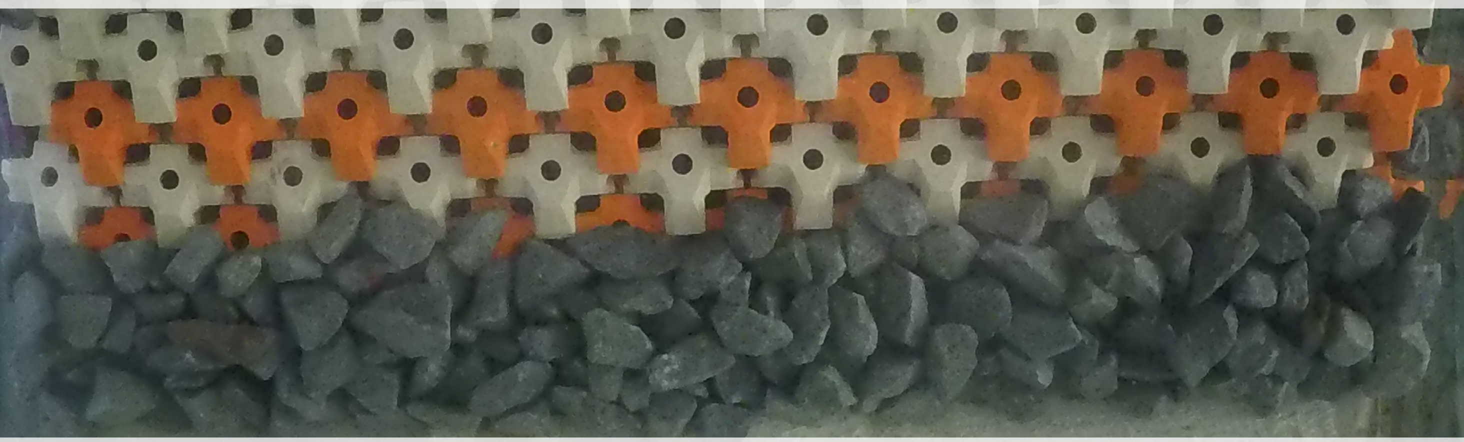
Stability of the first row of an XblocPlus armour layer



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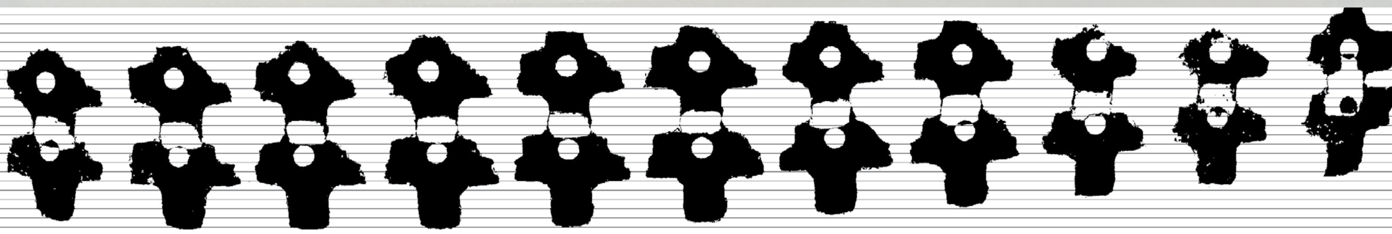

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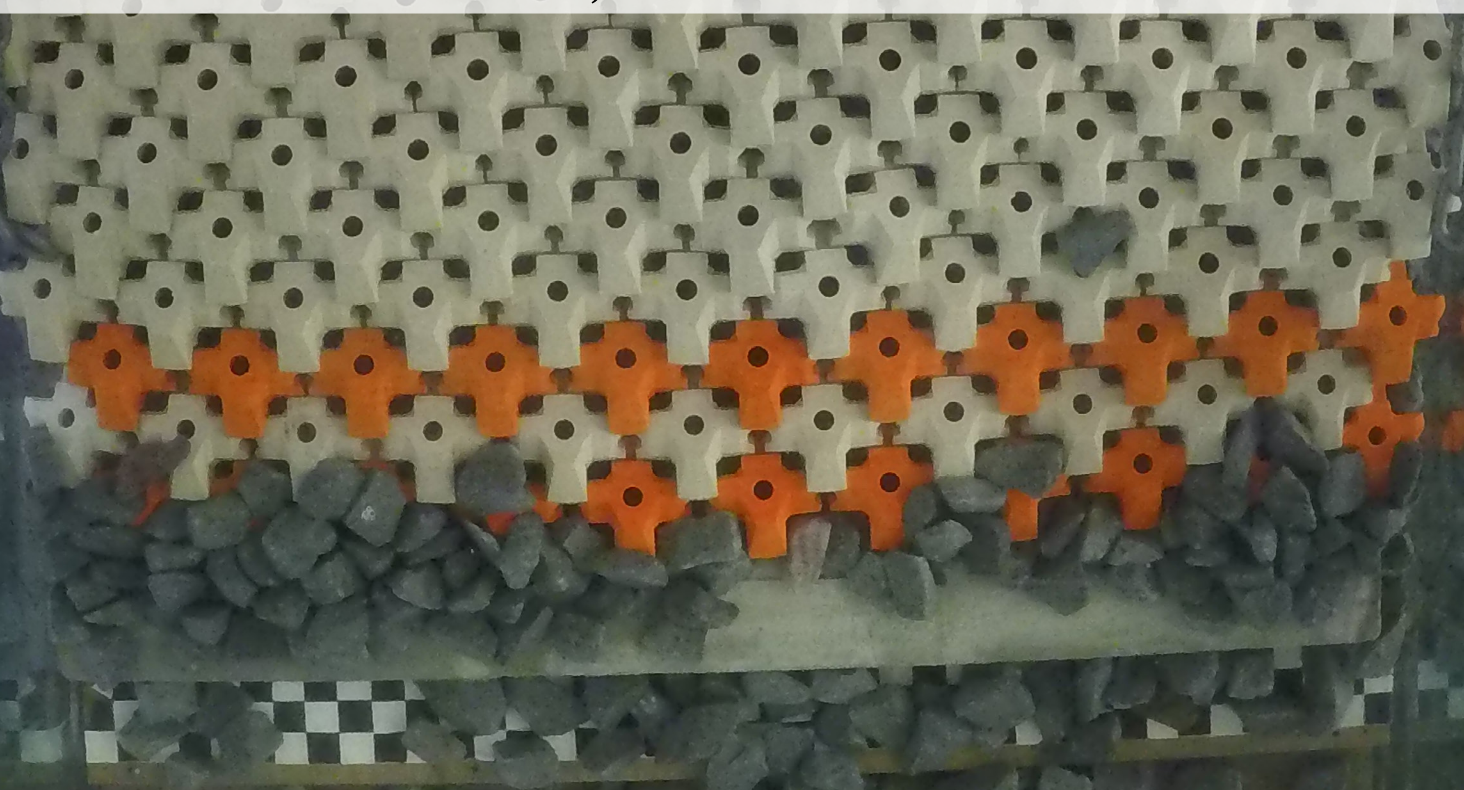

STABILITY OF THE FIRST ROW OF AN XBLOCPLUS ARMOUR LAYER

A PHYSICAL MODEL STUDY USING DIGITAL DISPLACEMENT ANALYSIS

Master of Science thesis

to obtain the title of Master of Science
at the Delft University of Technology,
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by

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An electronic version of this dissertation is available at
http://repository.tudelft.nl/.

Contents

Summary

XblocPlus is a new uniformly placed single layer armour unit, developed by Delta Marine Consultants (BAM Infraconsult). This report focuses on the stability of the first row of armour units, as this row is interlocked less then other rows. When the first row starts to slide, this is a big threat to the stability of the entire armour layer. Furthermore, not much is known what exactly influences the stability of the first row. In this study physical modelling is used to determine the stability the first row of an XblocPlus armour layer and investigate what influences this stability. The results of the tests were analysed with a method called Digital Displacement Analysis.

Two simplistic theoretical models were devised. The first model considers the destabilizing forces to consist only of the drag and uplift forces caused by the wave on the armour units of the first row itself, this is called **direct hydraulic loading**. The second model also considers destabilizing forces transferred by the armour layer to the first row, this is called **indirect hydraulic loading**.

From the model tests, it was found that indirect hydraulic loading plays an important role, as the movement clearly occurred during down-rush. When waves started to plunge, the waves failed to destroy the breakwater. Furthermore, loose armour units placed in front of the first row moved at higher heights than the first row, indicating that the armour layer pushes out the first row. Water depth influences stability, as lower water depth gave a lower stability. It is feasible that this effect is caused by the down rush being closer to the first row. The second model is most appropriate when taking these findings into account, see figure 1.

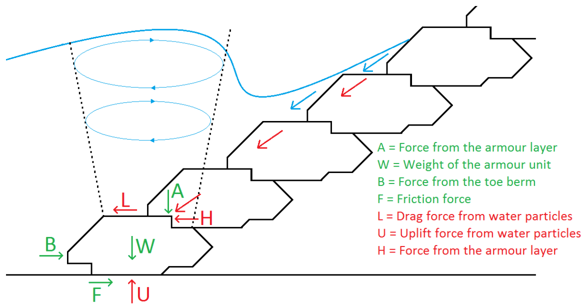

Figure 1: Balance of forces of the second model. Green arrows indicate stabilizing forces, red arrows indicate destabilizing forces. Blue arrows indicate the movement of the water particles.

From the tests it was found that a toe berm increases stability as it adds weight to the first row and thus increases friction resistance. A foundation layer also increases stability as friction between the foundation layer and the first row is higher than between the first row and the concrete slab that was used in other tests. Both measures also increase the damage threshold. The damage threshold is the amount of damage that a first row can take before brittle failure occurs. Brittle failure is an increase in damage of $0.2D_n$ (D_n is the nominal armour unit diameter). When both measures are used together, the foundation layer makes the toe berm more stable and this is why the stability of the first row is also increased. In table 1 the stability of the first row loaded with irregular waves with a wave steepness of 4% (s_{0p} = 4%) are compared. Based on the tests in this study the stability of the first row can be stated as $N_s = 3.87$ ($N_s = \frac{H_s}{\Delta D_n}$: the stability number is defined as the significant wave height divided by the nominal diameter of the armour units and the relative density of the armour units) as at the end of the tests the damage was only $0.13D_n$.

Toe Foundation layer	No No	Yes No	No Yes	Yes Yes
N_s ($\Delta x/D_n = 0.1$)	1.64	1.97	1.97	3.62
Percentage	100	120	120	221

Table 1: Overview of stability numbers for different configurations of the first row, loaded with irregular waves with $s_{0p} = 4\%$

A remarkable finding of this study is that two test series showed contradictory influences of wave steepness. The first test series showed that a lower wave steepness is detrimental for stability of the first row, while the second test series showed the opposite. More research should be conducted to investigate on this curiosity.

The test results were used to determine a working hypothesis for a design formula:

$$\begin{aligned} \frac{H}{\Delta D_n} &= 2.59 f_s f_w \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ \frac{H}{\Delta D_n} &= f_s f_w \left(0.3\frac{h_f}{D_n} - 0.26\right) \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{1}$$

More tests are necessary to validate this formula and determine it's accuracy. With f_s being a factor to correct for structural elements and f_w being a factor to change from regular to irregular waves (changing wave height H to significant wave height H_s):

Configuration	f_s [-]	f_w [-]
No toe berm, no foundation layer	1	0.6
Toe berm, no foundation layer	1.5	0.6
No toe berm, foundation layer	1.3	0.6
Toe berm, foundation layer	2.1	0.7

Table 2: Overview of factors to adjust the design formula

Preface

This thesis concludes my study of Hydraulic Engineering at the TU Delft. It wouldn't be complete however if I don't thank all the people who helped me take the final hurdle in order to obtain a MSc Degree.

First of all, I want to thank my thesis committee members. Stefan Aarninkhof, Bas Hofland, Alessandro Antonini and Bas Reedijk contributed to my research by giving valuable input at the committee meetings and whenever I had questions about the feedback I received from them. I want to especially thank my daily supervisor, Markus Muttray, as he was always available to help me. His patience and guidance helped me enormously.

I want to thank the people of the coastal engineering department at BAM Infraconsult for their support. Especially Ikbal Kelkitli, for helping me with my project and the struggles of writing a thesis in general. I want to thank BAM in a more general way for giving me the opportunity to execute this fascinating study and allowing me to combine it with my career in sports.

Furthermore I want to thank the friends I made during my studies. They gave me positive energy which helped complete my study and gave me a wonderful time in general. In D.S.S.V. ELS (the ice skating club for students in Delft) I found an awesome way to develop myself in all sorts of ways and also have a lot of fun. At the core of this experience are my fellow board members, Daniël, Wouter and Jelmer.

Finally I want to end this word of thanks with the people it all started with: my brother, sister and my parents. They gave me unconditional love and support, which made me confident in life.

Jan-Willem Broos
Delft, December 2019

Notation

This is a list of all the symbols used in this report:

α_b	Slope of the breakwater $[m/s]$
α_f	Slope of the foreshore (gradient) [-]
ΔX	Cumulative displacement of an armour unit $[mm]$
Δ	Relative density $(=\frac{\rho_s-\rho_w}{\rho_w})$ [-]
$\hat{u_\delta}$	Orbital velocity at the toe $[m/s]$
ρ_c	Density of concrete $[kg/m^3]$
ρ_s	Density of sediment $[kg/m^3]$
ρ_w	Density of water $[kg/m^3]$
ξ_{0p}	Surf similarity parameter $(=\frac{\tan\alpha_f}{\sqrt{H/L_o}})$ [-]
A	Surface perpendicular to the direction of the flow $[m^2]$
B_t	Width of toe berm $[m]$
C_l	Lift coefficient $[-]$
dX	Displacement of an armour unit per test run $[mm]$
d_{n50}	Median nominal diameter of sediment $[m]$
D_n	Nominal diameter of an XblocPlus armour unit $[m]$
F_p	Pulling force $[N]$
F_s	Unit weight $[N]$
g	Gravitational acceleration $[m/s^2]$
h	Water depth$[m]$
h_f	Depth of the first row (measured from the interface of the XblocPlus and the fore-shore to still water level) $[m]$
h_o	Off shore water depth$[m]$
H_s	Significant wave height $[m]$

h_t	Waterdepth above toe structure $[m]$
H_{rmd}	Design mean wave height (regular waves) $[m]$
H_{sd}	Design significant wave height $[m]$
$i_{2\%}$	Hydraulic gradient exceeded by 2% of the waves [-]
i_{cr}	Critical hydraulic gradient for motion [-]
k	wave number $[m^{-1}]$
L_b	Length of the breakwater section $[m]$
L_o	Deep water wave length $(=\frac{gT_p^2}{2\pi})$ $[m]$
L_p	Peak wave length in front of the structure $[m]$
$L_{m-1.0}$	Wave length belonging to the wave period calculated from the first negative moment of the spectrum $(=\frac{gT_{m-1.0}^2}{2\pi})$ $[m]$
N	number of displaced stones in a breakwater section [-]
n	Porosity [-]
$N_{\%}$	Percentage of displaced stones in a breakwater section [-]
N_s	Stability number $(=\frac{H_s}{\Delta d_{n50}})$ [-]
N_{od}	Damage number $(=\frac{N}{L_b/d_{n50}})$ [-]
P	Notional permeability) $[-]$
$R_{d2\%}$	Run down exceeded by 2% of the waves) $[m]$
RD	Relative displacement: the average of the middle three armour units divided by the nominal diameter of an armour unit $(=\frac{\overline{\Delta X_{5,6,7}}}{D_n})$ $[-]$
s_{0p}	Wave steepness for offshore waves of the peak period $(=\frac{2\pi H_s}{gT_p^2})$[-]
s_{om}	Fictitious wave steepness) $[-]$
T_p	Peak period $[s]$
t_t	Thickness of toe berm $[m]$
$T_{m-1.0}$	Wave period calculated from the first negative moment of the spectrum $[s]$
v	Water velocity $[m/s]$
W	Unit weight [kg]

1
INTRODUCTION

1.1. MOTIVATION

This report deals with the hydraulic stability of the XblocPlus armour units of the first row of a rubble mound breakwater with XblocPlus armour. The hydraulic stability of the first row is interesting as regular armour units (e.g. an armour unit that is not part of the first row) are interlocked by 4 neighbouring units, while armour units in the first row are only interlocked by 2 neighbouring units. Damage to the first row can lead to damage to the entire armour layer. Furthermore to the author's knowledge there is no published literature on stability of the first row of interlocking armour units (except for a study of loads on the first row, [1]). As XblocPlus is a novel armouring concept, this research gives valuable insight in the stability of the first row.

Please note: Armour layer stability can refer to structural strength or hydraulic stability of armour units. This study is dealing with hydraulic stability only, the term 'stability' refers in all places to hydraulic stability unless otherwise stated.

1.2. TOE TERMINOLOGY

Sometimes people mean different things when they say "toe structure", for clarity's sake the terminology used in this report is consistent with Figure 1.1. A breakwater toe can be defined as "the place where the breakwater touches the seabed", this is why a toe structure is considered to consist of the first row of the armour layer, the toe berm and the foundation (including the foundation layer and the granular filter layers).

Design of an XblocPlus breakwater should comply with guidelines composed by Delta Marine Consultants [2].

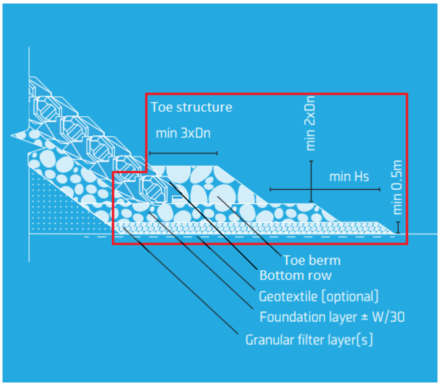

Figure 1.1: Typical toe structure on a sandy seabed (from Delta Marine Consultants [2])

Figure 1.2: Illustrating the interlocking mechanism

1.3. INTERACTION OF XBLOCPLUS ARMOUR UNITS

XblocPlus armour units have a certain interlocking mechanism. This means that movement of an armour unit is hindered by other armour units. This is illustrated in Figure 1.2. In a certain row X armour unit 0 is considered. For upward movement unit X0 is interlocked by armour units of a higher row H, which are L and R (respectively to the left and right of unit X0). For side ward movement X0 is interlocked by armour units above and below: LL, LR, HL and HR. For forward movement unit X0 is interlocked by two armour units of a lower row L, L and R. This means that for forward movement, armour units are partly dependent on armour units below them. In the first row however, armour units are not interlocked for forward movement by other armour units below them, so their stability is only generated by friction. In Figure 1.2 units of the first row F the middle unit F0 is loaded by units LL and LR.

1.4. AIM OF THE RESEARCH

The goal of this study is to gain insight in the hydraulic stability of the first row of an XblocPlus armour layer, scour is not considered. This research aims to find out which failure mechanism causes displacement of the first row.

1.5. RESEARCH QUESTIONS

In order to reach the goal that is specified above, research questions are determined:

1. What types of loading occur that cause the first row to fail?
2. What level of damage to the first row is critical for the stability of the entire armour layer?
3. In what way do wave height, water depth, wave period and wave steepness influence the hydraulic stability of the first row and can they be combined in a stability formula?

4. In what way does a foundation layer influence the hydraulic stability of the first row?

5. In what way does a toe berm influence the hydraulic stability of the first row?

6. What is the stability of the first row?

1.6. METHODOLOGY

In order to answer the research questions a methodology is put up. First a literature study (Chapter 2) was performed on the subject. On the base of this literature study a working hypothesis is developed by describing a simplistic model of stabilizing and destabilizing forces. This led to two theoretical models (Chapter 3). A physical model study was developed to investigate which of these models was more accurate (Chapter 4, the setup and specifics of the tests are presented in Chapter 5). A method to measure the displacement of the first row was developed: Digital Displacement Analysis (Chapter 6). The output of this analysis (Chapter 7) is evaluated (Chapter 8), discussed (Chapter 9) and used to validate the initial hypothesis, giving recommendations for toe structure design (Chapter 10). After this study, recommendations are given for further research (Chapter 11).

2

LITERATURE STUDY

The following is a summary of a more elaborate literature study, which can be found in Appendix A.

2.1. BALANCE OF FORCES ON AN ARMOUR UNIT

The forces acting upon an armour unit determine whether the armour unit will move or not. This means that understanding of these forces is important. Brebner and Donnelly [3] mention the following forces acting on rocks of a rubble mound breakwater:

- Self weight of the armour unit;
- Weight of armour units resting on the armour unit;
- Friction forces;
- Drag force;
- Lift force;
- Inertia force;

A valuable stability analysis based purely on physics is very hard because these forces are dependent on each other and are influenced or caused by processes that show a dynamic and turbulent behaviour. Brebner and Donnelly [3] tried to derive a function for the stability number of a rubble mound foundation, but they needed a lot of simplified assumptions and end up with an equation which contains a number of unknown coefficients.

The research of van de Koppel *et al.* [4] was aimed at gaining insight in these forces for Xbloc armour units by doing physical modeling, but the research showed that making a prediction of the loads based on the influencing parameters is not very reliable. He did find that the weight of the layers on top of the first row didn't increase when more than 10 rows were placed on top of each other.

2.2. XBLOCPLUS STABILITY

Single layer concrete armour unit stability is commonly described by the stability number $N_s = \frac{H_s}{\Delta d_{n50}} = c$, with c being a constant. This concept was introduced by Van der Meer [5]. Damage of XblocPlus armoured slopes starts at stability number of 3.5, but a design value of $N_s = 2.5$ is recommended by Delta Marine Consultants [2]. The XblocPlus stability is reduced in case of a steep foreshore, a low crested structure or a low core permeability [2].

2.3. DAMAGE TO THE FIRST ROW

One way of assessing how much damage has been dealt to a first row, is described by Hofland and Van Gent [6]. In their paper Hofland and Van Gent explain an image processing technique to detect very small movements. Parameters which influence this second phenomenon are wave height, water depth, number of rows, breakwater slope and possibly the rock size of the foundation layer and toe berm, wave length and wave steepness. They find a damage threshold of $0.2D_n$, as after this threshold armour unit extraction occurs.

2.4. TOE BERM

Stability of rock toe berm in front of a rubble mound breakwater was investigated by, amongst others, Gerding [7], Docters van Leeuwen [8], Van der Meer [5], Ebbens [9], Muttray [10], Van Gent and Van der Werf [11] and Muttray *et al.* [12]. Stability of rock toe berm may differ from the stability of the units of the first row, so the stability formulae of these studies are most probably not applicable for 1st row of interlocking armour units. Governing parameters for toe stability are wave height [7], [8], [5], [9], [10], [11], [12] and wave length [11], [12] on the driving side of motion and the depth of the toe berm and the rock size at the retaining side [7], [8], [5] [9], [10], [11], [12]. Toe stability might be further influenced by the wave steepness [9], the width of the berm [10], [11], the thickness of the berm [10], [11], the slope of the breakwater [11] and the slope roughness [11]. It can be concluded that there is a lot of discussion about which parameters play a role in toe berm stability as only a few parameters (wave height, water depth and rock size) are generally recognized as important parameters, while other parameters are sometimes mentioned to play a role, while other researchers don't agree and others don't even mention these parameters. See Appendix A for more elaboration on this topic.

