it has a diameter between 0.0625 mm to 2mm, one class larger is defined as gravel. Gravel itself can be divided into different classes such as granular (2 mm – 4 mm), pebble (4 mm – 64 mm), cobble (64 mm – 256 mm) and bolder or rock (>256 mm) (Wentworth, 1922).
**Stone shape**
The stone shape can have influence on the stability of gravel in the breaker zone. Van der Meer (1992) investigated if the shape has a significant influence, by using rounded gravel, angular stones and flat/long stones. From experiments, on slopes of 1:3 and 1:5, it can be seen that there is no difference for the angular stones compared to the flat/long stones. The rounded gravel, however, has the tendency to form a lower crest and a longer berm, but the difference are small (Van der Meer, 1992). Therefore Van der Meer (1992) concluded that the shape of the stones has only a minor influence on the profile.

**Grading**
Natural stones are always graded, because it is almost impossible to have natural stones all of the same size. This grading can have influence on the stability of the gravel. The grading is defined as $D_{85}/D_{15}$ which are the 85 and 15 percent non-exceedance values of the sieve curves.

**Mass density**
The stability of the stones depends on the weight which is a function of the mass density, $\rho$, of the stones. The mass density for natural stones ranges from 2300 kg/m$^3$ till 3000 kg/m$^3$ and for steel slags even up till 3600 kg/m$^3$ (CIRIA, et al., 2007).

**Slope angle**
Mild slopes are slopes milder than 1:6. Typically foreshores in the Netherlands have a slope in the order of 1:10. The slope angle influences the type of breaking of waves, which has a large influence on the stability of stones.

**Hydraulic conductivity versus permeability**
The hydraulic conductivity, $k$, is a property of both the porous media as the fluid that flows through it. This is different from the permeability which is only a function of the media itself (Turner & Masselink, 2012). The hydraulic conductivity describes the flux of water through the pore spaces of the porous media. This property plays an important role in the stability of gravel on a slope in breaking waves, it describes whether the wave penetrates into the structure or whether it reflects the wave, this results into different loads on the structure.

The hydraulic conductivity depends in the stone size and size distribution. Typical values for gravel in the field are in the order of $10^{-3}$ – $10^0$ (Turner & Masselink, 2012). Till now no accurate formula exists which can calculate the hydraulic conductivity for a gravel beach for a specific $D_{50}$. Different formulae can be used to give an estimate, for example the Hazen formula:

$$ k = C_H D_{10}^{2} $$

$k$ = Hydraulic conductivity [cm/s]  
$C_H$ = Hazen empirical coefficient  
$D_{10}$ = Particle size for which 10% of the soil is finer [cm]

Many different ranges are proposed for this coefficient ranging from 1 till 1,000 (CARRIER III & David, 2003).

**Base and under layers**
Generally a design of a protection has a base layer of sand with a filter layer on top. It can be investigated in what way this base layer influences the stability of the protection layer.
2.3.3 Damage parameters

Damage, after extreme conditions, can be defined in different ways, for example Van der Meer (1988) used the eroded area divided by the square of the stone diameter see equation (9).

\[
S = \frac{A}{D_{50}^2}
\]  

\( S \) = Damage level [-]
\( A \) = Eroded area \([m^2]\)

In this definition, it does not show how this area is distributed. It could be the case that over a relatively large length a small depth is eroded, this gives the same damage level as a relatively small length in combination with a large depth. This is why Melby & Kobayashi (1998) introduced the erosion depth, \( d_e \), defined as the maximum slope-normal distance between the initial slope and the damaged slope. In case of a protection for mild slopes, the eroded depth is a better indicator for a failing protection instead of the number of moved stones. This definition is also used for horizontal bottom protections. The behaviour of stones on a mild slope is probably more equivalent to a horizontal bottom then that of a very steep slope.

To determine whether a protection layer is failing, the parameter cover depth \( (d_c) \) can be used which is defined as the minimum slope-normal distance between the underlayer and damaged profile (Melby & Kobayashi, 1998).

2.3.4 Dimensionless parameters

Some of the above named parameters can be combined in order to create dimensionless parameters. Dimensionless parameter are convenient to compare results of different tests, not only in this study but also with outcomes from other studies.

**Relative density**

The relative density, \( \Delta \), of stones in water is defined as follows:

\[
\Delta = \frac{\rho_s - \rho_w}{\rho_s}
\]  

\( \rho_w \) = Density of water \([kg/m^3]\]
\( \rho_s \) = Density of rock \([kg/m^3]\]

**Stability parameter**

The stability parameter gives the ratio between the load, which is the wave height, and the strength which is the effective weight of the stones. It is defined as:

\[
\frac{H_s}{\Delta D_{n50}}
\]  

Van der Meer (1988) defined this parameter as wave height parameter and used it to categorize type of structures, for instance a statically stable breakwater has a wave height parameter between 1 - 4 and a gravel beach 15 - 500.
Breaker parameter

Other names used in literature are Iribarren number or surf similarity number, they all represent the ratio of the slope steepness and the wave steepness. The breaker parameter, $\xi$, is defined by Battjes (1974) as:

\[
\xi = \frac{\tan \alpha}{\sqrt{H/L_0}}
\]  

\[
\alpha = \text{Angle of the slope [\text{\textdegree}]} \\
L_0 = \text{Deep-water wave length [m]}
\]

The type of breaking can be described by this parameter, it shows if and how a wave breaks and where this takes place. Figure 6 illustrates the four main breaker types which are surging, collapsing, plunging and spilling. For this study only the last two types of breaking are of importance, because on mild slopes collapsing and surging breakers do not occur.

Figure 6: Breaker types (Battjes, 1974)
The breaker parameter determines when a wave breaks, in Figure 7 it can be seen that for milder slopes, waves break at higher depths. This influence the stability and determines where the erosion might occur. Another key process that is also described by the type of breaking is the dissipation of energy over the slope. For instance, a plunging breaker which forms a jet has a larger impact at one specific point at the slope and on the contrary a spilling breaker dissipates its energy more evenly over the slope without a point of high impact.

![Figure 7: Breaker depth as function of ξ (Schiereck & Verhagen, 2012)](image)

**Wave steepness**
The wave steepness, $s$, determines the wave load on the structure, it is the ratio between the wave height and deep water wave length. Generally in order to calculate the deep water wave length the wave period is used.

\[
s = \frac{H}{L_0} = \frac{2\pi H}{gT^2}
\]

$L_0$ = Deep water wave length [m]
3. Methodology

This chapter describes the used method to get answers to the research questions. First a selection of possible methods is given after which the most suitable model is chosen. Next the model description is given followed by the model set-up with an elaboration on the governing parameters as treated in chapter 2.

3.1 Selecting method

The Van der Meer (1988) formulae are used to calculate the stability of a stone protection on steep slopes (steeper than 1:6) under wave loads. Van der Meer has considered both statically stable and dynamically stable structures. For more gentle slopes on the other hand, it is not certain if a statically stable structure is the best option. Reliable design methods for designing statically stable stone protections on mild slopes are not yet available. That is why is chosen to study the area in between statically and dynamically stable structures, for mild slopes (from 1:5 to 1:50).

The search for a design method regarding profile development under wave loading can be determined by a three-stepped approach; a first step is a theoretical approach to investigate the governing physical processes involved. The second step is by using a numerical model to vary the governing parameters in order to investigate the influence of the individual parameters on the profile development and to develop a preliminary design method for the stone stability on mild sloped beaches. The third step is to perform physical model tests to validate the preliminary design method.

Chapter 2 showed that different researches have not been able to develop an accurate design rule, therefore the focus in this research is on step two. In a numerical experiment a lot of different cases can be investigated which are essential to investigate the basic principles of both statically and dynamically stable structures.

A lot of numerical models exists which can calculate the wave movement and thereby the loads on the structure, for example Volume of Fluid models, SWASH, XBeach and DUROSTA. Preferable the model can calculate both statically as dynamically stable structures, to make it possible to compare both types of structures. The model must also be able to calculate the stability of small stones till the size of small gravel. All these requirements together let to the numerical model XBeach-G, which is developed to model the morphology of gravel beaches during storm events.

3.2 Used numerical model: XBeach-G

Recently a numerical model was developed called XBeach-G (XBeach-Gravel) by McCall et al. (2014), in which several modification are made compared to the one-dimensional XBeach model which is used to model morphological changes of sandy beaches. XBeach-G can be used to calculate the response of a gravel beach to a storm event. Gravel beaches naturally occur on several high-attitude coasts throughout the world. Those beaches have varying slopes in the order of 1:5 – 1:10 with sediment size varying from a diameter of 0.01 m till 0.08 m. These dimensions are similar to the gravel used to protect mild slopes, because of these similarities it is chosen to use XBeach-G to model gravel stability on mild slopes.
3.3 Model description

XBeach-G is, just as XBeach, a process-based model. Compared to XBeach, three important modifications are implemented to develop XBeach-G. These modifications are concerning the wave equations, a groundwater model and sediment transport relations, which are explained in more detail in Appendix A.

This model is validated extensively by field and laboratory data. Large scale physical model tests were conducted in the Deltaflume and at three natural gravel beaches in the UK and one at the Brittany coast of France. In order to validate the morphological changes aspects such as berm formation, beach erosion and crest build-up were investigated. XBeach-G is able to predicts the morphological response qualitatively as well as quantitatively (Masselink, et al., 2014).

3.4 Model application

In this section a description of the conducted model experiment is given. In the model set-up the input parameters are described. Different tests were conducted in which the chosen parameters are varied. All possible combinations of input parameters are modelled and investigated.

3.4.1 Model set-up

The input parameters values of the model originate from data used for a design of a mild slope protection at the Western Scheldt and Lake IJssel so that realistic values are used. In order to investigate the influence of each parameter, the original data is used together with variations of it. Each input parameter is described separately in the same order as done in Chapter 2. First the hydraulic parameters, then the structural parameters and finally some parameters which are specific for the numerical model are described.

3.4.2 Hydraulic parameters

**Wave height**: For the numerical experiment the significant wave height is used. Wave heights in the range of 1 m till 2 m are investigated, those are in line with maximum wave heights that occur on Dutch estuaries and lakes. The $H_s$ is used instead of the $H_{2\%}$ because in XBeach-G the wave transformation is part of the model and takes into account the depth-limited conditions.

**Wave period**: In the numerical experiment the peak period is used. Different wave periods have been studied which form together with the chosen wave height typical wave steepness’s (explained in section 2.3.4). The wave periods range from 3.6 s till 11.3 s. This range is chosen as a result of chosen specific wave steepness’s of $s=0.01$, $s=0.03$ and $s=0.05$. The wave steepness $s=0.03$ corresponds to the design conditions of both locations (Eastern Scheldt and Lake IJssel). The other steepness’s are investigated to study the influence of the wave steepness/wave period. Table 1 shows the wave periods which belong to the chosen wave heights and wave steepness’s.

*Table 1: Wave characteristics used in the numerical experiment*

<table>
<thead>
<tr>
<th>$s$</th>
<th>$H_s = 1$ m</th>
<th>$H_s = 2$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.01$</td>
<td>8.0 s</td>
<td>11.3 s</td>
</tr>
<tr>
<td>$0.03$</td>
<td>4.6 s</td>
<td>6.3 s</td>
</tr>
<tr>
<td>$0.05$</td>
<td>3.6 s</td>
<td>5.1 s</td>
</tr>
</tbody>
</table>

**Water depth**: The water depth pertains to the numerical limits which are described in the section concerning the structural parameter.
**Storm duration:** In this study only the waves at the peak of the storm are modelled in combination with a constant high water level. For the design of a stone protection of the foreshore for the Western Scheldt a storm duration of 3 hours was used as maximum.

Van der Meer (1988) used instead of the storm duration the number of waves. After approximately 5000 waves he concluded that the damage does not change significantly anymore. However, in this numerical experiment only Dutch estuaries and lakes are investigated and normal extreme storms do not consist of 5000 waves. That is why is chosen to use a constant storm duration of five hours for each test. A five-hour storm is also used as a representative storm duration for calculating beach and dune erosion during storm for the Dutch coasts (Vellinga, 1986).

**Spectrum type:** It is chosen to use a JONSWAP spectrum which is characteristic for wind sea and is common to use as the design spectrum.

### 3.4.3 Structural parameters

**Stone diameter:** The stone diameter is one of the most important parameter which determines the final design. Three different stone sizes were studied, namely $D_{50} = 0.1$ m, $D_{50} = 0.05$ m and $D_{50} = 0.01$ m. The model is validated for gravel of a size in between $D_{50} = 0.002 – 0.08$ m which naturally occur (McCall, et al., 2015). The largest investigated size of $D_{50} = 0.1$ m is slightly larger than the validated size, this should be taken into account when analysing the results.

**Stone shape:** Van der Meer (1992) concluded that the shape of the stones has only a minor influence on the profile. Furthermore it is not possible to vary the shape in the numerical model yet and therefore it shall not be part of this study.

**Grading:** In XBeach-G it is not possible to define a grading. It makes use of a transport formula which does only make use of the $D_{50}$. In order to calculated the roughness the $D_{90}$ is used which is calculated by multiplying the $D_{50}$ by 1.5. In this study the grading is thus kept constant and therefore the influence is not studied.

