Stability of protective rock berms for pipelines and cables

Jeroen Peleman, Wouter Declercq

Supervisors: Prof. dr. ir. Peter Troch, Prof. dr. ir. Andreas Kortenhaus
Counsellor: Ir. Koen Schepens (Jan De Nul)

Master's dissertation submitted in order to obtain the academic degree of
Master of Science in Civil Engineering

Department of Civil Engineering
Chair: Prof. dr. ir. Peter Troch
Faculty of Engineering and Architecture
Academic year 2016-2017
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Preface

A thesis for graduation as civil engineer with a topic related to coastal engineering got our preferences because of the personally background and the interesting courses during the academic years at the University of Ghent. After the very time consuming period in the wave flume for doing the hydraulic model tests, we can proudly present our manuscript.

The opportunity for doing academic research about a present practical application was a great honour wherefore we would like to thank our promotors Prof. dr. ir. Peter Troch, Prof. dr. -ing. Andreas Kortenhaus, and ir. Koen Schepens. Their support and friendly advice during the investigation helped us significant during the tests and processing.

A special thank goes to our supervisors dr. ir. Vicky Stratigaki, ir. David Gallach Sanchez, and Minghao Wu. During the academic year they were always available to assist with numerous questions, doubts, and advice. The final review of our text was also a great help to came up with this work.

During the experimental tests, the contribution of the technicians Herman Van der Elst, Tom Versluys, Sam Meurez and David Derynck for the optimal working of the wave flume during the four weeks was very important. Even in stress-situations with setbacks, they guaranteed a cheerful atmosphere with their positive attitude.

In use of the EPro software we want to thank professor Thomas Lykke Andersen from the university of Aalborg for the quick and helpful aid with the encountered software problems.

Finally we would like to thank our girlfriends and family. Because of our parents we got the possibility to develop ourselves further as students in the last Master-years. The support of the closely related people in difficult times was undoubtedly necessary to finalize our education at the university with this manuscript.

Thanks a lot!

Jeroen Peleman
Wouter Declercq

June 2017
De auteurs geven de toelating deze masterproef voor consultatie beschikbaar te stellen en delen van de masterproef te kopiëren voor persoonlijk gebruik. Elk ander gebruik valt onder de bepalingen van het auteursrecht, in het bijzonder met betrekking tot de verplichting de bron uitdrukkelijk te vermelden bij het aanhalen van resultaten uit deze masterproef.

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June 2nd 2017

Jeroen Peleman and Wouter Declercq
The design methodology of rock berms according to the critical stability method using the Shields stability parameter, based on sediment transport and scour assessments, is presently a popular design method. It is based on the bed-shear stress which includes the velocity of the water particles and the friction interaction with the stones. A recent study of Roulund et al. (2016) provides recommendations for the quantification of the different variables involved in the computation of the stability parameter and the wave orbital velocities, focussing on irregular waves. The present study is established to investigate the hydraulic stability of the armour layer for specific wave conditions in the facilities of the large wave flume at Ghent University, as such kind of test were never performed before. It means that the damage profile method is applied and thus different parameters, related to the hydraulic boundary conditions and the geometric configurations of the rock berm, are investigated in function of a damage level. During execution two measurement methods are reviewed and analysed.

Keywords: rock berms, physical model test, Shields parameter, pipelines and cables, hydraulic armour stability
Stability of protective rock berms for pipelines and cables

Jeroen Peleman and Wouter Declercq

Supervisors: Prof. dr. ir. P. Troch, Prof. dr. ir. A. Kortenhaus, Dr. ir. Vicky Stratigaki, Ir. K. Schepens

Abstract – The design methodology of rock berms according to the critical stability method using the Shields stability parameter, based on sediment transport and scour assessments, is presently a popular design method. It is based on the bed-shear stress which includes the velocity of the water particles and the friction interaction with the stones. A recent study of Roulund et al. [1] provides recommendations for the quantification of the different variables involved in the computation of the stability parameter and the wave orbital velocities, focussing on irregular waves. The present study is established to investigate the hydraulic stability of the armour layer for specific wave conditions in the facilities of the large wave flume at Ghent University, as such kind of test were never performed before. It means that the damage profile method is applied and thus different parameters, related to the hydraulic boundary conditions and the geometric configurations of the rock berm, are investigated in function of a damage level. During execution two measurement methods are reviewed and analysed.

Keywords – Rock berms, physical model test, Shields parameter, pipelines and cables, hydraulic armour stability.

I. INTRODUCTION

The placement of on-bottom rock dumping over a pipeline is defined as a near-bed rubble mound structure as the ratio between the crest compared to the water depth is lower than 0.5 [3]. Stability of a pipeline on the seabed can become critical due to hydrodynamic loading by waves. Movement of stones occurs which results in damage towards the armour layer of the rock berm.

To find a relation between the damage level and the applied hydraulic boundary conditions for a general geometric configuration of a rock berm, non-dimensional stability parameters are developed [2]. A frequently used parameter today is the Shields stability parameter \( \theta \). It represents the non-dimensional bed shear stress by considering the median stone diameter \( d_{50} \) and the mass densities of the rock grading \( \rho_s \) and water \( \rho_w \), as shown in Equation (1).

\[
\theta = \frac{\tau}{g (\rho_s - \rho_w) d_{50}} \tag{1}
\]

The bed shear stress \( \tau \) is determined by considering two parameters [4]. First, the bottom orbital velocity represented by \( u_m \) (irregular sea state), based on the root-mean-square velocity \( u_{rms} \) [5]. Secondly the wave friction factor \( f_w \) which is a continuous expression formulated by Roulund et al. [1].

According to the present design methodology with the critical stability method, a critical boundary or threshold of motion needs to be defined for the related stability parameter. Figure 1 represents the critical Shield curve \( \theta_{cr} \) or boundary of the threshold of motion with respect to the dimensionless stone size \( D^* \) [6]. Where for \( D^* \) larger than 100 a continuous value of 0.055 is assumed, based on previous studies [4] this value seems conservative for waves.

![Fig. 1: Critical Shields curve indicating conservative values under wave only loading [6]](image)

Different damage definitions can be applied in defining damage. The eroded area \( A_e \) is used (see Equation (2)), which has its origin in the damage definition of breakwaters. The dimensionless damage parameter \( S \) depends on the eroded area \( A_e \) of the rock berm (removed area of stone material) and the nominal median stone diameter \( d_{50} \) of the grading.

\[
S = \frac{A_e}{d_{50}^2} \tag{2}
\]

The objective is to study the feasibility of physical modelling of rock berms according to the Shields parameter for specific hydraulic conditions with dimensionless grain sizes larger than 100 at the test facility of UGhent. This is done by means of hydraulic model tests where the influence of the berm width and damage evolvement of different sections of the rock berm are investigated. Additionally two available measurement methods are reviewed. The first one is a non-contact laser profiler developed by
the university of Aalborg. Secondly a new electromagnetic velocity meter is installed whose data is compared with different wave theories.

II. TEST FACILITY

The experiments are conducted in the large wave flume of Ghent university (30 m x 1 m x 1.2 m, Length x Width x Height). Irregular waves are generated by a piston-type wave paddle which is controlled by the GENESYS software [7].

During the tests, the reflection in the flume is controlled by allowing a maximum reflection coefficient of 0.25 by means of two wave absorption techniques: passive and active wave absorption. The first one is achieved by installing recycled honeycomb blocks at the opposite end of the wave flume, instead of the traditional parabolic spending beach. For the active wave absorption, an active wave absorption system (AWASYS) [8] is implemented in GENESYS. It adjusts the generation of waves by anticipating the reflected waves in the flume which are measured by two wave gauges in front of the wave paddle.

The horizontal wave orbital velocities are measured using an electromagnetic velocity recorder which is placed 4 metres in front of the tested rock and at a height which is equal to the crest level of the rock berm or the top of the armour layer.

Before, during, and after each test, the armour layer profile is measured with a non-contact laser profiler which is placed upon a mechanical motion vehicle, developed by Aalborg University. The operation and analysis with the profiler is performed with the software EPro. The profiler can be used for dry and wet measurements by the use of a water tight casing. When performing underwater measurements, attention should be paid to the refraction laser light when it travels from an air medium towards water.

III. METHODOLOGY OF EXPERIMENTAL MODEL TESTS

![Fig. 2: Prototype conditions and rock berm configuration as proposed by JDN](image)

The starting point for the experiments is a prototype configuration of a rock berm exposed to specific hydraulic boundary conditions. These conditions are proposed by the collaborative company Jan De Nul (JDN) and are represented in Figure 2.

Experiments

In order to conduct experiments in the wave flume, the prototype dimensions and hydraulic loadings are scaled. The principle of scaling is according to the Froude law, which ensures a correct scaling of the gravity forces. A scaling factor of 25 is applied based on the characteristics of the available facility.

The performed test matrix is presented in Table 1 with the setting of a constant peak wave steepness $s_p$ (0.035), berm height $h_{crest}$ (8 cm) and slope angle $(\cotg \alpha = 2.5)$ in all the test series. In ‘Test series 1, 2, and 3’ a berm width of 8 centimetres is applied, while in the final ‘Test series 4’ the berm width is increased to 20 centimetres. For each test approximately 1000 irregular waves have been generated. The tests are also executed at least twice to ensure reliability of the results.

<table>
<thead>
<tr>
<th>Test series 1, 3, and 4</th>
<th>Test series 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{w50}$ [mm]</td>
<td>$H_s$ [cm]</td>
</tr>
<tr>
<td>8.05 (series 1 and 4)</td>
<td>18.0</td>
</tr>
<tr>
<td>12.40 (series 3)</td>
<td>14.4</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_{w50}$ [mm]</td>
<td>$H_s$ [cm]</td>
</tr>
<tr>
<td>6.71</td>
<td>17.0</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>13.6</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In all test series, the influence on the damage level by changing the hydraulic boundary conditions will be investigated. More concrete, the significant wave height $H_s$, water depth $d$, and wave period $T_p$ will vary during the tests. Further, ‘Test series 1, 2 and 3’ are compared for the same hydraulic boundary conditions but with a different stone grading for the armour layer. Finally, ‘Test series 4’ refers to a larger berm width, thus a longer distance of less water depth, but with the same hydraulic boundary conditions as the other test series.

Wave analysis

The tests are performed with relatively large significant wave heights compared to the applied water depths (transitional water). As a results an amount of the generated irregular waves are breaking, which is limited to 5 % of the total amount of waves. The wave breaking occurs mostly several metres in front of the structure and according to relevant guidelines available from the Rock Manual [9], the influence of breaking waves can be neglected.

Nonlinear waves are clearly visible in the experiments which indicates that the linear wave
theory is not the most appropriate approach to estimate the horizontal orbital velocities under water. However, experiments with the electromagnetic velocity recorder indicate that the formula of Soulsby [5] which is based on linear wave theory for irregular waves offers a good approximation for further analysis. Those tests are performed for monochromatic waves with a wave height \( H \) of 0.18 metres, a wave period \( T \) of 1.635 seconds, and a water depth \( d \) of 0.55 metres. A graphical representations of the theoretical and measured velocity \( u \) towards the depth coordinate \( z \) is shown in Figure 3. For completeness the orbital velocity is compared with a Fenton Fourier approximation which is an alternative for higher order Stokes theories, these velocities are calculated with the software package ACES available at the civil engineering department of the UGhent.

**Damage analysis**

The software EPro calculates the erosion area \( A_e \) between two measured profiles to come up with a final 2D damage level. The area is determined by subtracting two averaged contour plots which results in a mean damage level, \( S_{\text{mean}} \), with a standard deviation. Further, the berm can be divided in three sections: the front slope which is the first slope encountered by the waves, the crest, and back slope orientated towards the passive absorption blocks. This is done for comparing damage levels of the sections

When analysing the results of the damage levels, it can be concluded that the tests with relative small damage levels result in significant values of the coefficient of variation \( \text{COV} \) within the results. This is because the amount of damage is lower or within the range of the measurement accuracy. For each test, the \( \text{COV} \) is determined in function of the mean damage level \( S \) (Figure 4).

The accuracy of recording the mean damage level and the effect of the motion vehicle on the laser profiling accuracy are assessed in this research. This has been achieved by measuring an identical rock berm profile three times, resulting in a standard variation of 11 percent.

**Shakedown test**

The quantification of the damage level of the berm should not include the initial stone settlements due to the propagation of the first waves over the structure generated in the laboratory. This would result in a larger damage number which does not provide representative “damage” numbers of the rock berm. To deal with this issue during the experiments, a shakedown test is performed before conducting a test by generating approx. 1000 waves. It means that about 333 waves characterized by a significant wave height \( H_s \) of 50 % of that of the design waves are propagating over the scale model.