2.5. FOUNDATION LAYER

Delta Marine Consultants [2] state that the averege rock weight of the foundation layer for XblocPlus has to be approximately W/30. This is based on experience and not on research. Delta Marine Consultants [2] state that the foundation layer should be supported by filter layers. Design formulas for filters are given by CURNET [13, CUR 233].

2.6. SUMMARY

From this literature study it can be concluded that very few starting point for this study are found. This is mainly because previous studies focused on either the stability of the armour layer or the stability of the toe berm and not on the stability of the first row. The literature study revealed what the stability number that is used by Delta Marine Consultants and an expectation for a damage threshold based on other armour units [6].

3

THEORETICAL MODEL

Failing of the first row failure can be caused by two types of loading. The wave can pull out an armour unit of the first row, this is called direct hydraulic loading. The armour layer can also push out the first row, this is called indirect hydraulic loading. The first phenomenon is detected by assessing whether armour units have been extracted. The second phenomenon is detected by assessing whether the first row has moved as a whole. In this chapter these two types of loading are examined by a theoretical model study. A simplistic theoretical model is proposed that is used to establish meaningful relations between the forces that are acting on the XblocPlus units of the 1st row. This simplistic theoretical model is in fact a supplement to a basic dimensional analysis; the latter is a common starting point for experimental studies.

Two models are devised, the first model can be seen as a simple and the second model as a more complete description of the first row failure.

3.1. MODEL 1

The first model (see Figure 3.1) considers the failure caused by the direct hydraulic load on the first row by the waves. In this model the armour layer only loads the armour unit in a vertical way (A), which is a stabilizing force. Together with the weight of the armour unit (W), an uplift force (U) and a friction factor they determine the friction resistance F of the armour unit. The other component of the resistance against movement is the resistance of the toe berm. The uplift force (U) is caused by the difference in water velocity under and above the armour unit. As the energy height at both places is the same, but the velocity of the water is different, this will result in an upward pressure. The destabilizing force L is made up out of two components: the water velocity caused by the waves and the water velocity caused by seepage out of the breakwater core. Movement (and thus failure) occurs when the horizontal destabilizing force is larger than the horizontal stabilizing forces.

Stabilizing forces depend amongst others on parameters D_{n50}, d_{n50}, t_f, t_t and B. Destabilizing forces depend amongst others on parameters H_s, L and h.

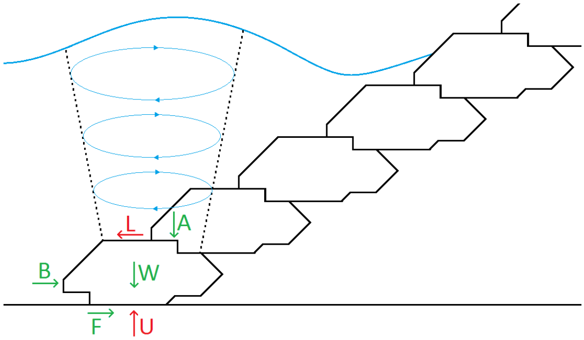

Figure 3.1: Balance of forces of the first model. Green arrows indicate stabilizing forces, red arrows indicate destabilizing forces. Blue arrows indicate the movement of the water particles.

3.2. MODEL 2

The second model (see Figure 3.2) considers an extra load, caused by the hydraulic load on the armour layer. The drag force of the wave is transferred through the armour layer to the first row. This is why this is called ’indirect’ hydraulic load. In this model the armour layer not only loads the armour unit in a vertical way (A), but also in a horizontal way (H), the latter of which is a destabilizing force. This horizontal load is caused by the waves which are trying to pull out the blocks. This is hindered by the interlocking of lower rows. Movement (and thus failure) occurs when the horizontal destabilizing forces are larger than the horizontal stabilizing forces. In this model both types of loading occur.
Stabilizing forces depend amongst others on parameters D_{n50}, d_{n50}, t_f, t_t, B, H_s, L and h. Destabilizing forces depend amongst others on parameters H_s, L, h, α_b and the number of rows.

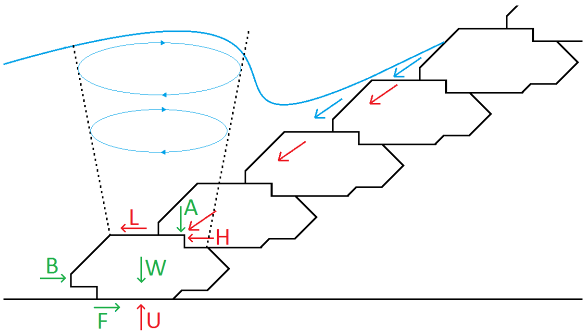

Figure 3.2: Balance of forces of the second model. Green arrows indicate stabilizing forces, red arrows indicate destabilizing forces. Blue arrows indicate the movement of the water particles.

4

Present study

4.1. Relevant parameters

From the literature study and the theoretical model some parameters have prevailed for possible further study. For failure by armour unit extraction by waves these are wave height, wave period, foreshore slope, water level, rock size of the toe berm, rock size of the foundation layer and foundation layer thickness. For failure by pushing of the first row by the armour layer the important parameters are water depth, the number of rows, breakwater slope, rock size of the foundation layer and toe berm, wave period and wave steepness. Investigating the influence of all these parameters is too much work, so a selection has to made. Wave period, wave steepness, water depth and wave height were chosen as hydraulic parameters. Structural parameters that were varied are the presence of a toe berm and/or a foundation layer. It is more important to investigate the effect of a conventional toe berm and/or foundation layer, than to know the precise influence of rock sizes and layer thicknesses at the moment. The influence of the number of rows was tested with pull-out tests. For the breakwater slope a typical breakwater slope was chosen. The foreshore slope was fixed as well.

4.2. Test program

In order to answer the research questions a test program is devised. The terminology used to define the tests can be found in table 4.3. The second research question required that the testing isn't stopped when the first row started to fail, but is continued to determine the consequences. The third research question required that water depth, wave period and wave steepness are varied in a test series. The first series of tests is used to answer these questions. This series consisted of the following tests:

T [s] / h_f [s]	1.25	1.5	2
0.276	-	x	-
0.324	x	x	x
0.376	x	x	x
0.426	-	x	-

Table 4.1: First series of tests

4

The wave heights that were used in each test can be seen in Appendix D. Almost all tests were done with regular waves for high testing efficiency and good observation possibilities. One extra test was done at $h_f = 0.276m$, $T_p = 1.5s$ as this appeared to be the most critical test, this time irregular waves were used.

The fourth and fifth research question required a second series of tests. This test series consisted of the following tests:

	Configuration				
2%	NT NF	WT NF	NT WF	WT WF	-
4%	NT NF	WT NF	NT WF	WT WF	WT FF
Irregular (4%)	NT NF	WT NF	NT WF	WT WF	-

Table 4.2: Second series of tests, 'N' means 'No', 'W' means 'With', 'T' means 'Toe berm', F means 'Foundation layer', 'F' means 'with a Fine'

All tests were useful for answering the first and lasts research questions. The first two rows are tests that are conducted using regular waves. All tests were done at $h_f = 0.276m$ as this was the most critical water depth of the previous series of tests. WTNF4 was redone as the output of the Python script was troublesome.

Component	Description
Test program	Complete testing program
Test series	Several tests with the intention of finding the influence of wave conditions or structural specifics
Test	Several test runs with increasing wave height, while other wave conditions and structural specifics remain constant, in between the test runs pictures are made
Test run	Application of a certain wave wave field with a certain (significant) wave height

Table 4.3: Definition of test terminology

4.3. FAILURE DETECTION

Failure can be detected in two ways. The first is by seeing when an amour unit is extracted from the first row. The second way is by photographing the bottom row before and after each test and using digital image processing to detect whether the first row has moved. Digital Displacement Analysis (DDA) is a method developed for this study in order to determine the displacement that has occurred to the first row. DDA is explained in chapter 6.

4.4. OTHER TESTS

Other tests were done as well. An amour unit was placed in front of the toe structure, in order to determine what direct wave load is needed to displace an armour unit that isn't loaded by the armour layer. Furthermore pull out test were performed. These were done to find the influence of armour slope and number of rows on the resistance to horizontal movement. Two slopes (1:2 and 3:4) and three amount of rows (5, 10 and 15 rows) are all tested 5 times. No under-layer was used. This means that the armour units were resting on dry wood.

Figure 4.1: The pull-out test set up

5

MODEL SETUP

In this chapter the model setup is explained. The model is aimed to represent a generic breakwater, with some deviations depending on the test series.

5.1. WAVE FLUME

The tests for this research were performed in the Bam flume in Utrecht. The length of the flume is 25 m, a width of 0.6 m and a height of 1.05 m. The minimum water level is 0.4 m, the maximum is 0.7 m and the maximum wave height that can be generated is 0.3 m. The waves are generated with a piston wave generator (of Edinburgh Designs). This wave generator can generate regular and irregular waves. The reflected waves are damped by the wave generator.

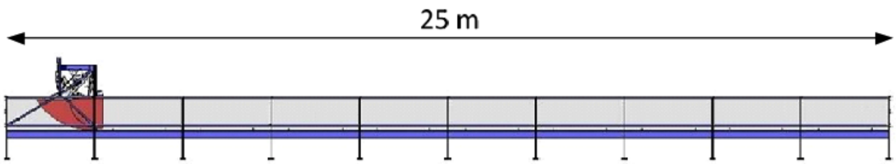

Figure 5.1: Longitudinal view of the flume (from Van Zwicht [14])

5.2. HYDRAULIC PARAMETERS

5.2.1. WAVE GENERATION

The hydraulic tests were mainly performed using regular waves. Advantages of regular waves are a high testing efficiency and good observation possibilities. In order to test whether irregular waves result in a higher load on the first row, some tests were performed using irregular waves. These irregular waves were made according to the JONSWAP spectrum. The JONSWAP spectrum requires three shape factors: γ, σ_a and σ_b.For a standard JONSWAP spectrum $\gamma = 3.3$, $\sigma_a = 0.07$ and $\sigma_b = 0.09$. As irregular testing takes a lot of time, irregular tests were only done at the most critical point found by regular wave testing.

5.2.2. WAVE ANALYSIS

The waves were analysed by 2 groups of three wave gauges positioned close to each other in a line parallel to the wave propagation. The first group of wave gauges was positioned before the foreshore slope, the second group was positioned 1.5 meters before the breakwater on the foreshore slope. The signal of these gauges were analysed using WaveLab 3, a program developed by the University of Aalborg. WaveLab3 uses the method developed by Mansard and Funke [15].

5.2.3. WAVE HEIGHT, H_s

The design significant wave height for XblocPlus model units is $H_{sd} = 0.0986m$. This study is done mostly with regular waves, so the significant design wave height has to be multiplied with a factor of 1.4 (rule of thumb) to get the mean design wave height: $H_{rmd} = 0.138m$. A test was started with a run at relatively low wave heights of approximately 45%. Then it was increased to approximately 60%, then to 75%. After this point the wave height was increased with steps of approximately 7% until failure.

5.2.4. WAVE PERIOD, T

For the first test series wave periods of 1.25, 1.5 and 2 seconds were generated. This results in surging waves for most of the wave heights, which is representative for a generic XblocPlus breakwater. Only the 1.25 s waves had plunging waves at the highest wave heights.

5.2.5. WAVE STEEPNESS, s_0

For the second test series waves with $s_0 = 2\%$, $s_0 = 4\%$ and $s_{0p} = 4\%$. This results in surging waves.

5.2.6. TEST DURATION

An advantage of testing with regular waves is that is very fast. It is not necessary to wait for the maximum wave, as all waves are the same. This means that most tests can be performed in a short period. During testing 2 minutes seemed to be enough to let all movement happen. For the tests with irregular waves duration is of importance. This means that a storm of 3 hours should be represented. A storm of 3 hours contains approximately 1000 waves, so tests with irregular waves should have a duration of approximately 1000 waves. This means that a test duration of 1 hour was required.

5.2.7. WATER DEPTH, h

Testing was done at four water depths. These are 0.55, 0.60, 0.65 and 0.70 m. The foreshore had a height of 0.274 m, so at the breakwater toe the water depths was 0.276, 0.326, 0.376 and 0.426 m respectively. With the wave lengths that are made, the waves were in intermediate water depth, so shoaling occurred.

5.3. STRUCTURAL PARAMETERS

The breakwater model has no real prototype. Still, an imaginary prototype was devised after which the breakwater model can be modelled.

5.3.1. ARMOUR LAYER

The prototype has a $V = 2.5m^3$ and a $d_n = 1.36m$. The smallest XblocPlus model armour unit available has a $d_n = 0.029m$. This results in a scaling factor of 1:46.9. The weight of the model unit $W = 0.0585kg$. The density $\rho_m = 2360kg/m^3$.

5.3.2. UNDER LAYER

Delta Marine Consultants advise a rock grading for the under layer of 300 - 1000 kg. Such a grading has a $d_{n50} = 0.56m$. Scaled this means that the model has an under layer with a $d_{n50} = 0.012m$.The under layer thickness is specified by DMC as 1.3 meters, so this scales to a model under layer thickness of 0.028 m.

5.3.3. CORE

A typical breakwater core was filled with quarry run. This means it is hard to determine what rock grading is used for the imaginary prototype, as this in practise depends on what is easily obtainable at the project site. For now a $d_{n50} = 0.4m$ is assumed. It is not possible to scale this correctly with respect to Froude's and Reynolds law simultaneously. Burcharth *et al.* [16] proposed a method to find a solution for this problem. The idea is that the characteristic velocity in the prototype is calculated using the Forchheimer equation. This characteristic velocity is then scaled using the Froude number. Then the d_{n50} of the model core is adjusted until the target characteristic velocity is reached. After this the Reynolds number of the flow is calculated. This number is preferably above 300 to remain turbulent and has to remain above 100. For the model for this study a $d_{n50} = 0.01m$ gives a representative characteristic velocity. The Reynolds number becomes 176, which will result in conservative but acceptable results.

5.3.4. CROSS SECTION

The breakwater model represents a generic XblocPlus breakwater. Such a breakwater typically has a slope of 3:4. The dimensions of the model are presented in Figure 5.2. The water level depicted in the figure is 0.5 m.

5.3.5. FORESHORE

A foreshore of 1:20 was chosen as this will represent a rather steep foreshore, giving conservative results as waves will shoal and break closer to the breakwater.

5.3.6. INNER SLOPE

The inner slope was not of interest for this research. It should not fail, so a combination of gabions and elastocoast was used.

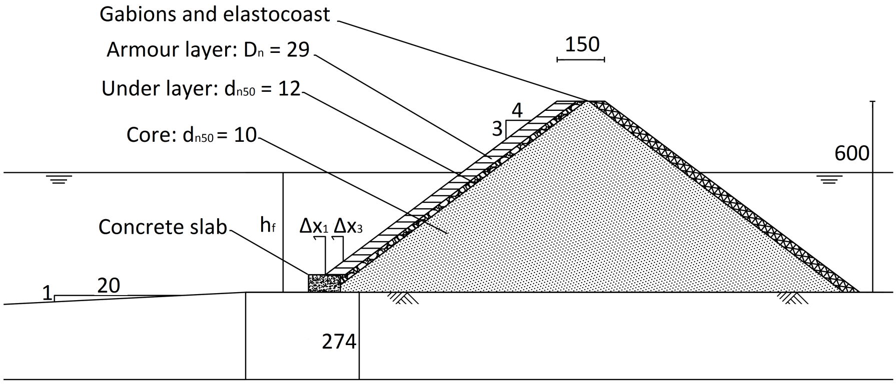

Figure 5.2: Cross section of the breakwater model with a water depth of 55 cm. Units in figure: [mm]

5.3.7. FOUNDATION LAYER

For the second series of testing, for some tests, a foundation layer was used. The foundation layer has a $d_{n50} = 0.012m$. As the foundation layer should at least be twice as thick as the d_{n50}, the layer thickness becomes 2.4 cm. The foundation layer was 17 cm long. For one of the tests a fine foundation layer was used. When foundation layer wasn't used, a concrete slab was used as foundation for the first row.

5.3.8. TOE BERM

For the second series of testing, for some tests, a toe berm was used. The heaviest rock grading available was used, as the research aims to find the stability of the first row and not the stability of the toe berm. This means that the rock grading that is used has a $d_{n50} = 0.018m$. As toe berms are typically '3 stones wide and 2 stones high', the toe berm is 3.6 cm high and 5.4 cm wide.

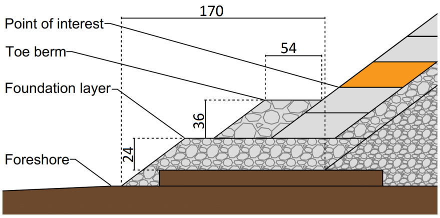

Figure 5.3: The toe structure with dimensions [mm]

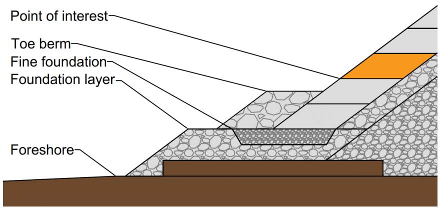

Figure 5.4: The toe structure with a fine foundation

5.4. TESTING PROCEDURE

A test was done according to the following procedure:

1. The flume is emptied.

2. The breakwater is restored.
3. The flume is filled.
4. Pictures from above and the side are made of the breakwater.
5. The camera on the side of the breakwater is set to film the test.
6. The wave maker is activated.
7. Observations are made.
8. If the breakwater hasn't failed and is not appearing to fail in two minutes, the wave maker is stopped.
9. The camera is stopped and a picture from above and the side are made when the water is quiet again.
10. Repeat step 4 to 9 until the breakwater fails. Then the procedure is repeated from step 1 for an new test.

The data from the wave gauges, observations, the film and the pictures are used to analyse the tests.

6

DIGITAL DISPLACEMENT ANALYSIS

Digital Displacement Analysis (DDA) is a method developed for this study to determine the displacement that has occurred to the first row. The method is based on Automatic Settlement Analysis [6]. DDA uses a python script to compare images before and after a test run to determine the displacement of armour units in the first row. In this chapter the method is explained.

6.1. IMAGES

The camera needs to be fixed to a frame, as it is important that it doesn't move during the tests. It is essential to use a camera that has a remote control, as touching the camera will move the camera and distort the measurements. It is advised to connect the camera to mains, as a test is ruined when the battery runs out. The camera needs to be positioned straight above and perpendicular to the point of interest, see figure 6.1. The distance from the camera to the water surface and the first row should be measured precisely. This will make it easier to calculate the influence of refraction due to the water surface and to determine the accuracy of the measurements. A camera with a high resolution is advised. When taking pictures after each test run, it could suffice to only take a picture before the first test run. Still, it is advised to take pictures before and after each test run, as the extra pictures before each test run could help determine accidental movement of the camera in between test runs. This saves a test from being completely ruined, as when extra movement is determined, this can be subtracted from the movement that is caused by the waves. To get usable images, it is important to paint the armour units of interest with a colour that has great contrast to the rest of the picture. Colours like red, purple and orange are useful, colours like green and blue are less suited.

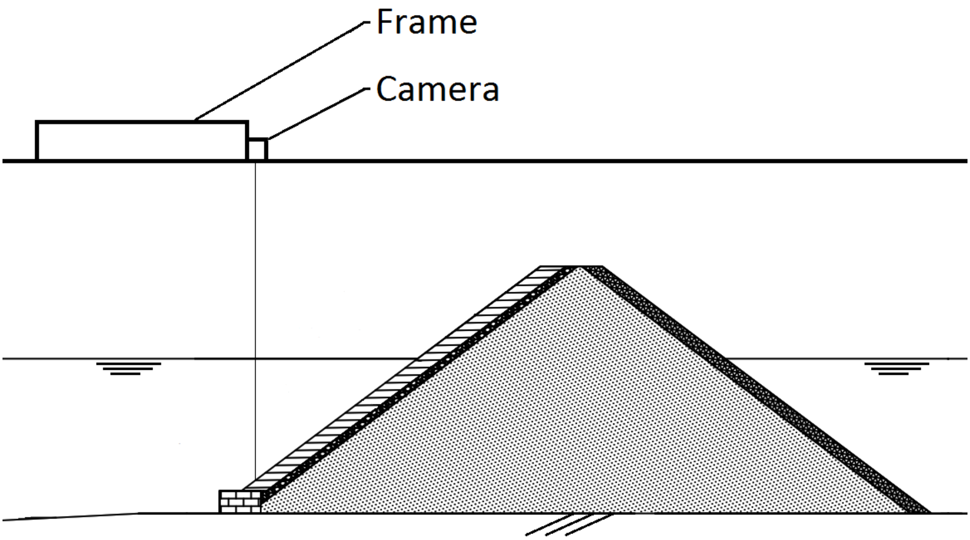

Figure 6.1: Proper camera set-up

6.2. ACCURACY OF THE MEASUREMENTS

In order to determine the accuracy of the measurements, reference lines are drawn at the foundation (in this case a concrete slab). One reference line needs to be placed straight under the middle of the camera. The other reference lines need to be drawn at distances that are relevant for the study (start of damage, intermediate damage, unacceptable damage). One picture is taken in an empty flume and one picture is taken in a filled flume. When the unfilled distance between the lines is multiplied with a refraction factor (that can be calculated using Snells Law), and is compared with the distance measured in the pictures of the filled flume, the accuracy can be determined.

6.3. PYTHON PROCESSING

The images before and after a test run are loaded in Python. The images are filtered for colour. A range in the colour spectrum is determined. All the pixels that fall in this range are made black, the rest is made white. Then the amount of white pixels is calculated from the edge of the picture to the first black pixel. Per armour unit this is done at 5 places in order to determine whether the measurement is reliable. This is done for both pictures. The difference between the amount of white pixels is the displacement that has occurred during the test run. The output of the python script is the displacement of each armour unit in the row of interest measured in pixels. This still has to be translated in a displacement in D_n. This can be done by determining how many pixels are needed per centimeter in the image of the filled flume with the reference lines.

6.4. FURTHER ANALYSIS

Now the images are converted into data, the data can be processed further to get to usable results. The following isn't specific for Digital Image Processing, but is necessary in this study to make the results useful.

6.4.1. REPRESENTATIVE RELATIVE DISPLACEMENT

As the flume is not very wide, so 11 armour units could be placed in the first row. This represents a section of a breakwater. The armour units on the side are less stable, as they are not interlocked from the side. This phenomenon is called wall effect. In order to stabilize the outer armour units, a chain is put over the outer armour units. Still, the wall effect is not undone, it has merely changed. When the relative displacement of all the armour units are observed, it is clear that the outer armour units move less then the armour units in the middle. The wall effect extends 2 armour units from the left side of the flume and 4 units from the right side. This can be seen in figure 6.2, where all the displacement of all armour units of the first row is compared to the middle armour unit for all regular tests after the last test run. Based on this information, it is decided that the average of the middle three armour units are representative for a generic armour unit in a breakwater.