**Mass density:** In this study it is chosen to use the value 2650 kg/m$^3$ for the mass density of the stones.

**Slope angle:** As already mentioned in the research question the main focus of this study is on mild slopes, which means that the slope angle is of primary importance. Uniform slopes are investigated during this study ranging from 1:5 to 1:50. The slope angles have been selected in a way that they overlap with the validity range of existing design methods. This means that also 1:5 is included to create an overlap with the statically stable method of Van der Meer (1988). Furthermore slopes equal to 1:10 and 1:25 are selected because these were studied by Sistermans (1993) and Ye (1996) who both conducted physical experiment. The top of the profile is chosen in such a way that no overtopping occurs, since the focus is on the development of damage and not on any overtopping.

The depth of the slope and cross-shore distance is determined by model limitations. It is not possible to use XBeach-G to model wave transformation accurately from deep water or from large distances from the coast (Masselink, et al., 2014). Therefore limits are given concerning the water depth, $kd < 3$ in which $k$ is the wave number, and concerning the cross-shore distance, $L_{model} < 20L_{wave}$ in which $L_{wave}$ is the characteristic wave length (Masselink, et al., 2014).
Hydraulic conductivity: The hydraulic conductivity largely depends on the stone size and the porosity. Because no accurate formula exists (see Chapter 2.3.2), it is chosen to use values which are measured during field experiments. However, the measured values still have a large range, for example for a beach with a stone size of \(D_{50} = 0.04\) m values for the hydraulic conductivity are found of \(k = 0.2 - 0.6\) m/s (McCall, et al., 2015). For the validation of the XBeach-G model the average values were used, which are also applied during this numerical experiment. An overview of the used values is given in Table 2.

**Table 2: Sediment characteristics**

<table>
<thead>
<tr>
<th>(D_{50}) [m]</th>
<th>(K) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.5</td>
</tr>
<tr>
<td>0.05</td>
<td>0.5</td>
</tr>
<tr>
<td>0.01</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Base and under layers: In the numerical experiment it is chosen to study a homogeneous layer of gravel in order to study the damage. In this way the damage is not limited to a certain level at which the base layer is present.

3.4.4 Model parameters

Grid size: A grid is automatically formed when defining the profile. Three variables can be adapted, namely 1) the minimum grid size, which is used near the water line and on the gravel beach, 2) the maximum grid size, applied in deeper sections of the model and 3) the minimum number of points per wavelength which describes the characteristics of the wave. It is not recommended to reduce this last number to less than 15 (Van Geer, 2014), during the experiment 25 points are used per wavelength.

3.5 List of used parameters

To summarize, an overview is given with all parameters which are investigated in this study. In the Table 3 the hydraulic and structural parameters are given together with the range to be considered. Table 4 shows the dimensionless parameters and their range.

**Table 3: Governing parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height</td>
<td>(H_s)</td>
<td>1 - 2 m</td>
</tr>
<tr>
<td>Wave period</td>
<td>(T_p)</td>
<td>3.6 - 11.3 s</td>
</tr>
<tr>
<td>Storm duration</td>
<td>(t)</td>
<td>5 hours</td>
</tr>
<tr>
<td>Stone diameter</td>
<td>(D_{50})</td>
<td>0.01 - 0.1 m</td>
</tr>
<tr>
<td>Slope angle</td>
<td>(\cot\alpha)</td>
<td>5 - 50</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td>(k)</td>
<td>0.01 - 0.5 m/s</td>
</tr>
</tbody>
</table>

**Table 4: Governing dimensionless parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stability parameter</td>
<td>(H_s/\Delta D_{50})</td>
<td>6 - 121</td>
</tr>
<tr>
<td>Wave steepness</td>
<td>(s = 2\pi H/gT^2)</td>
<td>0.01 - 0.05</td>
</tr>
<tr>
<td>Iribarren number</td>
<td>(\xi = \tan\alpha/(H/L_0)^{0.5})</td>
<td>0.1 - 2</td>
</tr>
<tr>
<td>Slope angle</td>
<td>(\cot\alpha)</td>
<td>5 - 50</td>
</tr>
</tbody>
</table>
4 Results

The following chapter presents the results of the conducted numerical experiment with XBeach-G. The results of all experiments can be found in Appendix B. After running the XBeach-G model, the model can provide different output data such as a cross-shore profile, a time series and run-up data. In this analysis the cross-shore profile is used, which gives the profile changes after the storm at a given point in time. From this information several analysis have been performed to derive profile development parameters, for example the amount of erosion or sedimentation and damage levels. The results of the tests are divided into two categories namely:

1. Tests in which no or minor damage (order erosion depth of $0.5D_{50} - 1D_{50}$) occurs, denoted as statically stable situations.

2. Tests in which a new equilibrium profile is formed, denoted as dynamically stable profiles.

First the definition of statically and dynamically stable structures is discussed in this chapter. Then the statically stable results as well as the dynamically stable results are described, of which the dynamically stable results are described in a quantitatively way as well as a qualitative way. Finally the most suitable parameter to describe the damage for design purposes is described for both type of profiles.

4.1 Static versus dynamic stability

In literature statically stable structures are structures for which no or minor damage is allowed under the design conditions, in which damage is the displacement of armour units (Van der Meer, 1988). For design purposes the damage is expresses by the damage level $S$, equal to:

$$S = \frac{A}{D_{50}^2}$$  \hspace{1cm} (14)

- $S$ = Damage level [-]
- $A$ = Eroded area [m$^2$]

The damage level $S$ is determined by means of empirical research. The ‘no damage’ criterion of Hudson (1953) is equal to a damage level $S$ in between 1 and 3, which is applicable to an armour layer consisting of two layers of stones and is depended on the slope angle; for milder slopes a higher damage level can be allowed. For milder slopes, waves act on a larger area which is present around the water level, as can be seen in Figure 8.

![Figure 8: Erosion area steep and gentle slope](image-url)
In order to obtain equal damage for gentle slopes as for steep slopes, more stones may be displaced without exposing the filter layer. Therefore higher damage levels can be allowed for gentle slopes. This higher damage level does not coincide with a higher risk, the remaining armour layer thickness shall be in the same order of safety. Van der Meer (1988) summarized this in Table 5.

<table>
<thead>
<tr>
<th>Slope angle (cot α)</th>
<th>S (Start of damage)</th>
<th>S (Failure = filter layer visible)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>17</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>17</td>
</tr>
</tbody>
</table>

The acceptable values used for S for the different slope angles resulted from experiments (Van der Meer, 1988). This makes it difficult to determine the values of S for even milder slopes which were not taken into account in these experiments. Therefore this study also investigates whether or not values for S of 2 or 3 are adequate for mild slopes (slopes milder than 1:6), or that these can be considered to be conservative.

### 4.1.1 Static damage level for mild slopes

To describe the static stability for milder slopes, the relation between damage level and slope is investigated. Van der Meer (1988) uses the damage level for a two-layer stone protection, which leads to a maximum eroded depth under extreme conditions equal to $1*D_{n50}$. It is commonly used to apply a double layer of stones. Thus, when 2 layers are eroded, the filter layer is visible and thus the armour protection is failing. This maximum eroded depth of one stone diameter is used to obtain the damage levels for milder slopes. It is assumed that the height over which damage occurs, $H_e$, stays the same for every slope, see Figure 8. This assumption is made in order to make a first estimate for a damage level for mild slopes. The same height combined with a different slopes angle leads to different lengths over which damage occurs ($L_e$). This leads to the following equation:

$$S(\alpha) = S_{\text{start}} \cdot \frac{\sin(\alpha_{\text{start}})}{\sin(\alpha)}$$ (15)

For the start values, the values shown in the second column in table 5 can be implemented, as is illustrated in Figure 9. From this figure it becomes clear that the start value has a significant influence on the damage levels for the milder slopes. These differences can be connected to different levels of reliability and robustness which can be defined by the client and depends on the type and location of the structure. This method is already in use for steep slopes (steeper than 1:6).
Figure 9: Damage level S in relation with the slope angle

4.1.2 Static damage level for small gravel
The smaller the gravel the larger the damage level, although the eroded area is in some cases almost zero. This is a results of the way of calculating the damage level in which the eroded area is divided by the square of the stone diameter. Because of this effect, the structure cannot be categorized as statically stable and therefore automatically is categorized as dynamically stable.

In addition to the previous point, the constructed layer thickness of gravel is commonly in the order of 30 cm, with tolerances which can reach 0.05 m for construction above the water level and 0.10 m below the water level (Schiereck & Verhagen, 2012). Gravel sizes of $D_{50}=0.1$ m, $D_{50}=0.05$ m and $D_{50}=0.01$ m are investigated in this thesis. During construction the armour layer for all three sizes consists of more layers than the 2 layers used by Van der Meer. Therefore, the allowed eroded depth can become larger providing that a sufficient cover depth is always present.

The construction thickness of the armour layer does not change for the investigated gravel sizes and therefore cannot become dimensionless. This is also chosen for the cover depth, which can be related to the construction depth or the gravel size. An option is to require that the cover depth is equal to half the construction depth (0.15 m). This results into larger allowable eroded depths in number of $D_{50}$, especially for the smaller gravel sizes.

4.1.3 Dynamically stable damage description
For a dynamic stable structure, generally a description of the equilibrium profile is used. After reaching an equilibrium profile, the profile is more or less statically stable again (Van der Meer, 1992). For structures consisting of only one stone size (for instance a berm breakwater), no problems occur when the profile changes or even shifts. This is, however, not the case for a protection on an existing sandy slope as described in this thesis. The profile may change within the installed armour layer, but not into the under-layer, because this will cause failure.

In a design it must be guaranteed that a minimum depth of the protection is always present to cover the mild slope. Therefore it is more convenient to use the eroded depth ($d_e$) or cover depth ($d_c$) instead of the description of an equilibrium profile. The definition of these depths are illustrated in Figure 10. The eroded depth is used in the next section about the quantification of the results.
4.2 Statically stable results

To describe statically stable structures, the adjusted damage level is used using the assumption of a layer thickness of \(2D_{50}\). The start values are chosen in such way that the damage levels for the milder slope coincide best with the values given by Van der Meer (1988). The damage levels are shown in Table 6 and are rounded to integers.

### Table 6: Adjusted damage level for mild slopes

<table>
<thead>
<tr>
<th>Cot((\alpha))</th>
<th>Damage level S</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>10</td>
<td>7</td>
</tr>
<tr>
<td>25</td>
<td>17</td>
</tr>
<tr>
<td>50</td>
<td>34</td>
</tr>
</tbody>
</table>

By means of an integral of the eroded depth, the eroded area is calculated. This area is compared with the acceptable damage level for statically stable structures, the situations in which this requirement is met are described.

4.2.1 Quantitative damage analysis of statically stable profiles

The used wave heights, wave periods and slopes, are parameters which can occur under extreme circumstances in Dutch lakes and estuaries. For some of the combinations, the results fulfil the statically stable requirements. For the largest stone size of 0.1 m in combination with the very mild slopes, 1:25 and 1:50, for all types of wave conditions the results are statically stable with only one exception which is the highest wave height combined with the largest wave period. The lower wave height of 1 m gives more statically stable results even for the steeper slopes, 1:5 and 1:10, in combination with the large stone size of 0.1 m and for the stone size of 0.05 m for the very mild slopes, 1:25 and 1:50. In total 25% of the results are statically stable. How these results are distributed over the different stone sizes and slopes can be seen in Table 7.

### Table 7: Number of statically stable results per slope and stone size

<table>
<thead>
<tr>
<th>Stone size</th>
<th>Slope 1:5</th>
<th>Slope 1:10</th>
<th>Slope 1:25</th>
<th>Slope 1:50</th>
</tr>
</thead>
<tbody>
<tr>
<td>(D_{50}=0.1)</td>
<td>1</td>
<td>2</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>(D_{50}=0.05)</td>
<td>-</td>
<td>-</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>(D_{50}=0.01)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
4.2.2 Amount of damage

The amount of damage for the statically stable profiles varies from a damage level of 5 for the 1:50 slope till no damage at all. From the 18 statically stable tests, 10 results show no damage. For the other tests the eroded depth and location of damage could be determined. The dimensionless eroded depth ($d_{e}/D_{50}$) is for all tests below 0.05, which is very low. This can mean that the method which is used to calculate the eroded area is maybe not the best method. Whilst the eroded depth is very low, still large eroded areas are calculated by using an integral and thereby even larger damage levels. To determine the eroded area, it might be better to implement a minimum damage depth (relative to the stone diameter) and only calculate the area belonging to this minimum depth. For example, XBeach-G shows erosion with an accuracy of 0.0001 m. Although the erosion depth may be small, when this erosion takes place over a large length, this still results into a relative large eroded area as is shown in Figure 11. For design purposes this accuracy is not required at all, therefore a minimum should be implemented, like the $D_{15}$.

![Figure 11: Impression of a small eroded depth (enlarged) over a large length](image)

4.2.3 Location of damage

In most tests the location of damage is spread over a relative large area with a length up to $1*H_{s}$. The erosion always takes place below the still water level and the sediment is transported upward. In most cases, the maximum eroded depth is located around a depth of $1*H_{s}$ which is relative and perpendicular to the still water line. It may also be more representative for the location of damage to set a minimum eroded depth.

4.3 Dynamically stable results: qualitative damage analysis

Visual comparison of all test runs show that the profile development can occur in two different ways, namely:

1. A crest profile: material is transported in upward direction, creating a crest and an erosion hole at the foot of this crest. The crest is formed around the still water level. In literature this type of profile can also be indicated with different names, such as summer, accretion, step or crest profile (Van Hijum, 1974).