A small case study is performed to check the relevance of the shakedown test with respect to the mean damage level of ‘Test series 1, 2 and 3’. The tests for which a ‘shakedown test’ has been conducted, resulted in a larger or equal mean damage level for ‘Test series 1, 2 and 3’. The larger values are remarkable and means that the placement of the stones do not have a noticeable influence on the mean damage levels. After doing this case study of each stone grading, the remaining experiments are conducted without a ‘shakedown test’.

**Mop in rock berm**

The necessity of using a mop in between the armour layer and the core material has been tested for practical reasons. The influence of the mop on the hydraulic stability is investigated, as the expectation is that the flow will be more blocked and pushed in the vertical direction. However, a small case study with three tests indicates that no clear conclusion can be drawn about the influence of the mop. Specifically two tests result in less damage (lower mean damage level) while the third one gives a larger value. For further analysis it is assumed that the tests are conservative in terms of the hydraulic stability of the
armour layer stones and the mop will be applied for the tests.

IV. RESULTS

Sensitivity study

The sensitivity study includes the investigation in the 2D damage level, S, when a geometric or hydraulic parameter during the experiments is changed. This is not always straightforward because of the possible large standard deviation on the low damage levels measured. For this reason, only the data sets with relatively large mean damage numbers (>10) are considered.

Influence of the water depth d: The damage levels show a variation when the water depth is adjusted. The bottom wave orbital motion velocity components increase with an decreasing water depth. This observation agrees with expectations based on the theoretical approach.

Influence of the significant wave height H_s: The change of significant wave height H_s is also noticeable in the velocity measurements u_w and mean damage levels S_mean. Again, larger significant wave heights cause larger bed shear stresses which induce more stone movements or higher mean damage levels.

Influence of the peak wave period T: Theoretically, an increase in peak wave period results in larger orbital velocities, which appears to be in agreement with the recorded data. However, the wave steepness is kept constant during the tests thus the influence of changing peak wave periods is accompanied by a change of the significant wave height. According to the formula of the horizontal orbital velocity component [5], the significant wave height has a larger influence than the wave period on the water particle velocities and the related bed shear stress.

Influence of the mean nominal stone diameter: Increase of the stone size results in a lower Shields parameter (Equation (1)) and thus in a lower mean damage level. Comparing ‘Test series 1, 2 and 3’, the trend of the mean damage is observed confirming that smaller stones are less stable.

Influence of the berm width: By comparing ‘Test series 1 and 4’, an evaluation of the rock berm width is performed. A larger width of the berm results in a larger mean damage level, S, because of the larger possible erosion area A_e as shown in Equation (2) [2].

To deal with this theoretical bias for comparisons with variable berm widths, a new formula is introduced from the literature. Van den Bos [2] proposes an expression for the adapted damage level S' which includes the berm width B (Equation (3)). It gives an evaluation of the eroded thickness of the armour layer.

\[ S' = \frac{A_e}{B d_{50}} = \frac{S}{B} d_{50} \]  

Considering the new damage definition (Equation (3)), it is concluded that the eroded thickness S' is smaller for a rock berm construction with a larger width B, unless a larger damage level S is present as can be seen in Figure 5.

Stability of rock berm sections

The initiation of motion starts at the crest where stones roll oscillatory or in the direction of the wave propagation. Also transport from the front slope towards the back slope occurs during the experiments. The opposite motion from the back slope towards the front slope is less pronounced, and thus only a few stones of the crest shift towards the front slope.

The visual observations coincide with the measured mean damage levels of the different sections S_mean_Section and can be represented against the total damage level S_Total (Equation (2)), see Figure 6. It is noted that the correlation coefficients of the back slope section is quite low which could be explained by the turbulence occurring at the back of the structure in combination with low damage levels.

Fig. 5: Influence of berm width on adapted mean damage level S' – test series 1 and 4

Influence of the berm width towards each section of the rock berm: When comparing ‘Test series 1 and 4’, the first conclusion is that the eroded thickness S' of the rock berm with a larger berm width is lower than that for a smaller one, as mentioned in ‘Influence of the berm width’.

A more detailed analysis of the crest section, separately from the rock berm, results in the same conclusion. The eroded thickness S' of the crest is smaller than for the wider berm. A possible explanation is that less turbulence effects occur over the berm cross-section. Oscillatory flow can be developed over a longer berm width. Another possibility is concerning the damage definition in Equation (2). As the eroded area A_e is used for the damage level quantification, a developed scour hole

\[ S'_c = \frac{A'_c}{B d_{50}} = \frac{S}{B} d_{50} \]
can be refilled by other stones. A larger rock berm width increases the possibility to have more accretion.

**Damage level in function of Shields stability parameter**

The mean damage levels are plotted against the Shields stability parameter in Figure 7. The shear stress \( \tau \) is estimated by using the horizontal orbital motion velocity measurements \( u_m \) and the theoretical value of the wave friction factor \( f_w \) [1].

Increasing Shields number indicates that the bed shear stresses are larger, which results in larger mean damage levels. The trend shows an exponential regression which is comparable with other experimental studies focussing on damage analysis in function of the bed shear stress [2].

A prediction formula for the mean damage level \( S_{\text{mean}} \) in function of the Shields stability parameter \( \Theta \) can be developed by using a power regression. It results in the expression of Equation (4), with a significant coefficient of variation (COV). Due to several uncertainties introduced during the measurements of the mean damage levels, the correlation coefficient is only 0.55. To give an illustration of the standard variation, the upper and lower boundary of the 90% confidence interval are drawn in Figure 7.

\[
S_{\text{mean}} = 8 \times 10^6 \Theta^{3.7}
\]  

(4)

**V. CONCLUSIONS**

Experimental tests of near bed structures in the large wave flume of UGhent are possible but rather limited until Shields numbers of 0.030. Using larger scaling factors as alternative (instead of 25) lead to smaller grains which are practical difficult for constructing the scale model. The hydraulic flow regime through the stones of the armour layer is more affected towards a laminar flow regime instead of a rough turbulent one. By using a smaller scaling factor on the other hand, it results in smaller significant waves heights (physical limitations) which means that smaller Shields numbers or bed shear stresses are achieved and thus lower mean damage levels at the rock berm.

During the tests, visual inspection shows that the main damage occurs at the crest where stones start to roll oscillatory or in the wave propagation direction. For the larger waves, transport of stones happens at the front and back section. However, stones move more in the wave propagation direction than towards the opposite direction. A possible reason can be a net mass transport of the fluid (Stokes drift). Regarding the total mean damage levels, on average 45% of the damage occurs at the crest, 35% at the front slope, and only 20% damage on the back slope. It means that the wave propagation has an influence on the sectional damage of the rock berm for transitional water.

The influence of changing the berm width should be accompanied by a new definition of the damage level. The adapted damage level or eroded thickness includes the berm width because a larger berm will always result in larger damage levels. However, the adapted damage definition concludes that larger berm-widths result in a lower value of the eroded thickness \( S^* \). A possible reason is that the present definition of the damage level with the parameter “eroded area” does not take into account a possible refill of a scour hole. In a larger berm width the scour holes have more chance to become refilled. For the crest and front slope section a decrease in eroded thickness is observed by increasing the berm width, while for the back slope section an increase is observed.

The non-contact laser profiler and electromagnetic velocity recorder fulfil their purpose in the research but have their limitations. The measurement accuracy of the profiler gives a coefficient of variation of 11% for the considered stone gradings. It results for each test in mean damage level with a significant standard deviation. The velocity recorder contrary is more
accurate between the same tests with a maximum standard deviation of 4 %. However, this device is rather robust and inflexible for measurements at other locations in the wave flume.

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List of abbreviations

AWA  Active Wave Absorption
AWASYS  Active Wave Absorption System
CEM  Coastal Engineering Manal
CETMEF  Centre d'Etudes Techniques Maritimes et Fluviales
CIRIA  Construction Industry Research and Information Association
CUR  Civieltechnisch Centrum Uitvoering, Research en Regelgeving
EPro  Erosion Profiling program
JDN  Jan De Nul NV
JONSWAP  Joint North Sea Wave Project
PIV  Particle image velocimetry
SWL  Still water level
UGhent  Ghent University – Universiteit Gent
WG  Wave gauge
FFT  Fast Fourier Transformation
DFT  Discrete Fourier Transformation
COV  Coefficient of variation - statistics
**List of symbols**

<table>
<thead>
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<th>Unit</th>
<th>Description</th>
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<td>[-]</td>
<td>Angle of slope</td>
</tr>
<tr>
<td>$\alpha_L$</td>
<td>[-]</td>
<td>Scaling factor at experiments</td>
</tr>
<tr>
<td>$\omega$</td>
<td>[rad/s]</td>
<td>Angular wave or radian frequency ($= 2\pi/T$)</td>
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<td>$\omega_p$</td>
<td>[rad/s]</td>
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<td>Scaling factor of laser profiling in wet conditions</td>
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<td>Morison-Type stability parameter</td>
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<td>[m$^2$/s]</td>
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<td>$\kappa$</td>
<td>[-]</td>
<td>Von Karman’s constant</td>
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<td>[rad]</td>
<td>Angle between current and wave propagation</td>
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<td>Specific density of stones</td>
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<td>Standard deviation of a dataset</td>
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<td>Mean total bed shear stress – current and waves</td>
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<td>Wave skin friction bed shear stress</td>
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<td>[rad/s]</td>
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<td>Turbulence-Type stability parameter</td>
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</table>
Dimensionless transport parameter

Density of stone – equal to $\rho_{\text{app}}$

Density of water

Wave boundary layer thickness

Amplitude orbital excursion

Amplitude horizontal orbital velocity component

Current average horizontal fluid velocity

Local horizontal flow acceleration

Local vertical flow acceleration

Local horizontal flow acceleration fluctuation

Erosion area

Wave horizontal orbital acceleration component

Wave vertical orbital acceleration component

Width of the berm or length of the crest

Constant of JONSWAP spectrum

Blockiness of stone

Wave celerity

Amplitude coefficient – wavemaker theory

Empirical bulk coefficient – effect of drag and lift forces combined

Current drag coefficient

Empirical added mass coefficient

Wave coefficient of reflection

Wave coefficient of transmission

Water level with respect to the seabed – water depth

Median sieve diameter of stone grading

Water level with respect to the crest level

Sieve diameter of stone grading – scale model

Nominal diameter of stone grading

Nominal diameter of stone grading - prototype

Median nominal diameter of stone grading

The x percent of the sieve curve from the rock grading

Dimensionless grain diameter

Total wave energy in one wave length per unit crest width

Wave frequency

Peak wave frequency

Wave friction factor

Lift force

Drag force

Resultant force force

Inertia force

Gravity constant

Wave height
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<td>Incident wave height</td>
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<td>Average wave height of the highest 1% of the waves (= maximum)</td>
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<td>$H^-$</td>
<td>[m]</td>
<td>Lowest significant wave heights in a test series</td>
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<td>[m$^{-2}$]</td>
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<td>[#]</td>
<td>Number of waves</td>
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<td>Porosity of bulk-placed materials</td>
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<td>Mean 2D adapted damage level over the length of the berm</td>
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<td>Duration of the time series – experimental test</td>
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<td>[m]</td>
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<td>Current velocity</td>
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<td>[m/s]</td>
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<td>Orbital velocity quantification of irregular waves</td>
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<td>[m/s]</td>
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<td>$z$</td>
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<td>Vertical or transversal axis/coordinate in 3D coordinate system</td>
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<td>[m]</td>
<td>Bed roughness length – zero-velocity level</td>
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Chapter 1  Introduction

1.1 Definition protective rock berms

Stability of a pipeline on the seabed can become critical due to hydrodynamic loading of waves and currents. The resistance against moving can be increased by making the pipe heavier, lying in a trench, or covering with a layer of stable rock. Using the last method means that the related protective rock berms should be designed to fulfil the requirements of the pipeline stability.

The placement of on-bottom rock dumping over a pipeline is defined as a near-bed rubble mound structure which can be approached as a submerged structure with a relatively low crest compared to the water depth. Vidal et al. (1998) mentioned that this is the case for ratios until 0.5 for the height of the crest over the water depth. Further it includes that the influence of possible wave breaking is not relevant on the stability (CIRIA, CUR, CETMEF, 2007) and that flow around the structure can be estimated by a wave theory. This is not the case with ‘submerged’ structures, which have a larger crest height. Remark that a clear definition of a near-bed structure with dimensions and hydraulic boundary conditions is not present in literature.

![Figure 1-1: Comparison near-bed and submerged structure (CIRIA, CUR, CETMEF, 2007)](image)
1.2 Design of granular near-bed structures

The design of a rock berm protection has no standard guideline or code available. Contrary, the design of pipelines under the rock is supported by the DNV-code (Det Norske Veritas (DNV), 2010). Conform the Rock Manual (CIRIA, CUR, CETMEF, 2007), a common parameter in rubble mound structures is the ‘stability’ which is defined by the ratio of load by resistance. In present design practice of any mechanical structure, the response of a granular near-bed structure is expressed in function of a load level or stability number which results in a stability-damage diagram. For the actual design it is possible to use two design approaches: critical stability design approach and allowable damage design approach.