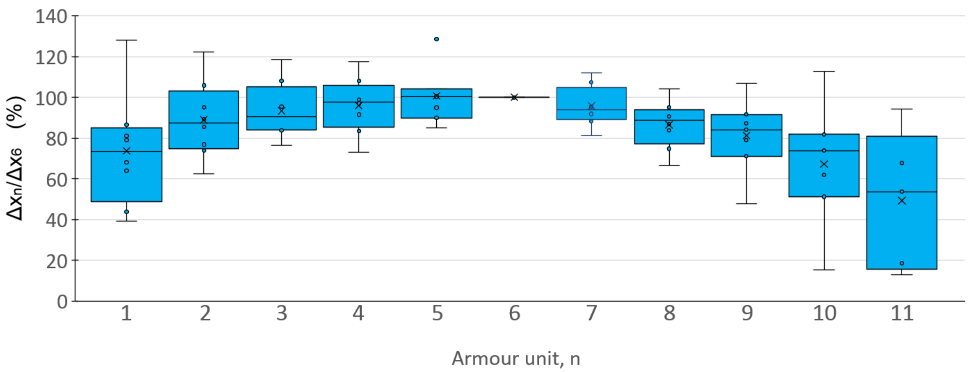

Figure 6.2: Movement of armour units of all regular tests compared to the middle armour unit

6.4.2. COMPARING THE FIRST AND THIRD ROW

This report focuses on the displacement of the first row. Some tests are conducted with a covered first and second row, as a toe berm is used in some tests. The third row is then the first row which can be used for determining displacements. It is necessary to determine whether the displacement of the third row can be used to determine the displacement of the first row. In order to do this, the displacement of the third row in tests without a toe berm are measured and compared to the displacement of the first row of the same test. This can be seen in figure 6.3. Even after the correction, a uncertainty of 10% remains.

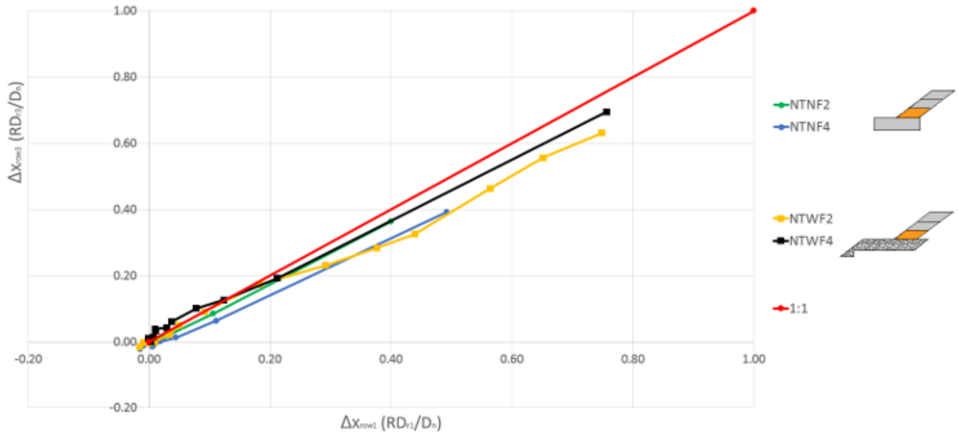

Figure 6.3: Displacement of the first row compared to the third row

The displacement of the third row will represent the displacement of the first row best if it is multiplied with a factor of 1.15. The result of this multiplication can be seen in figure 6.4

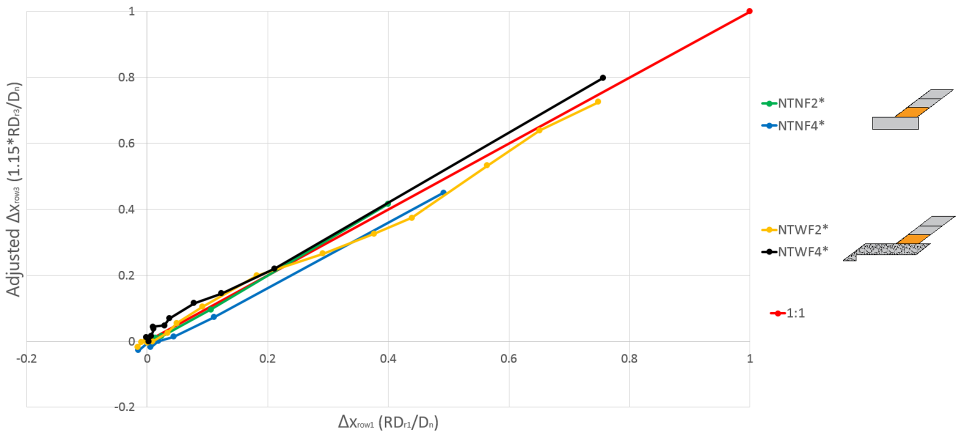

Figure 6.4: Displacement of the first row compared to the third row*1.15

6.4.3. REPRESENTATIVE WAVE HEIGHT

The waves are measured at two places: offshore and nearshore. The offshore wave gauges are useful to check whether the wave measurements nearshore are reliable. Offshore wave measurements contain measurements of waves when the reflected wave has not arrived at the wave gauges yet, giving a reliable measurement of the incoming waves. Incoming waves should shoal, so comparing the waves measured nearshore with the offshore waves multiplied with a shoaling factor, gives an indication of the reliability of the nearshore measurements. In figures 6.5, 6.6 and 6.7 this can be seen. The flume has some friction loss, so it is expected that the measured wave height near shore is a bit lower than the calculated waves, but this especially the irregular waves show strange behaviour near shore. This is why in this study the calculated wave height at the toe structure is used.

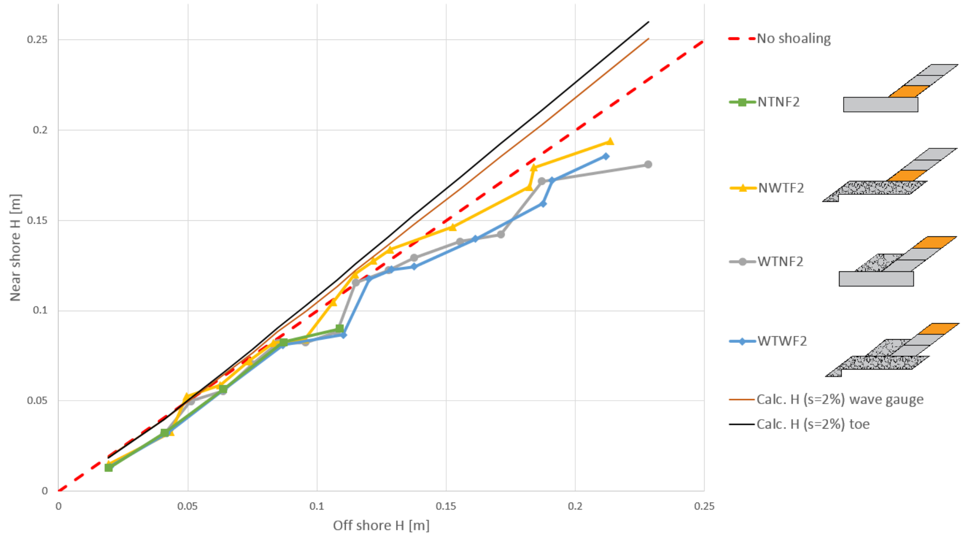

Figure 6.5: Wave height off shore compared to the wave height near shore for the tests with regular waves with a 2% steepness

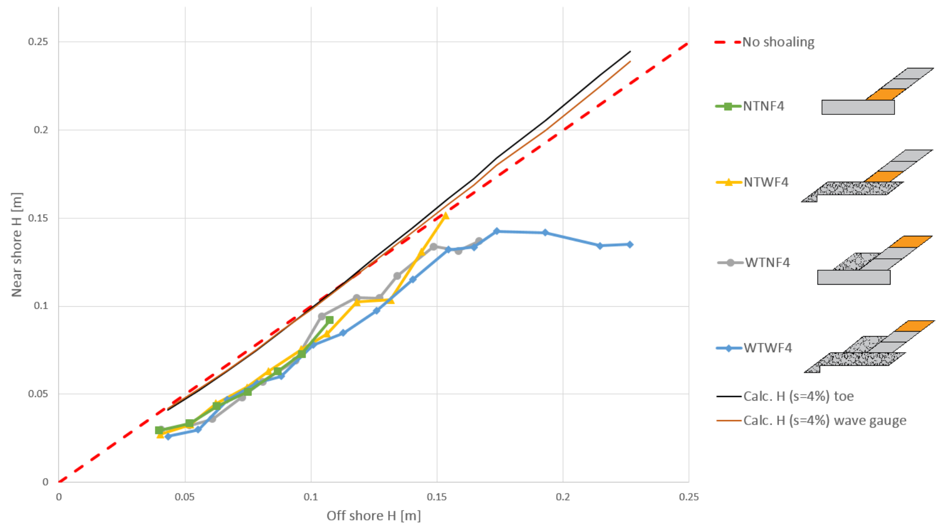

Figure 6.6: Wave height off shore compared to the wave height near shore for the tests with regular waves with a 4% steepness

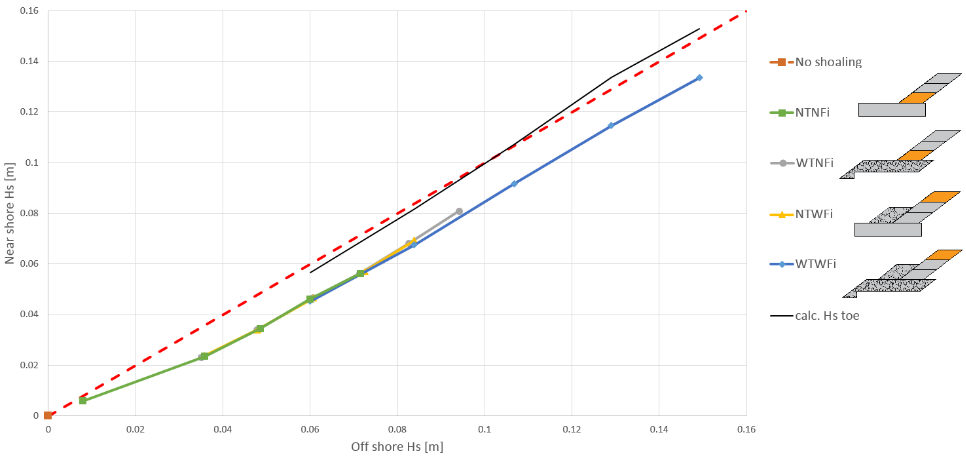

Figure 6.7: Wave height off shore compared to the wave height near shore for the tests with irregular waves

7

TEST RESULTS

7.1. HYDRAULIC TESTING

7.1.1. INFLUENCE OF THE HYDRAULIC PARAMETERS

The first test series comprised 10 tests. In all these tests the breakwater failed by the first row being pushed away. The Python script that is used to determine the displacement of the armour units, can be seen in Appendix B. The results of all the tests are presented in Appendix C. An example of the output of the Python script can be seen in Appendix D.

The tests with $h_f = 0.326m$ ($h_o = 0.6m$)can be compared using the figure 7.1. The test at $h_f = 0.3266m$ and $T = 1.25s$ was the only one where the first row wasn't pushed out far enough for the whole armour layer to collapse. This is probably because this test is the only test which ends with plunging waves. Plunging waves have less down-rush, so the load on the first row is decreased.

Furthermore for the test with a $T = 2s$ it is remarkable that although the wave height in run 3 ($\frac{H}{D_n\Delta} = 2.5$) wasn't higher than in run 2 ($\frac{H}{D_n\Delta} = 2.7$) the Relative Displacement ($RD = \frac{\overline{\Delta X_{5,6,7}}}{D_n}$) did increase with $0.03D_n$. the average displacement of the middle three armour units in the first row divided by the nominal diameter of an armour unit) did increase with $0.09D_n$. This phenomenon is observed in a few other tests as well. The reason the wave height did not increase was because the wave maker sometimes did not increase the wave height according to wave input file.

In all other tests the breakwater completely collapsed in the end, so RD is hard to determine. RD = 2.00 is used to indicate this.

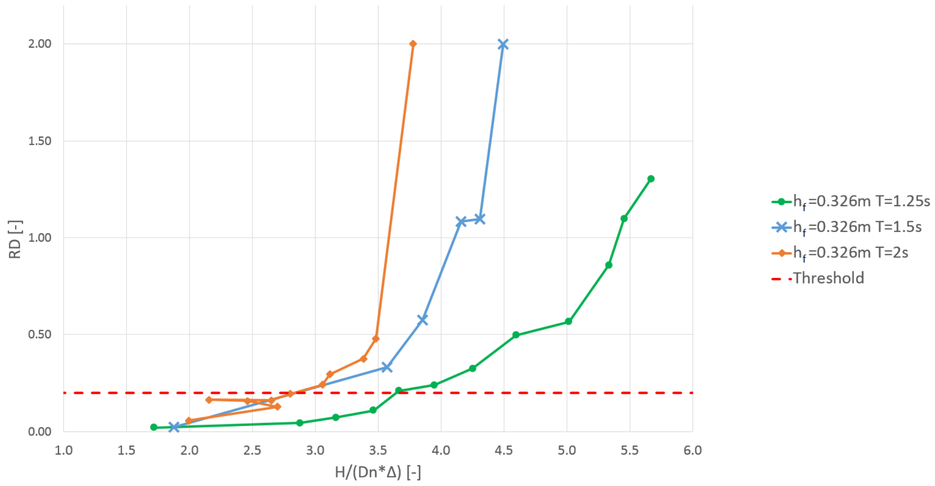

Figure 7.1: Relative movement of the first row vs. wave height over the nominal diameter with a $_f = 0.3266m$

In the following table different stability numbers are given of 4 damage levels. It is important to note that the test with a wave period of 1.5 s, accidentally had a second test run with a wave height which turned out to be rather large. This is why the first three damage levels of this test are all interpolated from the same two points, greatly reducing the validity of these points.

7

Wave period T [s]	1.25	1.5	2.0
N_s ($\Delta x/D_n = 0.05$)	2.94	2.03	1.78
Percentage	145	100	88
N_s ($\Delta x/D_n = 0.1$)	3.40	2.30	2.43
Percentage	147	100	106
N_s ($\Delta x/D_n = 0.2$)	3.65	2.85	2.83
Percentage	128	100	99
N_s ($\Delta x/D_n = \infty$)	NO	4.49	3.78
Percentage	-	100	84

Table 7.1: Overview of stability numbers for tests done with a depth of 60 cm, 'NO' means 'Not observed'

The tests with $h_f = 0.376m$ ($h_o = 0.65m$) can be compared using Figure 7.2. Complete failure occurred at slightly lower wave heights. Furthermore it is remarkable that in the end T = 1.5 s seems to be the most critical wave period for this water depth. This probably is caused by the fact that when the tests reach the critical wave heights, the test with T = 1.5 s has a wave steepness of 4.3%, resulting in surging waves ($\xi = 3.4$), while the test with T = 1.25 s at that moment has a wave steepness of 5.9%, resulting in collapsing waves ($\xi = 3.1$). Collapsing waves have a smaller down-rush than surging waves, so the load on the first row is smaller.

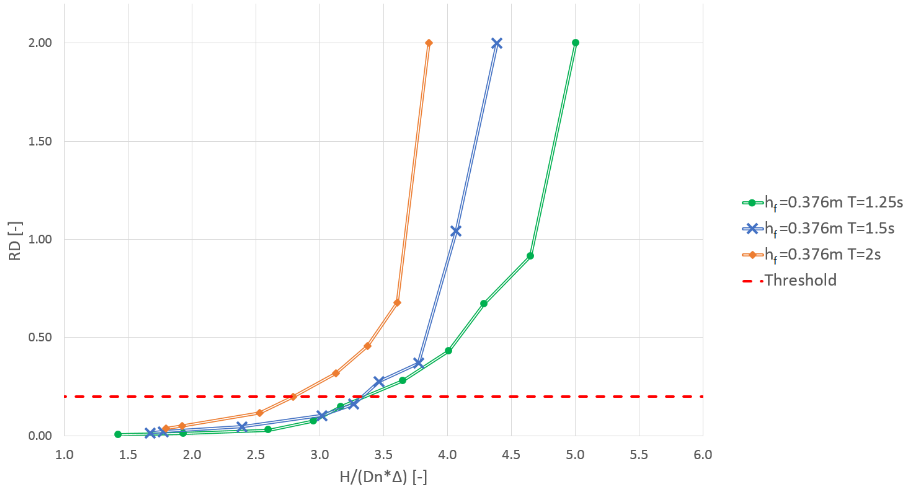

Figure 7.2: Relative movement of the first row vs. wave height over the nominal diameter with a depth of $h_f = 0.376m$

In the following table different stability numbers are given of 4 damage levels.

Wave period T [s]	1.25	1.5	2.0
N_s $(\Delta x/D_n = 0.05)$	2.76	2.45	1.92
Percentage	113	100	78
N_s $(\Delta x/D_n = 0.1)$	3.03	3.00	2.39
Percentage	101	100	80
N_s $(\Delta x/D_n = 0.2)$	3.36	3.34	2.80
Percentage	101	100	84
N_s $(\Delta x/D_n = \infty)$	5.00	4.39	3.86
Percentage	114	100	88

Table 7.2: Overview of stability numbers for tests done with $h_f = 0.376m$

In figure 7.3 the results of all the tests with a wave period of 1.5 s are presented. In the test with $h_f = 0.426m$ $(h_o = 0.7m)$ again showed that a test run with lower waves than the previous run can cause some additional damage. In the test with $h_f = 0.276m$ $(h_o = 0.55m)$ the breakwater collapsed completely by a failing first row at way lower wave heights than at other water depths. It also failed in a brittle fashion: a lot less displacement of the first row had occurred before the breakwater failed completely. As a water depth of 55 cm proved to be the most critical situation, a test was done using irregular waves. The results are presented in table C.9. The first wave run was only 10 minutes long as there was little time left and this wave height was used to calibrate the wave maker.

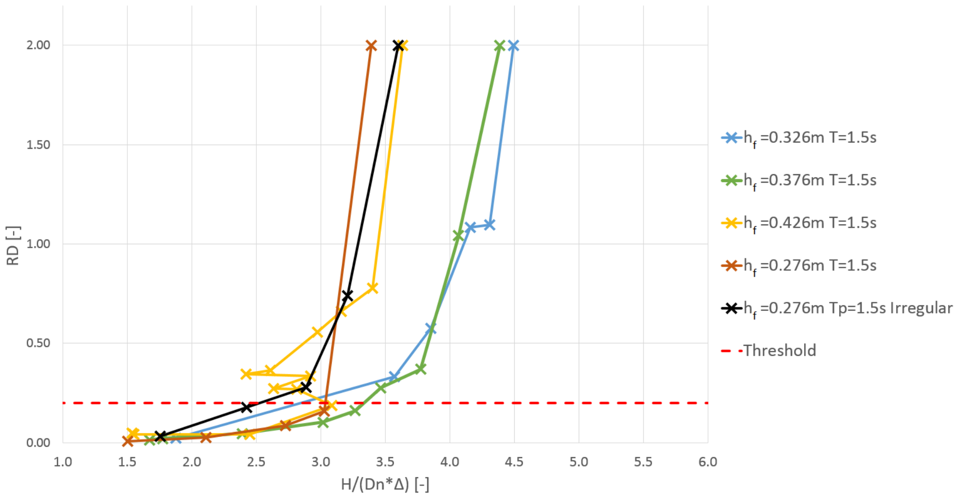

Figure 7.3: Relative movement of the first row vs. wave height over the nominal diameter

In the following table different stability numbers are given of 4 damage levels of tests with different depths but with the same wave period.

Depth h_f [m]	0.276	0.326	0.376	0.426	0.276 (irregular)
N_s $(\Delta x/D_n = 0.05)$	2.36	2.03	2.45	1.56	1.85
Percentage	100	86	104	66	78
N_s $(\Delta x/D_n = 0.1)$	2.79	2.30	3.00	2.70	2.07
Percentage	100	83	108	97	74
N_s $(\Delta x/D_n = 0.2)$	3.04	2.85	3.34	3.04	2.53
Percentage	100	94	110	100	83
N_s $(\Delta x/D_n = \infty)$	3.39	4.49	4.39	3.63	3.60
Percentage	100	132	129	107	106

Table 7.3: Overview of stability numbers for tests done with a wave period of 1.5 s

7.1.2. INFLUENCE OF THE STRUCTURAL PARAMETERS

The second series of tests comprised of 14 tests, consisting of 4 tests using a regular waves with a $s_0 = 2\%$, 6 tests using regular waves with $s_0 = 4\%$ and 4 tests using irregular waves with $s_{0p} = 4\%$. All the tests in this series were conducted at a water depth of $h_f = 0.276m$ ($h_o = 0.276m$). In the figures below the tested configurations are presented.

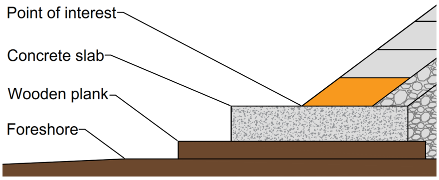

Figure 7.4: No toe berm, no foundation layer: NTNF

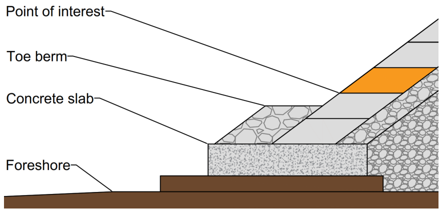

Figure 7.5: With a toe berm, no foundation layer: WTNF

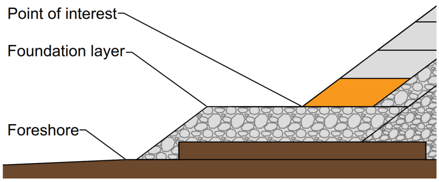

Figure 7.6: No toe berm, with a foundation layer: NTWF

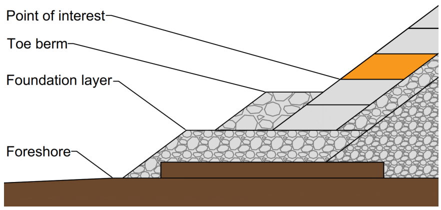

Figure 7.7: With a toe berm, with a foundation layer: WTWF

2% WAVE STEEPNESS TESTS

The following graph shows the tests that are conducted with regular waves and a $s_0 = 2\%$.