2. A bar profile: Material transport is in downward direction, creating submerged bar and an erosion hole above the bar, also located around still water level. Different names for this type of profile are winter, storm, erosion and bar profile (Van Hijum, 1974).

The profiles found for all wave conditions can be found in Appendix B. The two different patterns can be seen in Figure 12, left the bar profile is illustrated and the right figure shows the crest profile. The blue line indicates the mean water level. The dashed line in the initial slope and the red line the slope after a storm.
Which pattern occurs, is influenced by different parameters for instance the slope angle and the stone diameter, see for all parameters section 3.5. Why these different pattern occur are explained based on the influence on the different parameters, and summarized in section 4.3.4.

4.3.1 Influence of the hydraulic parameters

In order to investigate the influence of individual parameters, model runs were conducted in which only one parameter was changed and the other input parameter were kept constant.

Wave height

The wave heights which were tested were $H_s = 1$ m and $H_s = 2$ m. Obviously more damage occurred for the higher waves than for the lower waves, the erosion hole becomes larger and deeper and for the crest profile, the crest becomes higher and at the same time the erosion below the crest is located deeper (see Figure 13). However, the patterns of the profile changes are equal, as can be seen in Figure 13. The main differences are the amount of damage and the location of it. If damage occurs with significant changes of the profile, it can be seen that the angle of the profile is almost the same for both wave heights as shown in Figure 13. With only varying the wave height the pattern of the profile does not change from a crest profile to a submerged bar profile or the other way around.
Van Hijum (1974) states that the larger the wave period the larger the damage, because a long wave period coincides with a long wave length which contains more energy. This trend is also seen in every conducted test, for example in the following tests which are illustrated in Figure 14. The left figure shows that the crest height increases with increasing wave period, and thereby also deepening of the erosion hole. In the figure at the right the erosion length and depth both are increasing with increasing wave period.

No exceptions are found, which show less damage for longer wave periods. The variation of the wave period does not influence the pattern of the profile. Each test shows that only the amount of damage changes and not the pattern of it.

**Storm duration**

The duration of the storm determines the amount of damage, which increases with the duration of the storm. To illustrate this the profile change is plotted every hour for two different types of profiles as can be seen in Figure 15. Most of the damage occurs during the first two hours after which the increase of damage is reduced for the longer duration. For the crest profile (Figure 15 left panel) the
crest becomes higher for a longer storm duration, also the erosion hole shifts downward. For the submerged bar profile (Figure 15 right panel) the erosion hole becomes larger for a longer storm duration and thereby increasing the height of the submerged bar. Van der Meer (1988) also found that most profile development takes place in the first few hundred waves, which coincide with the first hours of the storm. Furthermore he noticed that with a longer storm duration the crest height increases and an increase in the length of the changed profile, this is also visible in Figure 15 both profiles are longest after the 18000 s.

![Influence of the storm duration D50=0.05 Hs=2 m s=0.01](image1)

![Influence of the storm duration D50=0.01 Hs=2 m s=0.01](image2)

Figure 15: Profile change per hour for a five-hour storm, left for a crest profile, right for a bar profile

4.3.2 Influence of the structural parameters

**Slope angle**

The initial slopes varied from 1:5 till 1:50. In order to plot different slopes in one plot, the profiles were shifted to one point which they all have in common, namely the intersection with the still waterline. Only the profiles in which a lot of damage occurred were compared. Profiles without visual changes cannot be compared. The following aspects were compared: the response angle of the sediment, the crest location and the submerged bar location.

For the slopes of 1:50 almost no visual damage occurred as can be seen in the bottom left panel of Figure 16, so no comparison to the other profiles could be made. For the used wave conditions in this thesis, no damage occurred but for higher waves conditions that may not be the case.

For the high wave steepness s=0.01 in combination with $D_{50}= 0.1$ m it seems that different profiles occur for different initial slopes (Figure 16). However, when these three different profiles are plot at the same scale, the same profiles occur for all slopes which means they all have the same response angle of the resulting scour hole on the upward side of the scour hole, see Figure 17. More details about the response angle is given in section 4.3.4.
Chapter 4 - Results

Figure 16: Different types of profiles for different slopes with the same wave conditions for $D_{50}=0.1 \text{ m}$

Figure 17: Three different slopes plot in one figure for $D_{50} = 0.1 \text{ m}$
Decreasing the stone size till $D_{50} = 0.05$ m, the same profiles occur, only a deviation is showing for the 1:25 slope which has a higher submerged bar but still has the same response angle, Figure 18 left illustrates this. Figure 18 right, shows that by decreasing the stone size till $D_{50} = 0.01$ m, totally different types of profiles occur. The response angle changes, and also the location of the crest and submerged bar deviate for the different profiles. This trend occurs also for the other combinations of wave steepness and wave height. For the wave conditions which are elaborated in this section, the profile changes are the most significant and therefore more easily to compare with each other.

**Figure 18:** Influence of the slope for $H_s=2$ m and $s=0.01$, left for $D_{50}=0.05$ m, right for $D_{50}=0.01$ m

**Stone diameter**

A lot of different profiles occur when only the stone diameter is changed. For instance a crest profile occurs for a $D_{50} = 0.1$ m and $D_{50} = 0.05$ m while for $D_{50} = 0.01$ m a submerged bar profile occurs, as is illustrated in the left part of Figure 19. For some tests the profile pattern changes in between $D_{50} = 0.1$ m, which has formed a small crest above the waterline, and $D_{50} = 0.05$ m and $D_{50} = 0.01$ m, in which only a submerged bar is formed. This is illustrated in the right part of Figure 19.

**Figure 19:** Influence of the stone diameter, left on a slope of 1:10, right on a slope of 1:5
For most wave conditions on the slopes 1:5, 1:25 and 1:50 the erosion increases for decreasing stone sizes. This is not always the case for the 1:10 slope, in which the damage stays the same or becomes even smaller for decreasing stone sizes. This can be partly explained by the occurrence of different profile patterns for the different stone sizes, which results into other erosion areas which are in this case smaller. This is further elaborated in section 4.3.4.

**Hydraulic conductivity**

Hydraulic conductivity is a property of the porous medium as well as of the fluid that flows through it, and describes how easy a fluid can flow through the porous medium (section 2.3.2). The hydraulic conductivity influences the groundwater dynamics in the structure. Different values varying from 0.01 till 0.5, which can occur at natural gravel beaches, were analysed and the most important results are shown in Figure 20 at the next page.

All three figures illustrate that the hydraulic conductivity does not influence the type of profile or the amount of damage. In the top left of Figure 20 it can be seen that the angle of response is slightly influenced and becomes steeper for a higher hydraulic conductivity. This may be because of that waves can penetrate very easily and have less power to transport the sediment back downwards. The top right figure shows that the location of the erosion hole depends on the hydraulic conductivity. However, the amount of damage stays the same. Figure 20 at the bottom left illustrated that the height of the crest is also slightly depended on the hydraulic conductivity, in which the crest height becomes larger for lower values for the hydraulic conductivity. An explanation can be that for the very high hydraulic conductivity the waves infiltrate faster than for the lower values and thereby cannot reach the height that can be reached in case the waves were not infiltrating into the structure. In this figure the change of the response angle is not visible. In order to investigate the influence of the hydraulic conductivity on the angle of response more research has to be conducted.
4.3.3 Influence of the Iribarren number

The tests conducted in the experiment consisted of different wave heights, wave steepness’s and different slope angles. The Iribarren number is the ratio of the slope angle and the square root of the wave steepness. The range of tested Iribarren numbers are summed up in Table 8.

Table 8: Iribarren numbers per test

<table>
<thead>
<tr>
<th>$\xi_\varphi$</th>
<th>Slope 1:5</th>
<th>Slope 1:10</th>
<th>Slope 1:25</th>
<th>Slope 1:50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S=0.01$</td>
<td>2</td>
<td>1</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>$S=0.03$</td>
<td>1.1</td>
<td>0.6</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>$S=0.05$</td>
<td>0.9</td>
<td>0.5</td>
<td>0.2</td>
<td>0.1</td>
</tr>
</tbody>
</table>

The range of the Iribarren number for the gentle slope angles does not change significantly. In order to investigate the influence of the Iribarren number, extra tests were conducted with a constant Iribarren number of $\xi=1.11$ and $\xi=0.56$ for plunging waves and $\xi=0.2$ for spilling waves. The Iribarren number is kept constant for the different slopes by only changing the wave period.

It can be concluded that solely the Iribarren number does not influence the pattern of the occurring profile. For example for $\xi=0.2$ and $D_{50}=0.01$ m, a bar profile occurs for a 1:5 and 1:10 slope and a crest profile occurs for the 1:25 slope, this is illustrated in Figure 21. The same occurs for the $D_{50}=0.05$ only
than the transition is in between the 1:5 and 1:10 slope, exactly as is found for all other tests elaborated in the next section.

Figure 21 Results for Iribarren number =0.2 for different slopes and $D_{50}=0.01$ m

4.3.4 Different profiles and processes

After analysing all different parameters it can be seen that some parameters only influence the amount of damage and that other parameters also influence the type of the profile (crest or bar). The two main parameters which determine the type of profile are the slope angle and the stone diameter. In Table 9 a summery is given of all conducted experiments and it shows that despite the changing wave conditions the same type of profile occurs. Only for the very mild slopes two patterns can occur, namely no change or a crest profile. For these situations only for the largest wave height in combination with the largest wave period a profile change occurred.

Table 9: Different profile patterns for different situations

<table>
<thead>
<tr>
<th></th>
<th>Slope 1:5</th>
<th>Slope 1:10</th>
<th>Slope 1:25</th>
<th>Slope 1:50</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{50}= 0.1$ m</td>
<td>Crest</td>
<td>Crest</td>
<td>No ( or Crest)</td>
<td>No</td>
</tr>
<tr>
<td>$D_{50}= 0.05$ m</td>
<td>Bar</td>
<td>Crest</td>
<td>Crest/No</td>
<td>No (or Crest)</td>
</tr>
<tr>
<td>$D_{50}= 0.01$ m</td>
<td>Bar</td>
<td>Bar</td>
<td>Crest</td>
<td>Crest</td>
</tr>
</tbody>
</table>
From the model test results it can be concluded that the investigated wave height and period does not influence the type of profile for the investigated values in contrast to the slope angle and the stone diameter which do influence the type of profile. This can be explained by the equilibrium profile that is formed to the wave loads after a sufficient long time. Each type and size of gravel forms a profile for a certain wave forcing in which the sediment transport is in equilibrium. The equilibrium profile is described by the response angle of the gravel material. Depending on whether the initial slope which needs to be protected is steeper or gentler than the response angle, the sediment will go upward or downward respectively. For the more gentle slopes the equilibrium slope of the investigated gravel is steeper and therefore the sediment transport is in an upward direction forming a crest profile. In Table 9 it can be seen that the crest profile for the steepest slopes (1:5) is only found for the largest stone size ($D_{50}=0.1\ m$) and that for a gentler slope (1:10) a crest profile is found for the smaller stone size ($D_{50}=0.05\ m$) which continues for the even more gentle slopes (1:25 and 1:50) for which all investigated stone sizes form a crest profile or give no profile change.

For the crest profiles, the response angle of a certain stone sizes can be determined based on the model tests, an overview is given in Table 10. In the second row the angle of response is given which is the average of the response angles for the different wave conditions. The third row of Table 10 shows the range of the response angle for the investigated wave conditions. The maximum slope angle occurs around the still water level. Small differences were found for the different wave conditions; the smallest response angles coincide with the steepest waves (i.e. the smallest wave periods), and conversely, the longest waves coincide with the steepest equilibrium profile. Wave conditions in which no damage occurred were not taken into account by determining the response angle and its range.

<table>
<thead>
<tr>
<th>Response angle</th>
<th>Mean angle in tan $\alpha$</th>
<th>Range tan $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{50}= 0.1\ m$</td>
<td>0.3 (1:3.33)</td>
<td>0.26 – 0.39</td>
</tr>
<tr>
<td>$D_{50}= 0.05\ m$</td>
<td>0.22 (1:4.5)</td>
<td>0.15 – 0.26</td>
</tr>
<tr>
<td>$D_{50}= 0.01\ m$</td>
<td>0.08 (1:12.5)</td>
<td>0.07 – 0.09</td>
</tr>
</tbody>
</table>

The found response angles are angels which can also be found at natural gravel beaches. Jennings and Shulmeister (2002) found that the slope of natural gravel beaches varies from $\tan \alpha = 0.08 – 0.25$.

For the largest gravel size tested in this thesis ($D_{50}=0.1\ m$) the equilibrium profile occurs for both the 1:5 and 1:10 slope and could therefore be compared with each other. However, no significant differences were found for the different slopes. This can be explained by the fact that the Iribarren number for those slopes were respectively equal to 2 and 1, which are both of the same type of wave breaking, consequently resulting in only minor differences.

For the different wave conditions, wave height and period, larger differences were found as can be seen in Table 10 indicated by the range of the response angles. The different wave conditions were al plot together in Figure 22. The statically stable profile is clearly visible, this is the result of the lowest wave height in combination with two highest steepness’s. For all other wave conditions, the profile around the still water line has almost the same angle, which can be explained by the associated Iribarren numbers which are in between $1 – 0.5$ and still are a plunging type of breaker. The same results were found for the other slopes and gravel diameters.
The other type of profile, the bar profile, occurs when the initial profile is steeper than the equilibrium profile of the grading. This is the case for the 1:5 slope for gradings with a $D_{50}=0.05$ m and $D_{50}=0.01$ m and for the 1:10 slope in combination with $D_{50}=0.01$ m. For these tests the sediment is transported downward, creating a submerged bar and erosion hole around the still water line. A remarkable result is that for $D_{50}=0.05$ the mean equilibrium slope is equal to 1:4.5 (see Table 10) which is steeper than the investigated 1:5. However, in all conducted tests for that specific situation, a submerged bar profile was found instead of a crest profile. A crest profile would be more in line with the hypothesis that the type of occurring profile depends on the equilibrium profile for a certain stone diameter. This results indicates that the initial profile or occurring wave conditions may have some influence on the equilibrium profile or equilibrium profile.