The first design approach is schematized by a flow chart in Figure 1-2. The chart corresponds with a black-white design and needs a critical level of the stability or threshold of motion. Below the threshold stones are assumed to be stable and does not move (zero response of the structure). Exceeding the critical stability level means movement and is considered as unacceptable.

A possible alternative is the allowable damage design approach which accord better with the real behaviour of structural damage. The damage is characterized by a smooth rising design curve in function of the stability number (Figure 1-3). This method has the advantage that smaller stones can be chosen with a certain allowed degree of damage. Further, when the damage can be estimated with a significant degree of accuracy, the need for maintenance can be assessed over time. Contrary to the first approach, the parameter ‘allowable damage’ is now introduced in the flow chart, Figure 1-4. When the design curve is defined, the critical stability parameter is estimated by extrapolating the damage to zero.
1.3 Present design methodology

The present design method of the company Jan De Nul NV uses the critical stability approach with the Shields stability parameter, based on sediment transport and scour assessments for larger grains. The Shields parameter analyses the bed-shear stress which includes the horizontal orbital velocity of the water particles and the friction interaction with the stones. Other factors as turbulence, accelerations, stone grading, permeability, etc. are not considered within this design method.

A recent publication of Roulund, Sutherland, Todd, and Sterner (2016) gives recommendations for the quantification of the different variables in the Shields stability parameter and wave orbital velocities. The paper concerns the domain of irregular waves which are eventually combined with a current interaction. Two important parameters; near bed wave orbital velocity and wave friction factor are estimated by parametric expressions. The first one should be represented by $u_m$ (in an irregular sea state) which is defined as the multiplication of the root of two and the root mean square value of the near bed velocity $u_{rms}$. This is according to findings of Sumer and Fredsøe (2001) which performed tests to investigate scour around a pile. For the second parameter, wave friction factor, a continuous expression is proposed. It combines existing formula and depends on the amplitude orbital excursion and bed roughness.

More information about the parameters of the present design methodology and the origin of the Shields stability parameter is explained in Chapter 2 about the principles on hydraulic stability of stones in an armour layer for a rock berm protection. This chapter comes also in contact with other theories which are more recent but still not used in design practices. However, it gives a better understanding about the physics and disadvantages of the present design theory by Shields.
1.4 Problem statement

According to the present design methodology with the critical Shields stability method, a critical boundary or threshold of motion needs to be defined. Figure 1-5 shows the classical work which is initially performed by Shields (1936) and later extended by experiments with waves and/or currents, investigated by Soulsby (1997). The curve in the figure plots the maximal dimensionless shear stress (Shields parameter) $\theta_c$, towards the dimensionless grain size $D_\ast$. It indicates that the critical Shields parameter of 0.055 is rather conservative for rocks with a $D_\ast$ larger than 100 and which are exposed to waves only.

No conclusive research is performed to verify the critical boundary of 0.055 for specific wave conditions such that a study will be executed in this manuscript.

![Figure 1-5: Threshold of motion for waves and/or currents (Soulsby, 1997)](image-url)
1.5 Research objectives

The explanation of the problem statement is accompanied with research objectives for the investigation of the rock berm protection. Different aspects of testing near-bed structures with waves will be dealt and evaluated in purpose of upcoming research. An overview of the research objectives is enlisted.

Damage evaluation for specific hydraulic boundary conditions

The main goal of the research is evaluating the damage levels of commonly used rock berms with a median dimensionless grain size larger than 100 (Chapter 2), which are exposed to specific hydraulic boundary conditions. The damage evaluation will be done in function of the Shield parameter and end up with a prediction formula of the damage level of the armour layer with respect to this Shields parameter.

Sectional damage analysis of rock berm

The damage development of a rock berm protection will be described by a sectional damage analysis. The different sections in a cross-section; front slope, crest, and back slope will be evaluated separately in order to get a better understanding of the damage evolution.

Influence of rock berm-width on damage level

In practice the berm-width of rock protections is allowed to change. The influence of changing this geometrical parameter on the damage levels of the rock berm will be investigated. Additionally, a sectional damage analysis will be performed on a changing berm-width.

Knowhow of measurement methods

The damage levels and wave orbital velocity components will be measured by two test devices of Ghent university. The first device for the damage quantification is a non-contact laser profiler which is developed by Aalborg University. Secondly, a new electromagnetic velocity recorder is installed a method to process the obtained data will be proposed. Both devices will be analysed by and reviewed by multiple case studies.

Comparison with similar tests in literature

In literature, some research about comparable tests but with different boundary conditions is available. The experimental data of this research will be evaluated and compared with other prediction formulas or findings.
Chapter 2  Principles on hydraulic stability

2.1 Introduction

This chapter explains the basic theories which are related to the hydraulic stability of a near-bed structure. The first subchapter discusses the structural performance and properties of a near-bed structure. A next step is to explain the hydraulic performance in a sea state with irregular waves. The influence and interference with currents are briefly described, with experimental findings in literature. However, currents are not considered in the hydraulic model tests later. The following paragraph gives an overview about the stability parameters that are already investigated or still need some extra research with respect to the structural performance. Finally, it ends with a description of different damage definitions and the developed design methodologies.

2.2 Structural performance

2.2.1 Armour layer stability

Hydrodynamic loads as waves and currents can cause movement of the first layers of the armour layer units of the rock berm protection. The relevant displacements can lead to hydraulic instability until the protection of the pipeline is not present anymore. A second phenomenon of instability is the structural integrity of the stones itself. In this manuscript the focus goes to the first topic. It is assumed that the shape of the stones is invariable. The different stability approaches are discussed later in chapter 2.4.

Typical movements of the armour units are separated by different motions which are rocking, displacement of units out of the armour layer, sliding of a packet of armour units, and settlement due to compaction of the armour layer (Burcharth, 1993). The failure modes are summarized in Figure 2-1. The correlated definition of damage will be discussed later in chapter 2.5.

The structural parameters of the near-bed cross-section which are influencing the hydraulic stability of armour layers is discussed in next paragraphs.
2.2.2 Parameters related to cross-section

The structural dimensions and properties of the different materials in a rock berm protection have an influence on the hydraulic stability. A general overview about possible parameters with respect to the structure is represented in Figure 2-2.

The slope angle of the front and back slope is defined by symbol $\alpha$ and assumed to be equal and constant in the further analysis. Further, the height of the crest, with respect to the seabed, is defined by $h_{crest}$ and also constant. Contrary, the width of the crest is represented by $B$ and varies in this research. The last relevant parameter of the cross-section is the relative permeability of underlays and core to the armour layer. Van der Meer (1988) has tried to classify the permeability of a coastal structure in function of the different rock layers by a notional permeability factor $P$. This variable is determined by the ratio between the permeability parameters of the armour layer and core grading. An overview of the different values of $P$ is given in Figure 2-3.

Figure 2-1: Typical armour layer displacements (Burchart, 1993)
2.2.3 Parameters related to armour layer

The top layer of the rock berm or armour stones is directly exposed to the hydrodynamic loads. This layer can be composed of infinite combinations of stone grading but with respect to the availability of standard classes, gradings of the code EN 13383 are commonly used in design.

A first important ‘resistance’ parameter against instability concerns the weight of stones which is defined by the apparent stone mass density $\rho_{\text{app}}$ and the median nominal stone diameter $d_{50}$. Using a fixed ratio (a value of 1.15 (De Vos, 2008) is determined for the stones at the Ghent university laboratory) between the sieve diameter $d$ and the nominal diameter $d_{n}$ of an equivalent cube, the
average mass $M$ is determined with Eq. 2-1. The apparent mass density varies with the saturation level of the stone and needs to be used for the effective design. The value for this density is constant in the research.

$$M_{50} = (d_{50}/1.15)^3 \rho_{\text{app}}$$  
Eq. 2-1

With:
- $\rho_{\text{app}}$ [kg/m$^3$]  
  Apparent density of stone assumed 2650 kg/m$^3$
- $d_{50}$ [m]  
  Median stone diameter of grading
- $M_{50}$ [kg]  
  Median mass of stone grading

With respect to the weight of the stones under water and the use of stability formula later in this research, a relative buoyant density $\Delta$ is defined in Eq. 2-2.

$$\Delta = \frac{\rho_{\text{app}} - \rho_w}{\rho_w} = \frac{\rho_{\text{app}}}{\rho_w} - 1$$  
Eq. 2-2

With:
- $\Delta$ [-]  
  Relative buoyant density
- $\rho_{\text{app}}$ [kg/m$^3$]  
  Apparent density of stone
- $\rho_w$ [kg/m$^3$]  
  Density of water

Another relevant parameter is the blockiness $\text{BLc}$ which is a shape descriptor that equals the volume of a stone divided by the volume of the enclosed beam/cube, Eq. 2-3. The parameters are defined in Figure 2-4 with $M$ the mass of the stone. This parameter has an influence on the packing density and interlock.

$$\text{BLc} = \left(\frac{M}{\rho_{\text{app}}} \cdot \frac{1}{X \cdot Y \cdot Z}\right) \cdot 100$$  
Eq. 2-3

Figure 2-4: Left to right, $\text{BLc} = 80\%, 60\%, 40\%$ (CIRIA, CUR, CETMEF, 2007)

A third parameter is the mass distribution of the grading. The ratio $d_{85}/d_{15}$ of a grading gives an indication about the composition. $d_{85}$ and $d_{15}$ are representing the respectively values of the 85 percent and 15 percent of the relevant sieve curve. This topic has an influence of the permeability with respect to the water flow. The following definitions are expressed in The Rock Manual (CIRIA, CUR, CETMEF, 2007);

- Very wide grading with a relative small permeability: $d_{85}/d_{15} > 2.5$
- Wide grading: $1.5 < d_{85}/d_{15} < 2.5$
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- Narrow grading with a larger permeability: $d_{85}/d_{15} < 1.5$

Some last relevant parameters are the packing density and layer thickness. With the variable density it is mentioned to verify what the overall density of the armour layer is. The layer thickness plays an important role in the allowable damage of the armour layer, as an increase of armourstone coverings results in higher allowed damage (CIRIA, CUR, CETMEF, 2007).

2.3 Hydraulic performance

A rock berm protection is during the life time mainly exposed to near-bed velocities generated by hydrodynamic loads. The loads can be separated in two phenomena namely waves and currents which interfere in reality. The main focus and explanation will go about the hydraulic performance of waves, especially irregular waves in a sea state. Later in the chapter, a general insight of currents and the interaction with waves will be discussed to complete the topic of hydraulic loads.

2.3.1 Waves

Waves can be separated in monochromatic/regular waves or irregular waves, depending on the environmental conditions. Only for a swell - waves propagating outside wind field - the waves can be modelled by a regular wave type with longer period. In a fully developed sea state, waves in a wind field, the behaviour of waves is clearly irregular. The first group assumes constant wave characteristics in the time domain while the second has a specific range of different occurring waves in the time domain. Irregular waves are thus traditionally quantified by a certain spectrum, in this research the JONSWAP spectrum is used. For sediment transport purposes and the application in limited water depths where the waves interact with the sediments, it is the most appropriate spectrum (Soulsby, 1997). In this manuscript irregular wave theory are applied because this is closer related to the reality.

Irregular waves

The frequency spectrum of propagating waves in one direction (2D structure), gives the one-dimensional contribution of the energy level $E$ in function of the wave period $T$ or angular frequency $\omega$. Two most important spectra are the Pierson-Moskowitz (PM) spectrum (fully-developed waves in deep water) and the JONSWAP spectrum, represented in Figure 2-5. Measured spectra can be approximated by a semi-empirical formula with general parameters. The general variance-density spectra $S$ in function of the radian frequency $\omega$ is given by Eq. 2-4 and Eq. 2-5 (Soulsby, 1997). The peak-shape factor $\gamma$ in the equations is assumed to have a constant value of 3.3 which is supposed by Soulsby and used in this manuscript.
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\[ S(\omega) = B \left( \frac{H_s}{4} \right)^2 \frac{\omega_p^4}{\omega^5} \exp \left[ -\frac{5}{4} \left( \frac{\omega}{\omega_p} \right)^4 \right] \gamma \phi(\omega/\omega_p) \]  \hspace{1cm} \text{Eq. 2-4}

\[ \phi(\omega/\omega_p) = \exp \left[ -\frac{1}{2\beta^2} \left( \frac{\omega}{\omega_p} - 1 \right)^2 \right] \]  \hspace{1cm} \text{Eq. 2-5}

With:
- \( \omega \) \ [rad/s] Angular wave frequency
- \( \beta \) \ [-] Peak-shape parameter (\( \beta = 0.07 \) for \( \omega \leq \omega_p \); \( \beta = 0.09 \) for \( \omega > \omega_p \))
- \( \gamma \) \ [-] Peak-shape parameter
- \( \omega_p \) \ [rad/s] Peak angular wave frequency
- \( B \) \ [-] Constant of JONSWAP spectrum (\( B = 3.29 \))
- \( H_s \) \ [m] Significant wave height

Describing natural waves in the time domain is commonly done by the parameters; significant wave height \( H_s \) and mean period \( T_m \). As the analysis is done in the frequency domain, the parameters are approached by the zeroth moment \( m_0 \) and second moment \( m_2 \) of the wave density spectrum (frequency domain analysis). The significant wave height is often estimated in deep water by the integral of the variance in the spectrum and is defined as the spectral domain significant wave height \( H_{m0} \) (Eq. 2-6). The mean wave period \( T_m \) is determined by the zero and second order moment of the spectra (Eq. 2-7).

\[ H_s \approx H_{m0} = 4\sqrt{m_0} \]  \hspace{1cm} \text{Eq. 2-6}
\( T_m = \sqrt{m_0/m_2} \)  
Eq. 2-7

To have some quantifications in the time domain, the following relations can be derived for waves which are away of the breaking zone (USACE (U.S. Army Corps of Engineers), 2008);

- Average wave height of the highest one-third of the waves: \( H_{1/3} \approx H_s \),
- Zero crossing wave period: \( T_z \approx T_m \),
- Zero crossing wave period: \( T_z = 0.781 T_p \) (JONSWAP),
- Root-mean-square wave height: \( H_{rms} = H_s/\sqrt{2} \).