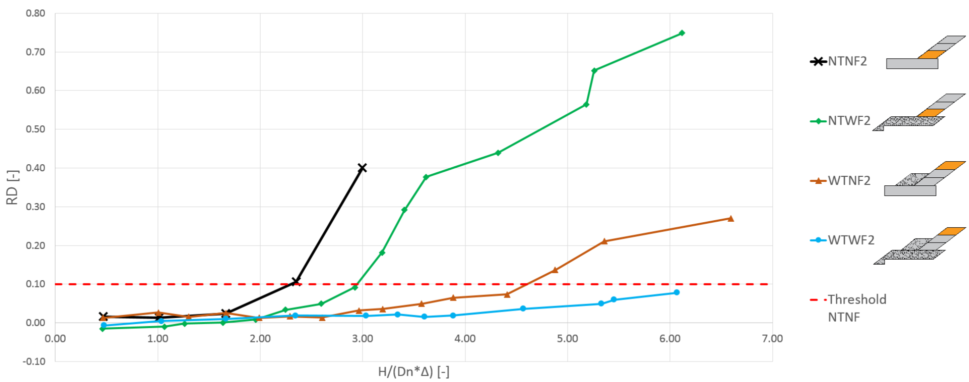

Figure 7.8: Relative movement of the first row vs. the stability number

In the following table different stability numbers are given of 3 damage levels.

Configuration	NTNF2	WTNF2	NTWF2	WTWF2
N_s ($\Delta x/D_n = 0.05$)	1.88	3.59	2.60	5.35
Percentage	100	191	138	284
N_s ($\Delta x/D_n = 0.1$)	2.30	4.61	2.95	NO
Percentage	100	200	128	-
N_s ($\Delta x/D_n = 0.2$)	2.56	5.13	3.23	NO
Percentage	100	201	126	-

Table 7.4: Overview of stability numbers, 'NO' means 'Not observed'

In graph 7.8 the total movement is used, in the following graph the movement per wave run is used.

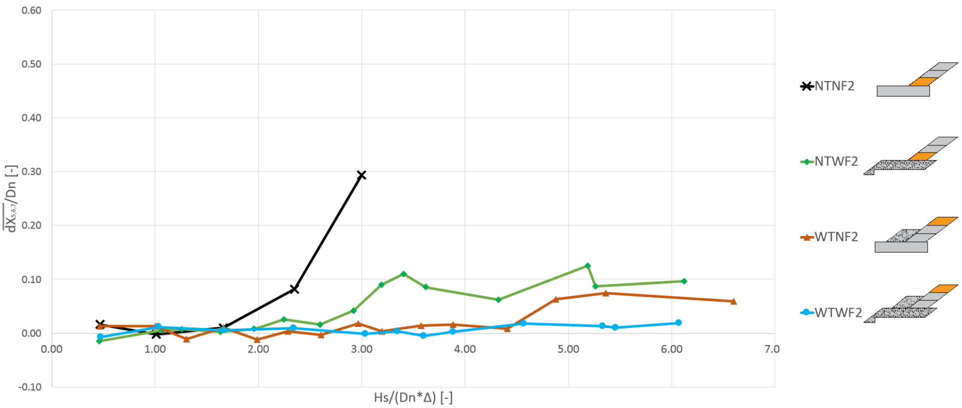

Figure 7.9: Relative movement per wave run of the first row vs. the stability number

4% WAVE STEEPNESS TESTS

The following graph shows the tests that are conducted with regular waves and a s_0 = 4%. The fourth test run of the WTWF4 test gives a lower RD than the third test run. This is probably caused because the third row of this test was painted green, which has less contrast than orange armour units with the rest of the picture, giving troublesome results.

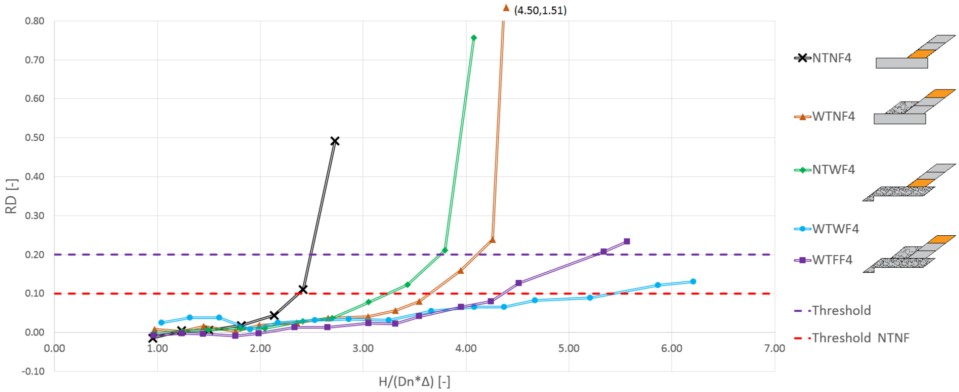

Figure 7.10: Relative movement of the first row vs. the stability number

In the following table different stability numbers are given of 3 damage levels.

Configuration	NTNF4	WTNF4	NTWF4	WTWF4	WTFF4
N_s ($\Delta x/D_n = 0.05$)	2.16	3.20	2.81	3.58	3.69
Percentage	100	148	130	166	171
N_s ($\Delta x/D_n = 0.1$)	2.37	3.65	3.24	5.43	4.36
Percentage	100	154	137	229	184
N_s ($\Delta x/D_n = 0.2$)	2.49	4.11	3.75	NO	5.27
Percentage	100	165	151	-	212

Table 7.5: Overview of stability numbers, 'NO' means 'Not observed'

In graph 7.10 the total movement is used, in the following graph the movement per wave run is used.

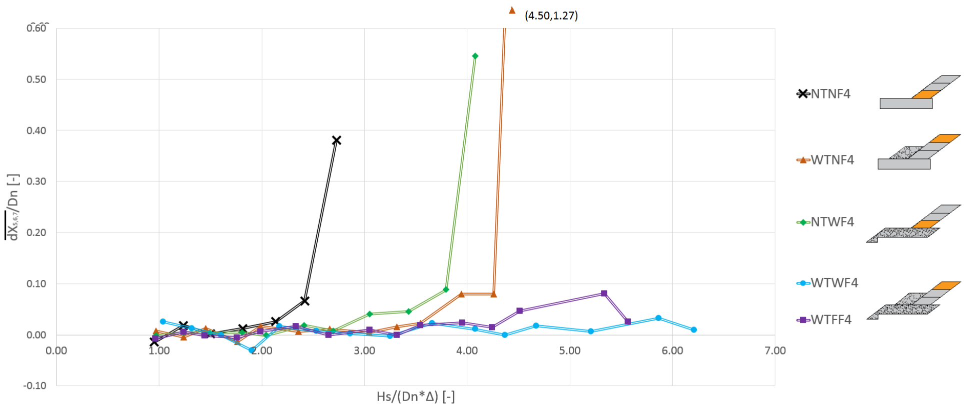

Figure 7.11: Relative movement per wave run of the first row vs. the stability number

IRREGULAR WAVE TESTS

The following graph shows the tests that are conducted with irregular waves and a s_{0p} = 4%.

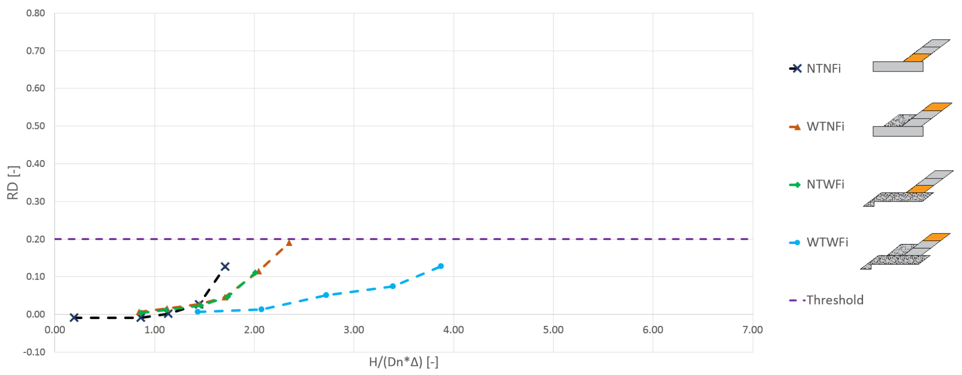

Figure 7.12: Relative movement of the first row vs. the stability number

In the following table different stability numbers are given of 3 damage levels.

Configuration	NTNFi	WTNFi	NTWFi	WTWF4i
N_s ($\Delta x/D_n = 0.05$)	1.51	1.72	1.74	2.70
Percentage	100	115	116	179
N_s ($\Delta x/D_n = 0.1$)	1.64	1.97	1.97	3.62
Percentage	100	120	120	221
N_s ($\Delta x/D_n = 0.2$)	NO	NT	NT	NT
Percentage	-	-	-	-

Table 7.6: Overview of stability numbers, 'NT' means 'Not tested'

In graph 7.12 the total movement is used, in the following graph the movement per wave run is used.

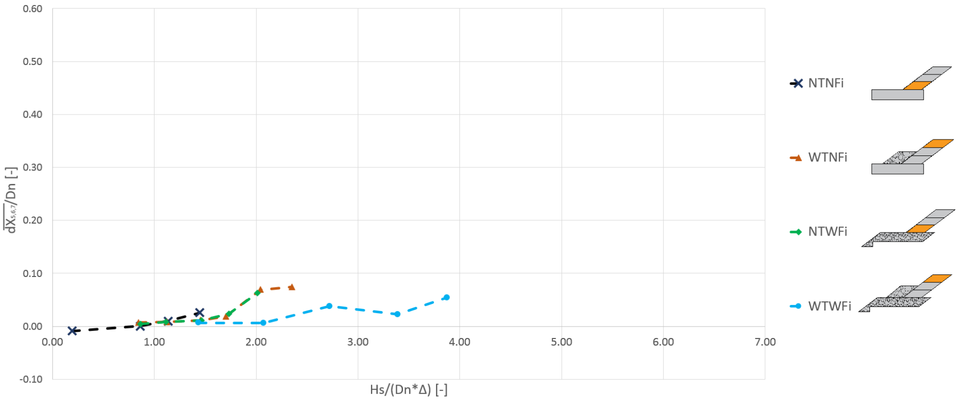

Figure 7.13: Relative movement per wave run of the first row vs. the stability number

7.1.3. DISPLACEMENT IN TIME

The tests of the second series were also filmed from the side. Two of these films were useful for determining when the displacement occurs. These were films that were made during test WTNF4 and NTWF4. A python file was used to measure the displacement as waves run up and down the slope. In this file the displacement of an orange armour unit was determined at 4 locations at every frame of the film. The mean of these 4 locations is showed with the black lines in graph 7.14 and 7.15. The standard deviation of these 4 measurements is shown with a red line.

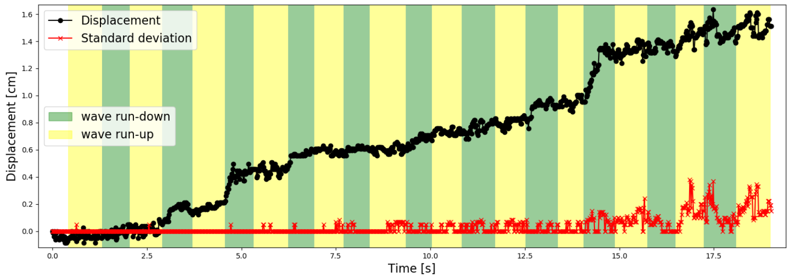

Figure 7.14: Movement of the first row of test WTNF4 plotted against the time

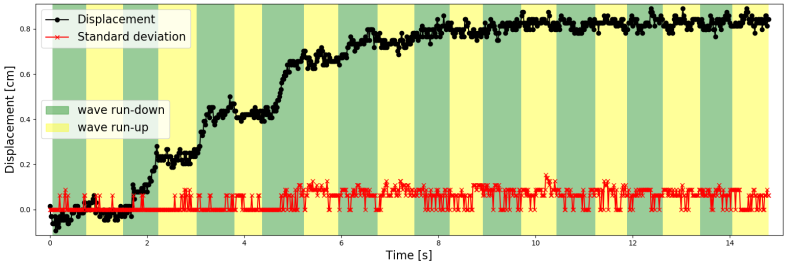

Figure 7.15: Movement of the first row of test NTWF4 plotted against the time

7.2. LOOSE UNIT TESTS

Each test of the first series was done with one loose armour unit in front of the first row. This armour unit stayed in its place during all tests except for the tests with a water depth of 55 cm and one test with a water depth of 60 cm and a wave period of 1.5 s. In all cases the armour units in the front row had already moved a bit before the loose armour unit was displaced. See table 7.7. Calculations show that the highest particle velocity at the bottom didn't occur during the tests with a water depth of 55 cm. In this calculation linear wave theory is assumed, but this probably isn't a good indicator of orbital velocity at this place.

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h [m]	T [s]	u_{bot} [m/s]	Movement?	$N_{unstable}$ $[H/(D_n*\Delta)]$	N_{stable} $[H/(D_n*\Delta)]$
0.55	1.5	0.33	Yes	2.46	-
0.60	1.25	0.36	No	-	4.59
0.60	1.5	0.36	Yes	3.98	-
0.60	2	0.41	No	-	4.23
0.65	1.25	0.29	No	-	4.30
0.65	1.5	0.28	No	-	3.63
0.65	2	0.35	No	-	4.00
0.70	1.5	0.29	No	-	4.13
0.55	1.5	Irregular	Yes	2.03	-

Table 7.7: Movement of the loose armour unit ($D_n = 0.029$)

7.3. PULL-OUT TESTS

The outcome of the pull-out tests is presented in tables 7.8 and 7.9. It is clear from the tests that both slopes add to the resistance to movement of an armour unit as a loose armour unit only needs 0.6*F_s to be displaced.

Slope: 3:4 [-]						
Number of rows	F_p/F_s [-]					Average [-]
5	3.0	3.0	1.8	2.4	2.4	2.5
10	2.4	3.0	3.0	2.4	3.0	2.7
15	2.4	3.0	2.4	2.4	2.4	2.5

Table 7.8: The pulling force divided by the unit weight needed to pull out an armour unit from the first row when an armour slope of 3:4 is applied

Slope: 1:2 [-]						
Number of rows	F_p/F_s [-]					Average [-]
5	1.8	1.8	1.8	1.8	1.8	1.8
10	2.4	1.8	1.8	1.8	1.8	1.9
15	1.2	1.8	1.8	1.8	1.2	1.5

Table 7.9: The pulling force divided by the unit weight needed to pull out an armour unit from the first row when an armour slope of 1:2 is applied

8

ANALYSIS

The analysis of the results is performed by comparing the observations to the expected behaviour according to the theoretical models proposed in chapter 3. The observations are used for validation of the models and subsequently the models are used for interpretation of the observations and the underlying processes.

8.1. TYPES OF LOADING

In all tests the first row was pushed out by the armour layer. Not a single unit was extracted by direct hydraulic loading in any test. This means that indirect hydraulic loading is present. Movement occurs when the wave rushes down the slope, which can be seen in graph 7.14 and 7.15, this confirms the finding that the wave pulls down the armour layer, which is illustrated in figure 3.2. Another argument that the armour layer pushes out the first row is the results of the loose unit tests, as the first row always moved before the loose armour unit was displaced. The fact that the loose unit in some tests moved as well, shows that direct hydraulic loading occurs as well.

8.2. DAMAGE THRESHOLD

A large part of this study relies on assessing the amount of damage (movement) that has been dealt to the first row. Before this research was conducted, a rule of thumb that is used for a damage threshold is $\Delta x/Dn = 0.2$. This research can be used to determine a more substantial damage threshold. Acceptable damage can be defined as the amount of damage that the first row can take before brittle failure occurs, see figure 8.1. In this study brittle failure is defined as an increase in displacement of the first row of at least $0.2D_n$. NTNF2, NTNF4, WTNF4 and NTWF4 show this kind of failure behaviour, see figure 7.8 and 7.10. Other tests seem to not reach the damage threshold. Based on these tests, it can be argued that for an armour layer with a toe berm or foundation layer $\Delta x/Dn = 0.2$ is an appropriate threshold, as NTWF4 and WTNF4 show such failure after $\Delta x/Dn = 0.21$ and $\Delta x/Dn = 0.24$ have occurred respectively. When both a toe berm and a foundation layer are used, high stability numbers were found without reaching brittle failure. The WTFF4

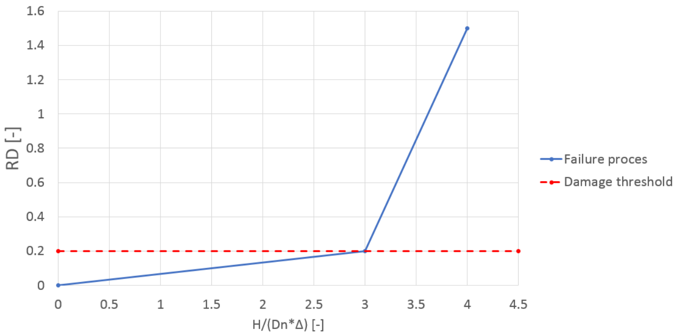

Figure 8.1: Schematized failure proces

test reached $\Delta x/Dn = 0.23$ on a stability number of 4.48. This means that maybe the damage threshold for breakwaters with a toe berm and foundation layer is even higher than $0.2D_n$, but that remains to be proven. Without toe berms and foundation layers less damage is acceptable before brittle failure occurs, so $\Delta x/Dn = 0.1$ is advisory, as NTNF2 and NTNF4 fail after $\Delta x/Dn = 0.11$ has occurred.

8.3. EFFECT OF A FAILING FIRST ROW

There is some difference in the failure behaviour of the first row. When the conditions become more critical, brittle failure starts occurring at lower stability numbers and the damage threshold also reduces, which can be seen clearly in figure 7.3. Most test conditions of the first test series allowed for more than $0.5D_n$, but at the most critical depth (h_f = 0.276), only 0.16 relative armour unit displacement occurred before When this happened, the armour layer started sliding down the slope, exposing the under layer: the breakwater completely collapsed.

8

8.4. INFLUENCE OF HYDRAULIC VARIABLES

In this section the influence of the investigated parameters is analysed. It is important to notice that for each parameter the influence is examined on the two theoretical models that are proposed in Chapter 3.

8.4.1. EFFECT OF WAVE BREAKING

Wave height has the largest influence on the stability of the first row, this is why it is included in the stability number. The wave height can influence the stability of the first row in two ways, a direct and an indirect way. The direct hydraulic loading increases with higher wave heights. In graph 7.14 and 7.15 it can be seen clearly that movement occurs when the wave is running down the slope, indicating the important role of indirect loading. Higher wave heights cause a greater down rush, it seems like this creates a larger pull on the armour layer. This larger pull gives a higher indirect hydraulic loading on the first row. This is the case until a] the waves become so high that depth-induced breaking occurs (tests were stopped when waves started to break, as no increase in damage was found), or b] (in case of testing with a constant wave period) the waves become steeper with growing wave height and start to plunge on the breakwater, which greatly reduces the indirect hydraulic loading, see figure 7.1.

8.4.2. WATER DEPTH

Comparing the deepest three water levels from figure 7.3 and table 7.3 with the shallowest water level ($h_f = 0.276m$) it can be seen that complete failure happens at the lowest stability number at $h_f = 0.276m$. This can be explained in two ways. The first way is that the direct hydraulic loading on the first row is higher. This theory is supported by the loose armour unit as it starts moving when the water depth decreases. Still, at that moment the armour layer has already moved. The second way is that the indirect hydraulic loading on the first row is higher, as the still water level is closer to the first row, the down rush comes closer to the the first row.

8.4.3. WAVE PERIOD

In table 7.1 and table 7.2 it can be seen that wave period has a clear influence on the stability of the first row. Longer waves have a higher orbital velocity, resulting in a higher direct loading on the first row. Longer waves also mean that the water is displaced more, leading to more water rushing up the slope, which in turn leads to more water rushing down the slope. This will lead to a higher indirect loading as well.

8.4.4. WAVE STEEPNESS

Wave steepness has a clear influence on the stability of the first row as plunging waves fail to completely destroy the breakwater, see figure 7.1. Plunging waves have less down rush than surging waves and thus have less pull on the armour layer.

But even when the waves have a wave steepness that result in surging waves, wave steepness has an influence on the stability of the first row. According to the second test series $s_0 = 4\%$ gives a lower stability than $s_0 = 2\%$. This can be seen in figure 8.2 and 8.3. Still, it can be seen that the difference is rather small at acceptable damage levels ($RD < 0.2$). At higher damage levels the difference becomes more pronounced though. Furthermore it is clear that brittle failure occurs at lower wave heights for a wave steepness of 4%: brittle failure didn't even occur at the WTNF2 and NTWF2 tests, while it did in the WTNF4 and NTWF4 tests.

The problem is that according to the first test series, the relation between wave steepness and stability is completely opposite. When looking at figures 7.1 and 7.2, it is clear that longer waves (having a lower wave steepness at comparable wave heights) have a lower stability. The tests with $T = 2s$ have wave steepnesses ranging from 1% at the beginning of the test to 2% at the end of the test, while tests with $T = 1.5s$ have wave steepnesses of 2% to 5%. All these waves are surging, but the test with longer waves cause more displacement of the first row.

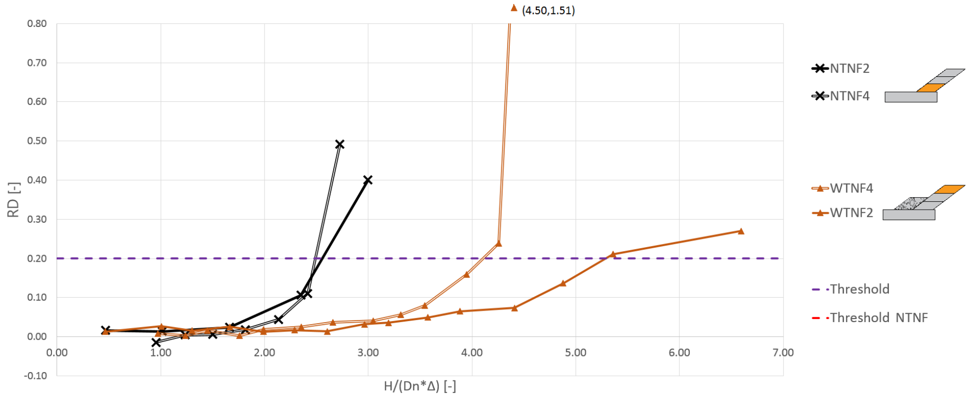

Figure 8.2: Comparing the effects of the wave steepness on NTNF and WTNF

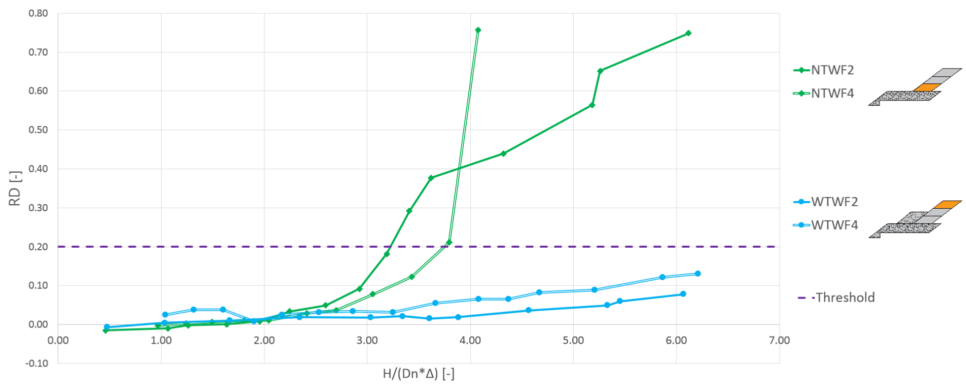

Figure 8.3: Comparing the effects of the wave steepness on NTWF and WTWF

Configuration	NTNF2	NTNF4	WTNF2	WTNF4
N_s $(\Delta x/D_n = 0.05)$	1.88	2.16	3.59	3.20
Percentage	100	115	100	89
N_s $(\Delta x/D_n = 0.1)$	2.30	2.37	4.61	3.65
Percentage	100	103	100	79
N_s $(\Delta x/D_n = 0.2)$	2.56	2.49	5.13	4.11
Percentage	100	97	100	80
N_s Brittle failure	2.35	2.13	NO	4.26
Percentage	100	91	-	-

Table 8.1: Comparison of the effect of wave steepness on stability numbers of NTNF and WTNF

Configuration	NTWF2	NTWF4	WTWF2	WTWF4
N_s ($\Delta x/D_n = 0.05$) Percentage	2.60 100	2.81 108	5.35 100	3.58 67
N_s ($\Delta x/D_n = 0.1$) Percentage	2.95 100	3.24 110	NO -	5.43 -
N_s ($\Delta x/D_n = 0.2$) Percentage	3.23 100	3.75 116	NO -	NO -
N_s Brittle failure Percentage	NO -	3.80 -	NO -	- -

Table 8.2: Comparison of the effect of wave steepness on stability numbers of NTWF and WTWF

8.5. INFLUENCE OF STRUCTURAL VARIABLES

8.5.1. TOE BERM

For the test with $s_0 = 2\%$ at three three mentioned damage levels the average improvement of stability was 97% (std=5.5%). For $s_0 = 4\%$ the average improvement was 56% (std=8.5%). For irregular waves the average improvement was 17% for the lowest two damage levels (std=4%). An additional effect of adding a toe berm is that the damage threshold increases, this can be seen in figure 7.8 and 7.10. NTNF4 shows brittle failure when $\Delta x/Dn = 0.11$ is reached, while WTNF4 only shows brittle failure when $\Delta x/Dn = 0.24$ is reached. This means a 115% increase. WTNF2 doesn't even show brittle failure.