### 4.4 Dynamically stable results: quantitative damage analyses

Similar to the statically stable results, the dynamically stable results can be quantified in different ways. For instance the damage level, which becomes significantly large for the dynamically stable profiles (order 1000 and higher). Other ways to quantify the damage is in terms of the eroded area ($A_e$) and eroded depth ($d_e$) which is described in this section. Subsequently, the location of the damage on the slope is described and finally some remarks are given in which a damage parameter is determined which is best to describe the damage for design purposes.

#### 4.4.1 Amount of damage

**Eroded area**

In the same way as for the statically stable results, the eroded area is calculated by taking the integral of the eroded depth over the eroded profile. For the statically stable results the dimensionless eroded area is calculated named the damage level ($S$). Figure 23 shows the dimensionless eroded depth in relation with the stone diameter. For dynamically stable profiles, the damage levels become very large for the smallest stone diameter. Because the numbers become so large, it is difficult to distinguish the damage levels for the larger stone sizes which are more in the area of interest. Therefore it is chosen not to plot the dimensionless form of the eroded area and just the eroded area in square meters.
In Figure 24 also the statically stable results are plotted, which can be recognized by the small eroded area (almost zero). The left panels of Figure 24 show the results of the one-meter wave height and the right panels show the two-meter wave height. The first row gives the lowest wave steepness, the next row the intermediate wave steepness and the lowest row gives the steepest waves.
Figure 24: Eroded area of all experiments, left panels $H_s=1$m, right panels $H_s=2$m, for different wave steepness’s (indicated above the figure).

For most of the slopes the eroded area decreases for larger stone sizes. However, this is not the case for the 1:10 slope. For this slope in combination with the lower wave steepness’s the eroded area even increases for larger stone sizes. The top right figure shows this trend also for the 1:25 slope. This trend was already mentioned in section 4.3.2 in which the influence of the stone diameter was investigated, the combination of slope and stone diameter shows it more clearly.

Generally, the tests for the 1:10 slope show that the difference between the initial slope and the equilibrium slope is largest for the largest stone size and thus creating the largest eroded area as shown
in Figure 24. This is contrary to statically stable structures for which a larger stone size is always more stable. However, some tests show that for the smaller wave heights in combination with high steepness’s the damage slightly increases for larger stone diameters, or stays the same. This is probably because these profiles are almost statically stable for $D_{50}=0.05$ m and fully statically stable for $D_{50}=0.1$ m.

**Eroded depth**

For all preformed tests, including the statically stable tests, the eroded depth is plot against the stone diameter for the different slopes and wave conditions. The tests consisted of a homogeneous structure, so the eroded depth is not limited by an under-layer. The eroded depth has been made non-dimensional with the $D_{50}$ of the armour material. The results can be seen in Figure 25 till Figure 28.

Figure 25 presents the 1:5 slope which shows a significant eroded depth (100 till 200 times the $D_{50}$ which is equal to 1 – 2 m) for the smallest stone size ($D_{50}=0.01$ m), which is not acceptable for an armour layer. The eroded depth already decreases for $D_{50}=0.05$ m, but still is too large (order $10*D_{50}$) and therefore not desirable. This means that for this type of slope larger stones are needed, depended on the wave conditions. For the smallest wave height in combination with the highest wave steepness’s the stone size of $D_{50}=0.1$ m already gives a statically stable results. Statically stable results have a maximum depth of $1*D_{50}$. For the dynamically stable results a stone size can be found for which the eroded depth stays below a required level, because the trend shows a decrease in eroded depth for larger stone sizes.

**Figure 25: Maximum eroded depth, 1:5 slope**

Figure 26 illustrates the 1:10 slope and shows just as for the eroded area (Figure 24) that the larger stone sizes do not result into a smaller eroded depth. The dimensionless depth parameter is almost the same value for the two larger stone sizes, which means that the eroded depth in meters increases. This effect is best observable for the largest wave lengths (lowest wave steepness). For the dynamically stable profiles it is not certain if for every wave condition a stone diameter can be found which fulfils the requirements concerning a maximal eroded depth, because the erosion depth for the smallest stone size is probably too large and the erosion depth only increases with increasing stone size. This is only possible in case a very thick armour layer is constructed which allows a large eroded depth. For the 1:10 slope two statically stable results are found for the largest stone sizes in combination with the lower wave heights.
In Figure 27 many statically stable results are shown for the $D_{50}=0.1$ m as well as for the $D_{50}=0.05$ m. It is also possible to construct a dynamically stable armour layer, if this is desirable because the damage stays between limits which can be set. This results into smaller stone sizes. The eroded depth for the smallest stone size is already within a reasonable limit when for instance a layer thickness of 30 cm is constructed, which means that it is possible to design a dynamically stable structure. The only remarkable result is for the highest wave height combined with the lowest wave steepness, which shows a much higher erosion depth than the other wave conditions.

The 1:50 slope, see Figure 28, has even more statically stable results and equally important the dynamically stable results are within limits which can be used for protection of mild slopes. This means that a dynamically stable profile may be an option for a armour layer on a 1:50 slope. Again, deviations are found for the highest wave height combined with the lowest wave steepness, but the trend is the same as for the other wave conditions. Only the amount of damage is larger, which coincides with the larger wave period.
Figure 28: Maximum eroded depth 1:50 slope

In Appendix C, an overview of all damage results can be found.

4.4.2 Location of damage
Damage in XBeach-G is calculated by using the Nielson (2002) formula for sediment transport. When the critical Shields value is exceeded, the amount of damage is calculated. Based on this calculated transport, the erosion is calculated which can be in the range of order $10^{-5}$ till $10^{0}$. The smallest number of damage can only occur if very small particles are present in the grading. However, these size of gravel cannot cause severe damage. That is why is chosen to define damage as at least $0.5*D_{50}$ to determine the eroded area.

For design purposes it may be interesting to differentiate different gradings along the profile to provide the most economical design. Therefore it is useful to investigate at what range the damage on the slopes occurs. Table 11 shows the highest and lowest points of damage for different slopes and different stone sizes. The values are expressed in numbers of the occurring wave height ($H_{s}$) and relative and perpendicular to the still water line. The maximum values of all investigated wave conditions are shown, most values resulted from the most severe wave conditions. In Appendix C all values can be found for the location of damage for every wave condition.

Table 11: Location of the damage relative to the still water line in term is $H_{s}$

<table>
<thead>
<tr>
<th>Slope</th>
<th>Limit</th>
<th>$D_{50}=0.1$ m</th>
<th>$D_{50}=0.05$ m</th>
<th>$D_{50}=0.01$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:5</td>
<td>upper limit</td>
<td>+0.4$H_{s}$</td>
<td>+1.3$H_{s}$</td>
<td>+2$H_{s}$</td>
</tr>
<tr>
<td></td>
<td>lower limit</td>
<td>-1.9$H_{s}$</td>
<td>-1.5$H_{s}$</td>
<td>-1$H_{s}$</td>
</tr>
<tr>
<td>1:10</td>
<td>upper limit</td>
<td>-0.5$H_{s}$</td>
<td>-0.3$H_{s}$</td>
<td>+1$H_{s}$</td>
</tr>
<tr>
<td></td>
<td>lower limit</td>
<td>-2.8$H_{s}$</td>
<td>-2.7$H_{s}$</td>
<td>-3.5$H_{s}$</td>
</tr>
<tr>
<td>1:25</td>
<td>upper limit</td>
<td>-0.5$H_{s}$</td>
<td>-0.6$H_{s}$</td>
<td>-0.5$H_{s}$</td>
</tr>
<tr>
<td></td>
<td>lower limit</td>
<td>-2$H_{s}$</td>
<td>-2.8$H_{s}$</td>
<td>-4.8$H_{s}$</td>
</tr>
<tr>
<td>1:50</td>
<td>upper limit</td>
<td>0$H_{s}$</td>
<td>-0.8$H_{s}$</td>
<td>0$H_{s}$</td>
</tr>
<tr>
<td></td>
<td>lower limit</td>
<td>0$H_{s}$</td>
<td>-2.4$H_{s}$</td>
<td>-5$H_{s}$</td>
</tr>
</tbody>
</table>
To describe the location of damage, a distinction is made between a crest profile and a bar profile. For the crest profiles, only erosion occurs below the still water line because sediment is transported upwards to form a crest just above the still water line. Only the most severe wave condition in combination with the 1:5 slope shows damage above the still water line. This special case shows a profile which is in between a crest and a bar profile (see Figure 29 right, indicated in red) and for that special case some erosion takes place around the still water level. The location from where the sediment is taken to form a crest profile varies with the used stone diameter, as shown in Figure 29.

In the model the influences of the stone diameter cannot be linked to the occurring profile changes by analysing the used formulae. It was already known that the used transport formula does not predict the location of erosion well. In Figure 30 a graph of the model validation is shown. The location of the crest and the response angle can be predicted really well, only an improvement can be made for the submerged bar erosion prediction. A recommendation for better results concerning the location of erosion is to use XBeach-G with a different transport formula.
For the bar types of profile, the erosion takes place around the still water level. Sediment is eroded and subsequently transported downwards. For the smallest stone sizes, the maximum depth can be relatively large compared to the other stone sizes. A reason for this can be that the chosen definition of damage (0.5*D_{50}) is not suitable for smaller stone sizes. Especially for dynamic profiles it should be determined whether the damage and its location must be depended on the stone diameter.

### 4.5 Design based remarks on damage analyses

For statically stable structures it is common to use the damage level. As already discussed in section 4.1 the damage level is not suitable for dynamically stable structure especially not for smaller gravel sizes. For dynamically stable structures an equilibrium profile is formed, however, this profile cannot move or shift to much because the protected slope cannot be exposed. Therefore, the description of an equilibrium profile is also not an option for this kind of structure. Other possibilities are the eroded area and the eroded depth. For mild slopes it does not matter if the erosion is spread over a longer area, which increases the eroded area. What is important is the eroded depth, the protected slope must never be exposed. With this requirement a maximum eroded depth can be determined.

For statically stable structures on steep slopes generally an armour layer consists of two layers of stones. For milder slopes in combination with dynamically stable structures, the size of stones decreases significantly. Small stone sizes, in the order of gravel, cannot be constructed with a thickness of two stone diameters. The accuracy of the construction method used are less than two times the stone diameter, in particular when constructing below the water level. Therefore it is common to construct a 30 cm thick layer (CIRIA, et al., 2007). This means that the layer thickness is not related to the stone size anymore, which is also a possibility for the description of allowable damage.

An option is to allow the damage to a maximum equal to half the constructed layer thickness. This may also be an option for statically stable structures in case a thicker layer is constructed than the common two-stone layer.
5 Comparison numerical results with current design methods

Many design methods exist for statically stable stone protections, e.g. Hudson, Van der Meer and Van Gent et al., which are primarily applicable to steep slopes (steeper than 1:6). Design methods to determine the stability of stones on mild slopes are not yet available. The design formulae used today to determine both statically and dynamically stable structure on mild slopes, are usually those of Van der Meer (1988). Therefore, in this chapter a comparison is made between the results of the XBeach-G model and the two existing design formulae of Van der Meer (1988) for statically and dynamically stable stone protections.

5.1 Statically stable methods

In the literature review it became clear that the method used to calculate the required stone size for a statically stable structure is the Van der Meer formulae (1988). Only the formula for plunging waves is given, see equation 26, since no surging breakers were investigated in the conducted numerical experiment.

\[
\frac{H_s}{\Delta D_{n50}} = 6.2 P^{0.18} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5} \quad \text{(plunging breakers)}
\]  

(16)

- \(D_{n50}\) = Median nominal stone diameter [m]
- \(H_s\) = Significant wave height [m]
- \(P\) = Permeability of the structure [-]
- \(S\) = Damage level [-]
- \(\xi_m\) = Iribarren number based on mean wave period \(T_m\) [\(^{\circ}\)]
- \(N\) = Number of waves [-]
- \(\alpha\) = Slope angle [\(^{\circ}\)]

To compare the numerical results from XBeach-G with the Van der Meer formula, the parameters have to be chosen in such a way that these parameters best coincide with the parameters used in the numerical experiment. The same values are used for the wave height, relative density and the slope angle. For the permeability of the structure \(P=0.6\) is chosen which can be used for structures with a homogenous armour and core, which corresponds to the model used in the Xbeach-G. The number of waves is calculated and differs per conducted test. It is calculated by using the mean period instead of the peak period. The mean period \((T_m)\) can be calculated with the following equation.

\[
T_p = 1.2 \times T_m \quad \text{(CIRIA, et al., 2007) (Schiereck & Verhagen, 2012)}
\]  

(17)

For the number of waves 4000 waves is chosen, which is equal to the average number of waves for all wave conditions. Although some tests consisted of less waves than 4000, this number does not influence the outcome significantly. The nominal stone diameter \(D_{n50}\) has been calculated using the empirical relation from Laan (1981) using:

\[
D_{n50} \approx 0.84 \times D_{50}
\]  

(18)
The mean period is also used to determine the Irribarren number $\xi_m$ which is used in the Van der Meer formula. Lastly, the damage number to calculate the required stone size is set to be $S=2$, which corresponds to only minor damage.