Away from the wave breaking zone the average wave height of the highest one-third of the waves \( H_{1/3} \) (time domain analysis) is approximately equal to \( H_s \). Further, the zero crossing wave period \( T_z \) is approached by \( T_m \). A measure of the average wave energy is the root-mean-square wave height \( H_{rms} \). This parameter is related to the significant wave height \( H_s \) by root of two (apart from near breaking). In very shallow water, near breaking, the ratio \( H_s/H_{m0} \) changes from 1 to about 1.1 for steep waves and from 1 to 1.3-1.4 at breaking for less steeper waves (Thompson & Vincent, 1985).

**Linear wave theory**

To describe wave characteristics as wave length, wave height, water particle velocities, etc., a certain wave theory should be used. For wave models according to the linear wave theory, the surface elevation can be described with a sinusoidal function (Eq. 2-8), derived from the velocity potential \( \phi \) (Dean & Dalrymple, 1991). The phase angle \( \theta \) is determined by the wave number \( k (= 2\pi/L) \) and the wave angular frequency \( \omega (= 2\pi/T) \). The amplitude in this function is half of the wave height \( H \). In this notation waves are fully described when the wave height \( H \), wave period \( T \) and water depth \( d \) are known.

\[
\eta(x,t) = \frac{1}{g} \left( \frac{\partial \phi}{\partial t} \right)_{z=0} = \frac{H}{2} \cos(\theta) \quad \text{Eq. 2-8}
\]

<table>
<thead>
<tr>
<th>With:</th>
<th>Unit</th>
</tr>
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<tbody>
<tr>
<td>( \eta )</td>
<td>[m]</td>
</tr>
<tr>
<td>( g )</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>( \phi )</td>
<td>[m²/s]</td>
</tr>
<tr>
<td>( H )</td>
<td>[m]</td>
</tr>
<tr>
<td>( \theta )</td>
<td>[rad]</td>
</tr>
</tbody>
</table>

The corresponding wave length \( L \) is larger for waves with a longer period, while waves become shorter when the water depth \( d \) decreases. These physical phenomena are expressed by a dispersion relation in function of the wave number and angular frequency, Eq. 2-9. The equation should be solved iteratively to calculate the wave length \( L \).
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\[ L = \frac{gT}{\omega} \tanh(kd) \]  
Eq. 2-9

With:  
- \( L \) [m] Wave length  
- \( g \) [m/s²] Gravity acceleration constant  
- \( \omega \) [rad/s] Angular wave frequency  
- \( d \) [m] Water depth  
- \( k \) [m⁻¹] Wave number

Notice that for deep water conditions \((kd > \pi/2)\) the wave length fully depends upon the wave period. In shallow water \((kd < \pi/10)\) conditions, expression Eq. 2-9 could be simplified to:

\[ L = \frac{T(gd)^{1/2}}{\omega} \]

Two types of velocity can be considered in waves. The first one is the propagation of a wave, also known as ‘wave celerity’ \( C \). In one wave period \( T \) a wave could travel its wave length \( L \), thus the celerity is expressed by \( L/T \). The second type of velocity is the motion of the water particles which can influence the seabed or near-bed structures. Some velocity profiles are shown in Figure 2-6.

![Velocity profiles at 4 locations (Troch P., 2007)](image)

Figure 2-6: Velocity profiles at 4 locations (Troch P., 2007)

According to airy theory, it is assumed that the water particles don’t interact with other water motions, by stating this only normal forces are important and shear forces are negligible. These types of flows are called irrotational because the water particles do not rotate. Due to this assumption and the velocity-components in the \( x \) and \( z \) direction could be expressed by means of the velocity potential \( \Phi \), whose gradient is at any point the velocity vector in the fluid. The corresponding velocity vector has a horizontal component ‘\( u \)’ (along \( x \)-axis) and vertical component ‘\( w \)’ (along \( z \)-axis), which are respectively given by equations Eq. 2-10 and Eq. 2-11. Considering the velocity potential which respect the linear boundary conditions, it gives the local velocity components in function of the water depth, wave height, and wave period (Eq. 2-12 & Eq. 2-13). These velocities vary with changing water depth and position \((z + d)\) along the depth-axis with the seabed as reference.
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\[ u = \frac{\delta \phi}{\delta x} \quad \text{Eq. 2-10} \]
\[ w = \frac{\delta \phi}{\delta z} \quad \text{Eq. 2-11} \]

The horizontal component reaches maximum values (both in positive and negative direction) when the phase angle becomes \(0, \pi, 2\pi, 3\pi\ldots\). The vertical component becomes maximum when the phase angle becomes \(\pi/2, 3\pi/2, 5\pi/2, 7\pi/2\ldots\).

In shallow water conditions, the horizontal velocity component is more dominant, water particles move back and forward. In deeper water, the particles have rather a circular motion. The water particle movements for deep water, transitional and shallow water conditions are summarized in Figure 2-7.

\[ u = \frac{HgT}{2L} \cosh \left( \frac{2\pi \left( z + d \right)}{L} \right) \cos \theta \quad \text{Eq. 2-12} \]
\[ w = \frac{HgT}{2L} \sinh \left( \frac{2\pi \left( z + d \right)}{L} \right) \sin \theta \quad \text{Eq. 2-13} \]

For completeness, the related accelerations \(a_x\) and \(a_z\) are given by equations Eq. 2-14 and Eq. 2-15. These are the first derivatives of the relevant velocity equation with respect to the time domain.

\[ a_x = \frac{g\eta H}{L} \cosh \left( \frac{2\pi \left( z + d \right)}{L} \right) \sin \theta \quad \text{Eq. 2-14} \]
\[ a_z = -\frac{g\eta H}{L} \sinh \left( \frac{2\pi \left( z + d \right)}{L} \right) \cos \theta \quad \text{Eq. 2-15} \]

---

**Figure 2-7**: Dependency of the orbital movement towards the water depth
The combination of linear waves with an irregular wave spectrum result in a continuous variation of the horizontal wave orbital velocities in each point beneath the water elevation. The best representation of the near bed orbital velocity in an irregular sea state $U_m$ is given by Eq. 2-16 (Roulund, Sutherland, Todd, & Sterner, 2016). The formula contains the standard deviation of the velocity or the root-mean-square value. An approximation of the root-mean-square horizontal orbital velocity $u_{rms}$ from a JONSWAP spectrum is proposed by Soulsby (2006) which is an exponential expression, Eq. 2-17. The accuracy of the equation is explained in his report and can have relative errors of up to 35 %, depending upon the ratio $T_n/T_z$. $T_n$ represents the natural scaling period and is equal to the root of the water depth $d$ divided by the gravity constant $g$ (Soulsby, 2006). Thus the accuracy depends on the water depth and wave period.

$$u_m = \sqrt{2}u_{rms}$$ \hspace{1cm} Eq. 2-16

$$u_{rms} = \frac{H_s}{4} \sqrt{\frac{g}{d}} \exp\left(-\frac{3.65}{T_z} \sqrt{\frac{d}{g}} \right)^{2.1} \hspace{1cm} Eq. 2-17$$

With: $\omega$ [rad/s] Angular wave frequency  
$d$ [m] Water depth  
$g$ [m/s$^2$] Gravity acceleration constant  
$H_s$ [m] Significant wave height  
$T_z$ [s] Zero crossing wave period  
$u_{rms}$ [m$^{-1}$] Root mean square of wave horizontal orbital velocity component

**Non-linear wave theory**

The use of airy wave theory is only acceptable for waves in deep water which have a small wave height (low wave steepness) but can give an initial idea about the quantification of orbital velocities. Depending on the conditions, other theories as higher orders of Stokes or Cnoidal theory can become more appropriate. A classification of the different wave theories is supposed by Le Méhauté (1976) and represented in Figure 2-8. The conditions that will be investigated later are mostly in the range of ‘intermediate depth’ with relatively large wave heights $H$.

An alternative for the higher order theory of Stokes with perturbation factors or the Cnoidal theory is the Fourier approach method. This procedure solves the Laplace equations and the fully non-linear boundary conditions by a numerical method which uses a Fourier series development with a certain order in combination with a stream function (Fenton, 1999). The advantage of this method is the applicability in both deep and shallow water.

The velocity potential $\varphi$ is unsteady for this method and the series approximation is visible in the potential equation (Eq. 2-18), horizontal velocity component $u$ (Eq. 2-19), and the vertical velocity component $w$ (Eq. 2-20). Where the wave lengths are calculated according to Cnoidal or Stokes theory depending if the wave length is respectively larger or smaller than a threshold wavelength $L_F$, which is determined upon experimental data defined by Fenton (CEM, 1996). The calculation of these parameters according to Fenton can be done by a software package ACES.
which is available at the civil engineering department of the UGhent. An important remark is that
the horizontal velocity components under the crest are larger than these under the through due to
the asymmetry of the waves.

\[
\phi(x,z,t) = (c - \bar{U})x + \sqrt{\frac{g}{k^2}} \sum_{j=1}^{N} B_j \frac{\cosh jkz}{\cosh jkd} \sin jk(x - ct) + C(t) \quad \text{Eq. 2-18}
\]

\[
u(x,z,t) = (c - \bar{U}) + \sqrt{\frac{g}{k}} \sum_{j=1}^{N} jB_j \frac{\cosh jkz}{\cosh jkd} \cos jk(x - ct) \quad \text{Eq. 2-19}
\]

\[
w(x,z,t) = \sqrt{\frac{g}{k}} \sum_{j=1}^{N} jB_j \frac{\sinh jkz}{\cosh jkd} \sin jk(x - ct) \quad \text{Eq. 2-20}
\]

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\phi)</td>
<td>(m^2/s)</td>
<td>Wave velocity potential</td>
</tr>
<tr>
<td>(\bar{U})</td>
<td>(m/s)</td>
<td>Current average horizontal fluid velocity</td>
</tr>
<tr>
<td>(B)</td>
<td>[-]</td>
<td>Dimensionless constants for a particular wave</td>
</tr>
<tr>
<td>(c)</td>
<td>(m/s)</td>
<td>Wave celerity</td>
</tr>
<tr>
<td>(d)</td>
<td>(m)</td>
<td>Water depth</td>
</tr>
<tr>
<td>(g)</td>
<td>(m/s^2)</td>
<td>Gravity acceleration constant</td>
</tr>
<tr>
<td>(k)</td>
<td>(m^{-1})</td>
<td>Wave number</td>
</tr>
<tr>
<td>(N)</td>
<td>[-]</td>
<td>Finite integer for series development</td>
</tr>
<tr>
<td>(t)</td>
<td>(s)</td>
<td>Time</td>
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<tr>
<td>(u)</td>
<td>(m/s)</td>
<td>Wave orbital horizontal velocity component</td>
</tr>
<tr>
<td>(w)</td>
<td>(m/s)</td>
<td>Wave orbital vertical velocity component</td>
</tr>
<tr>
<td>(x)</td>
<td>(m)</td>
<td>Horizontal coordinate</td>
</tr>
<tr>
<td>(z)</td>
<td>(m)</td>
<td>Vertical coordinate</td>
</tr>
</tbody>
</table>
Wave boundary layer

In transitional and shallow water, the waves interfere with the bed/structure and frictional effects are introduced. An oscillatory boundary layer appears in which the wave orbital velocity amplitude increases rapidly from zero at the bed to the peak value \( u \) at a certain vertical position \( z \) (van Rijn, 1993). The phenomenon is graphically shown in Figure 2-9, remark that \( \hat{u}_\delta \) represents the amplitude of the horizontal velocity component at the top of the boundary layer \( z \).