8.5.2. FOUNDATION LAYER

For the test with $s_0 = 2\%$ at the three mentioned levels the average improvement of stability was 31% (std=6.5%). For $s_0 = 4\%$ the average improvement was 39% (std=10.5%). For irregular waves the average improvement was 19% for the lowest two damage levels (std=3%). An additional effect of adding a foundation layer is that the damage threshold increases, this can be seen in figure 7.8 and 7.10. NTNF4 shows brittle failure when $\Delta x/Dn = 0.11$ is reached, while NTWF4 only shows brittle failure when $\Delta x/Dn = 0.21$ is reached. This means a 91% increase. WTNF2 doesn't even show brittle failure.

The rock size of the foundation layer does influence the stability, as a test with a much smaller foundation layer and a toe berm resulted in a 10% lower stability for the first row at a damage level of $0.1D_n$ compared to the case with a normal foundation and a toe berm.

8.5.3. TOE BERM AND FOUNDATION LAYER

For the test with $s_0 = 2\%$ at the improvement of stability was 184%. In this test, only the lowest damage level was reached. For $s_0 = 4\%$ the average improvement was 97% for the lowest two damage levels (std=44%). For irregular waves the average improvement was 100% for the lowest two damage levels (std=30%). The stability is increased so much is because the foundation layer results in toe berm that is much more stable. During testing it became quite clear that more stones of the toe berm washed away at comparable wave heights when no foundation layer was present. The toe berm stones become more stable because there is interlocking between the foundation layer and the toe berm. This effect can be seen by comparing the figures below.

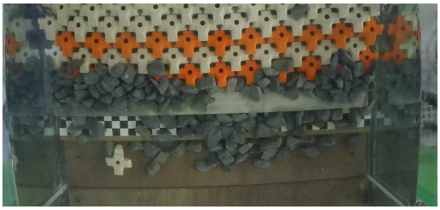

Figure 8.4: Damage of the toe berm at $H/D_n * \Delta = 4.36$, $\Delta x/D_n = 1.10$

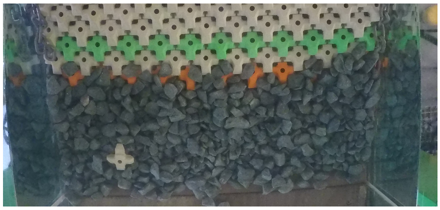

Figure 8.5: Damage of the toe berm at $H/D_n * \Delta = 4.8$, $\Delta x/D_n = 0.10$

8.6. DESIGN FORMULA

The results found in this study can be used to determine a design formula. Appendix F elaborates on how the design formula is determined. The design formula that is found, is:

$$\begin{aligned} \frac{H}{\Delta D_n} &= 2.59 f_s f_w \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ \frac{H}{\Delta D_n} &= f_s f_w \left(0.3 \frac{h_f}{D_n} - 0.26\right) \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{8.1}$$

With factors f_s and f_w being:

Configuration	f_s [-]	f_w [-]
No toe berm, no foundation layer	1	0.6
Toe berm, no foundation layer	1.5	0.6
No toe berm, foundation layer	1.3	0.6
Toe berm, foundation layer	2.1	0.7

Table 8.3: Overview of factors to adjust the design formula

This formula reasonably predicts the stability number of the tests at the damage threshold, see figure 8.6. Unfortunately the influence of wave steepness couldn't be integrated in the formula, as the two test series show a contradictory influence of wave steepness on the stability.

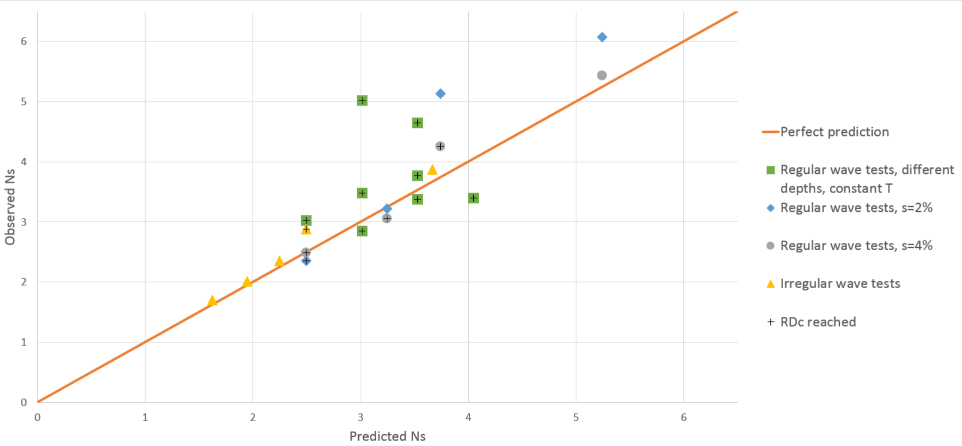

Figure 8.6: Predicted N_s compared to the tested N_s for all tests

8

9

DISCUSSION

9.1. RELIABILITY OF THE STUDY

Uncertainties are mainly related to wave measurements, armour movement measurements, the model set-up and the test program.

- Wave measurements: The measurements of the wave heights of the regular wave tests near the structure seem incorrect, so the near-shore wave height is computed by multiplying the off-shore wave heights with a shoaling factor. This is inconvenient as it gives a good estimation of the wave height at the toe of the breakwater, but still some uncertainty remains as the real wave height is unknown.

- Armour movement measurement: The reliability of the measuring technique is quite high, when all damage characterisation criteria are considered (see De Almeida *et al.* [17]). The method has a low random error, as the maximum inaccuracy that was found, was only 7% of critical displacement. This inaccuracy is an absolute error. The displacements that are measured from the third row are less reliable however, as these have an added relative uncertainty of 10%. The method has no bias when the first row can be photographed directly and low bias when the third row is used, but this bias can be corrected for. The damage range can be distinguished very well: start of damage, intermediate damage and failure can all be distinguished clearly. Different structures can be analysed as well.

- Model set-up: The wall-effect had a large impact on the displacement of the outer armour units. This is why only the middle three units are used in the processing of the results, it is likely that the wall-effect doesn't influence the displacement there, as the displacement for these units is roughly the same. Tests with a wider flume could confirm this.

- Test program: The reliability of the results is hard to quantify as tests aren't repeated.

All of this means that the numbers presented in this report can only be used for an indication of stability, and to recognize what parameters influence stability. The numbers are not really suitable for determining design formula, although a presumption for a design formula can be proposed. The results across the different tests are quite consistent, indicating the reliability is good enough to justify the conclusions.

9.2. Influence of hydraulic variables

In this section the influence of certain investigated parameters are discussed.

9.2.1. Wave height

Both direct and indirect hydraulic loading increase when wave height increases. Based on the tests it's impossible to say which one is dominant. It should be noted however that when the waves start plunging (and the indirect load decreases due to reduced down-rush), the waves fail to completely destroy the breakwater, indicating the important contribution of indirect loading.

9.2.2. Water depth

Both direct and indirect hydraulic loading increase when water depth decreases. Based on the tests it's impossible to say which one is dominant.

9.2.3. Wave steepness

It is remarkable that the results of the two test series contradict each other with respect to the influence of wave steepness on stability. It is hard to come up with an explanation, as the only difference between the two tests series is that in the first one the wave steepness gradually built up and in the other it was held constant. It isn't likely that such a difference would explain such a different outcome. Further study would be needed to find the explanation.

9.3. Indirect loading

In this study it is concluded that the indirect load is very important for the stability of the first row. The indirect load on the first row can be caused in a few ways, which are discussed in this paragraph. Based on this study, it can't be determined if one is dominant and if so, which one is dominant.

9.3.1. Drag force

When the wave rolls down the slope, the water has a certain velocity. This creates a drag force on the armour layer, pulling it down and pushing the first row out. If this is the case, then run-down is an important parameter. Before this study was conducted, run-down wasn't recognized as an important parameter. Van der Meer even reported: 'Run-down often is less or not important compared to wave run-up' [18]. The Rock Manual gives a formula from Van der Meer to calculate wave run-down, but has no recommendation to use it for stability calculations [19]. In this study however, it is found that run-down is important. This means that understanding which parameters influence run-down, will give understanding in how much the first row is loaded. The formula of Van der Meer in the Rock Manual includes wave steepness, armour slope and structure notional permeability: $\frac{R_{d2\%}}{H_s} = 2.1\sqrt{\tan\alpha_b} - 1.2P^{0.15} + 1.5\exp(-60s_{om})$. The first two parameters are recognized in this study as important for wave run-down. The third isn't subject to investigation in this study.

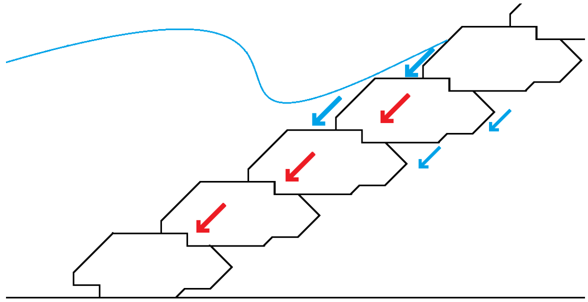

Figure 9.1: Drag force visualized. Blue arrows indicate the movement of the water particles. Red arrows indicate forces in the armour layer.

9.3.2. UPLIFT

Uplift can be caused in two ways. The first is when the wave rolls up the slope, water seeps into the breakwater. When the wave then rolls back down the slope, the water flows out again. This causes an uplift force on the armour layer, potentially increasing the weight on the first row, as it is then less supported by the under layer. The second way is that when the wave rolls down the slope, water flowing outside the armour layer flows faster then water flowing inside. This creates a pressure gradient, resulting in uplift. Larger down-rush and higher down-rush velocities will result in higher uplift pressures.

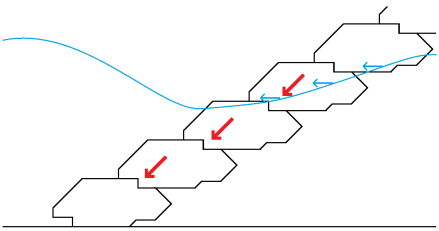

Figure 9.2: The first uplift force visualized. Blue arrows indicate the movement of the water particles. Red arrows indicate forces in the armour layer.

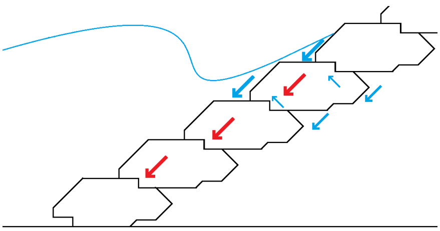

Figure 9.3: The second uplift force visualized. Blue arrows indicate the movement of the water particles. Red arrows indicate forces in the armour layer.

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9.4. INFLUENCE OF STRUCTURAL VARIABLES

9.4.1. TOE BERM

The improvement in stability can be explained as the toe berm increases the resistance of the first row to movement. The first row is pushed out by the armour layer, so adding a toe berm adds more weight to the first row, resisting the pushing force of the armour layer. An additional effect of adding a toe berm is that the damage threshold increases. This effect might be caused by the fact that the toe berm keeps resisting movement, even when the first row itself has moved so much that compared to the case without a toe berm, the first row would have slid away.

9.4.2. FOUNDATION LAYER

The most probable explanation for the improvement in stability of the first row by the foundation layer is that resistance is increased. As the first row is pushed out by the armour layer, supporting the first row with a foundation layer gives more grip. Even when it slides away, it can still find support on the rough surface of the foundation layer. It is possible that the foundation layer reduces the direct loading on the first row, as water particles are now able to move through the foundation layer as well. It isn't possible to determine how big this effect is, as the water velocity isn't measured at the toe structure.

9.4.3. TOE BERM AND FOUNDATION LAYER

The stability is increased so much because the foundation layer makes the toe berm much more stable. The toe berm stones become more stable because there is interlocking between the foundation layer and the toe berm.

9.5. COMPARISON WITH LITERATURE

In chapter 2 it was noted that according to Delta Marine Consultants the design value of an XblocPlus armour layer $N_s = 2.5$. In the light of this study it can be said that the stability is much higher, even as high as $N_s = 3.87$, as long as both a toe berm and a foundation layer are used. The last measure isn't common practice when a breakwater is built on a rocky seabed. This should be avoided at any time, as this is detrimental to the stability of the breakwater, lowering the stability to $N_s = 1.97$.

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In chapter 2 it was also noted by Hofland and Van Gent [6] that for other armour unit types a displacement of $0.2D_n$ was critical and would lead to armour unit extraction. In this study $0.2D_n$ is also found to be the damage threshold, but armour unit extraction didn't occur. It was however a threshold after which brittle failure (displacement increase of $0.2D_n$) could occur. This is very dangerous for the breakwater as the first row loses all its stability and the whole armour layer can come sliding down.

9.6. OVERALL OR FIRST ROW STABILITY?

It might seem like overall stability is investigated in this report, as the whole armour layer can slide down if the first row is pushed too far out. Also, this study shows that the armour layer contributes to the instability of the first row. Still, it is important to note that the subject of the whole study is the first row. That the first row fails, and makes the whole armour layer fail, doesn't mean that other mechanisms can't make the armour layer fail before the first row fails. This isn't subject of this study however, so it can't be stated that the stability of the first row equals the stability of the complete armour layer. The only thing that can be stated is that when the first row fails, the armour layer will fail as well, so it needs proper attention.

9.7. SUMMARY

Based on this research, it can be concluded that of the two simplistic models mentioned in chapter 3 the second one, the one including the indirect hydraulic loading, is best applicable. This means that the down-rush of a wave is partly responsible for pulling down the armour layer. The armour layer pushes out the first row in this way. If water depth is lower, the down-rush gets closer to the first row and the load on the first row increases, until depth-induced breaking occurs. Waves that are too steep will start to plunge, greatly reducing down-rush and thus greatly reducing the load on the first row.

An effective way to increase the stability of the first row is to add a toe berm and a foundation layer. The toe berm functions as extra weight on the first row to keep this row in its place. The foundation layer increases the stability mainly by interlocking the toe berm but also by improving the resistance to sliding of the first row.

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Conclusion

The goal of this study is to gain insight in the hydraulic stability of the first row of an XblocPlus armour layer. To reach this goal, the research questions are answered:

1. *What types of loading occur that cause the first row to fail?*

   Waves can load the first row in two ways. Waves can load the first row directly, as the orbital velocity of the water particles around the first row can drag it out. Waves can also load the first row indirectly: when the waves roll down the slope, uplift and drag forces are created, resulting in an armour layer that pushes the first row out. Both of these types of loading are working on the first row. This is represented in model 2 of chapter 3, it is visualised again in figure 10.1. Based on the tests it can be stated that indirect loading is important, as the complete first row is pushed out, and not the least stable armour unit. Also, when waves start to plunge, the breakwater won't be destroyed. Furthermore it can be clearly seen in test results that movement happens during down-rush. Also, if direct loading would be dominant, a toe berm wouldn't be effective, as toe berm stones have less resistance to direct hydraulic loading than armour units, as they have considerably less weight.

2. *What level of damage to the first row is critical for the stability of the entire armour layer?*

   Critical damage is defined as the amount of damage the first row can sustain just before brittle failure occurs. Brittle failure is defined as a damage increase of $0.2D_n$. In the tests, this appeared to be dangerous for the whole breakwater, as when brittle failure had occurred, the first row lost its stability and therefore the whole armour layer came sliding down. Based on these tests, it can be argued that for an armour layer with a toe berm and/or foundation layer $\Delta x/D_n = 0.2$ is an appropriate threshold. Without a toe berm and foundation layer less damage is acceptable, so $\Delta x/D_n = 0.1$ is advisory.

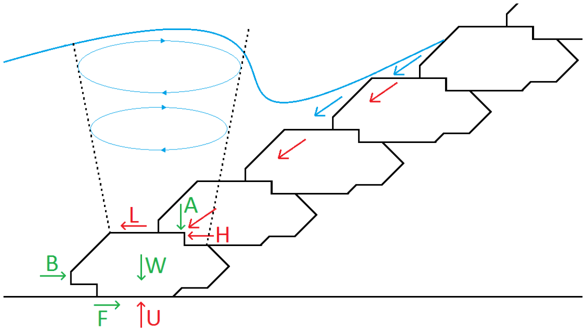

Figure 10.1: Balance of forces of the second model. Green arrows indicate stabilizing forces, red arrows indicate destabilizing forces. Blue arrows indicate the movement of the water particles. For a more complete description, see chapter 3

.

3. *In what way do wave height, water depth and wave steepness influence the hydraulic stability of the first row and can they be combined in a stability formula?*

   Higher waves will result in a lower stability of the first row, as higher waves give a larger down-rush and a higher orbital velocity, increasing the load. When waves become so high that they start to break, the down-rush becomes less, decreasing the load on the first row.

   A lower water depth will result in a lower stability of the first row. This could be caused by the fact that with a lower water depth, the down-rush gets closer to the first row. This gives the armour layer less space to dissipate the excess forces into the under layer. When the water becomes to shallow however, the waves will break before they get to a critical height.

   Based on this study, the influence of wave steepness can't be determined, as the results of the two test series are contradictory. The first test series (containing tests with constant wave period) showed a lower stability for longer (and thus less steep) waves, while the second test series (containing tests with a constant wave steepness) showed a lower stability for steeper waves.

   It can be stated however that when waves become so steep that they start to plunge, the load on the first row is reduced a lot. Plunging waves have less down-rush. This is seen in the results of this study, conforming the important role of indirect loading.

   The following stability formula has been derived from the tests, but it has to be clarified that more tests are needed to validate this formula and improve it.

$$\begin{aligned} \frac{H}{\Delta D_n} &= 2.59 f_s f_w \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ \frac{H}{\Delta D_n} &= f_s f_w \left(0.3 \frac{h_f}{D_n} - 0.26\right) \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{10.1}$$

10

With factors f_s and f_w being:

Configuration	f_s [-]	f_w [-]
No toe berm, no foundation layer	1	0.6
Toe berm, no foundation layer	1.5	0.6
No toe berm, foundation layer	1.3	0.6
Toe berm, foundation layer	2.1	0.7

Table 10.1: Overview of factors to adjust the design formula

4. *In what way does a foundation layer influence the hydraulic stability of the first row?*

   A foundation layer increases the resistance of the first row. It has more friction than a concrete foundation, also when the first row has displaced already, resulting in a higher damage threshold. For irregular waves the increase in stability is approximately 20%.

5. *In what way does a toe berm influence the hydraulic stability of the first row?*

   A toe berm increases the resistance of the first row as it adds weight to the first row, increasing the friction forces. The damage threshold also rises, as the toe berm keeps resisting movement, even when the first row has moved so much that without the toe berm the first row would have slid away. For irregular waves the increase in stability is approximately 20%.

   When both a foundation layer and a toe berm are used, the effects increase each other, as the toe berm makes the foundation layer more stable, and the toe berm is made more stable by the foundation layer, as the stones have a lot more interlocking. This results in a first row that is a lot more stable then a first row without a toe berm and/or a foundation layer. For irregular waves the increase in stability is approximately 120%.

6. *What is the stability of the first row?*

   In this study only one test with irregular waves, a toe berm and a foundation layer was performed, so the stability that was found can only be used as an indication of stability of the first row. Still, the stability number $N_s = \frac{H_s}{\Delta D_n} = 3.87$ was found with a damage of only $0.13D_n$. This means that the real stability could be even higher, but that remains to be proven. When only a toe berm or a foundation layer is used, the stability is reduced to $N_s = \frac{H_s}{\Delta D_n} = 1.97$ (based on one test per configurations, allowing for a conservative amount of damage, $0.1D_n$).

11

Recommendations

Using the experience gained from testing, the tests can be improved. Next time, the first test series next time should be tested with constant wave steepness during a test and more depths can be investigated. The tests with irregular waves should have been continued with higher significant wave heights. Also, every parameter mentioned in this study (wave steepness, toe berm and foundation layer specifics, breakwater and foreshore slope, breakwater height, permeability of the structure) can of course be investigated with physical model testing. But as this study can be seen as a first exploration into first row stability, certain aspects in this study look promising enough that it can be considered feasible to conduct a focused follow-up research to find a design formula for first row stability and get a better understanding of the processes that influence first row stability. A more general recommendation is to research the influence of wave steepness more thoroughly.

11.1. Design formula

In order to arrive at a useful design formula the following steps are needed:

1. Tests should be conducted where the influence of water depth and wave steepness is investigated more thoroughly.
2. Tests should be conducted where more armour units can be placed next to each other, in order to ensure that the wall-effect is not influencing the results.
3. Tests should be repeated a number of times. This will give great insight in the spreading of the results and the accuracy of a design formula obtained from those results.

11.2. Understanding of first row stability

The first row stability is determined by the loading and the resistance. In this study it is found that the loading is largely determined by the down rush of the waves. The resistance is determined by the weight of the first row and the toe berm and the interlocking between the toe berm and the foundation layer. Already a lot of research has been conducted to investigate toe berm stability, so it is recommended to research down-rush. Can the amount of water running down the slope, the run-down velocity or the distance between max run-down and the first row be related to first row stability?