The results of XBeach-G obtained in Chapter 4 are not optimal for a comparison, because a lot of statically stable results had no damage ($S=0$). In order to compare the numerical results with Van der Meer, new tests were conducted to obtain the stone size which corresponds to minor damage (around $S=2$). This is a slow and iterative process, therefore it is chosen to accept damage levels between $S=1$ and $S=3$ which gives adequate results for a first approximation. Furthermore some extra wave conditions have been added to obtain more results for the 1:5 slope, for which only one statically stable point with an value between $S=1$ to $S=3$ was calculated during the numerical experiment. The extra numerical calculations were performed with wave conditions consisting of a lower wave height of 0.8 m in combination of the same wave steepness as used during the numerical calculations in Chapter 4. All results are plotted in Figure 31. The blue line gives the Van der Meer formula for statically stable structures, the dashed line shows the extrapolated part of the formula for which part the formula is out of applicable range. The coloured stars show the numerical results for different wave conditions. Which wave condition belong to which point can be found in Table 12, the points are numbered by increasing Irribarren number $\xi_m$.

![Experimental results compared with the plunging formula of Van der Meer](image)

*Figure 31: Results of XBeach-G compared with the Van der Meer formula for plunging waves (1988)*
### Table 12: The named points with their characteristics

<table>
<thead>
<tr>
<th>Number</th>
<th>Slope</th>
<th>S [-]</th>
<th>(\xi_m) [-]</th>
<th>(D_{50}) [m]</th>
<th>(H_s) [m]</th>
<th>(T_p) [s]</th>
<th>(H_s/\Delta D_{50})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1:50</td>
<td>2.5</td>
<td>0.075</td>
<td>0.021</td>
<td>1</td>
<td>3.6</td>
<td>28.9</td>
</tr>
<tr>
<td>2</td>
<td>1:50</td>
<td>1.5</td>
<td>0.075</td>
<td>0.06</td>
<td>2</td>
<td>5.1</td>
<td>20.2</td>
</tr>
<tr>
<td>3</td>
<td>1:50</td>
<td>1.4</td>
<td>0.093</td>
<td>0.07</td>
<td>2</td>
<td>6.3</td>
<td>17.3</td>
</tr>
<tr>
<td>4</td>
<td>1:25</td>
<td>2.9</td>
<td>0.150</td>
<td>0.035</td>
<td>1</td>
<td>3.6</td>
<td>17.3</td>
</tr>
<tr>
<td>5</td>
<td>1:25</td>
<td>1.9</td>
<td>0.150</td>
<td>0.08</td>
<td>2</td>
<td>5.1</td>
<td>15.2</td>
</tr>
<tr>
<td>6</td>
<td>1:50</td>
<td>2.0</td>
<td>0.167</td>
<td>0.06</td>
<td>1</td>
<td>8.0</td>
<td>10.1</td>
</tr>
<tr>
<td>7</td>
<td>1:25</td>
<td>1.7</td>
<td>0.186</td>
<td>0.09</td>
<td>2</td>
<td>6.3</td>
<td>13.5</td>
</tr>
<tr>
<td>8</td>
<td>1:25</td>
<td>1.1</td>
<td>0.192</td>
<td>0.05</td>
<td>1</td>
<td>4.6</td>
<td>12.1</td>
</tr>
<tr>
<td>9</td>
<td>1:25</td>
<td>1.1</td>
<td>0.333</td>
<td>0.09</td>
<td>1</td>
<td>8.0</td>
<td>6.7</td>
</tr>
<tr>
<td>10</td>
<td>1:10</td>
<td>1.5</td>
<td>0.375</td>
<td>0.08</td>
<td>1</td>
<td>3.6</td>
<td>7.6</td>
</tr>
<tr>
<td>11</td>
<td>1:10</td>
<td>2.1</td>
<td>0.497</td>
<td>0.105</td>
<td>1</td>
<td>4.6</td>
<td>5.8</td>
</tr>
<tr>
<td>12</td>
<td>1:5</td>
<td>2.4</td>
<td>0.745</td>
<td>0.06</td>
<td>0.8</td>
<td>3.2</td>
<td>8.1</td>
</tr>
<tr>
<td>13</td>
<td>1:5</td>
<td>1.2</td>
<td>0.750</td>
<td>0.09</td>
<td>1</td>
<td>3.6</td>
<td>6.7</td>
</tr>
<tr>
<td>14</td>
<td>1:5</td>
<td>1.4</td>
<td>0.955</td>
<td>0.085</td>
<td>0.8</td>
<td>4.1</td>
<td>5.7</td>
</tr>
<tr>
<td>15</td>
<td>1:5</td>
<td>2.4</td>
<td>0.958</td>
<td>0.115</td>
<td>1</td>
<td>4.6</td>
<td>5.3</td>
</tr>
</tbody>
</table>

As can be seen in Figure 31 all points found are located above the (extrapolated) line obtained with the Van der Meer formula. This implies that smaller stone sizes are found in XBeach-G than when using the Van der Meer formula (1988) for the same damage criterion (around) \(S=2\). This means that using the Van der Meer formula outside its applicable range might give rather conservative values. For the milder slopes – and thereby lower Iribarren number – the stability number \((H_s/\Delta D_{50})\) increases significantly, following the same trend as the extrapolated formula of Van der Meer. The numerical results show that even smaller stone diameters can be stable than the stone diameter calculated using the extrapolated Van der Meer formula for start of damage criterion. Similar results were found by Sistermans (1993) and Ye (1996), as presented in Figure 7.

The results that show large deviations from the Van der Meer Formula between \(\xi_m=0.7\) and \(\xi_m=1.0\), which is remarkable since these values fall into the validity area of the Van der Meer formula for plunging waves. In theory these points should therefore not differ much from the Van der Meer formula, however, for some points almost a half the stone size is found to be stable using Xbeach-G. In order to explain this difference both the reliability of the numerical outcomes and the Van der Meer formula is described by conducting a sensitivity analysis in a qualitative way.

#### 5.1.1 Sensitivity analysis

For a sensitivity analysis all input parameter should be investigated together with the structural parameters. Most of the parameters such as the \(H_s\) and the slope angle are part of both the Van der Meer formula and used in XBeach-G and therefore were not investigated. Only the parameters which differ in both methods were investigated.
Sensitivity of Van der Meer formula

In the Van der Meer formula the constant value of \( C_{pl} \) is used. According to CIRIA et al. (2007) this coefficient has an average value of 6.2, assuming a normal distribution this values has a standard deviation of 0.4. Normally only the lower limit is of importance which is assessed by \( \mu - 1.65 \times \sigma \), giving the 5% limit of being smaller than this value. But to explain the differences between both methods, the upper limit is of interest. This upper limit is calculated in the same way only adding the \( 1.65 \times \sigma \) resulting in the 5% exceedance value, plotted in Figure 32 with the dotted line. As can be seen, the differences cannot be explained by this coefficient only.

Experiment results compared with the plunging formula of Van der Meer

![Graph showing sensitivity of the \( C_{pl} \) factor in the Van der Meer formula (dotted line) and the test results of Van der Meer for homogenous structures (black stars).](image)

**Figure 32:** Sensitivity of the \( C_{pl} \) factor in the Van der Meer formula (dotted line) and the test results of Van der Meer for homogenous structures (black stars)

Second point concerning the Van der Meer formula is the validity for slope in the range of 1:5 and 1:6. Van der Meer has conducted 26 tests for a 1:6 slope with only an impermeable core of which only 6 tests have a damage level between 1 and 3. Yet, the point of interest for this theses is in homogeneous structures and Van der Meer only conducted tests with a homogeneous structure for slopes of 1:2. This means that points found by Van der Meer correspond to much higher Iribarren numbers than those which were investigated in this thesis. This is illustrated in Figure 32. The black stars show the point found by Van der Meer for a homogeneous structure.

On the basis of relations between the 1:2 and the 1:6 slope, the formula is extrapolated to the lower Iribarren numbers for homogeneous structures. This extrapolation may lead to more uncertainties for slopes milder than 1:2. This is especially true for very mild slope, milder than 1:5. A point of interest is if the permeability is influenced by the slope angle or Iribarren number, which is the case for surging breakers. More research is needed to investigate this hypothesis. It cannot be excluded that uncertainties for the homogeneous mild slopes can explain the differences between the numerical model and the Van der Meer formula.
Sensitivity of XBeach-G

Different parameters, in XBeach-G, can influence the amount of damage and thereby the stability number. The used transport formula of Nielsen, see Appendix A, makes use of two calibration factors namely the sediment friction factor, $f_s$, and the phase lag angle, $\varphi$. The sensitivity of both parameters was already conducted by McCall (2015) and are described separately.

The friction factor ($f_s$) can influence the outcome of the amount of damage significantly, as can be seen in Figure 33. Research has been done to determine realistic values for gravel. McCall (2015) recommends to use values between 0.005 and 0.025. In this thesis already the 0.025 value is used which results into the highest sediment transport rates and is thus a conservative value. Although this value can increase to 0.1, which was found for a test with a very shallow foreshore in combination with sand, this value is not taken into account.

![Figure 33: Sensitivity to friction factor $f_s$, (McCall, 2015)](image)

The phase lag angle ($\varphi$) influences the friction velocity ($u_*$) by increasing or decreasing the acceleration term. Research has been done to obtain the value for this coefficient, but only for sand. Nielsen (2006) found that for sand the optimal value is $\varphi=51^\circ$ and that for coarser sand, $\varphi$ decreased significantly. However, this is contrary to the expectation in which the acceleration effects should become larger for larger sediment. McCall (2015) recommends to use a value between 20 and 35 degrees for gravel. During the numerical experiment a value of 25 degrees is used. By increasing this value, the sediment transport will increase as well, as shown in Figure 34.
To determine values for the two coefficients, calibrations can be done by modelling the tests conducted by Van der in XBeach-G. Difficulties are that Van der Meer only conducted tests for an impermeable layer for the 1:6 slope, this should be taken into account when calibrating the coefficients. Another way of calibrating the coefficient is by using a different transport formula in XBeach-G to compare the outcomes with.

In comparison, the differences found between the Van der Meer formula and the outcomes of XBeach-G can have different reasons. First, the uncertainty of the Van der Meer formula is probably larger than presently is assumed for the milder slopes (1:6) in combination with a homogenous structure. Second, the calibration factors of the Nielsen formula in XBeach-G influence the outcome significantly. Therefore more research is needed to quantitatively explain the large deviations between the results found by Van der Meer and XBeach-G within the validity region of Van der Meer.

### 5.2 Dynamically stable methods

The comparison for dynamically stable slopes is based on the method of Van der Meer (1992). This method is included in the design tool called BREAKWAT and is used to calculate the structural response to one wave condition which was also investigated in the conducted numerical experiment. This is done for the 1:5 and 1:10 slope and for the three different stone sizes. The structural response is calculated by a set of equations described in Van der Meer (1992). The response profile can be described by four parameters and is shown in Figure 35. The results obtained from BREAKWAT can be seen in Figure 35 in green, red shows the results obtained by XBeach-G and the dotted line indicated the initial profile. First the 1:5 slope is shown for the different stone sizes, next the 1:10 slope is shown also for the different stone sizes.
Chapter 5 - Comparison numerical results with current design methods

Figure 35: Comparison results XBeach-G and Van der Meer formula for dynamic profiles
The first noticeable difference is that for the BREAKWAT results for every stone size a same type of profile occurs, namely the crest profile. In XBeach-G this only occurs for the larger stone sizes or for the mild slopes. Even for the smallest stone size of $D_{50}=0.01$ m the sediment is transported upwards. Although it is possible to create a bar profile with the equations used to calculate the response profile, this only occurs for very steep initial profiles in the order of 1:1.5 – 1:3 as is shown in Figure 36.

![Figure 36: Influence of the initial slope (Van der Meer, 1988)](image)

The second aspects that differs from the outcomes of the numerical results is the response angle of the crest that is been built. The found response angles are for all investigated stone sizes much higher than the angles found in the numerical model. Sometimes even larger than the angle of internal friction of 35 degrees (equal to $\tan \alpha = 0.7$), which is not stable and should collapse. The angle of response does become smaller for decreasing stone sizes, only the values are much higher than the values found with XBeach-G and also higher than values found on natural gravel beaches. The found response angles are given in Table 13.

<table>
<thead>
<tr>
<th>Response angle (tan $\alpha$)</th>
<th>$D_{50}=0.1$ m</th>
<th>$D_{50}=0.05$ m</th>
<th>$D_{50}=0.01$ m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope 1:5</td>
<td>0.7</td>
<td>0.6</td>
<td>0.4</td>
</tr>
<tr>
<td>Slope 1:10</td>
<td>0.8</td>
<td>0.7</td>
<td>0.4</td>
</tr>
</tbody>
</table>

The results for the dynamically stable structures differ a lot between the numerical results and the BREAKWAT results, not only the amount of damage differs significantly, also the type of profile is sometimes totally different. This is probably because the method is not applicable to gentle slopes. Although it is stated that the formulas for dynamically stable structure are applicable to every slope angle, they only have been tested on slopes till 1:5 for the method which is used in BREAKWAT (Van der Meer, 1992). Yet, this still cannot clarify the differences found for the results on a slope of 1:5.