The thickness of the wave boundary layer \( \delta_w \) depends on the hydraulic flow regime. This condition is a function of different parameters which are outlined by two main parameters (Figure 2-10); the wave Reynolds number \( \text{Re}_w \) (Eq. 2-22) and the relative roughness \( r \) (Eq. 2-23). In case of turbulent flow, the thickness of the boundary can be approached by Eq. 2-21, with an error of about 20% (Fredsøe J., 1984). Besides this, the Nikuradse roughness \( k_s \) represents the physical grain roughness and depends on the surface prevailing during bed-load movements. According to literature this parameter is set equal to 2.5 times the median grain diameter \( d_{50} \) (Engelund & Hansen, 1967) and (Fredsoe & Deigaard, 1992). Remark that the assumption is based on experiments with sand instead of rock.

\[
\frac{\delta_w}{\hat{A}_\delta} = 0.15 \left( \frac{\hat{A}_\delta}{k_s} \right)^{-0.25} \quad \text{Eq. 2-21}
\]

\[
\text{Re}_w = \frac{\hat{u}_\delta \hat{A}_\delta}{v} \quad \text{Eq. 2-22}
\]

\[
r = \frac{\hat{A}_\delta}{k_s} \quad \text{Eq. 2-23}
\]

With:
- \( v \) [m\(^2\)/s] Kinematic viscosity coefficient
- \( \hat{A}_\delta \) [m] Amplitude orbital excursion (\( = \hat{u}_\delta/\omega \))
- \( \hat{u}_\delta \) [m/s] Amplitude horizontal velocity component just outside boundary layer
- \( k_s \) [m] Nikuradse roughness (\( = 2.5 d_{50} \))
- \( r \) [-] Relative roughness
- \( \text{Re}_w \) [-] Wave Reynolds number
- \( \delta_w \) [m] Wave boundary layer thickness

![Figure 2-9: Wave boundary layer (van Rijn, 1993)](image-url)
Bed shear stress and bed friction

In the boundary layer of previous paragraph, the waves exert friction forces during propagation. This lead to stresses, called the skin-friction shear stress $\tau_{ws}$, which are determined by a friction coefficient $f_w$. The physical phenomenon is quantified by Eq. 2-24 (Jonsson, 1966) and (Soulsby, 1997). The total bed shear stress related to waves $\tau_w$ is assumed to be equal to the defined skin-friction shear stress $\tau_{ws}$ ($\tau_w = \tau_{ws}$). The explanation is be given in next chapter 2.3.2 about currents. Thus the form-drag component and sediment transport component are neglected. The factor $f_w$ depends on the flow regime (turbulent or laminar) and can be estimated by empirical formula, based on measurements. A small overview is be given by Eq. 2-25 until Eq. 2-31. An error analysis of this coefficient $f_w$ concludes that the uncertainty of this parameter can have deviations of 50 % and larger (Soulsby, 1997). The last parameter, wave bottom horizontal orbital velocity $u_d$, is in literature mostly calculated by the linear wave theory.

$$\tau_{ws} = \frac{1}{2} \rho_w f_w u_d^2$$

Eq. 2-24

![Figure 2-10: Hydraulic regimes in oscillatory flow (van Rijn, 1993)](image)

**Laminar flow**

(Jonsson, 1966)

$$f_w = 2 \left( \frac{\bar{u}_d \bar{A}_d}{v} \right)^{-0.5}$$

Eq. 2-25

**Smooth turbulent flow**

(Jonsson, 1966)

$$f_w = 0.09 \left( \frac{\bar{u}_d \bar{A}_d}{v} \right)^{-0.2}$$

Eq. 2-26
Principles on hydraulic stability

Rough turbulent flow

\( 0.25 f_w^{-0.5} + \log(0.25 f_w^{-0.5}) = -0.08 + \log r \)

Eq. 2-27

With \( f_w = 0.3 \) for \( r \leq 1.57 \)

\( f_w = \exp[-6 + 5.2 \ r^{-0.19}] \)

Eq. 2-28

With \( f_w = 0.3 \) for \( r \leq 1.57 \)

\( f_w = 0.237 \ r^{-0.52} \)

Eq. 2-29

All flow conditions

\[
\frac{0.32}{f_w} = \left\{ \ln(6.36 \ r^{0.5}) - \ln \left[ 1 - \exp \left( -0.0262 \ \frac{\text{Re}_w f_w^{0.5}}{r} \right) \right] \right\}^2 + 1.64
\]

Eq. 2-30

\( f_w = \begin{cases} 
0.32(r)^{-0.8} & 0.2 < r < 2.92 \\
0.237(r)^{-0.52} & 2.92 < r < 727 \\
0.04(r)^{-0.25} & r \geq 727 
\end{cases} \)

Eq. 2-31

With:

- \( \nu \) \hspace{1cm} [m^2/s] \hspace{1cm} \text{Kinematic viscosity coefficient} \\
- \( \rho_w \) \hspace{1cm} [kg/m^3] \hspace{1cm} \text{Density of water} \\
- \( \tau_{ws} \) \hspace{1cm} [N/m^2] \hspace{1cm} \text{Wave skin friction bed shear stress} \\
- \( \tilde{A}_s \) \hspace{1cm} [m] \hspace{1cm} \text{Amplitude orbital excursion (} = \bar{u}_w / \omega \text{)} \\
- \( \bar{u}_w \) \hspace{1cm} [m/s] \hspace{1cm} \text{Amplitude wave bottom horizontal orbital velocity component} \\
- \( f_w \) \hspace{1cm} [-] \hspace{1cm} \text{Wave friction coefficient} \\
- \( r \) \hspace{1cm} [-] \hspace{1cm} \text{Relative roughness (} = \tilde{A}_s / k_s \text{)} \\
- \( \text{Re}_w \) \hspace{1cm} [-] \hspace{1cm} \text{Wave Reynolds number} \\
- \( u_b \) \hspace{1cm} [m/s] \hspace{1cm} \text{Wave bottom horizontal orbital velocity} \)
Breaking waves

Waves will break when the steepness (H/L) reaches a critical value. For shallow water conditions, horizontal bed and, irregular waves the limiting value is supposed by Eq. 2-32. In terms of stability of the rock berm protection, breaking waves play not a significant role (CIRIA, CUR, CETMEF, 2007). Vidal et al. (1998) mentioned also that for structures with a ratio of 0.1 until 0.5 for the height of the crest $h_{crest}$ over the water depth $d$, the flow around the structure is not affected by breaking waves and can be described for a fist approximation by a wave theory.

$$\gamma_{br} = \frac{H_s}{d} = 0.55$$  Eq. 2-32

With:
- $H_s$ [m] Significant wave height
- $d$ [m] Water depth
- $\gamma_{br}$ [-] Breaking coefficient

2.3.2 Currents

Currents may occur in the sea by tidal motion, wind-stress, atmospheric pressure gradients, wave-induced forces, etc. In the nearshore region wave-induced currents are mainly present whereas offshore a combination of tidal and meteorological forcing is dominant. The depth average velocity $\bar{U}$ is a popular parameter to quantify a current. It is often estimated by measurements and mathematically defined by Eq. 2-33. The bottom boundary of the integral depends on the bed roughness length $z_0$ which is defined in next paragraph and visually represented in Figure 2-11.

$$\bar{U} = \frac{1}{d} \int_{z_0}^d U(z) \, dz$$  Eq. 2-33

With:
- $\bar{U}$ [m/s] Current velocity depth-averaged
- $d$ [m] Water depth
- $U$ [m/s] Current velocity at height $z$
- $z$ [m] Vertical coordinate – height above seabed
- $z_0$ [m] Zero-velocity level ($u = 0$ at $z = z_0$) – bed roughness length

Velocity profile and boundary layer

A boundary layer near the bed occurs because of the friction with the bed, equivalent as explained for waves. In case of currents the thickness of this layer can have a height of several metres in deep water and appear over the total water column in shallow water. Due to this thickness a velocity profile exists that has a maximum just below the water surface. This velocity profile is commonly expressed with a logarithmic function, Eq. 2-34. In this function the Von Karman’s constant is set equal to 0.4 for sediment-induced effects on the velocity profile (Soulsby, 1997). Further, the current friction velocity $u_*$ is related to the total bed shear-stress $\tau_c$ which is explained in next paragraph.
Principles on hydraulic stability

\[ U = \frac{u_*}{\kappa} \ln \left( \frac{z}{z_0} \right) \quad \text{Eq. 2-34} \]

With:
- \( \kappa \) [\(-\)] Von Karman’s constant
- \( u_* \) [m/s] Current friction velocity (\( \tau_c = \rho_w u_*^2 \))
- \( U \) [m/s] Current velocity at height z
- \( z \) [m] Vertical coordinate – height above seabed
- \( z_0 \) [m] Zero-velocity level (\( u = 0 \) at \( z = z_0 \)) – bed roughness length

The bed roughness length \( z_0 \) depends on different parameters as current speed, viscosity of the water and the composition of the bed material. Experimental results by Nikuradse (1933) are mathematically fitted by Eq. 2-35 (Christoffersen & Jonsson, 1985) and (van Rijn, 1993). It is a general expression which is valid for all Reynolds numbers. In the situation of hydrodynamically rough flow (\( u_* k_s / \nu > 70 \)), the expression can be simplified to Eq. 2-36. The same can be considered for hydrodynamically smooth flow (\( u_* k_s / \nu < 5 \)), represented by equation Eq. 2-37.

\[ z_0 = \frac{k_s}{30} \left[ 1 - \exp \left( \frac{-u_* k_s}{27 \nu} \right) \right] + \frac{\nu}{9 u_*} \quad \text{Eq. 2-35} \]

\[ z_0 = \frac{k_s}{30} \quad \text{Eq. 2-36} \]

\[ z_0 = \frac{\nu}{9 u_*} \quad \text{Eq. 2-37} \]

With:
- \( u_* \) [m/s] Current friction velocity (\( \tau_c = \rho_w u_*^2 \))
- \( \nu \) [m\(^2\)/s] Kinematic viscosity coefficient
- \( k_s \) [m] Nikuradse roughness (=2.5 \( d_{50} \))
- \( z_0 \) [m] Zero-velocity level – bed roughness length

Bed shear stress and bed friction

The total bed shear stress \( \tau_c \) acting on a seabed is a function of different contributions due to:
- Skin friction component \( \tau_{cs} \): related to (and acting) on the sediment grains,
• Form drag component \( \tau_{cf} \): related to pressure field, induced by flow over ripples on the bed,

• Sediment-transport component \( \tau_{ct} \): related to momentum transfer to mobilise grains.

Assuming a flat bed and a relative small sediment transport combined with coarse grains (rocks instead of sand), the distinction between total and skin friction quantities is not relevant \( (\tau_c = \tau_{cs}) \) (Soulsby, 1997).

The bed shear stress or skin friction shear stress is related to the depth-average current velocity \( \bar{U} \) by the drag coefficient \( C_D \). This physical phenomenon is approached by a quadratic friction law, Eq. 2-38. For a flat bed, the coefficient \( C_D \) is defined by the bed roughness length and water depth, and can be written by a widely used logarithmic relationship (Eq. 2-39) (Soulsby, 1997).

\[
\tau_c = \rho_w C_D \bar{U}^2 ; \quad \text{Eq. 2-38}
\]

\[
C_D = \left[ \frac{\kappa}{Q + \ln(z_0/d)} \right]^2 ; \quad \text{Eq. 2-39}
\]

With:
- \( \kappa \) [-] Von Karman’s constant
- \( \rho_w \) [kg/m³] Density of water
- \( \bar{U} \) [m/s] Current velocity depth-averaged
- \( \tau_c \) [N/m²] Current total bed shear stress
- \( Q \) [-] Empirical constant of drag coefficient
- \( C_D \) [-] Current drag coefficient
- \( d \) [m] Water depth
- \( z_0 \) [m] Zero-velocity level – bed roughness length

2.3.3 Waves and currents combined

In real situations waves are often accompanied by currents which have an influence on the exerted hydrodynamic stresses and sediment dynamics. The interaction between both is not simply a linear summation of their individual behaviour. A first impact is the modification of the phase speed and wavelength of the waves what is leading to refraction. Further, the interaction of a steady layer (current) and an oscillatory boundary layer (waves) modifies the bed shear stress.