11.3. The influence of wave steepness

This study has found contradictory results for the influence of wave steepness on first row stability. The conventional method of testing, using tests with constant wave steepness, was used in the second test series, which indicated a negative influence of wave steepness on stability. The first test series consisted of tests with constant wave period (meaning the wave steepness gradually increased during the test), finding a positive influence of wave steepness on stability. It would be beneficial to repeat the tests in order to see if the results can be replicated and to try and find an explanation.

A

LITERATURE STUDY

A.1. BALANCE OF FORCES

A.1.1. BALANCE OF FORCES ON AN ARMOUR UNIT

The forces acting upon an armour unit determine whether the armour unit will move or not. This means that understanding of these forces is important. Brebner and Donnelly [3] mention the following forces acting on rocks of a rubble mound breakwater:

- Self weight of the armour unit (F_g);
- Weight of armour units resting on the armour unit (One of the smaller arrows);
- Friction forces (One of the smaller arrows);
- Drag force (F_d);
- Lift force (F_l);
- Inertia force (F_i);
- Seepage force (F_s)

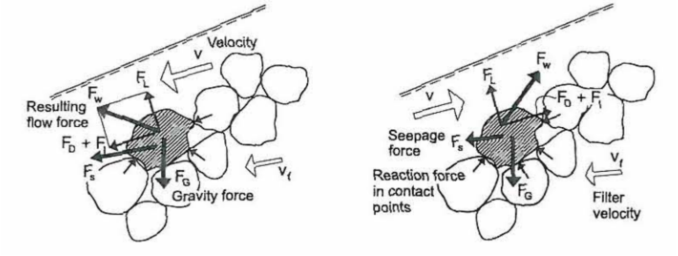

Figure A.1: Illustration of forces on an armour stone (from Hald [20])

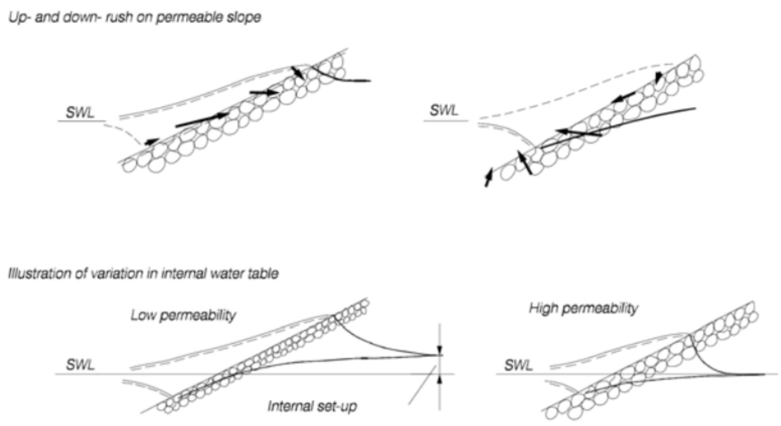

Figure A.2: Influence of permeability on the run-up and internal waterlevel (from Burcharth [21])

A valuable stability analysis based purely on physics is very hard because these forces are dependent on each other and are influenced or caused by processes that show a dynamic and turbulent behaviour. Brebner and Donnelly [3] tried to derive a function for the stability number of a rubble mound foundation, but they needed a lot of simplified assumptions and end up with an equation which contains a number of unknown coefficients. The validity and usefulness of such a formula is questionable, so this is why Brebner and Donnelly resorted to physical modelling.

A.1.2. FORCES ON THE BOTTOM ROW

The research of van de Koppel *et al.* [4] was aimed at gaining insight in these forces for Xbloc armour units by doing physical modeling, but his research showed that making a prediction of the loads based on the influencing parameters isn't very reliable. This is why the stability of armour units is derived by physical modelling. He did gain a lot of insight in the forces loading the bottom row this way. He found that the governing loads are the static and the mean wave load (wave averaged dynamic load). The static load is caused by the weight of the rows on top of the bottom row and this weight increases with each row up until the tenth row, after which the static load becomes a constant value. The mean wave load depends mostly on the wave height. Other influencing parameters are the permeability of the core, the smoothness of the underlayer and the steepness of the breakwater. The peak load occurs at the downwash and is also related to the wave height. Still, peak loads are significantly smaller than the static and mean wave load. The peak load occurs at the downwash because at this moment the water is flowing out of the breakwater (this is illustrated in Figure A.2), creating a drag force on the armour units in such a way that units can be dragged out of the breakwater. The permeability of the foundation layer might play a role as this can reduce the outflow of water in the bottom row. The amount of downwash is greatly influenced by whether a wave breaks or not, breaking waves have less downwash than non-breaking waves. The slope of the breakwater affects how the wave breaks on the slope. The breaker-type determines how much energy is dissipated and in that way also influences the load on the bottom row. Breakwaters built with interlocking concrete armour units are generally applied at slopes which result in surging waves. The static, mean wave and peak loads make up respectively 30%. 60% and 10% of the total load. Van de Koppel found an average load of approximately 3.7 times the unit weight in a design storm and a max load of 5.6 times the unit weight.

One important note has to be placed at Van de Koppels research: he measured the forces at the bottom row, but this row wasn't touching the seabed. It appears that Van de Koppel tried to have the bottom row at maximum run down, in order to be able to measure the highest forces during downwash. For this report this isn't considered a representative case.

A.1.3. ARMOUR LAYER STABILITY

Single layer concrete armour unit stability is commonly described by the stability number $N_s = c$, with c being a constant. This concept was introduced by Van der Meer [5]. Damage of XblocPlus armoured slopes starts at stability number of 3.5, but a design value of $N_s = 2.5$ is recommended by Delta Marine Consultants [2]. The XblocPlus stability is reduced in case of a steep foreshore, a low crested structure or a low core permeability [2].

Still, by rating the stability of an interlocking armour unit only by dividing the wave height by the weight of the block, one of the key aspects of this type of armour is ignored, namely interlocking. Van Zwicht [14] showed that steeper slopes rely more on interlocking for stability and flatter slopes rely more on the weight of an armour unit. The effect of interlocking and friction are hard to quantify however and this is not the focus of this research, so this report will use the existing stability formula for XblocPlus.

A.1.4. FAILURE FIRST ROW

The experiments upon which Delta Marine Consultants based the design stability number were performed with a fixed toe. Since then more experiments have been performed to gain more insight in the performance of the XblocPlus. Of those experiments, two were performed with a representative toe structure. Rada Mora [22] and Jiménez Moreno [23] did tests with a toe berm stability number of 4.1. They assumed that the start of damage was acceptable to the toe berm and didn't report failure of the first row.

The first row can fail in two ways. The wave can pull out an armour unit of the first row, or the armour layer can push out the first row. The first phenomenon is detected by assessing whether armour units have been extracted. The second phenomenon is described by Hofland and Van Gent [6]. In this paper Hofland and Van Gent describe how the second failure mechanism can be detected by an image processing technique. Parameters which influence this second phenomenon are the wave height, the water depth, number of rows, the breakwater slope and possibly the rock size of the foundation layer, the wave length and the wave steepness.

A

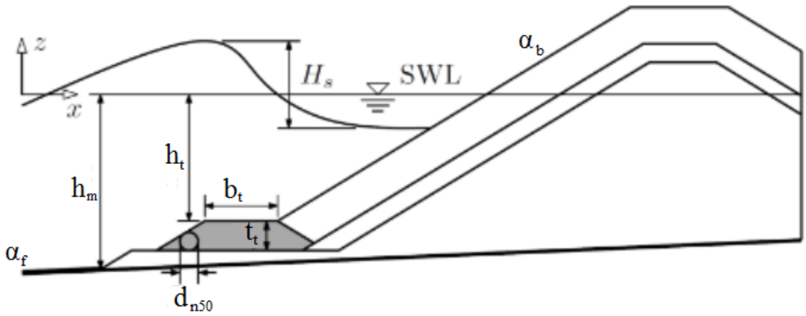

Figure A.3: Definition of toe berm geometry and adjacent slopes (from Muttray *et al.* [12])

A.2. TOE BERM STABILITY

A.2.1. INTRODUCTION

Although the toe berm isn't the main point of interest of this study, a lot of research has been done to investigate the processes that are influencing the stability of the toe berm. These processes will also influence the stability of the first row of XblocPlus as well. This means that a lot can be learned from studying this research.

A.2.2. GENERAL

In this literature study only research on rubble mound breakwaters tested by irregular wave fields are reviewed. Gerding [7] did physical model tests and tried to find a formula based from his results. The formula he derived from his research is:

$$\frac{H_s}{\Delta d_{n50}} = \left(0.24\left(\frac{h_t}{d_{n50}}\right) + 1.6\right) N_{od}^{0.15} \qquad \text{for} \quad 0.4 < \frac{h_t}{h} < 0.9 \quad \text{and} \quad 3 < \frac{h_t}{d_{n50}} < 25 \qquad \text{(A.1)}$$

This formula implies that stability depends linearly on the depth of the toe as a stabilizing factor. The effect of rock size is inconsistent, as it is stabilizing on the left hand side and destabilizing on the right hand side. The acceptable damage has a minor influence, as it has a power of 0.15. Furthermore the acceptable damage is physically incorrect, as zero damage results in zero stability.

Docters van Leeuwen [8] did more tests to validate Gerdings research and tried to extend its validity by varying the density of the rocks used. Van der Meer [24] re-analysed the formula of Gerding and came up with the following:

$$\frac{H_s}{\Delta d_{n50}} = \left(6.2\left(\frac{h_t}{h}\right)^{2.7} + 2\right) N_{od}^{0.15} \qquad \text{for} \quad 0.4 < \frac{h_t}{h} < 0.9 \quad \text{and} \quad 3 < \frac{h_t}{d_{n50}} < 25 \qquad \text{(A.2)}$$

This formula has the same problem for the acceptable damage as the formula of Gerding. Furthermore it is strange that an increasing local water depth has a destabilizing effect, this is counter intuitive and is probably done in order to get a dimensionless result.

The formula of Van der Meer is often used in design for toe berms, as well as toe berms that are designed for a breakwater built with XblocPlus [2].

Ebbens [9] confirmed the formula of Van der Meer with his own tests (although he used Xblocs in stead of double layered rock), while further investigating some (extra) parameters. He also derived a formula for very shallow water:

$$\frac{H_s}{\Delta d_{n50}} = 3\frac{N_{\%}^{1/3}}{\sqrt{\xi_{0p}}} \quad \text{for} \quad \frac{h_m}{H_s} < 2$$
$$\text{and} \quad N\% = 5 \quad \text{for} \quad s_{0p} = 0.01 \quad \text{and} \quad N\% = 10 \quad \text{for} \quad s_{0p} = 0.035 \tag{A.3}$$

Muttray [10] remarks that all the previous studies are purely experimental and lack physical support. The influence of these shortcomings can be seen by the large scatter when the predictions by the design formulas are plotted against the test results. Muttray tries to find a formula from an Izbash type of approach, by finding a ratio between the driving force (the flow induced drag force) and the main retaining force (the weight). This results in the following formula:

$$\frac{H_s}{\Delta d_{n50}} = \left(\frac{2.4N_{od}^{1/3}}{1.4 - 0.4h_t/H_s}\right) \quad \text{for} \quad h_t < 3H_s \tag{A.4}$$

Muttrays formula appears to be more accurate and have a better physical basis than the formulas of Gerding and Van der Meer. The acceptable damage has a minor influence, with a power function of 1/3.

Van Gent and Van der Werf [11] also tried to find a physical basis for a design formula and found that a prediction method based on the orbital (characteristic) velocity was most promising. It is remarkable however that the orbital velocity was calculated in deep water (using linear wave theory), irrespective of the real situation. Other (more complex) estimates of the orbital velocity (in other words, taking into account non-linear effects caused by shallow water) didn't result in better predictions of the toe stability. Furthermore they investigated the influence of berm width and thickness, breakwater slope, slope roughness and the type of toe berm blocks (e.g. natural rock or concrete blocks). This resulted in the following formula:

$$\frac{H_s}{\Delta d_{n50}} = \left(0.032\frac{t_t}{H_s}\left(\frac{B_t}{H_s}\right)^{0.3}\frac{\hat{u}_\delta}{\sqrt{gH_s}}N_{od}\right)^{1/3}$$
$$\text{for} \quad 1.2H_s < h_m < 4.5H_s \quad \text{and} \quad 0.1h_t < t_t < 0.3h_t \quad \text{and} \quad 1/1.5 < \alpha_b < 1/2 \tag{A.5}$$

where $\hat{u}_\delta = \frac{\pi H_s}{T_{m-1.0}}\frac{1}{\sinh kh_t}$ and $k = \frac{2\pi}{L_{m-1.0}}$

Muttray *et al.* [12] analysed the validity of equation A.2, A.4 and A.5 by using them to predict all of the test results (rock grading, toe berm stability and damage). This is possible because the test conditions were reasonably comparable. It turned out that the formulas were reasonably capable of predicting the test results that were used to derive the formulas, but not for all of the test results, indicating a lack of validity. The formulas turned out to be specifically faulty in predicting the damage (see Figure A.3). Muttray suggested some explanations for these findings, the main one being that the formulas contained dependent variables. The main parameters of toe stability: wave height and

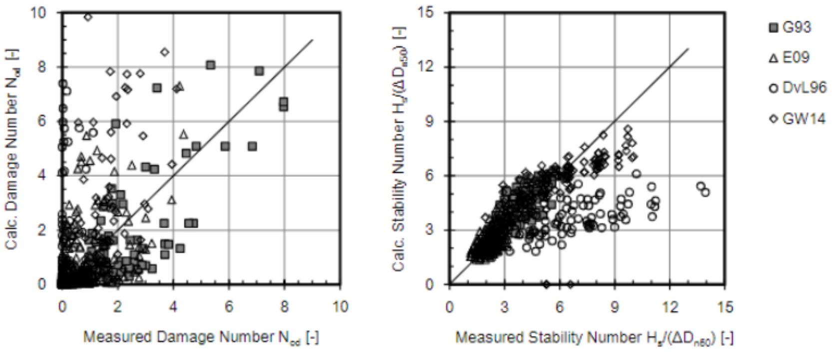

Figure A.4: Predicted damage (left) and stability number (right) according to equation A.2 plotted against experimental results (from Muttray *et al.* [12])

the depth of the toe, appeared to be dependent. This could be caused by depth limited conditions, but also deeper conditions confirmed this dependency. Muttray suggests the dependency could be caused by the choices of the experimenters in their model setup and test program. After this observation, he derived a new formula:

$$\frac{H_s}{\Delta d_{n50}} = \left(1.8 + \frac{h_t}{L_p} m\right) N_{od}^{1/3} \tag{A.6}$$

It is remarkable that although pointing out the interdependence of h_t and H_s, Muttray still includes these parameters in his formula. Furthermore, this formula is physically incorrect as the stability number becomes zero if the acceptable damage is zero.

Their research gives insight in influencing parameters the stability of the toe berm. The influencing parameters have been listed below and per parameter the insights of each research have been listed.

A.2.3. PARAMETERS OF INFLUENCE

WAVE STEEPNESS, s_{0p}

Wave steepness is an important parameter when determining how a wave breaks. This is a very important parameter for the stability of armour units around the water line, where the breaker-type has a big influence on the load. The toe structure however is protected by the water on top of it, making the breaker type a lot less important. This is why wave steepness is expected to have little influence on the stability of the toe berm.

Gerding [7] concludes from his research that s_{0p} has no clear influence on stability of the toe berm.

Van der Meer [24] confirms the findings of Gerding.

Ebbens [9] notes that s_{0p} does have influence on the stability of the toe berm, he finds that higher wave steepness results in less damage. The influence of wave steepness is noticed to be dependent on whether the waves are in shallow or deep water. He doesn't incorporate this in a formula though, probably because not enough tests were executed to draw clear conclusions.

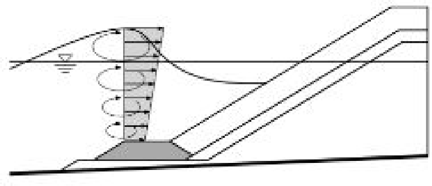

Figure A.5: Motion of the water particles visualized (from Muttray *et al.* [12])

Muttray [10] concludes that wave steepness seems to be of minor importance.

Van Gent and Van der Werf [11] mention s_{0p} to have influence on toe berm stability, but doesn't include it clearly in his stability formula. This is probably because the parameters that determine the wave steepness are already included in the calculation of the orbital velocity of the water particles.

WAVE LENGTH, L

Gerding [7], Docters van Leeuwen [8], Van der Meer [24] and Muttray [10] don't mention the effect of wave length.

Ebbens [9] notices that long waves (representing swell waves) do different damage than short waves (representing wind waves). Both kind of waves move rock of the toe berm downwards, but only long waves move rock upwards. This being said, the influence of the wave length isn't mentioned on the stability of the toe berm.

Van Gent and Van der Werf [11] relate the toe stability to the orbital velocity. The orbital velocity is larger for longer waves, so longer waves result in a lower toe berm stability. This is visualized in Figure A.5.

Muttray *et al.* [12] uses the same reasoning as Van Gent and Van der Werf. Muttray uses the peak wave length in front of the structure.

NOMINAL ROCK DIAMETER (ROCK SIZE), d_{n50t}

The weight of the rock used for the toe berm causes the main retaining force. As the weight of a rock scales with the rock size this means that it is an important parameter. This is why it is included in the stability number.

Gerding [7] found damage on the toe with a linear relation between the d_{n50t} and the H_s, but is not linear through the origin. Gerding suggests that this is because the wave attacks the armour layer directly and the toe berm is protected by it's submerged position. This means that there are more parameters influencing the damage than just the wave height and the rock grading. Gerding also includes the rock size on the right hand side of his formula, but does this in order to make his right hand side dimensionless, which is not a strong argument. This is even more so, as now the rock size has a stabilizing effect according to the left hand side of the formula and a destabilizing effect according to the right hand side of the formula.

Docters van Leeuwen [8] confirms the findings of Gerding.

Van der Meer [24] re-analysed the results obtained by Gerding and came to the conclusion that d_{n50t} is already present in the stability number and shouldn't be used again

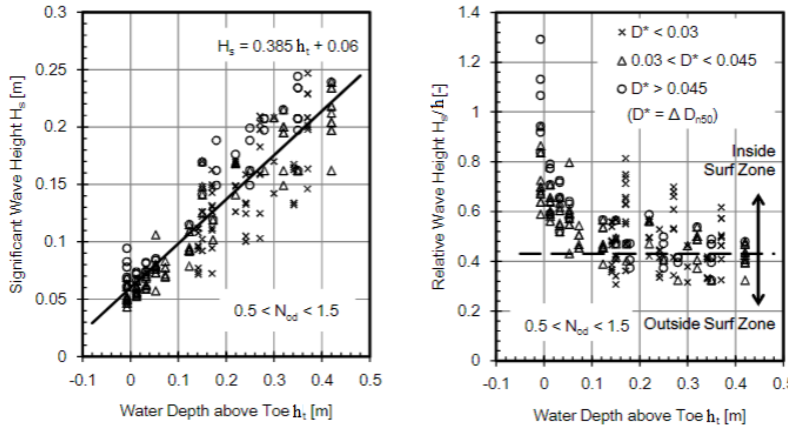

Figure A.6: Relation of wave height and water depth on the toe berm: Interrelation in tests with moderate damage (left), occurrence of depth limited wave conditions in these tests (right) (from Muttray *et al.* [12])

on the right-hand side of the formula.

Ebbens [9] confirms the findings of Van der Meer.

Muttray [10], Van Gent and Van der Werf [11] and Muttray *et al.* [12] find a linear influence of the d_{n50t} on the stability number.

LOCAL WATER DEPTH, h_m

In deep water the waves don't move the water particles at the bottom, so in deep water a toe berm isn't necessary. In shallow water the water particles at the bottom do have a horizontal velocity, so the toe berm will be less stable when h_m decreases.

Gerding [7] finds that deeper local water depth results in a more stable toe berm. Gerding doesn't include the local water depth in his formula, probably because the local water depth closely relates to the depth of the toe berm.

Docters van Leeuwen [8] also concludes that h_m influences the stability linearly, but different depths give a different coefficient for the linear relation between h_t and d_{n50t}.

Van der Meer [24] does use h_m on the right-hand side of the stability formula.

Ebbens [9] confirms the findings of Van der Meer, but makes a new formula for very shallow water. In this case the waves break on the foreshore slope and not on the breakwater slope. The stability number can be calculated using the damage and the Iribarren number. This means that h_m has an influence as a threshold for toe berm stability.

Muttray [10], Van Gent and Van der Werf [11] and Muttray *et al.* [12] don't include h_m in their formulas, suggesting it is of minor importance or that it is taken into account with the depth of the toe, as this closely relates to the water depth.

WATER DEPTH OF THE TOE BERM, h_t

A shallower toe berm shall be less stable than a deeper toe berm, as the water particles will have a lower horizontal velocity when they are in deep or transitional water. In shallow water the horizontal velocity of the water particles doesn't vary along the depth. This should mean that a toe structure will be more stable when it is situated deeper in the water.

Gerding [7] confirms this and finds a linear influence, although the experimental data doesn't support the linear influence that strongly. Still, Gerding uses h_t in his formula to determine the stability of the toe berm.

Docters van Leeuwen [8], Van der Meer [24] and Ebbens [9] confirm Gerdings findings.

Muttray [10] identifies the ratio of the water depth of the toe berm over the incoming wave height as the governing parameter for toe berm stability. He reports a linear influence.

Van Gent and Van der Werf [11] use h_t to calculate the orbital velocity, giving it some influence on the stability. As the depth of the toe increases, the orbital velocity decreases, increasing the stability of the toe.

Muttray *et al.* [12] also consider h_t important for the same reason as Van Gent and Van der Werf do, but uses the ratio with the wave length instead of the precise calculation of the orbital velocity. The inter dependency found by Muttray of h_t and H_s is shown in Figure A.6.

WIDTH OF THE TOE BERM, b_t

A wider berm should be more stable than a smaller toe, as in a wider toe more stones have the opportunity to find a stable position. Furthermore for a wider toe more damage is tolerable.

Gerding [7] concludes from his research that the width of the toe doesn't influence the stability of the toe berm. Still, he remarks that for wider berms more damage is tolerable. This means that a higher N_{od} is acceptable for wider toes than for narrow toes. This suggests that a damage percentage ($N_{\%}$) can be a better criteria for wide berms.

Van der Meer [24] confirms Gerdings findings.

Muttray [10] reports that berm size, and thus the berm width, is of minor importance.

Van Gent and Van der Werf [11] find that berm width does have influence, their test results show higher stability for wider berms.

THICKNESS OF THE TOE BERM, t_t

A thicker berm should be more stable than a thinner berm, as for a thicker berm more damage is tolerable.

Docters van Leeuwen [8] does investigate the influence of the thickness of the toe, but doesn't find a clear influence. This result might be caused because the depth of the toe isn't kept constant with varying toe thicknesses.

Muttray [10] reports that berm size, and thus the berm thickness, is of minor importance.

Van Gent and Van der Werf [11] find that thicker toe berms are more stable.