Loman et al. (2010) already compared physical tests results with the empirical formula of Van der Meer (1988) for the gravel beach constructed on the Maasvlakte 2. They concluded that the measured maximum eroded depth and also the crest berm height were both considerably less than calculated with the formula of Van der Meer. This is in line with the findings of the conducted numerical tests with XBeach-G.
The objective of this MSc Thesis is the establishment of a method which can be used to determine the stability of gravel on a mild slope in breaking waves. In order to accomplish this objective, research questions were identified which are of importance for the design method for an armour layer on a mild slope. On the basis of the research questions a numerical experiment was set up, from which conclusions were drawn and answers to the research questions were provided.

This chapter presents conclusions based on the results, drawn from the conducted numerical experiment. Furthermore, answers to the research questions are given and recommendations for further research are provided.

6.1 Conclusions

Several research questions were identified. The questions raised in section 1.2 are repeated in this chapter and are answered separately.

1. What method (scale model tests, numerical model or extension of existing empirical design methods) can be used to establish an accurate way to determine the stability of gravel on a mild slope in breaking waves?

It was found that no theoretical or empirical method exists which is directly applicable to design a stone protection for mild slopes (milder than 1:10). In addition, it was found that an armour layer can also be designed as a dynamically stable structure, in which some damage is allowed. To be able to compare results, both types of structures, dynamically and statically stable, were investigated. As stated in Chapter 1, a numerical model is preferred over a physical model, because of its efficiency. XBeach-G has proven to provide reliable results for gravel beaches with slopes of 1:5 – 1:10. Therefore XBeach-G has found to be an adequate method to design a stone protection on a mild slope (milder than 1:5) for both statically and dynamically stable structures.

2. What is a suitable model to design a statically stable structure with gravel for mild slopes with shallow foreshores?

The model XBeach-G is developed to predict morphological changes during extreme events for gravel beaches. The sediment transport is calculated with a sediment transport formula of Nielsen (2002) using the critical Shields value. It is thereby possible to model statically stable structures for which the critical Shields value is only exceeded at some places resulting in a low eroded area. This eroded area is also described by Van der Meer, using a non-dimensional damage level, S, being the ratio between the eroded area and the nominal stone diameter. It is possible with XBeach-G to obtain stone protections with low transport rates, or damage levels close to the start of damage (e.g. S=2) which can be denoted as statically stable structures.

XBeach-G is validated for stone sizes with a diameter up to 0.08 m and therefore suitable to calculate the stability of stones with the size of gravel. Even though it is not possible to model stones larger than gravel in XBeach-G, for protections on mild slopes in the Netherlands the stones sizes which can be calculated are proven to be of a sufficient size to reach stability (static or dynamic).
The used model is capable of modelling mild slopes (milder than 1:6) and shallow foreshores. Gravel beaches with slopes between 1:5 and 1:10 were validated for XBeach-G, milder slopes can be modelled, provided that the limits concerning the wave transformation are not exceeded. The same is true for shallow foreshores.

The challenge of using XBeach-G to design a protection of gravel is that it is an iterative and slow process. It is not possible to set a certain damage level after which the model can calculate the required stone size. Different stone sizes have to be modelled to find the stone size which corresponds to the required damage level. Despite this limitation, the use XBeach-G is found to be an adequate method for designing a statically stable protection for mild slopes in breaking waves.

3. Which (different) parameters are of importance for the design of a stone protection on mild slopes, by comparing XBeach-G with current literature?

The answers for this are twofold, one concerning the statically stable structures and one for the dynamically stable structures and are given separately.

**Statically stable structures**

The conducted numerical experiment confirms trends and relations found in current literature. The following parameters were investigated: wave height, wave steepness, storm duration, (initial) slope angle, stone diameter, hydraulic conductivity and the Iribarren number. Furthermore no different parameters were found which influence the stability of statically stable structures.

However, when comparing the outcomes of XBeach-G and of the current used method of Van der Meer (1988) for plunging waves, significant differences were found, see Figure 37. The Van der Meer formula for plunging waves is valid till a slope of 1:6, which corresponds to a Iribarren number with a minimum of 0.7 (see region 2 and 3 in Figure 37). The conducted numerical experiment partly coincides with this validity region, but most part is outside this region. In order to compare all results, the Van der Meer formula is extrapolated outside its validity region (area 3). The comparison shows that the numerical results follow the line of the extrapolated part of the Van der Meer formula, only slightly higher. All numerical results lay above the (extrapolated) line of Van der Meer formula, which means that lower stone diameters are found to be stable even in the applicable range of Van der Meer.

Two possible reasons which may explain the differences are found. Firstly, by investigating the derivation of the Van der Meer formula (1988). It was found that for homogeneous structures, which are modelled in XBeach-G, Van der Meer only validated the formula for 1:2 slopes (see points in region 3 in Figure 37). The same is true for permeable structures. This limits the validity of the Van der Meer formula for homogenous structures, especially for the mildest slopes (1:6), for which Van der Meer only conducted tests for impermeable structures. For milder slopes the influence on the permeability may be larger than for steeper slopes because of the larger effective area, resulting in a more stable structure for milder slopes. This is also the case in the Van der Meer formula (1988) for surging waves, in which the permeability is related to the Iribarren number and thereby the slope angle. More research is needed to investigate this hypothesis.
Secondly, in order to predict the amount of transport in XBeach-G, the Nielsen formula (2002) is used. However, results are sensitive to the different calibration factors which influence the calculated amount of damage. The used values for these calibration factors, which are the wave friction factor and the phase lag angle, are considered to be representative for gravel beaches, although their values are still point of discussion. Their sensitivity may influence the sensitivity of the outcomes of XBeach-G and thereby explaining the differences found between XBeach-G and Van der Meer (2002). Therefore, more research is needed to find the exact values for the calibration factors.

Dynamically stable
For dynamically stable profiles one specific characteristic is found which influences the damage significantly, namely the shape of the equilibrium profile. When the equilibrium profile is steeper than the initial profile, sediment is transported upward and vice versa. The difference between the initial profile and the equilibrium profile determines for an important part the amount of damage, in which a larger difference results into an increased damage level. The equilibrium profile is described by the response angle of the equilibrium profile and depends on the stone size and to a small extent to the wave conditions.

The results for the dynamically stable structures differ a lot between the numerical results and the results from current used method for reshaping breakwaters as implemented in BREAKWAT, both the amount and location of damage differs significantly. XBeach-G models less erosion than BREAKWAT for dynamically stable profiles on mild slopes. This is in line with earlier studies e.g. for the cobble beach of Maasvlakte 2.
4. What is the location of the occurring damage and is it possible to optimise gravel size on the foreshore?

To be able to define the location of damage, first the definition of damage was established. XBeach-G calculates damage in case the critical Shields values is exceeded, sometimes resulting in very small amounts of damage (eroded depths in the order of 0.0001). For statically stable structures no minimum eroded depth was defined, in order to compare results with current used methods. For dynamically stable structures a fraction of $D_{50}$ was chosen as a minimum depth.

The area over which damage takes place is relatively large for statically stable structures, but the corresponding eroded depth is relatively small. Only a small amount of sediment is transported, all of which in an upward direction, causing erosion below the still water line. The maximum erosion takes in most of the cases place at a depth equal to $-1H$, perpendicular to the still water line. The location over which damage occurs is roughly between $-0.5H$ and $-1.5H$, relative to the still water line, which is equal to the place where most waves break. Yet, because of limitations of the used transport formula, the precise location of erosion cannot be determined accurately.

For dynamically stable profiles the location of erosion depends mostly on which type of profile occurs, a crest or a bar profile. Crest profiles have erosion below the still water line, from an average depth of $-0.5H$ till an average depth of $-3H$. For the smallest stone sizes these values are larger and are found at larger depths. For bar profiles the erosion takes place around and above the still water line, the location of the erosion is takes place between $+1.5H$ and $-1.5H$.

The findings from the numerical experiment show that it is possible to optimise the gravel sizes along a mild slope on a foreshore. Depending on the equilibrium profile, larger diameters for gravel are needed below or above the still water line. Yet, because of tides and possible storm surges, the water level is not constant during a storm. This will result into a larger actual area over which the damage will occur than tested in this thesis. In case a very long mild slope needs to be protected, it can therefore be economical to apply different gravel sizes, although physical tests need to be conducted in order to validate these numerical findings.

5. Is a statically stable structure for a protection on a mild slopes the most economical solution?

For statically stable structures, it is required that the damage, and thus the morphological changes, stays within low transport ranges. For steep slopes this strict level is equal to a damage level of $S=2-3$. In this thesis it was found that the allowed damage level related to the start of damage criterion may be increased for milder slopes, since the area over which wave load can act is larger, more stones can be moved in order to maintain the same safety level. The damage level was linked to the length over which damage takes places, which becomes larger for milder slopes. How large this increase of the damage level may be depends also on the desired safety level which can be set by the client. Depending on the required safety, a start value is set and the formula given in section 4.1.1, can be used to calculate the corresponding damage levels for the milder slopes.

A higher allowed damage level already result into smaller stone sizes, even smaller stone sizes are found for dynamically stable protections, wherein even more damage is allowed. Yet, a stone protection cannot be fully dynamically stable, because the damage should always stay within certain
limits to prevent erosion of the under-layer. The damage description with the damage level is not representative anymore for these structures, because the S-value becomes large (order 1000 or higher) for mild slopes in combination small gravel diameters and with a relative small eroded area. Therefore, a different damage parameter must be used. By using the eroded depth (\(d_e\)) or the cover depth (\(d_c\)) it can be guaranteed that the under-layer is always covered sufficiently. In case it is possible to design a stone protection which fulfils this requirement, it is more economical to use a dynamically stable structure.

Ultimately, based on the answers given at the research questions, an answer can be formulated to this thesis’ objective, which was to establish a method which can determine the stability of gravel on a mild slope in breaking waves more accurately to lower investment costs. It was found that XBeach-G can perform the key functions to model the stability of gravel on a mild slope in breaking waves, both for statically and dynamically stable profiles. Currently, the formula of Van der Meer for plunging breakers (1988) is often used, and was found to be rather conservative for designing a statically stable structure for mild slopes. The extrapolated Van der Meer formula does however give a better first estimate, but XBeach-G shows that even smaller stone diameters provide enough stability. Besides, with a higher allowed damage level for mild slopes, even smaller stone sizes provide adequate protection. However, for smaller stone sizes the damage level S is not representative anymore, resulting into high damage levels for small eroded areas. Therefore a new description of erosion should be adapted for stone protections on mild slopes. Furthermore it is found that for mild slopes a dynamically stable profile is more economical, provided that the damage stays between certain limits which can be described in the form of a maximum eroded depth.

6.2 Recommendations

The items listed below include possible improvements of the results and potential subjects for further research.

a. All tests were only calculated with one transport formula, the Nielsen (2002) formula. It is recommended to investigate the differences between the Nielsen formula and the Van Rijn formula in XBeach-G.

b. Furthermore it is recommended that the essential cases and outcomes are tested and validated by physical model tests. Essential cases are cases with a 1:5 slope, which coincide with the region of validity of Van der Meer. The 1:10 slopes are also of importance for which a transition in type of profile occurs, from bar to crest profile.

c. For dynamically stable profiles it was found that the damage depends on the equilibrium profile of the stones in combination with the wave conditions. It must be kept in mind that in nature the forcing is absolutely not constant and therefore a stable equilibrium profile is never reached, however, the profile still remains between a steady envelope and is called the dynamic equilibrium profile (Bosboom & Stive, 2013). More research has to be done to investigate the influence of varying wave conditions and water levels, for example induced by the tide or storm surges.

d. It must also be investigated how the profile, obtained after the storm, changes during normal wave conditions.
e. No longshore transport of sediment was taken into account. More research can be carried out to investigate the influence of longshore transport by currents and waves.

f. In this thesis only a homogeneous layer of gravel is investigated. For armour layers an essential aspect is the permeability of the under-layer which needs to be investigated.

g. Throughout this thesis no minimum level to the eroded depth was given for statically stable profiles leading to relative high eroded area’s compared to the eroded depth. This problem occurred because the accuracy of XBeach-G is much higher than measurement techniques used by Van der Meer, which leads to very small eroded depths which are taken into account by calculating the eroded area. Although these eroded depths are very small, the sum still is relative large, therefore a lot of tests were not categorized as statically stable. It might be better to set a minimum eroded depth in order to calculate the damage level, for example the D_{15}.

h. Regarding the description of the eroded depth still some option are possible. For large stones this is normally done in terms of the D_{50}. However, for very small gravel with a diameter of D_{50}=0.01 m this is not realistic anymore. Firstly, because the accuracy of the construction is much lower than only 0.01 m, especially when constructing below the water level. Secondly, for a construction thickness which is mostly in the order of 0.2 - 0.3 m an eroded depth of 2*D_{50} is disproportion for very small stone sizes. Therefore it is suggested to couple the maximum eroded depth to the construction depth instead of the stone size. For instance for a 0.3 m thick layer it may be allowed to have an erosion depth equal to half the layer thickness. This leads to the following question:

i. Are the safety requirements fulfilled when instead of connecting the eroded depth to stone size the eroded depth is connected to the construction layer thickness?

j. Another method to describe damage can be a normalized damage parameter S_{norm}, which can be defined as the eroded area divided by the stone diameter times the length over which erosion takes place S_{norm}= A_{eroded}/(D_{50}*L_{erosion}). In this way the length is taken into account, which is key for mild slopes. However, with the normalized damage level it is not guaranteed that the maximum eroded depth is lower than 1*D_{50} which is essential for a statically stable, two-layer stone protection. More research is needed to take this into account.
References


Stability of Gravel on Mild Slopes in Breaking Waves


Appendices

Appendix A: Model description XBeach-G

The three important modifications when comparing XBeach-G with XBeach are described in this appendix. These modifications are involved in the wave equations, a groundwater model and sediment transport relations.