Wave length

An opposing or following current has an impact on the steepness of the waves. The related wave length \( L \) is calculated with an alternative expression than defined by Eq. 2-9. For the case of waves with a fixed wave period \( T \), the wave length decreases and the wave height \( H \) increases when an opposing (propagating in opposite direction) current is present. The contrary occurs when a current has the same flow direction as the wave-propagation. The expression to calculate the wavelength is represented by Eq. 2-40 (Soulsby, 1997). The left side is called the relative
radian wave frequency. Another conclusion is that for a sufficiently large opposing current ($\bar{U} > \omega/k$), a wave cannot propagate.

\[(\omega - \bar{U}k\cos\phi)^2 = gk \tan(kd)\]

Eq. 2-40

<table>
<thead>
<tr>
<th>With:</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\omega$</td>
<td>[rad/s]</td>
<td>Angular wave frequency</td>
</tr>
<tr>
<td>$\phi$</td>
<td>[rad]</td>
<td>Angle between current and wave propagation</td>
</tr>
<tr>
<td>$\bar{U}$</td>
<td>[m/s]</td>
<td>Current velocity depth-averaged</td>
</tr>
<tr>
<td>$d$</td>
<td>[m]</td>
<td>Water depth</td>
</tr>
<tr>
<td>$g$</td>
<td>[m/s²]</td>
<td>Gravity acceleration constant</td>
</tr>
<tr>
<td>$k$</td>
<td>[m⁻¹]</td>
<td>Wave number</td>
</tr>
</tbody>
</table>

**Velocity profile and bed shear stress**

Laboratory experiments have been done by researchers to investigate the interaction between waves and a current. The case of rough beds (rocks), the water particle velocities near the bed decrease when waves are superimposed to a following or an opposing current. The related relevant results are shown in Figure 2-13, remark the water depth $d$ is symbolized by ‘h’. Further, the current related bed shear stress can increase to a factor of 3 larger than for a current alone. The main reason is the change of the bed roughness which is not anymore related to the geometrical properties of the roughness elements. The apparent bed roughness $k_a$ for combined current and waves is thus larger than the physical bed roughness $k_s$, depending on the wave characteristics Figure 2-12.

A lot of theories are available in literature to describe the non-linear interaction between the wave and current boundary layers. Comparing different predictions of the mean $\tau_m$ and maximum $\tau_{\text{max}}$ bed shear stress, these parameters give differences of 30 to 40 percent between the models (Soulsby, 1993). Some comparison is graphically shown in Figure 2-14.

*[Figure 2-12: Influence of combined waves and current on bed roughness (van Rijn, 1993)]*
Figure 2-13: Influence of waves on current profile (Kemp & Simons, 1983)

More recently, a data-based method is derived (Soulsby, 1995) which induce a relatively simple equation to approach the mean (Eq. 2-41) and maximum (Eq. 2-42) bed shear stress for the situation of waves combined with a current.

\[
\tau_m = \tau_c \left[ 1 + 1.2 \left( \frac{\tau_w}{\tau_c + \tau_w} \right)^{3.2} \right]
\]  \hspace{1cm} \text{Eq. 2-41}

\[
\tau_{\text{max}} = \sqrt{\left[ (\tau_m + \tau_w \cos \phi)^2 + (\tau_w \sin \phi)^2 \right]}
\]  \hspace{1cm} \text{Eq. 2-42}

With:
- \(\tau_c\) [N/m²] Current total bed shear stress
- \(\tau_m\) [N/m²] Mean total bed shear stress – current and waves
- \(\tau_{\text{max}}\) [N/m²] Maximum total bed shear stress – current and waves
- \(\tau_w\) [N/m²] Wave total bed shear stress
- \(\phi\) [rad] Angle between current and wave propagation

As represented in literature, determination of the resulting bed shear is not straightforward to determine and specific for different situations. More detailed information of the comparison between the different methods can be found at the thesis of van den Bos (2006).
2.3.4 Interference with structure

A near-bed structure is exposed to waves and currents with varying energy levels through the time. The hydraulic performance of this structure with respect to waves can be described by the phenomena as wave transmission and wave reflection.

Wave transmission can be expressed by the coefficient of transmission $C_t$, calculated with the incident ($H_i$) and transmitted wave height ($H_t$) in Eq. 2-43. In case of a near-bed structure, the wave transmission should be in the range of 0.8 – 1.0 which depends on the interaction between the structure and the waves (CIRIA, CUR, CETMEF, 2007). For shallower water the transmission coefficient becomes smaller, compared with deep water circumstances.

$$C_t = \frac{H_t}{H_i} \quad \text{Eq. 2-43}$$

The effect of wave reflection is limited for rock berms in deep water, with a relative very large negative freeboard distance $R_f$. The wave reflection coefficient $C_r$ has values of 0 – 0.1 which are based on the reflected $H_r$ and incident wave height $H_i$ (Eq. 2-44).

$$C_r = \frac{H_r}{H_i} \quad \text{Eq. 2-44}$$
2.4 Stability parameters

A stability parameter is a parameter that is composed of different variables which are related to the stability of near-bed structures. For these designs, the ratio of load over strength is called the stability. The related threshold of motion or the critical stability defines when hydraulic instability occurs and the stones move.

In literature different stability parameters are defined, coming from different approaches and theories. A small overview is given in next subchapters about the present available methods or topics which are still under research. A first group (and oldest) of stability parameters are related to the hydraulic conditions of the waves or currents. Later on a second and still the present design method uses the bed shear stresses as the governing parameter. Finally, two theories which are based on the Morison theory (Morison J. R., 1950) are explained.

The experimental investigation later is limited to the load conditions of waves. In this manuscript, only the impact of waves on the near-bed structure is researched. The influence of the current in terms of velocities and bed shear stresses is very briefly introduced in previous chapter 2.3.3. Further, the Rock Manual (CIRIA, CUR, CETMEF, 2007) assumes that for ratios of the average current velocity to the horizontal wave orbital velocity with values smaller than 2.2, the effect of a current can be neglected. This means that the stability parameters in next chapters are mainly focussed on the impact of waves.

2.4.1 Mobility parameters

The first researchers of near-bed structures have used the parameters of the waves to come up with a stability parameter such as the amplitude value of the horizontal orbital velocity component near the bed \( \bar{u}_h \). Expressions with this variable are called Izbash-type stability parameters and can be noted in a dimensionless form \( \Omega \), based on the maximum wave orbital velocity (Eq. 2-45).

\[
\Omega = \frac{\bar{u}_h^2}{g \Delta d_{n50}} \tag{2-45}
\]

With:
- \( \Omega \) [\text{-}] Mobility stability parameter
- \( \bar{u}_h \) [m/s] Amplitude wave bottom horizontal orbital velocity component
- \( \Delta \) [\text{-}] Specific density \( = (\rho_s - \rho_w)/\rho_w \)
- \( d_{n50} \) [m] Median nominal diameter of stone grading
- \( g \) [m/s\(^2\)] Gravity constant

2.4.2 Shields parameter

A second parametrisation is the dimensionless Shields parameter \( \theta \) which uses the bed shear stress (section 2.3.1) as the main variable to define stability (Eq. 2-46). The Shields parameter takes into account the drag forces due to a pressure difference and the viscous skin friction forces induced by hydraulic loads (loads). The inertia forces related to accelerations are not considered. Lifting
forces are assumed to be negligible, compared to the submerged particle weight (resistance). Remark that initially the Shields parameter was introduced for experiments with steady currents and later the principle of bed shear stresses was extended for waves and combined waves plus current (Soulsby, 1997).

\[
\theta = \frac{\tau_w}{g (\rho_s - \rho_w) d_{50}} \quad \text{Eq. 2-46}
\]

With:
- \(\theta\) [\text{-}] Shields stability parameter
- \(\rho_s\) [\text{kg/m}^3] Density of stone – equal to \(\rho_{\text{app}}\)
- \(\tau_w\) [\text{N/m}^2] Wave total bed shear stress
- \(\rho_w\) [\text{kg/m}^3] Density of water
- \(d_{50}\) [\text{m}] Median stone diameter of grading
- \(G\) [\text{m/s}^2] Gravity constant

2.4.3 Morison-type

The Shield stability parameter is physically only correct in steady and uniform flow conditions; no accelerations are present. Considering an individual bed particle under flow accelerations, the stability can become less than for the case of steady flow (Dessens, 2004). Figure 2-15 gives an overview of the physical forces on a bed particle with:

- \(F_L\) Lift force,
- \(F_D\) Drag force,
- \(F_R\) Resultant force (Lift + Drag),
- \(F_M\) Inertia force,
- \(G\) Gravity force.

Including the inertia force in a stability-expression for stones, conform the method of Morison (1950), it results in a new stability parameter \(\Theta\) (Eq. 2-47). The coefficients \(C_M\) and \(C_a\) in expression are related to the bed and not to the individual stones.
The experiments show that the behaviour of stones is distinctively different for situations with the same velocities but different accelerations. Dessens (2004) did also a comparison with the shear stress based approach and concluded that a critical Shields parameter $\theta_{cr}$ of 0.025 – 0.030 is a reasonable measure to describe the threshold of motion in accelerated flow. This is considerably lower than the proposed value of 0.055 for uniform flow, see chapter 2.6.1.

$$\Theta = \frac{\frac{1}{2} C_B u^2 + C_M \frac{Du}{Dt} d_{n50}}{\Delta g d_{n50}}$$

Eq. 2-47

With:
- $\Theta$ [-] Morison-type stability parameter
- $\Delta$ [-] Specific density $= (\rho_s - \rho_w)/\rho_w$
- $C_B$ [-] Empirical bulk coefficient – effect of drag and lift forces combined
- $C_M$ [-] Empirical added mass coefficient
- $d_{n50}$ [m] Median nominal diameter of stone grading
- $\frac{Du}{Dt}$ [m/s$^2$] Material derivative $= \partial u/ \partial t + u \partial u/ \partial x$
- $g$ [m/s$^2$] Gravity constant
- $u$ [m/s] Local flow velocity

This approach which includes inertia forces is a recent development in this field of research. Some limitations obstruct a successful application in practice;

- A clear definition of the velocity and acceleration in the vicinity of stones is missing,
- Not clear how the theory should be applied in the combination of waves and a current,
- A reasonable estimation of the coefficients $C_M$ and $C_B$ is necessary and there is no reason to assume that the values are constant.

The use of this stability parameter requires the information about the variation of the accelerations in time (wave cycles) which is measured in the hydraulic model tests later. Further, the values of the constants $C_M$ and $C_B$ still need to be more investigated to get a better indication of these values.

### 2.4.4 Turbulence-based stability

In hydraulic structures turbulence can play an important influence in terms of hydraulic stability. In a turbulent flow the water motion is more violent and the damage to a structure becomes larger. A theoretical analysis by Hofland (2005) introduces two sources of turbulent forces: quasi-steady forces QSF and turbulence wall pressures TWP.

The QSF describes the instantaneous flow velocity as a fluctuation ($u'$) around a mean value: $u = u_{\text{mean}} + u'$. Thus the related drag and lift forces as described in previous chapters have a mean and fluctuating part as well with a certain probability distribution. Secondly, TWP are related to fluctuating accelerations ($a'$) due to fluctuation in pressure because of the Bernoulli effect. The pressures forces work on the entire volume of the stone and are related to the inertia forces. Finally, the two phenomena result in a new defied theoretical stability parameter $\psi$ (Hofland, 2005), Eq. 2-48, which should be interpreted as an end-goal of the currently ongoing research.
\[
\psi = \frac{C_B (u + u')^2 + C_M d_{50} (a + a')}{\Delta g d_{50}} \quad \text{Eq. 2-48}
\]

With:
- \(\psi\) [-] Turbulence-type stability parameter
- \(\Delta\) [-] Specific density of stones (= \((\rho_s - \rho_w)/\rho_w\))
- \(C_B\) [-] Empirical bulk coefficient – effect of drag and lift forces combined
- \(C_M\) [-] Empirical added mass coefficient
- \(d_{50}\) [m] Median diameter of stone grading
- \(g\) [m/s\(^2\)] Gravity constant
- \(u\) [m/s] Local horizontal flow velocity
- \(u'\) [m/s] Local horizontal flow velocity fluctuation
- \(a\) [m/s] Local horizontal flow acceleration
- \(a'\) [m/s] Local horizontal flow acceleration fluctuation

To use the turbulence-based expression a lot of questions still need to be answered and need more investigation;

- A clear definition of the local horizontal flow velocity \(u\), velocity fluctuation \(u'\), acceleration \(a\), and acceleration fluctuation \(a'\). Is the use of an analytical expression possible?

- What are the values of the coefficients \(C_B\) and \(C_M\), and what is their accuracy?

### 2.5 Damage quantification

A clear definition should be made to quantify damage of a hydraulic near-bed structure. Different options are possible to specify damage, two popular definitions are explained in this chapter; transport and erosion area.

#### 2.5.1 Transport

A first option is the use of a transport parameter, meaning the measurement of a certain volume of material that moves through a cross-section per unit width and unit time, denoted \(q_s\). In literature mostly a dimensionless form is used with the mathematical expression given by Eq. 2-49. This parameter \(\Phi\) can be linked to a stability parameter by a transport formula. In the specific case of waves, Madsen and Grant (1979) suggest some transport expressions from their experimental tests with ranges of validity.