DENSITY OF ROCK MATERIAL, ρ_s

As stated at the rock grading paragraph, the main retaining force of the toe berm is the weight of the rocks. As the density of rock material influences the weight of the rocks, it is an important parameter. This is why it's included in the calculation of the stability number. In this way all researchers include this parameter in their formula [7], [24], [9], [10], [11], [12].

Docters van Leeuwen [8] varies ρ_s and confirms the formula found by Gerding.

FORESHORE SLOPE, α_f

A steeper foreshore will result in a narrower surf zone, which will result in more breaking waves close to the toe. This will probably result in a less stable toe berm.

Ebbens [9] does mention α_f and finds that steeper foreshore slopes result in more damage on the toe berm. The influence is noticed in very shallow water but in deep water as well. He doesn't incorporate this in a formula though.

Muttray [10] says that the foreshore slope is of minor importance. Muttray has some remarks on the way Ebbens measured the wave height, and when correcting for this, he doesn't find a clear influence of the foreshore slope.

Muttray *et al.* [12] does include the foreshore slope in his formula, pointing out it has a rather big influence.

BREAKWATER SLOPE, α_b

The formula of Van Gent and Van der Werf [11] was derived for 1:2 slopes. As breakwaters can have steeper slopes, they also tested it for 1:1.5 slopes. Dikes can have toe berms as well, so they also tested 1:4 slopes. The result of their testing was that their formula applied best for the 1:2 slopes, was still applicable for 1:1.5 slopes and not accurate enough for 1:4 slopes. They suggest that breakwater slope influences the orbital velocity, although no clear trend can be seen (as the 1:4 slope gives cases with significantly more and significantly less damage).

SLOPE ROUGHNESS

Down-rush is considered important for the stability of the toe according to Van Gent and Van der Werf [11]. Down-rush is influenced by the roughness of the slope. This is why they did tests with a smoother slope, namely single layer cubes. These tests resulted in more damage, the predicted damage had to multiplied with a factor 1.5 to match the measured damage. This means that smoother slopes have a lower toe stability.

A.2.4. CONCLUSION

It can be concluded that experimenters have great difficulty in solving the problem of predicting the toe stability. Experimental formulas lack validity and attempts to solve the problem from a physical understanding didn't succeed all to well. Researchers did manage to find the main parameters of influence, which are the wave height and wave length on the driving side of motion and the depth of the toe berm and the rock grading at the retaining side. Another important parameter seems to be the slope of the foreshore, but for the moment it is decided it is better to investigate this parameter in follow-up research. Parameters of minor importance seem to be the wave steepness, the width of the berm, the thickness of the berm, the slope of the breakwater and the slope roughness.

A

A.3. FOUNDATION LAYER

A.3.1. FILTER LAYERS

There are two types of granular filters: geometrically closed and geometrically open filters. In geometrically closed filters the subsequent layers of sediment are unable to move through the layers. This is achieved by choosing the grain size of the sediment in such a way that the grains are too big to pass through the voids of the sediment in the next layer. Geometrically open filters are designed in such a way that the sediment is able to pass through the layers, but the hydraulic gradient is lower than the critical gradient. This means that the hydraulic loading is taken into consideration, leading to a more economic design [25].

A.3.2. FILTER LAYERS FOR BREAKWATERS

Traditionally breakwater foundations are built with geometrically open filters, as constructing geometrically closed filters is rather expensive and difficult to do. This is why research has been performed in order to investigate the use of geometrically open filters in breakwater foundations.

Wolters and Van Gent [26] mention that in the 1980's and 1990's a lot of research has been conducted to determine the criteria for initiation of motion in granular filters. The research resulted in various formulae which are incorporated in CUR Report 161 (1993). New criteria for interface stability are added in CUR Report 233 (2010), but these are yet to be verified by experimental results. Wolters and Van Gent remark that very little is known about base material transport (and critical hydraulic gradients) in filters under cyclic (unsteady) loading. Their research focused on this topic. They concluded that if $\frac{i_{2\%}}{i_{cr}} < 3$ only a thin granular filter was needed to prevent erosion of the sand bed. In this ratio i_{cr} is the hydraulic gradient threshold for motion and $i_{2\%}$ is the hydraulic gradient which is exceeded for 2% of the waves. Under storm conditions erosion will occur however, with a strong increase in transport with hydraulic gradients from $\frac{i_{2\%}}{i_{cr}} > 3.7$. Filter layer thickness appeared to be dominant in preventing erosion in these tests.

Even more radical is the research of Den Bieman *et al.* [27]. In this research it was questioned whether stones could be placed directly on a sandy seabed. A numerical method (OpenFOAM) was used to investigate this. The results were quite remarkable as sedimentation was predicted in the toe structure, and a unrealistically narrow scour hole was predicted at the interface of the toe structure (just in front of the toe structure). It was concluded that physical model tests are needed to check on the results. Den Bieman and fellow researchers suspect that one of the assumptions made on which the numerical model is based might be invalid. This assumption was that only bed load transport plays a role, but probably suspended sediment transport plays a role as well inside the structure.

A.3.3. FOUNDATION FOR XBLOCPLUS

Delta Marine Consultants [2] state that the averege rock weight of the foundation layer for XblocPlus has to be approximately W/30. This is based on experience and not on research. It would be beneficial to research the influence of the foundation layer on the stability of the first row of XblocPlus. Delta Marine Consultants [2] state that the foundation layer should be supported by filter layers. Design formulas for filters are given by CURNET [13, CUR 233].

At the moment, not much is known about the role a foundation layer has on the stability of the bottom row of an armour layer. Generally speaking the foundation has to be stable (in other words, not eroding) so that the bottom row of an armour layer is secure. This is accomplished by designing the foundation according to the design formulas for filter layers. But it is unknown whether the permeability of the foundation layer has an influence on the stability of the bottom row of an armour layer itself. It is possible that a higher permeability of the filter layer reduces the hydraulic load on the bottom row of an armour layer. The permeability of the foundation layer can be changed by adjusting the rock grading and layer thickness. Other influences might also play a role, for example a finer rock grading can result in a more correct placement of the bottom layer.

B

Python script

In this Appendix the Python script is presented which was used to execute a Digital Displacement Analysis, using the images taken of the first row.

B

File - D:\beeldmateriaal 2e serie\Boven\DDA.py

```python
import cv2
import numpy as np
from skimage.measure import compare_ssim
import argparse
import imutils
import statistics
import csv

ap = argparse.ArgumentParser()
ap.add_argument("-f", "--first", required=True,
    help="first input image")
ap.add_argument("-s", "--second", required=True,
    help="second")
args = vars(ap.parse_args())

#load the two input images
imageA = cv2.imread(args["first"])
imageB = cv2.imread(args["second"])

croppedA = imageA[1200:1600, 300:3600]
croppedB = imageB[1200:1600, 300:3600]

hsv1 = cv2.cvtColor(croppedA, cv2.COLOR_BGR2HSV)
hsv2 = cv2.cvtColor(croppedB, cv2.COLOR_BGR2HSV)

lower_ora = np.array([0, 130, 130])
upper_ora = np.array([40, 255, 255])

mask1 = cv2.inRange(hsv1, lower_ora, upper_ora)
res1 = cv2.bitwise_and(croppedA, croppedA, mask=mask1)

mask2 = cv2.inRange(hsv2, lower_ora, upper_ora)
res2 = cv2.bitwise_and(croppedB, croppedB, mask=mask2)

#Put in coordinates of the armour units
start = [365,620,900,1170,1460,1745,2035,2310,2580,2865,3125]
PM = np.zeros((55,), dtype=int)
n = 0

#determine the displacements in 5 locations per armour unit
for q in range(0,11):
    st = [start[q]+10,start[q]+20,start[q]+30,start[q]+40,start[q]+50]
```

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Figure B.1: The Digital Displacement Analysis script

File - D:\beeldmateriaal 2e serie\Boven\DDA.py

```python
    bv = (mask1[:, st])
    bn = (mask2[:, st])
    for w in range(0, 5):
        cv = np.where(bv[:, w] > 0)
        cvi = cv[0][::-1]
        cn = np.where(bn[:, w] > 0)
        cni = cn[0][::-1]
        PM[n] = cni[0] - cvi[0]
        n = n+1

def Avg(PM):
    return sum(PM) / len(PM)

#determine the standard deviation in the displacement measurements
AMB = np.zeros((11,))
StDv = np.zeros((11,))
for block in range(1,12):
    MIS = [PM[block*5-5],PM[block*5-4],PM[block*5-3],PM[block*5-2],PM[block*5-1]]
    avgb = Avg(MIS)
    stdv = (statistics.stdev(MIS))
    AMB[block-1] = avgb
    StDv[block-1] = stdv
    print(MIS)

avgt = Avg(PM)
print(AMB)
print('average movement is', avgt)
print('standard dev is:', StDv)

#store the data
with open('verschuiving3.csv','a', newline='') as f:
    writer = csv.writer(f)
    writer.writerow([args["second"],AMB[0],AMB[1],AMB[2],AMB[3],AMB[4],AMB[5],AMB[6],AMB[7],AMB[8],AMB[9],AMB[10],avgt])
    writer.writerow([args["second"], StDv[0], StDv[1], StDv[2], StDv[3], StDv[4], StDv[5], StDv[6], StDv[7], StDv[8], StDv[9], StDv[10]])

#create an image
tsA = cv2.cvtColor(res1, cv2.COLOR_HSV2BGR)
tsB = cv2.cvtColor(res2, cv2.COLOR_HSV2BGR)
grayA = cv2.cvtColor(tsA, cv2.COLOR_BGR2GRAY)
```

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Figure B.2: The Digital Displacement Analysis script

File - D:\beeldmateriaal 2e serie\Boven\DDA.py

```
grayB = cv2.cvtColor(tsB, cv2.COLOR_BGR2GRAY)

output = grayA.copy()

grayA_inv= cv2.bitwise_not(grayA)
totaalplaatje = grayA_inv - grayB
thresh = 127
im_bw = cv2.threshold(totaalplaatje, thresh, 255, cv2.THRESH_BINARY)[1]
cv2.imshow('resultaat',im_bw)

GRID_SIZE = 20

im_bwnol= cv2.threshold(totaalplaatje, thresh, 255, cv2.THRESH_BINARY)[1]

height, width = im_bw.shape
for y in range(0, width -1, GRID_SIZE):
     cv2.line(im_bw, (0, y), (width, y), (0, 0, 0), 1, 1)

cv2.imshow('result', im_bw)
cv2.imwrite('6ZTMF4/nieuwv/r16.jpg', im_bw)
cv2.waitKey(0)
```

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Figure B.3: The Digital Displacement Analysis script

C

Results

C.1. Results of the first test series

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n * \Delta)$ [-]	RD [-]
0.068	49	2.8	1.72	0.02
0.114	82	4.7	2.88	0.04
0.125	91	5.2	3.17	0.07
0.137	99	5.7	3.46	0.11
0.145	105	6.0	3.66	0.21
0.156	113	6.5	3.95	0.24
0.168	122	7.0	4.25	0.32
0.181	131	7.6	4.60	0.50
0.198	143	8.2	5.02	0.57
0.210	152	8.8	5.33	0.86
0.215	156	9.0	5.45	1.10
0.224	162	9.3	5.67	1.30

Table C.1: Test at $h_f = 0.326m$ and T = 1.25 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.074	54	2.1	1.88	0.02
0.141	102	4.0	3.57	0.33
0.152	110	4.3	3.85	0.58
0.164	119	4.6	4.16	1.08
0.170	123	4.8	4.31	1.10
0.177	128	5.0	4.49	2.00

C

Table C.2: Test at $h_f = 0.326m$ and T = 1.5 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.079	57	1.2	2.00	0.06
0.106	77	1.6	2.70	0.13
0.097	70	1.4	2.46	0.16
0.085	62	1.3	2.16	0.16
0.105	76	1.5	2.65	0.16
0.110	80	1.6	2.80	0.19
0.121	87	1.8	3.06	0.24
0.123	89	1.8	3.12	0.29
0.133	97	2.0	3.38	0.38
0.137	100	2.0	3.48	0.48
0.149	108	2.2	3.78	2.00

Table C.3: Test at $h_f = 0.326m$ and T = 2 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.056	41	2.4	1.42	0.01
0.076	55	3.2	1.93	0.01
0.103	74	4.3	2.60	0.03
0.116	84	4.9	2.95	0.08
0.125	90	5.3	3.16	0.15
0.144	104	6.0	3.65	0.28
0.158	115	6.6	4.01	0.43
0.169	123	7.1	4.29	0.67
0.183	133	7.7	4.65	0.91
0.197	143	8.3	5.00	2.00

Table C.4: Test at $h_f = 0.376m$ and T = 1.25 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.066	48	1.9	1.67	0.01
0.070	51	2.0	1.77	0.02
0.094	68	2.6	2.39	0.04
0.119	86	3.3	3.02	0.10
0.129	93	3.6	3.27	0.16
0.137	99	3.8	3.47	0.28
0.149	108	4.2	3.77	0.37
0.160	116	4.5	4.07	1.04
0.173	125	4.8	4.39	2.00

Table C.5: Test at $h_f = 0.376m$ and T = 1.5 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.071	51	1.1	1.79	0.04
0.076	55	1.1	1.92	0.05
0.100	72	1.5	2.53	0.11
0.110	80	1.7	2.79	0.20
0.123	89	1.8	3.13	0.32
0.133	96	2.0	3.38	0.46
0.142	103	2.1	3.61	0.68
0.152	110	2.3	3.86	2.00

Table C.6: Test at $h_f = 0.376m$ and T = 2 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.061	44	1.7	1.54	0.05
0.061	44	1.7	1.55	0.04
0.097	70	2.8	2.45	0.04
0.122	88	3.5	3.08	0.19
0.111	80	3.2	2.81	0.27
0.104	75	3.0	2.64	0.27
0.115	83	3.3	2.92	0.33
0.095	69	2.7	2.42	0.34
0.103	75	2.9	2.61	0.36
0.117	85	3.3	2.97	0.56
0.125	90	3.5	3.16	0.66
0.134	97	3.8	3.40	0.78
0.143	104	4.1	3.63	2.00

Table C.7: Test at $h_f = 0.426m$ and T = 1.5 s

$H[m]$	H/H_{rmd} [%] [%]	s_0[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.059	43	1.6	1.50	0.01
0.083	60	2.3	2.11	0.03
0.107	78	2.9	2.72	0.08
0.119	87	3.3	3.03	0.16
0.134	97	3.6	3.39	2.00

Table C.8: Test at $h_f = 0.276m$ and T = 1.5 s

C

Duration [min]	$H_s[m]$	H_s/H_{smd} [%]	s_{0p}	$H_s/(d_n*\Delta)$ [-]	RD [-]
11.2	0.069	70	1.9	1.76	0.03
61	0.096	97	2.6	2.43	0.18
63.8	0.114	115	3.1	2.88	0.28
63.5	0.126	128	3.4	3.21	0.74
35.7	0.142	144	3.8	3.60	2.00

Table C.9: Test with irregular waves at $h_f = 0.276m$ and $T_p = 1.5$ s

C.2. RESULTS OF THE SECOND TEST SERIES

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.018	13	1.94	97	0.47	0.02
0.040	29	2.09	104	1.01	0.01
0.066	47	2.10	105	1.66	0.02
0.093	67	2.15	107	2.35	0.11
0.118	86	2.22	111	3.00	0.40

Table C.10: Test with regular waves, $s_0 = 2\%$, no foundation, no toe berm: NTNF2

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.038	27	3.96	99	0.95	-0.01
0.049	35	4.21	105	1.24	0.00
0.059	43	4.14	104	1.50	0.01
0.072	52	4.36	109	1.82	0.02
0.084	61	4.40	110	2.13	0.04
0.095	69	4.24	106	2.42	0.11
0.108	78	4.34	109	2.73	0.49

Table C.11: Test with regular waves, $s_0 = 4\%$, no foundation, no toe berm: NTNF4

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.018	13	1.93	97	0.46	0.01
0.040	29	2.07	103	1.01	0.03
0.051	37	2.00	100	1.30	0.02
0.066	48	2.09	105	1.66	0.03
0.078	57	2.16	108	1.99	0.01
0.090	65	2.11	106	2.29	0.02
0.103	74	2.16	108	2.60	0.01
0.117	85	2.20	110	2.96	0.03
0.126	91	2.07	104	3.20	0.04
0.141	102	2.11	105	3.57	0.05
0.153	111	2.05	103	3.88	0.07
0.174	126	2.17	109	4.41	0.07
0.192	139	2.24	112	4.88	0.14
0.211	153	2.31	116	5.36	0.21
0.260	189	2.53	127	6.60	0.27

Table C.12: Test with regular waves, $s_0 = 2\%$, no foundation, with toe berm: WTNF2

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.038	28	4.04	101	0.97	0.01
0.049	35	4.21	105	1.24	0.00
0.057	41	4.95	124	1.45	0.02
0.069	50	4.24	106	1.76	0.00
0.078	57	4.11	103	1.99	0.02
0.093	67	4.11	103	2.36	0.03
0.105	76	4.04	101	2.66	0.04
0.120	87	4.24	106	3.05	0.04
0.131	95	4.21	105	3.31	0.06
0.140	101	4.04	101	3.54	0.08
0.156	113	4.27	107	3.95	0.16
0.168	122	4.15	104	4.26	0.24
0.177	128	4.17	104	4.50	1.51

Table C.13: Test with regular waves, $s_0 = 4\%$, no foundation, with toe berm: WTNF4

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.018	13	1.92	96	0.46	-0.02
0.042	30	2.20	110	1.07	-0.01
0.050	36	1.93	97	1.26	0.00
0.064	47	2.06	103	1.63	0.00
0.077	56	2.13	107	1.96	0.01
0.089	64	2.07	104	2.25	0.03
0.102	74	2.13	107	2.60	0.05
0.115	84	2.17	108	2.92	0.09
0.126	91	2.04	102	3.19	0.18
0.134	97	2.04	102	3.41	0.29
0.143	103	1.91	96	3.62	0.38
0.171	124	2.12	106	4.32	0.44
0.204	148	2.39	119	5.18	0.56
0.207	150	2.21	111	5.26	0.65
0.241	175	2.47	124	6.12	0.75

Table C.14: Test with regular waves, $s_0 = 4\%$, with foundation, no toe berm: NTWF2

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.038	28	4.03	101	0.97	0.00
0.049	35	4.22	106	1.24	0.00
0.059	43	4.12	103	1.49	0.01
0.071	52	4.35	109	1.81	0.01
0.081	58	4.22	106	2.04	0.01
0.095	69	4.21	105	2.41	0.03
0.106	77	4.32	108	2.70	0.04
0.120	87	4.22	105	3.05	0.08
0.135	98	4.36	109	3.43	0.12
0.150	108	4.33	108	3.80	0.21
0.161	117	4.40	110	4.08	0.76

Table C.15: Test with regular waves, $s_0 = 4\%$, with foundation, no toe berm: NTWF4

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.019	14	1.99	99	0.48	-0.01
0.041	30	2.12	106	1.04	0.00
0.066	48	2.13	106	1.67	0.01
0.092	67	2.16	108	2.34	0.02
0.120	87	2.24	112	3.03	0.02
0.132	96	2.13	107	3.34	0.02
0.142	103	2.19	109	3.60	0.02
0.153	111	2.04	102	3.88	0.02
0.180	131	2.26	113	4.57	0.04
0.210	152	2.47	124	5.33	0.05
0.215	156	2.35	117	5.45	0.06
0.239	173	2.45	122	6.07	0.08

Table C.16: Test with regular waves, $s_0 = 2\%$, with foundation, with toe berm: WTWF2

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.041	30	4.33	108	1.04	0.03
0.052	38	4.48	112	1.32	0.04
0.063	46	4.41	110	1.60	0.04
0.075	54	4.60	115	1.90	0.01
0.086	62	4.46	112	2.17	0.02
0.100	72	4.42	110	2.53	0.03
0.113	82	4.57	114	2.86	0.03
0.128	93	4.50	112	3.25	0.03
0.144	105	4.65	116	3.66	0.05
0.161	117	4.64	116	4.08	0.07
0.172	125	4.73	118	4.37	0.07
0.184	133	4.46	111	4.67	0.08
0.205	149	4.82	121	5.21	0.09
0.231	168	4.82	121	5.86	0.12
0.245	177	4.92	123	6.21	0.13

Table C.17: Test with regular waves, $s_0 = 4\%$, with foundation, with toe berm: WTWF4

C

$H[m]$	H/H_{rmd} [%]	s_0[%]	s_0/s_{targ}[%]	$H/(d_n*\Delta)$ [-]	RD [-]
0.038	28	4.01	100	0.96	-0.01
0.049	35	4.21	105	1.24	0.00
0.057	41	4.91	123	1.44	0.00
0.069	50	4.23	106	1.76	-0.01
0.078	57	4.07	102	1.99	0.00
0.092	67	4.07	102	2.33	0.01
0.105	76	4.05	101	2.65	0.01
0.120	87	4.25	106	3.05	0.02
0.131	95	4.23	106	3.31	0.02
0.140	101	4.05	101	3.54	0.04
0.156	113	4.25	106	3.95	0.07
0.167	121	4.15	104	4.24	0.08
0.178	129	4.15	104	4.51	0.13
0.211	153	4.68	117	5.34	0.21
0.219	159	4.56	114	5.56	0.23

Table C.18: Test with regular waves, s_0 = 4%, with a fine foundation, with toe berm: WTFF4

Dur. (min)	$H_s[m]$	H_s/H_{smd} [%]	s_{0p}	s_{0p}/s_{targ}[%]	$H_s/(d_n*\Delta)$ [-]	RD [-]
34.4	0.008	8	1.12	28	0.19	-0.01
54.7	0.034	34	3.63	91	0.86	-0.01
46.4	0.045	45	3.94	98	1.14	0.00
52	0.057	58	4.09	102	1.45	0.03
52.3	0.067	68	4.09	102	1.71	0.13

Table C.19: Test with irregular waves, s_{0p} = 4%, no foundation, no toe berm: NTNFi

Dur. (min)	$H_s[m]$	H_s/H_{smd} [%]	s_{0p}	s_{0p}/s_{targ}[%]	$H_s/(d_n*\Delta)$ [-]	RD [-]
59.6	0.033	34	3.56	89	0.84	0.01
59.4	0.044	45	3.71	93	1.12	0.02
59.4	0.057	58	4.08	102	1.46	0.03
59.4	0.067	68	4.08	102	1.71	0.05
59.4	0.081	82	4.05	101	2.04	0.12
59.4	0.093	94	4.29	107	2.35	0.19

Table C.20: Test with irregular waves, s_{0p} = 4%, no foundation, with toe berm: WTNFi

Dur. (min)	$H_s[m]$	H_s/H_{smd} [%]	s_{0p}	s_{0p}/s_{targ}[%]	$H_s/(d_n*\Delta)$ [-]	RD [-]
40.5	0.034	35	3.49	87	0.86	0.00
45.4	0.044	44	3.89	97	1.11	0.01
49.5	0.058	59	4.14	103	1.47	0.02
51.5	0.068	69	4.15	104	1.73	0.05
56.5	0.079	80	4.49	112	2.01	0.11

Table C.21: Test with irregular waves, s_{0p} = 4%, with foundation, no toe berm: NTWFi

C

Dur. (min)	$H_s[m]$	H_s/H_{smd} [%]	s_{0p}	s_{0p}/s_{targ}[%]	$H_s/(d_n*\Delta)$ [-]	RD [-]
15.2	0.057	57	4.21	105	1.43	0.01
60	0.082	83	4.11	103	2.07	0.01
59.4	0.107	109	4.18	104	2.72	0.05
59.4	0.134	136	4.00	100	3.39	0.07
59.4	0.153	155	4.62	116	3.87	0.13

Table C.22: Test with irregular waves, s_{0p} = 4%, with foundation, with toe berm: WTWFi

D

EXAMPLE OF PYTHON OUTPUT

In this appendix the results of the test at a $h_f = 0.276m$ with regular waves with a wave period T = 1.5 s are presented. Notice how after the third run the loose armour unit has disappeared.