Wave equations

XBeach-G simulates a non-hydrostatic flow, similar to the one-layer version of the SWASH model, in order to solve not only long waves but also wave-by-wave flow which are of importance for gravel slopes (Masselink, et al., 2014). To determine the depth-average flow the non-linear shallow water equations are used combined with a term which corrects for the non-hydrostatic pressure and groundwater exchange which is shown in equations 14 & 15 (Masselink, et al., 2014).

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial h u}{\partial x} + S = 0 \\
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} - \frac{\partial}{\partial x} \left( \nu_h \frac{\partial u}{\partial x} \right) = - \frac{1}{\rho} \frac{\partial (\rho \bar{q} + \rho g \zeta)}{\partial x} - \frac{\tau_b}{\rho h}
\]

\[
\begin{align*}
\zeta & = \text{Free surface elevation above an arbitrary horizontal plane} \\
u & = \text{Depth-average cross-shore velocity} \\
h & = \text{Total water depth [m]} \\
S & = \text{Surface water-groundwater exchange flow} \\
\nu_h & = \text{Horizontal viscosity} \\
\bar{q} & = \text{Depth-average dynamic pressure} \\
\tau_b & = \text{Bed shear stress} \\
x & = \text{Spatial, horizontal coordinate} \\
t & = \text{Temporal coordinate}
\end{align*}
\]

Groundwater model

The second main difference is the added groundwater model. Infiltration and exfiltration do not play an important role for sandy beaches, but for gravel beaches it does because the hydraulic conductivity of gravel beaches is much higher than that of sandy beaches. These processes together with the groundwater dynamics have a significant influence on predicting the morphology. To account for these effects McCall et al. (2012) developed a groundwater model using the law of Darcy and corrections for turbulent water flow, the formulae are as follows:

\[
\frac{\partial h_{gw} u_{gw}}{\partial x} + w_{gw,s} = 0
\]

\[
u_{gw} = -k \frac{\partial \bar{H}}{\partial x}
\]

\[
\begin{align*}
u_{gw} & = \text{depth average horizontal groundwater velocity} \\
h_{gw} & = \text{Height of the groundwater surface above the bottom of the aquifer} \\
w_{gw,s} & = \text{Vertical groundwater velocity at the groundwater surface} \\
k & = \text{Hydraulic conductivity} \\
\bar{H} & = \text{Hydraulic head, depth averaged}
\end{align*}
\]
The correction for the turbulent water flow is down by correcting the value of $k$, which decreases as the flow becomes more turbulent. The used formula can be found in (McCall, et al., 2012).

The last modifications are concerning the sediment transport relations which are different for gravel than for sand. XBeach-G uses the bed shear stress, $\tau_{bd}$, in order to describe the stability of gravel. The shear stress is used to calculate the Shields parameter for the sediment transport. The bed shear stress, due to drag, is calculated with the following equation:

$$\tau_{bd} = c_f \frac{u|u|}{h}$$

$$c_f = \frac{g}{(18 \log \left(\frac{18h}{k}\right))^2}$$

The dimensionless friction factor is calculated by using equation (24). The characteristic roughness height $k$, it is assumed to be equal to $3D_{90}$ (Van Rijn, 1982).

**Sediment transport**

It is assumed that for the transport of gravel only occurs via bed load transport and sheet flow transport. Suspended transport does not take place, because of the size of the gravel. The Nielsen (2002) formula for bed load transport is used, which is a modification of the Meyer-Peter and Müller (1948) formula for sediment transport.

$$q_s = 12(\theta - 0.05)\sqrt{\frac{\rho_s - \rho}{\rho}} \sqrt{\frac{g D_{50}^3}{\rho}}$$

$q_s$ = Volumetric sediment rate [m$^2$/s]

$\theta$ = Adjusted Shields parameter [-]

The Shields parameter is modified with a correction for the bed slope effect according to Fredsøe and Deigaard (1992) by using the following formula:

$$\theta = \frac{u^2}{\rho_s - \rho} \frac{g D_{50}}{\rho} * \cos\beta (1 + \frac{\tan \beta}{\tan \phi})$$

$\beta$ = Angle of the bed [-]

$\phi$ = Angle of response of the sediment [-]

In order to make this formula applicable to multiple situations, calibration factors are added which can improve the accuracy of the model when data is available. The following formula, derived by Nielsen (2002), takes into account the expansion and contraction of the boundary layer, together with effects created by pressure gradients and turbulent fronts (Masselink, et al., 2014).

$$u_* = \sqrt{\frac{f_s}{2}} \left( \cos(\varphi) \ast u + \frac{T_{m-1.0}}{2\pi} \sin(\varphi) \frac{\partial u}{\partial t} \right)$$

$f_s$ = Friction factor

$T_{m-1.0}$ = Offshore spectral period [s]

$\varphi$ = Phase lag angle [-]
The friction factor is a function of the sediment and can vary between 0.005 and 0.025. The value of 0.025 is used, which gives the best results according to the validation data used for different storms (McCall, 2015). The phase lag angle represents the phase lag between the boundary layer velocity and the free stream velocity, resulting from an approximation of the bed boundary layer in the swash zone (Nielsen, 2002). For gravel it is estimated to be in between 20 -30 °, the value of 25° is used.

As a last step the bed level change is calculated using the calculated bed load transport in the following formula:

\[
\frac{\partial \xi}{\partial t} + \frac{1}{(1 - n)} \frac{\partial q_b}{\partial x} = 0
\]

(28)

\(\xi\) = Elevation of the bed  
\(n\) = Porosity

Finally it has to be checked whether the calculated bed level is stable or if it will collapse by comparing it with the angle of response of the sediment.
Appendix B: Test results

This appendix shows all the profile changes for all wave conditions conducted during the numerical experiment. The different wave conditions are named A till F and in Table 14 the corresponding wave conditions can be found. This is also the order in which the profiles are presented in this appendix.

*Table 14: Investigated wave conditions*

<table>
<thead>
<tr>
<th></th>
<th>(H_s) (m)</th>
<th>(T_p) (s)</th>
<th>(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1</td>
<td>8.0</td>
<td>0.01</td>
</tr>
<tr>
<td>B</td>
<td>2</td>
<td>11.3</td>
<td>0.01</td>
</tr>
<tr>
<td>C</td>
<td>1</td>
<td>4.6</td>
<td>0.03</td>
</tr>
<tr>
<td>D</td>
<td>2</td>
<td>6.3</td>
<td>0.03</td>
</tr>
<tr>
<td>E</td>
<td>1</td>
<td>3.6</td>
<td>0.05</td>
</tr>
<tr>
<td>F</td>
<td>2</td>
<td>5.1</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Per wave condition the slope and stone diameter is varied and are presented in the following way shown in Table 15. On the horizontal the profiles from one slope can be found and in the vertical the profiles of one stone diameter can be found. In this way the profiles can be compared easily.

*Table 15: The order of presenting the different slopes and stone diameter throughout this appendix*

<table>
<thead>
<tr>
<th></th>
<th>(D_{50}=0.1) m</th>
<th>(D_{50}=0.05) m</th>
<th>(D_{50}=0.01) m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope 1:5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope 1:10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope 1:25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope 1:50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In each figure is the still water line indicated with blue, the initial profile with a dotted line and the changed profile in red.
Wave condition A

$H_s = 1 \text{ m}$
$T_p = 8.0 \text{ s}$
$s = 0.01$

### Slope 1:5

- Slope 1.5 $D_50=0.1 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.5 $D_50=0.05 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.5 $D_50=0.01 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$

### Slope 1:10

- Slope 1.10 $D_50=0.1 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.10 $D_50=0.05 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.10 $D_50=0.01 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$

### Slope 1:25

- Slope 1.25 $D_50=0.1 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.25 $D_50=0.05 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.25 $D_50=0.01 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$

### Slope 1:50

- Slope 1.50 $D_50=0.1 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.50 $D_50=0.05 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
- Slope 1.50 $D_50=0.01 \text{ m} \times 18000 \text{ s} H_s=1 \text{ m} s=0.01$
Wave condition B

$H_s = 2\, \text{m}$

$T_p = 11.3\, \text{s}$

$s = 0.01$

**Slope 1:5**

**Slope 1:10**

**Slope 1:25**

**Slope 1:50**
Appendix B – Test results

Wave condition C

\[ H_s = 1 \text{ m} \]
\[ T_p = 5.1 \text{ s} \]
\[ s = 0.03 \]

Slope 1:5

Slope 1:5 \( D_0 = 0.1 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:5 \( D_0 = 0.05 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:5 \( D_0 = 0.01 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:10

Slope 1:10 \( D_0 = 0.1 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:10 \( D_0 = 0.05 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:10 \( D_0 = 0.01 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:25

Slope 1:25 \( D_0 = 0.1 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:25 \( D_0 = 0.05 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:25 \( D_0 = 0.01 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:50

Slope 1:50 \( D_0 = 0.1 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:50 \( D_0 = 0.05 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)

Slope 1:50 \( D_0 = 0.01 \text{ m} \) \( t = 18000 \text{ s} \) \( H_s = 1 \text{ m} \) \( s = 0.03 \)
Wave condition D

$H_s = 2m$
$T_p = 6.3s$
$s = 0.03$

Slope 1:5

Slope 1:10

Slope 1:25

Slope 1:50
Wave condition E

$H_s=1m$
$T_p=3.6s$
$s=0.05$

Slope 1:5

Slope 1:10

Slope 1:25

Slope 1:50
Appendix B – Test results

Wave condition F

$H_s = 2\text{m}$

$T_p = 5.1 \text{ s}$

$s = 0.05$

Slope 1:5

Slope 1:10

Slope 1:25

Slope 1:50
### Appendix C: Damage overview

For every wave conditions an overview is given of all results obtained by the numerical experiment. First the lowest wave steepness’s are presented (wave conditions A&B) on the next page the intermediate wave steepness is found (wave conditions C&D) an at the last page the results which correspond to the highest wave steepness are shown (wave conditions E&F).

First the parameters concerning the damage are presented for each slope and stone size. Next the dimensionless parameters are shown and finally the location of the damage is described.

#### Table 16: Damage overview for wave conditions A

<table>
<thead>
<tr>
<th>Wave condition A: Hs = 1m, Tp = 8s, s=0.01</th>
<th>Damage/profile parameters and their dimensionless form</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stone diameter [m]:</td>
<td>1:5</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>0.05</td>
</tr>
<tr>
<td>A_e</td>
<td>Eroded Area [m²]</td>
<td>1.33</td>
</tr>
<tr>
<td>S</td>
<td>Damage level [-]</td>
<td>133</td>
</tr>
<tr>
<td>d_e</td>
<td>Maximum depth [m]</td>
<td>0.24</td>
</tr>
<tr>
<td>d_e/D_m</td>
<td>Dimensionless maximum depth [-]</td>
<td>2.4</td>
</tr>
<tr>
<td>tan α</td>
<td>Maximum slope [-]</td>
<td>0.32</td>
</tr>
<tr>
<td>type</td>
<td>Profile type (crest/bar/no)</td>
<td>crest</td>
</tr>
</tbody>
</table>

#### Dimensionless parameters

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ξ</td>
<td>6.06</td>
</tr>
<tr>
<td>s</td>
<td>2.0</td>
</tr>
</tbody>
</table>

#### Location damage, relative and perpendicular to the still water level. Damage is defined as $0.5DL$

<table>
<thead>
<tr>
<th>Location Maximum depth in fraction H_s [-]</th>
<th>-0.57</th>
<th>0.94</th>
<th>1.4</th>
<th>-0.75</th>
<th>-0.81</th>
<th>0.68</th>
<th>-0.59</th>
<th>-1.02</th>
<th>-</th>
<th>-</th>
<th>-0.99</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper limit [m]</td>
<td>-1.32</td>
<td>-0.99</td>
<td>-0.19</td>
<td>0.36</td>
<td>1.63</td>
<td>0.03</td>
<td>-1.31</td>
<td>-1.25</td>
<td>-1.54</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

#### Table 17: Damage overview for wave conditions B

<table>
<thead>
<tr>
<th>Wave condition B: Hs = 2m, Tp = 11.3s, s=0.01</th>
<th>Damage/profile parameters and their dimensionless form</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stone diameter [m]:</td>
<td>1:5</td>
</tr>
<tr>
<td></td>
<td>0.1</td>
<td>0.05</td>
</tr>
<tr>
<td>A_e</td>
<td>Eroded Area [m²]</td>
<td>13.70</td>
</tr>
<tr>
<td>S</td>
<td>Damage level [-]</td>
<td>1370</td>
</tr>
<tr>
<td>d_e</td>
<td>Maximum depth [m]</td>
<td>0.95</td>
</tr>
<tr>
<td>d_e/D_m</td>
<td>Dimensionless maximum depth [-]</td>
<td>9.5</td>
</tr>
<tr>
<td>tan α</td>
<td>Maximum slope [-]</td>
<td>0.39</td>
</tr>
<tr>
<td>type</td>
<td>Profile type (crest/bar/no)</td>
<td>crest</td>
</tr>
</tbody>
</table>