In terms of waves, two situations are possible; symmetrical and non-symmetrical waves. In the first case, no net transport occurs in the wave propagation theoretically because of the equal orbital velocities under the crest and trough. Stones move only back and forth. The situation with different maximum orbital velocities in one wave cycle, a net transport can be estimated between \(q_{1/2}\) under the wave crest and \(q_{1/2}\) under the wave trough.
The transport based damage parameter is not used in this manuscript due to limitations of measurements in the test set-up. Further, transport rates are more appropriate for current-related phenomenon.

\[ \Phi_q = \frac{q_s}{\Delta g d_{n50}^2} \]  
Eq. 2-49

With:
- \( \Delta \) [-] Specific density (= \( \rho_s - \rho_w \)/\( \rho_w \))
- \( \Phi_q \) [-] Dimensionless transport parameter
- \( d_{n50} \) [m] Median nominal diameter of stone grading
- \( g \) [m/s\(^2\)] Gravity constant
- \( q_s \) [m\(^2\)/s] Volume transport through a cross-section per unit of time and width

2.5.2 Erosion area

The erosion area \( A_e \) is defined as the area of ‘missing’ material in a cross-section after the loads have been applied. This parameter has his origin in the design of breakwaters and revetments with the use of a 2D dimensionless erosion area/damage level \( S \), Eq. 2-50. An advantage of this definition is the possible link of near-bed structures with breakwaters. Besides this, the parameter of erosion area is a direct measure of the damage.

\[ S = \frac{A_e}{d_{n50}^2} \]  
Eq. 2-50

With:
- \( A_e \) [m\(^2\)] Erosion area
- \( d_{n50} \) [m] Median nominal diameter of stone grading
- \( S \) [-] Damage level

2.6 Design methodology

The principle of design methodology can be approached by two principles as explained in the introduction. The first method is a black box theory which uses a theoretically clear boundary between movement and no movement, the critical stability method. An alternative is the damage profile method where the damage is expressed in function of the stability number by a smooth function. This principle allows to define a boundary of allowable damage.
2.6.1 Critical stability method

Critical mobility parameter

When the mobility parameter or the horizontal orbital velocity component is used as stability parameter, a critical value of the threshold of motion can be defined. Until now, research in terms of hydraulic instability of near-bed structures with the horizontal orbital velocity as stability parameter have been studied by several investigators. According to van Rijn (1993), the available experimental data is limited to sand particles of 90 to 3300 μm and wave periods in the range of 4 to 15 seconds. One of the more popular equations, resulting from this dataset and with an average accuracy of about 25 %, is represented in Eq. 2-51 (Komar & Miller, 1975). The influence of an increasing wave period T in the expression is such that the critical peak value of orbital velocity near the bed \( u_{\delta,cr} \) also increases (Figure 2-16). Remark that other experimental results have shown that an opposite trend is also possible (van Rijn, 1993). Finally, when a certain wave theory of chapter 2.3.1 is applied in Eq. 2-51, the critical velocity can be related to a critical wave height.

\[
\begin{align*}
\left( \frac{u_{\delta,cr}}{(s-1)gd_{50}} \right)^2 &= 0.21 \left( \frac{2A_{\delta,cr}}{d_{50}} \right)^{0.5} & d_{50} < 500 \mu m \\
\left( \frac{u_{\delta,cr}}{(s-1)gd_{50}} \right)^2 &= 1.45 \left( \frac{2A_{\delta,cr}}{d_{50}} \right)^{0.25} & d_{50} \geq 500 \mu m
\end{align*}
\]

Eq. 2-51

With:
- \( u_{\delta,cr} \) [m/s] Critical amplitude horizontal orbital velocity component
- \( A_{\delta,cr} \) [m] Critical amplitude orbital excursion (= \( u_{\delta}/\omega \))
- \( d_{50} \) [m] Median stone diameter of grading
- \( g \) [m/s\(^2\)] Gravity acceleration constant
- \( s \) [-] Ratio of densities of stone and water (= \( \rho_s/\rho_w \))

Figure 2-16: Critical peak velocity - initiation of motion over a plane bed (van Rijn, 1993)
Critical Shields parameter

A second and a traditional design method is based on the critical shear stress concept by the Shields parameter $\theta$ (chapter 2.4.2). Unless the Shields curve is initially generated for steady current flows, the concept of shear stress is applied for waves. The variable indicates a maximum shear stress boundary where movement is initiated when the boundary $\theta_{cr}$ is exceeded. The related popular design curve, modified Shields curve, is given in Figure 2-17 with the critical Shields parameter in function of a dimensionless stone diameter. Shields has defined the critical stage as the situation when a minor part – 1 to 10% – of the bed is moving. Using the diagram for design purpose, there is a clear theoretical boundary between displacement or not. In reality this is less pronounced because of the stochastic character of the bed shear stress, stone size, and protrusions.

The critical Shields parameter $\theta_{cr}$ can be analysed in function of a dimensionless stone size $D_s$. The variable takes into account the median stone size diameter, densities of stone and water, and the kinematic viscosity of the fluid medium (Eq. 2-53). A first remark is on the influence of the stone grading, which has an influence on the permeability and is not considered in the expression. The origin is because Shields has done his experiments with uniform grains (sand grains) and on a flat horizontal bed. Secondly, the median stone diameter of a grading $d_{50}$ is used instead of the median nominal stone diameter $d_{n50}$ in the investigation of Shields. For practical reasons, it is commonly justified to replace $d_{50}$ by $d_{n50}$ in the Shields formula (van den Bos, 2006).

An algebraic expression of the critical Shields parameter $\theta_{cr}$ is initially fitted by Shields (Shields, 1936) and later adapted (Soulsby & Whitehouse, 1997), the final equation which is represented in Figure 2-17 is given by Eq. 2-52.

$$\theta_{cr} = \frac{0.30}{1 + 1.2 D_s} + 0.055 \left[ 1 - \exp(-0.02 D_s) \right]$$  \hspace{1cm} Eq. 2-52

$$D_s = \left[ \frac{g (s - 1)}{v^2} \right]^{1/3} d_{50}$$  \hspace{1cm} Eq. 2-53

With:
- $v$ [m$^2$/s] Kinematic viscosity coefficient
- $\theta_{cr}$ [-] Shields parameter
- $D_s$ [-] Dimensionless grain diameter
- $d_{50}$ [m] Median stone diameter of grading
- $g$ [m/s$^2$] Gravity constant
- $s$ [-] Ratio of densities of stones and water ($= \rho_s/\rho_w$)

For large stones the curve is thus assumed to be constant with a Shields stability number $\theta$ of 0.055. According to the Rock Manual (CIRIA, CUR, CETMEF, 2007), it is recommended to use the following values for the design of armourstone layers:

- $\theta_{cr} = 0.030 – 0.035$: Initiation of movement
- $\theta_{cr} = 0.05 – 0.055$: limited movement
2.6.2 Damage profile method

A possible design analysis is the damage profile method which has its roots in offshore engineering, where different researchers have investigated the development of damage to a specific type of near-bed structure such as a pipeline cover under wave attack. More concrete, the method predicts the damage in function of the dimensionless damage parameter $S$ for a certain condition and geometry of the structure.

This method has not lead to common design formula yet. Contrary, every researcher has made a design formula which includes their own variables and fitted through their experiments. A comparison shows that it is still not clear which parameters are most relevant and how these should be defined. For example, some researchers prefer to use a stability parameter based on shear stress while other prefer a parameter based on the wave velocity. Also the related definition of the governing velocity is not clear defined. Unless these uncertainties, researchers make the conclusion that it is fruitful to use a ‘black box’ approach were the damage is related to basic fluid properties. It is not useful to predict transport rates for these type of structures (van den Bos, 2006).

In Table 2-1 an overview of four papers is summarized to analyse the performed hydraulic tests with similar tests. The important parameters are compared and discussed in chapter 5.4.
### Table 2-1: Overview existing papers and design methods

<table>
<thead>
<tr>
<th></th>
<th>Vidal et al</th>
<th>Lómonaco and Klomp</th>
<th>Gent and Wallast</th>
<th>Sears</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Year</strong></td>
<td>1998</td>
<td>1997</td>
<td>2001</td>
<td>2005</td>
</tr>
<tr>
<td><strong>Current</strong></td>
<td>No</td>
<td>Steady</td>
<td>Steady</td>
<td>No</td>
</tr>
<tr>
<td><strong>Waves</strong></td>
<td>Regular</td>
<td>Irregular - JONSWAP</td>
<td>Irregular – JONSWAP</td>
<td>Irregular - JONSWAP</td>
</tr>
<tr>
<td><strong>Stability parameter investigated</strong></td>
<td>Shields</td>
<td>Mobility</td>
<td>Morison, Shields, Mobility</td>
<td>Mobility</td>
</tr>
<tr>
<td><strong>Preferred stability parameter</strong></td>
<td>Shields</td>
<td>Mobility</td>
<td>Mobility</td>
<td>Mobility</td>
</tr>
<tr>
<td><strong>Damage parameter investigated</strong></td>
<td>Damage S</td>
<td>Transport, Damage S</td>
<td>Damage</td>
<td>Damage</td>
</tr>
<tr>
<td><strong>Relations investigated</strong></td>
<td>1) Shields – Damage</td>
<td>1) Shields – Transport</td>
<td>1) Shields – Damage</td>
<td>1) Mobility – Damage</td>
</tr>
<tr>
<td><strong>Influence parameters</strong></td>
<td>/</td>
<td>Number of waves fixed ( N = 1000 ), slope ( \alpha )</td>
<td>Number of waves ( \sqrt{N} )</td>
<td>Number of waves by ( \log N )</td>
</tr>
<tr>
<td><strong>Definition velocity</strong></td>
<td>Undisturbed velocity at level of crest ( (u_{crest}) )</td>
<td>Orbital velocity at the bottom based on ( H_s ) and ( T_m ) ( (u_\delta) )</td>
<td>Orbital velocity at the crest based on ( H_s ) and ( T_m ) ( (u_{\delta,crest}) )</td>
<td>Orbital velocity at the crest based on ( H_s ) and ( T_o ) ( (u_{\delta,crest}) )</td>
</tr>
</tbody>
</table>
Chapter 3    Test facility

Before the description of the methodology of the hydraulic model tests, an overview of the available test facilities is given in this chapter. At the Department of Civil Engineering of Ghent University (UGhent), a wave flume is available since 2003 to perform hydraulic model tests. With the start of the PhD of Leen Devos at 2004 with the topic of scour protection around monopiles, extra test equipment to investigate damage developments of rubble mound structures became available.

3.1 Wave flume set-up

3.1.1 Wave flume

The 2D wave flume is mainly constructed of reinforced concrete with one side wall that is composed out of vertical transparent glass panels, supported by a steel frame over a distance of 15 metres. The inside dimensions of the rectangular channel are: length: 30 metres, width: 1 metres, and height: 1.2 metres. Further, a certain part of the flume over a distance of about 21 metres is provided with a return current flow canal. This element has a height of 0.20 metres and results in a reduction of the available wave flume height. Taking into account the capacity of the pump to generate a steady current, a maximum flow velocity in a section of 0.089/water depth (m/s) can be achieved.

The coordination system of the wave flume is general defined with the x-axis parallel to the horizontal wave propagation, y-axis orthogonal to the x-axis in the horizontal plane, and the z-axis orthogonal on the x-axis in the vertical direction. The level of the x-axis (z-ordinate equal to zero) coincides with the still water level (SWL).

3.1.2 Wave generation technique

The generation of waves be executed by a wave generator with a piston-type wave paddle. The paddle is fixed to a moving framework with a maximum stroke length $S_0$ of 1.50 metres and
moves on linear bearings. The generator is an electronic servo motor wherefrom the rotation is translated to a horizontal motion by a spindle drive. The frequency of the paddle position has a rate of 40 positions per second which approaches a fluent motion. Due to the horizontal motion a certain water volume in front of the paddle displaces and waves are generated in the wave flume. The generator can produce both regular and irregular waves, but during the tests only irregular waves are produced because of a better approximation of the sea state in reality.

![Wave paddle and sketch](image)

*Figure 3-1: Left: wave paddle at wave flume of UGhent - Right: sketch piston type paddle*

A relation between the wave maker and generated waves allows to describe the generation theoretically. Two theories are developed; the simplified theory for plane wave makers in shallow water (Galvin, 1964) and the complete wave maker theory for plane wave makers (Dean & Dalrymple, 1991). Based on the technical background material for the wave propagation in the flume (Troch P., 2000), the last type of wave maker theory is used in the configuration. Remark that the theory has been developed for both small amplitude motions of the paddle and small wave heights. Significant nonlinear effects occur when the paddle moves with large displacements, it results in waves of different size and shape at different locations away from the wave maker.

The formula of the surface elevation \( \eta(x, t) \), Eq. 3-1, is a superposition of a far field and near field solution. The first term is the far-field solution where the amplitude doesn’t change theoretically with location. The second term expresses the amplitude just in front of the wave paddle and takes into account active wave absorption (section 3.1.3). The disturbance or second term exists only in the zone near the paddle for \( x < 2L \) and has an exponential decrease with the distance.