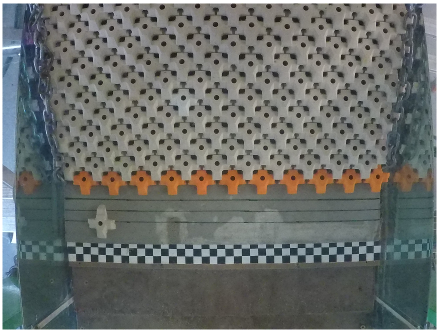

Figure D.1: Before the first run

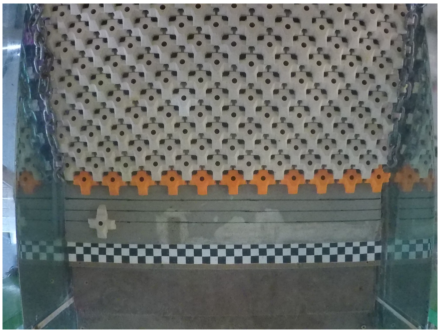

Figure D.2: After the first run (H = 0.059 m)

Figure D.3: Movement of the first row after the first run (H = 0.059 m)

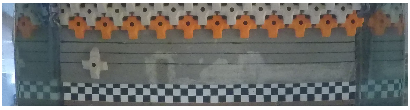

Figure D.4: After the second run (H = 0.083 m)

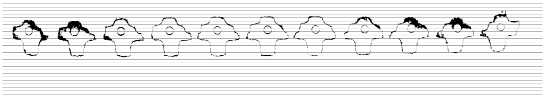

Figure D.5: Movement of the first row after the second run

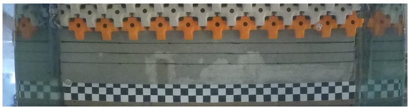

Figure D.6: After the third run (H = 0.107 m)

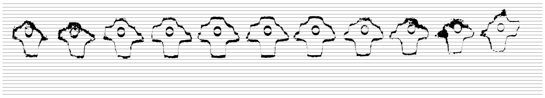

Figure D.7: Movement of the first row after the third run

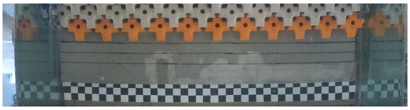

Figure D.8: After the fourth run (H = 0.119 m)

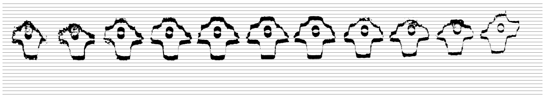

Figure D.9: Movement of the first row after the fourth run

Figure D.10: After the fifth run (H = 0.133 m)

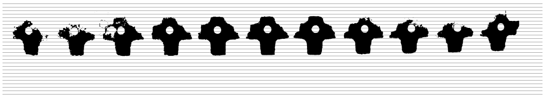

Figure D.11: Movement of the first row after the fifth run

E

REFRACTION CORRECTION

E.1. THEORETICAL ERROR

The camera set up is done in a way that minimizes errors. Still, the results need to be adjusted as a result of refraction. Luckily, this error can be calculated.

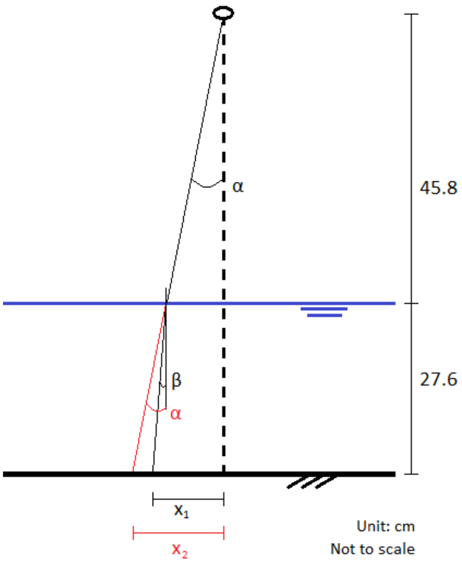

Figure E.1: Influence of the refraction

The relations between angle α and β is given by Snell's Law:

$$n = \frac{\sin(\alpha)}{\sin(\beta)} \tag{E.1}$$

For water $n = 1.333$. The error that is made (x_2/x_1) can be calculated in the following way:

$$\frac{x_2}{x_1} = \frac{73.4 * \tan(\alpha)}{50.8 * \tan(\alpha) + 22.6 \tan(\arcsin(\frac{\sin(\alpha)}{1.333}))} \tag{E.2}$$

This gives the following graph:

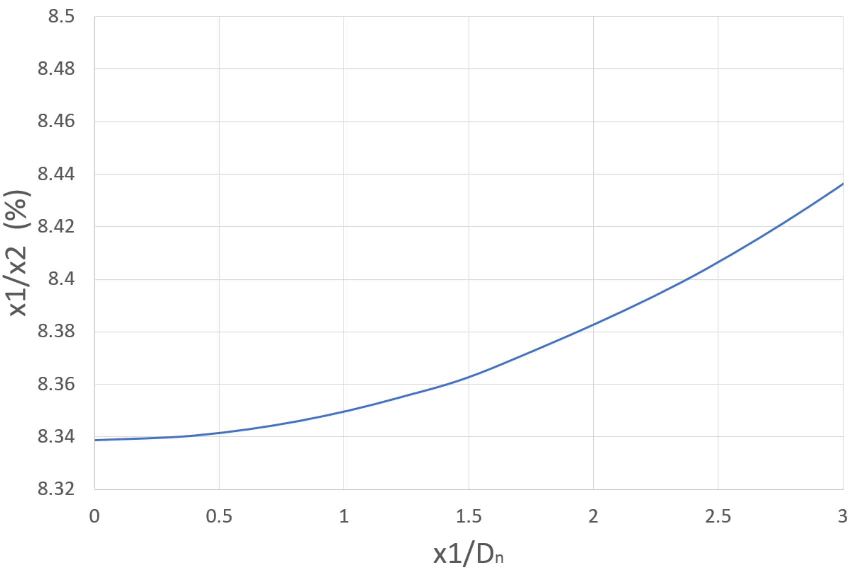

Figure E.2: Refraction error for d = 55 cm

The error is approximately 8.34% when movement is below D_n, which is the region of interest.

E.2. REAL ERROR

The theoretical error should be checked with actual measurements from photos. To test this, a photo without water in the flume is compared with water in the flume. The flume was filled for 53.5 cm, and the frame holding the camera was changed. This means the water depth over the foreshore was 21.1 cm and the distance from the camera to the water surface was 48.8 cm. This resulted in a theoretical error of 8.16% for this set up.

Filled		Empty		Difference	
Distance [px]	Dist. [cm]	Dist. [px]	Dist. [cm]	Relative [%]	Absolute [cm]
3	0.06	2	0.04	50	0.02
13	0.27	12	0.25	8.33	0.02
23	0.47	21	0.43	9.52	0.04
36	0.74	34	0.70	5.88	0.04
213	4.02	196	4.02	8.67	0.00

Table E.1: Error measured from photos

The distance in centimeters is computed by dividing the distance measured in pixels by a known distance over the number of pixels required to span this distance. This gives a conversion rate [px/cm] for the empty and the filled flume (the values of which are different). These measurements are taken from the following below. The black lines represent the measurements.

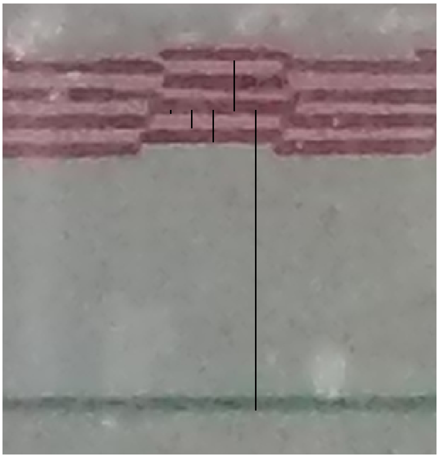

Figure E.3: Measurements from the filled flume

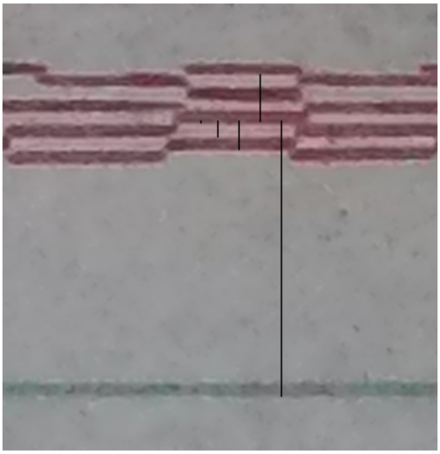

Figure E.4: Measurements from the empty flume

It can be seen that the error in pixels in the second, third and fifth measurement are quite close to the theoretical error (8.16% for this setup). The first and fourth measurement seem to be 1 pixel different from the expected value. In the case of the first measurement this gives a really large relative error because the distance is really small (only 2 pixels), but the absolute error is quite small: only 0.02 cm. The same applies for the fourth measurement. It seems acceptable to correct the images with the theoretical error and ignore the extra error because of the inaccuracy of the used method because it is too small to be relevant.

F

DESIGN FORMULA

This study has produced insights into first row stability which can be used for proposing a design formula. This formula should be seen as a working formula, which has to be verified and/or improved with new tests.

F.1. THEORY

The stability of an object can be determined using a force balance: $Z = R - S$. In which Z is the sum of the forces, R is the resistance of the object to movement and S is the load on the object. If $Z > 0$, movement occurs, if $Z < 0$, no movement occurs, so the critical point is when $Z = 0$, or in other words $R = S$. In this case, a little bit of movement is acceptable, as long as movement is limited. When the difference in movement between two wave runs exceeds $0.2D_n$, it is defined as brittle failure. The amount of damage that has occurred before brittle failure happens, is defined as critical damage. The amount of critical damage is the point of interest as in at this point the destabilizing forces are just in balance with the stabilizing forces.

F.2. STABILIZING FORCE

In the case of the first row of a breakwater, based on this study, it can be stated that the main parameter determining the resistance to movement is the depth of the first row, h_f. In this study it is found that if there is more distance between Still Water Level (SWL) and the first row, the first row can sustain more damage before brittle failure occurs. This can be seen in figure F.1. The test with $h_f = 0.326m$ and $h_f = 0.376m$ deviate quite a bit from the fitted formula, and this could be compensated for by multiplying $\frac{h_f}{D_n}$ with $s^{0.15}$, but this relation with wave steepness is contradictory with the relation that is found in the second test series. This means that improving the prediction here, would deteriorate the prediction of the test results in the second test series. Critical damage is noted as RD_c.

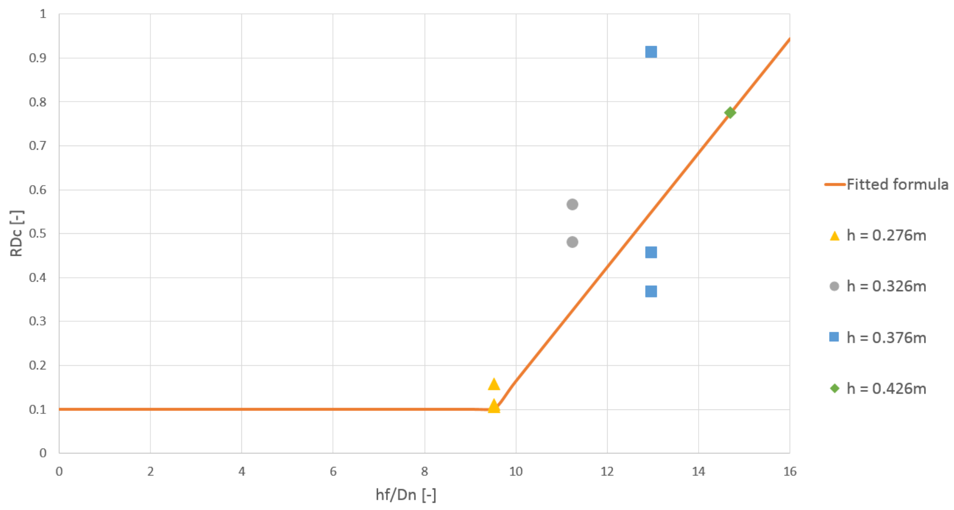

Figure F.1: The depth of the first row divided by the nominal diameter of different tests compared to the critical relative displacement

F

This relation can be used to determine a formula for the critical damage:

$$\begin{aligned} RD_c &= 0.1 \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ RD_c &= 0.13\frac{h_f}{D_n} - 1.135 \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{F.1}$$

There were no tests that had $\frac{h_f}{D_n} < 9.5$, so that part of the formula is not that substantial, but it is expected that when the water depth decreases further, waves experience depth-induced breaking before reaching the toe, so the toe isn't loaded with the critical wave heights.

F.3. DESTABILIZING FORCE

The main destabilizing parameter is the wave height. A higher wave height, means a higher run-down, meaning a greater load on the first row. In figure F.2 the stability number is plotted against the critical damage for different tests.

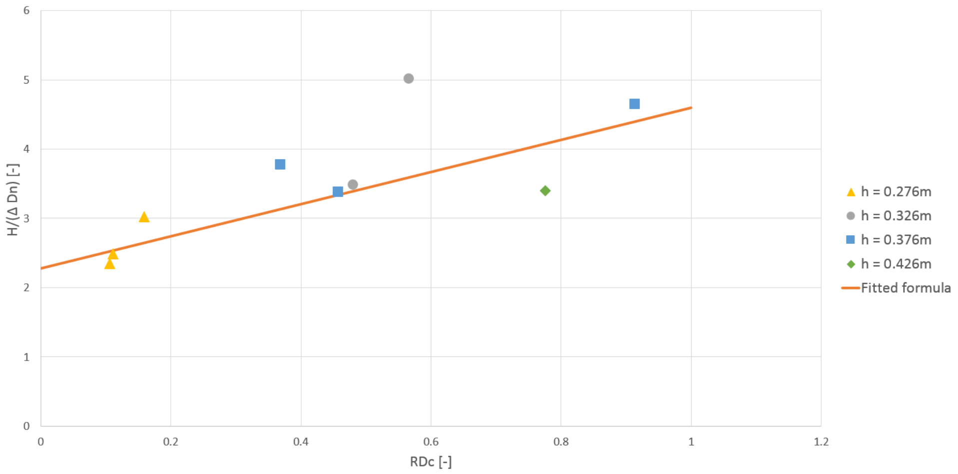

Figure F.2: Critical damage of different tests compared to the stability number

The following formula is fitted:

$$\frac{H}{\Delta D_n} = 2.322 RD_c + 2.275 \tag{F.2}$$

F.4. DESIGN FORMULA

When F.1 is filled into F.2, the following design formula rolls out:

$$\begin{aligned} \frac{H}{\Delta D_n} &= 2.59 \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ \frac{H}{\Delta D_n} = 0.3\frac{h_f}{D_n} &- 0.36 \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{F.3}$$

This formula is tested in figure F.3. It shows reasonable predictions. For a few tests the stability is overestimated, which is a bad thing; the test with $h_f = 0.426$ is predicted 22% more stable than it actually is.

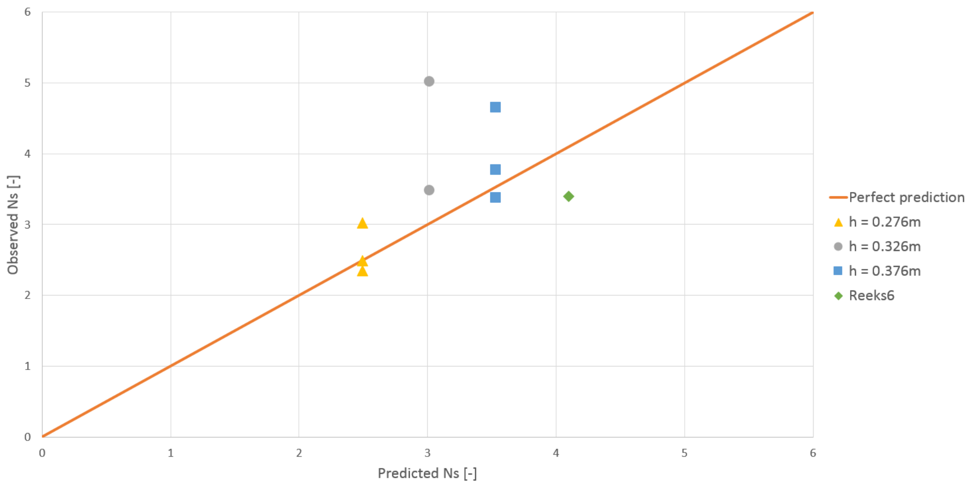

Figure F.3: Predicted N_s compared to the tested N_s

F

The result until now is a design formula which is made for breakwaters without a toe berm and a foundation layer, loaded with regular waves. Such a design formula is pretty much useless, so factor have to be determined to adjust it so it can be used for breakwaters with a toe berm and/or a foundation layer and irregular waves. This means the following formula is proposed:

$$\begin{aligned} \frac{H}{\Delta D_n} &= 2.59 f_s f_w \quad \text{if} \quad \frac{h_f}{D_n} < 9.5 \\ \frac{H}{\Delta D_n} &= f_s f_w \left(0.3\frac{h_f}{D_n} - 0.26\right) \quad \text{if} \quad \frac{h_f}{D_n} > 9.5 \end{aligned} \tag{F.4}$$

In this equation f_s is a factor to take structural parameters into account and f_w is a factor to convert regular waves to irregular waves, which also is dependent of structural parameters. The other tests showed no brittle failure, so there was no RD_c to tune these factors to, so the factors were determined in such a way that the stability number at $RD = 0.2$ was predicted for breakwaters with a toe berm and/or a foundation layer and $RD = 0.1$ for breakwaters without a toe berm and foundation layer, as the research showed that when conditions are most critical, these damage levels would be critical. This means that there is room for improvement especially in the irregular wave tests as for example the NTWFi and WTWFi test didn't reach $RD = 0.2$, as the test was stopped before that amount of damage was reached. Based on hereon the following parameters would give good results:

Configuration	f_s [-]	f_w [-]
No toe berm, no foundation layer	1	0.6
Toe berm, no foundation layer	1.5	0.6
No toe berm, foundation layer	1.3	0.6
Toe berm, foundation layer	2.1	0.7

Table F.1: Overview of factors to adjust the design formula

In table F.2 an overview is given of all the found and predicted stability numbers of the first row in tests in this study. This table is plotted in figure F.4. For safety's sake, a safety factor could be introduced to be able to get safe predictions for an outlier like the test at $h_f = 0.426m$, but this isn't done here as this is just a working hypothesis for a design formula. It can be concluded that the design formula is suited to predict a useful stability number for the first row of the tests. The predicted stability number is useful as it succeeds to determine the minimal stability number for all water depths, although there is a significant chance that the stability number is predicted to low. For design purposes, this is considered to be a good quality of a design formula.

Test	$N_{s,test}$ [-]	RD_c reached?	RD [-]	$N_{s,pred}$	$\frac{N_{s,pred}}{N_{s,test}}$ [%]
$h_f = 0.276m$ $T = 1.5s$	3.03	Yes	0.16	2.50	82
$h_f = 0.326m$ $T = 1.25s$	5.02	Yes	0.57	3.01	60
$h_f = 0.326m$ $T = 1.5s$	2.85	N.B.	0.20	3.01	106
$h_f = 0.326m$ $T = 2s$	3.48	Yes	0.48	3.01	87
$h_f = 0.376m$ $T = 1.25s$	4.65	Yes	0.91	3.53	76
$h_f = 0.376m$ $T = 1.5s$	3.77	Yes	0.37	3.53	94
$h_f = 0.376m$ $T = 2s$	3.38	Yes	0.46	3.53	105
$h_f = 0.426m$ $T = 1.5s$	3.40	Yes	0.78	4.05	119
$h_f = 0.276m$ irr.	2.88	Yes	0.28	2.50	87
NTNF2	2.35	Yes	0.11	2.50	106
WTNF2	5.13	No	0.20	3.74	73
NTWF2	3.23	No	0.20	3.24	101
WTWF2	6.07	No	0.08	5.24	86
NTNF4	2.49	Yes	0.11	2.50	100
WTNF4	4.26	Yes	0.24	3.74	88
NTWF4	3.05	Yes	0.21	3.24	106
WTWF4	5.43	No	0.10	5.24	96
NTNFi	1.71	No	0.13	1.62	95
WTNFi	2.35	No	0.19	2.25	96
NTWFi	2.01	No	0.11	1.95	97
WTWFi	3.87	No	0.13	3.67	95

Table F.2: Overview of tested stability numbers and predicted stability numbers for all tests in this study.
N.B.: RD_c can't be determined based on test, because the wave height was increased too much in the beginning, so $RD = 0.2$ is assumed

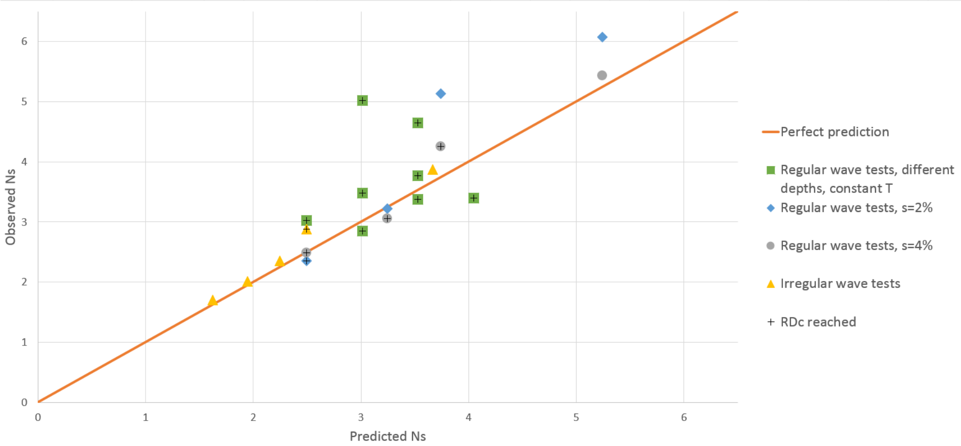

Figure F.4: Predicted N_s compared to the tested N_s for all tests

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