#### Dimensionless parameters

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>ξ</td>
<td>12.12</td>
</tr>
<tr>
<td>s</td>
<td>2.0</td>
</tr>
</tbody>
</table>

#### Location damage, relative and perpendicular to the still water level. Damage is defined as $0.5DL$

<table>
<thead>
<tr>
<th>Location Maximum depth in fraction H_s [-]</th>
<th>-0.77</th>
<th>2.59</th>
<th>3.91</th>
<th>-1.19</th>
<th>-0.88</th>
<th>1.29</th>
<th>-1.07</th>
<th>-1.03</th>
<th>-1.91</th>
<th>-</th>
<th>-1.71</th>
<th>-1.62</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper limit [m]</td>
<td>-0.385</td>
<td>1.295</td>
<td>1.955</td>
<td>-0.595</td>
<td>-0.44</td>
<td>0.645</td>
<td>-0.535</td>
<td>-0.515</td>
<td>-0.955</td>
<td>-</td>
<td>-0.855</td>
<td>-0.81</td>
</tr>
<tr>
<td>Lower limit [m]</td>
<td>-3.44</td>
<td>-2.75</td>
<td>-1.92</td>
<td>-5.56</td>
<td>-5.35</td>
<td>-6.11</td>
<td>-3.93</td>
<td>-5.57</td>
<td>-9.6</td>
<td>-</td>
<td>-4.7</td>
<td>-10</td>
</tr>
<tr>
<td>Lower limit in fraction H_s [-]</td>
<td>-1.72</td>
<td>-1.375</td>
<td>-0.96</td>
<td>-2.78</td>
<td>-2.675</td>
<td>-3.055</td>
<td>-1.965</td>
<td>-2.785</td>
<td>-4.8</td>
<td>-</td>
<td>-2.35</td>
<td>-5</td>
</tr>
<tr>
<td>Location Maximum depth [m]</td>
<td>-2.52</td>
<td>-1.48</td>
<td>-0.3</td>
<td>-3.45</td>
<td>-2.45</td>
<td>-0.02</td>
<td>-1.54</td>
<td>1.93</td>
<td>-3.02</td>
<td>-</td>
<td>2.34</td>
<td>-2.56</td>
</tr>
<tr>
<td>Location d_e in fraction H_s [-]</td>
<td>-1.26</td>
<td>-0.74</td>
<td>-0.15</td>
<td>-1.725</td>
<td>-1.225</td>
<td>-0.01</td>
<td>-0.77</td>
<td>-0.965</td>
<td>-1.51</td>
<td>-</td>
<td>-1.17</td>
<td>-1.28</td>
</tr>
</tbody>
</table>
### Table 18: Damage overview for wave conditions C

<table>
<thead>
<tr>
<th>Wave condition C: ( H_s = 2m, \ T_p = 6.3s, s=0.03 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Slope:</strong></td>
</tr>
<tr>
<td><strong>Stone diameter [m]</strong></td>
</tr>
<tr>
<td>0.1 0.05 0.01</td>
</tr>
<tr>
<td><strong>Damage/profile parameters and their dimensionless form</strong></td>
</tr>
<tr>
<td>( A_e ) Eroded Area ([m^2])</td>
</tr>
<tr>
<td>0.05 0.98 4.55</td>
</tr>
<tr>
<td>( S ) Damage level [-]</td>
</tr>
<tr>
<td>5 390 45470</td>
</tr>
<tr>
<td>( d_s ) Maximum depth ([m])</td>
</tr>
<tr>
<td>0.05 -0.93 -4.54</td>
</tr>
<tr>
<td>( d_s/D_{10} ) Dimensionless maximum depth [-]</td>
</tr>
<tr>
<td>0.46 -18.52 -53.70</td>
</tr>
<tr>
<td>( \tan \alpha ) Maximum slope [-]</td>
</tr>
<tr>
<td>0.23 - - -</td>
</tr>
<tr>
<td><strong>Type</strong> Profile type (crest/bar/no)</td>
</tr>
<tr>
<td>crest bar bar no crest no crest</td>
</tr>
<tr>
<td><strong>Dimensionless parameters</strong></td>
</tr>
<tr>
<td>( H_s/\Delta D_{10} ) Stability number [-]</td>
</tr>
<tr>
<td>6.06 12.12 60.06</td>
</tr>
<tr>
<td>( \xi ) Iribarren number [-]</td>
</tr>
<tr>
<td>1.15 1.15 1.15</td>
</tr>
<tr>
<td><em><em>Location damage, relative and perpendicular to the still water level. Damage is defined as 0.5</em>( D_{10} )</em>*</td>
</tr>
<tr>
<td><strong>Upperlimit [m]</strong></td>
</tr>
<tr>
<td>- 0.29 1.26</td>
</tr>
<tr>
<td><strong>Upperlimit in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>- 0.29 1.26</td>
</tr>
<tr>
<td><strong>Lower limit [m]</strong></td>
</tr>
<tr>
<td>- -0.61 -0.59</td>
</tr>
<tr>
<td><strong>Lowerlimit in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>- -0.61 -0.59</td>
</tr>
<tr>
<td><strong>Location maximum depth [m]</strong></td>
</tr>
<tr>
<td>-1.2 -0.26 -0.05</td>
</tr>
<tr>
<td><strong>Location ( d_s ) in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>-1.2 -0.26 -0.05</td>
</tr>
</tbody>
</table>

### Table 19: Damage overview for wave conditions D

<table>
<thead>
<tr>
<th>Wave condition D: ( H_s = 2m, \ T_p = 6.3s, s=0.03 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Slope:</strong></td>
</tr>
<tr>
<td><strong>Stone diameter [m]</strong></td>
</tr>
<tr>
<td>0.1 0.05 0.01</td>
</tr>
<tr>
<td><strong>Damage/profile parameters and their dimensionless form</strong></td>
</tr>
<tr>
<td>( A_e ) Eroded Area ([m^2])</td>
</tr>
<tr>
<td>2.47 6.26 15.97</td>
</tr>
<tr>
<td>( S ) Damage level [-]</td>
</tr>
<tr>
<td>247 2505 159680</td>
</tr>
<tr>
<td>( d_s ) Maximum depth ([m])</td>
</tr>
<tr>
<td>0.37 0.71 1.51</td>
</tr>
<tr>
<td>( d_s/D_{10} ) Dimensionless maximum depth [-]</td>
</tr>
<tr>
<td>3.70 14.20 151.00</td>
</tr>
<tr>
<td>( \tan \alpha ) Maximum slope [-]</td>
</tr>
<tr>
<td>0.35 - - -</td>
</tr>
<tr>
<td><strong>Type</strong> Profile type (crest/bar/no)</td>
</tr>
<tr>
<td>crest bar bar no crest no crest</td>
</tr>
<tr>
<td><strong>Dimensionless parameters</strong></td>
</tr>
<tr>
<td>( H_s/\Delta D_{10} ) Stability number [-]</td>
</tr>
<tr>
<td>12.12 24.24 121.21</td>
</tr>
<tr>
<td>( \xi ) Iribarren number [-]</td>
</tr>
<tr>
<td>1.11 1.11 1.11</td>
</tr>
<tr>
<td><em><em>Location damage, relative and perpendicular to the still water level. Damage is defined as 0.5</em>( D_{10} )</em>*</td>
</tr>
<tr>
<td><strong>Upperlimit [m]</strong></td>
</tr>
<tr>
<td>- 1.25 2.11</td>
</tr>
<tr>
<td><strong>Upperlimit in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>- 0.09 0.625</td>
</tr>
<tr>
<td><strong>Lower limit [m]</strong></td>
</tr>
<tr>
<td>-1.61 -1.28 -1.56</td>
</tr>
<tr>
<td><strong>Lowerlimit in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>-0.805 -0.64 -0.78</td>
</tr>
<tr>
<td><strong>Location maximum depth [m]</strong></td>
</tr>
<tr>
<td>-0.91 -0.48 0.16</td>
</tr>
<tr>
<td><strong>Location ( d_s ) in fraction ( H_s ) [-]</strong></td>
</tr>
<tr>
<td>-0.455 -0.24 0.08</td>
</tr>
</tbody>
</table>
### Table 20: Damage overview for wave conditions E

<table>
<thead>
<tr>
<th>Slope: 1:5</th>
<th>1:10</th>
<th>1:25</th>
<th>1:50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone diameter [m]: 0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0.1</td>
</tr>
<tr>
<td><strong>Damage/Profile parameters and their dimensionless form</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_e$</td>
<td>0.0068</td>
<td>0.296</td>
<td>2.587</td>
</tr>
<tr>
<td>$S$</td>
<td>1</td>
<td>118</td>
<td>25870</td>
</tr>
<tr>
<td>$d_e$</td>
<td>0.0043</td>
<td>0.1</td>
<td>0.67</td>
</tr>
<tr>
<td>$d_e/D_{10}$</td>
<td>0.04</td>
<td>2.00</td>
<td>67.00</td>
</tr>
<tr>
<td>$\tan \alpha$</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>type Profile type (crest/bar/no)</td>
<td>crest crest bar bar</td>
<td>crest crest bar bar no no crest</td>
<td>no no crest</td>
</tr>
</tbody>
</table>

**Dimensionless parameters**

<table>
<thead>
<tr>
<th>$H_s/D_{10}$</th>
<th>Stability number [-]</th>
<th>6.06</th>
<th>12.12</th>
<th>24.24</th>
<th>121.21</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi$</td>
<td>Iribarren number [-]</td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
<td>0.45</td>
</tr>
<tr>
<td>Location damage, relative and perpendicular to the still water level. Damage is defined as $0.5 * D_e$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upperlimit [m]</td>
<td>-</td>
<td>0.18</td>
<td>0.51</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Upperlimit in fraction $H_s$ [-]</td>
<td>-</td>
<td>0.18</td>
<td>0.51</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lower limit [m]</td>
<td>-</td>
<td>-0.47</td>
<td>-0.47</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lowerlimit in fraction $H_s$ [-]</td>
<td>-</td>
<td>-0.47</td>
<td>-0.47</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Location maximum depth [m]</td>
<td>-1.03</td>
<td>-0.2</td>
<td>0.01</td>
<td>-1</td>
<td>0.69</td>
</tr>
<tr>
<td>Location $d_e$ in fraction $H_s$ [-]</td>
<td>-1.03</td>
<td>-0.2</td>
<td>0.01</td>
<td>-1</td>
<td>0.69</td>
</tr>
</tbody>
</table>

### Table 21: Damage overview for wave conditions F

<table>
<thead>
<tr>
<th>Slope: 1:5</th>
<th>1:10</th>
<th>1:25</th>
<th>1:50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone diameter [m]: 0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0.1</td>
</tr>
<tr>
<td><strong>Damage/Profile parameters and their dimensionless form</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_e$</td>
<td>0.69</td>
<td>3.99</td>
<td>12.90</td>
</tr>
<tr>
<td>$S$</td>
<td>69</td>
<td>1597</td>
<td>292</td>
</tr>
<tr>
<td>$d_e$</td>
<td>0.14</td>
<td>0.59</td>
<td>1.4</td>
</tr>
<tr>
<td>$d_e/D_{10}$</td>
<td>1.4</td>
<td>11.8</td>
<td>140.0</td>
</tr>
<tr>
<td>$\tan \alpha$</td>
<td>0.27</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>type Profile type (crest/bar/no)</td>
<td>crest crest bar bar</td>
<td>crest crest bar bar no no crest</td>
<td>no no crest</td>
</tr>
</tbody>
</table>

**Dimensionless parameters**

<table>
<thead>
<tr>
<th>$H_s/D_{10}$</th>
<th>Stability number [-]</th>
<th>12.12</th>
<th>24.24</th>
<th>121.21</th>
<th>12.12</th>
<th>24.24</th>
<th>121.21</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi$</td>
<td>Iribarren number [-]</td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>Location damage, relative and perpendicular to the still water level. Damage is defined as $0.5 * D_e$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upperlimit [m]</td>
<td>-0.29</td>
<td>0.8</td>
<td>1.97</td>
<td>-1.06</td>
<td>-1.16</td>
<td>0.95</td>
<td>-</td>
</tr>
<tr>
<td>Upperlimit in fraction $H_s$ [-]</td>
<td>-0.145</td>
<td>0.4</td>
<td>0.985</td>
<td>-0.53</td>
<td>-0.58</td>
<td>0.475</td>
<td>-</td>
</tr>
<tr>
<td>Lower limit [m]</td>
<td>-1.07</td>
<td>-1.02</td>
<td>-1.25</td>
<td>-2.49</td>
<td>-2.87</td>
<td>-0.47</td>
<td>-</td>
</tr>
<tr>
<td>Lowerlimit in fraction $H_s$ [-]</td>
<td>-0.535</td>
<td>-0.51</td>
<td>-0.625</td>
<td>-1.245</td>
<td>-1.435</td>
<td>-0.235</td>
<td>-</td>
</tr>
<tr>
<td>Location maximum depth [m]</td>
<td>-0.85</td>
<td>-0.26</td>
<td>0.17</td>
<td>-1.54</td>
<td>-2.1</td>
<td>0.1</td>
<td>-</td>
</tr>
<tr>
<td>Location $d_e$ in fraction $H_s$ [-]</td>
<td>-0.425</td>
<td>-0.13</td>
<td>0.085</td>
<td>-0.77</td>
<td>-1.05</td>
<td>0.05</td>
<td>-0.045</td>
</tr>
</tbody>
</table>