Between the far-field wave height \( H \) and the stroke of the paddle \( S_0 \), a relation is expressed by the Bie\ssel transfer function (Bie\ssel & Suquet, 1951). For a piston-type wave paddle, the function is given by Eq. 3-1 and graphically represented in Figure 3-2. Remark in the picture that the production of waves with longer wave periods (frequency converge to zero), becomes problematic or limited.
\[ \eta(x, t) = c \cdot \sinh(kd) \cos(\omega t - kx) + \sum_{n=1}^{\infty} c_n \sin(k_n d)e^{-k_n x} \sin(\omega t) \]  
Eq. 3-1

\[ K_f = \frac{H}{S_0} = \frac{2 \cdot \sinh(kd)^2}{\sinh(kd) \cdot \cosh(kd) + kd} \]  
Eq. 3-2

With:
- \( \eta \) [m] Surface elevation
- \( \omega \) [rad/s] Wave angular or radian frequency
- \( c \) [-] Amplitude coefficient – wave maker theory
- \( d \) [m] Water depth
- \( H \) [m] Wave height
- \( k \) [m\(^{-1}\)] Wave number
- \( K_f \) [-] Biéssel transfer function parameter
- \( n \) [-] Integer
- \( S_0 \) [m] Stroke of wave paddle
- \( t \) [s] Time
- \( x \) [m] Horizontal axis/coordinate in 2D coordinate system

*Figure 3-2: Biéssel transfer function for piston-type wave maker (Troch P., 2016)*

**Software**

The generation of waves according to the Biéssel transfer function for a stroke wave paddle is implemented by the software script GENESYS (Troch & Versluys, 2004). With this PC-based application it is possible to produces both regular and irregular waves. To operate the wave generation and the measurement devices, the related script (as GENESYS for wave production) is grouped together in the software package LabVIEW, develop by National Instruments™.
3.1.3 Wave absorption

The reinforced concrete walls of the wave flume have a wave reflection coefficient (Eq. 2-44) of almost 100% and thus generating an unrealistic sea state for the experiments. To prevent this accumulation of wave energy in the wave flume as much as possible, wave absorption measures are installed. A first intervention is the use of passive wave absorption and a second one is the active wave absorption.

Passive wave absorption

A popular absorption technique is the use of a granular parabolic spending beach. Due to the shape, the water depth under the wave decreases exponentially and at a certain depth the waves start to break. The energy in the waves reduces significantly and thus also the reflection coefficient. A disadvantage is the initially installation and maintenance to restore the shape after every test.

An alternative is the use of recuperated plastic blocks which have perforated openings, in combination with absorption matrasses (blue elements in between the blocks), see Figure 3-3. Depending on the wave conditions of the test set-up (see chapter 4.3), this absorption technique gives reflection coefficients of about maximum 0.25. Conform historical tests in the wave flume, this is an acceptable result for the wave flume and the decision is made to use these elements. As is explained in section 4.4.1, the energy and wave height of the reflected waves (with a different frequency spectra) have no influence on the armour layer stability of the rock berm protection.

Figure 3-3: Passive absorption – perforated blocks in combination with absorption matrasses
Active wave absorption

The wave paddle is producing waves with a certain wave height, according to a certain spectrum. Further, the generation of waves can be influenced because of the presence of reflected waves in the wave flume. To maintain the intended incident waves, the related water level elevations should be compensated. This can be done by the use of the module AWASYS (Active Wave Absorption System) which is developed by Frigaard and Christensen (1994).

The system for active wave absorption involves digital filters and the placement of two wave gauges in front of the wave paddle and out of the near wave field (Eq. 3-1). Filters are used for the filtering of the measured surface elevations which are determined by the two wave gauges. After that, the reflected wave signal can be used to correct the wave paddle displacement, implemented in the GENESYS module. The specific disquisition of the AWASYS working method can be read in the paper of Frigaard and Andersen (2010).

3.2 Measurement devices and techniques

3.2.1 Wave gauges

A fundamental element in the wave flume operation is the use of wave gauges. A single wave gauge is composed out of two vertical metal rods, referred to as electrodes. The distance between the rods should remain constant during each test and is prevented due to the red top and bottom tip. Further, the electrodes are partially submerged until about one third of their length beneath the still water level (SWL). This set-up is important and should be adapted for every test to ensure that the wave crest and trough of every wave can be measured. The principle is to measure the conductivity between two parallel electrodes which produces an output voltage that is proportional to the immersed part of the electrodes. The wave gauges are of type 202 and manufactured by DHI.

The set-up for a rock berm stability investigation needs 8 wave gauges for analysis. The first two (AWA1 and AWA 2) are used for the active wave absorption in front of the wave paddle, as was explained in previous section. Further, three wave gauges should be placed in front and after the model in the wave flume, in total this leads to six elements.

The three wave gauges – WG1, WG2, and WG3 – in front of the scale model measure the incident and reflecting surface elevations from the structure. The numbering of wave gauges respects the
consecutive distance to the wave paddle, the closest one to the wave paddle has number 1. To determine the distance between the devices, attention should be paid that one wave should include the 3 wave gauges for the reflection analysis later. Recommendations of the specific distances for irregular waves are given by Mansard and Funke (1980), and summarized by Eq. 3-3, Eq. 3-4 and Eq. 3-5. Remark that the distance between wave gauge ‘n’ and wave gauge ‘m’ is symbolized by $x_{n,m}$. Further, the distance between WG1 and WG3 should be adapted for every test with a different peak period $T_p$.

After the scale model, three wave gauges – WG4, WG5, and WG6 are installed to measure the reflecting waves from the passive wave absorption. Besides this, the eventually wave modification due to the scale model can be investigated. The distances between these gauges have the same recommendations as mentioned for the three wave gauges in front of the scale model.

\[
x_{1,2} = x_{4,5} = \frac{L_p}{10} \quad \text{Eq. 3-3}
\]

\[
\frac{L_p}{6} < x_{1,3} = x_{4,6} < \frac{L_p}{3} \quad \text{Eq. 3-4}
\]

\[
x_{1,3} = x_{4,5} \neq \left( \frac{L_p}{5}; \frac{3L_p}{10} \right) \quad \text{Eq. 3-5}
\]

With:
- $L_p$ [m] Wave length for peak wave period
- $n$, $m$ [-] Integer
- $x_{n,m}$ [m] Distance between wave gauge $n$ and wave gauge $m$

### 3.2.2 Electromagnetic velocity recorder

On the bottom of the wave flume, an electromagnetic velocity recorder is installed to measure the water particle velocities in a certain local volume (spherical around sensor) of the wave flume. The principle – based in Faraday’s law of induction – is that a voltage will be generated when a conductor is moving in an electromagnetic field. The model type is a Model 802 with a spherical sensor and suitable for non-laminar flow applications, which is manufactured by Valeport.

![Model 802 - Electromagnetic flow sensor positioned in wave flume](image)

The location of the spherical sensor is set at the crest-level (8 cm above the bottom) of the rock berm protection in an undisturbed velocity profile. The choice of this set-up is according to the
tests of Vidal et al. (1998). Alternative locations, such as the placement on the crest of the rock berm, are not possible because of the instrument dimensions and the related significant influence on the flow pattern of the water over the structure. A picture of the device (Model 802) with positioning in the wave flume, is shown in Figure 3-5. Remark that the sensor is not located in the middle of the wave flume due to the dimensions of the equipment. To measure the water particle velocities parallel with the wave propagation direction, the sensor is located more to the concrete wall side at a distance of about 20 centimetres. Theoretically, the wave orbital velocities are the same over the whole wave flume-width in a 2D wave flume.

The technical specifications of the device with a spherical sensor (diameter 3.2 cm) mention a measurement accuracy of ± 1% in general. The data recording is done by the open-source software PuTTY.

3.2.3 Laser profiler

The determination of the erosion area $A_e$ or the equivalent damage number $S_d$ (section 2.5.2) is determined by measuring the armour layer profile in a 3D-space, before and after each wave test. To obtain this profile a distance measurement with a laser sensor - in combination with an automated motion vehicle – is executed.

Coordination system

The laser profiler with its software uses a different coordination system than defined by the wave flume;

- X-axis: direction parallel with the berm length and parallel with the wave flume width,
- Y-axis: direction parallel with berm height and parallel with the wave flume height,
- Z-axis: direction parallel with the berm width and parallel with the flume length.

The scanning area is defined as; the scan length parallel with the x-axis, and the scan width parallel with the z-axis.

Motion vehicle

The machine or laser profiler can move along three axes by pinion and rack mechanisms. The vehicle is driven by a high precision step-motor with a resolution of 1 millimetre. To operate the profiling, the software EPro (Erosion Profiling program) is which is fabricated in 2006 at the Department Hydraulic and Coastal Engineering group of the Aalborg University.
In the laboratory, a watertight framework is foreseen to allow measuring under water. This allows a faster execution of the tests – without pumping in and out of the water in the wave flume – to perform the laser profiling. Further, the scan-procedure can be executed by different patterns of motion, represented in Figure 3-7. According to the thesis of Van Tongeren and Dewitte (2016), path number two (right picture of Figure 3-6) from the figure is advised in order to reduce the profiling and the operation time to a minimum. A second choice for profiling is the operation in “fast” mode. Due to this setting the profiler does not stop at each measurement point, thus moving and collecting data continuously.
**Laser sensor**

The model of the laser sensor is a ‘optoNCDT 1402’ which is fabricated by Micro-Epsilon and designed for; displacement, distance, and position measurement. The principle is a laser triangulation which is the combination of a laser diode (projecting of light spot) and an optical receiver. An important advantage is that relatively small parts can be measured by a non-contact method. Some important technical characteristics of the device are:

- Measuring range (distance to target):
  - Above water: 55 – 256 millimetres,
  - Below water: 76 – 343 millimetres,
- Precision of ±2 millimetres above and below water,
- Digital data output.

The distance resolution between two measurement points in the x and z direction is set to 5 mm because of the smaller stones (compared with the thesis of Van Tongeren and Dewitte). A smaller resolution would result in an extremely long measurement time of 40 minutes per profile. Another important setting is the use of a reference point with a coordinate (x, y, z) to compare different profiles with each other later. Respecting the coordinates of the software (shown in Figure 3-6), a bolt is installed at the bottom of the flume with a certain (x, z) coordinate. The height of the laser is marked on the vertical bar which indicates the y-coordinate of the laser.

The recorded data of the laser and the control of the motion vehicle is processed by the software EPro. It is possible with EPro to show graphically the profiles of the rock berms (in 2D- or in 3D-space) and calculate/generate afterwards the damage level $S_d$ between two measured profiles. The determination of the damage number is explained in section 4.4.5.

**Measuring dry and under water**

When measuring a profile with the laser, there is a difference in settings between measuring dry (water pumped out of the flume) and measuring under water. This is due to refraction in the transfer from a gas medium (air) in the framework to a fluid medium (fresh water). It means that measured distances under water are not representative for analysis later and thus a certain scaling factor $\xi$ needs to be applied. A calibration block is used and measured with a calliper. With the assumption of a linear relationship between the measured distance dry and wet, the scaling factor is determined by a small statistical research. Profiling the block 3 times wet and 3 times dry gives an average scaling factor $\xi$ with a ratio of 0.764 (distance wet/distance dry) with a standard deviation $\sigma$ of 2.6 %. This scale factor, which coincides with the refractive coefficient of water is used in the analysis to use both dry and wet profiles.
3.2.4 More tools

Water characteristics

The kinematic viscosity is used to make the grain diameter dimensionless as mentioned in section 2.6.1, this parameter is calculated by dividing the absolute viscosity with the fluid mass density. The density depends on its turn upon the temperature of the water.

While performing tests, the wave flume is emptied and filled after each test as the scale model needs to be reconstructed. The transfer of water towards an underground storage tank could lead to differences in temperature of the water. To reduce the related variation as good as possible the water was kept inside the wave flume at the end of the day.

Additionally, the water density and temperature are measured once during testing. The temperature is measured with a FLUKE 51II thermometer which indicates an average temperature of 16 °C. Density measurements can be done with a hydrometer H100 of VWR® international and indicates an average density of 1000 kg/m³. The results determine a kinematic viscosity of $1.1 \times 10^{-6}$ m²/s in model conditions.

Berm construction and damage visualisation

Before and after each test a picture of the constructed scale model is captured. The location of each picture is kept constant by means of an angular profile over the cross-section of the flume,
this makes it easy to compare the evolution of the damage after passing of a wave train. The pictures are taken with a Nikon digital camera D5100, 16.2 mega pixels.

Other tools like angular profiles and rulers are used to construct the berm each time in the same way or to measure the intermediate distance between the different wave gauges. The water depth can also be measured to check the water level conditions.

### 3.3 Overview experimental set-up and wave flume operation

A schematic overview of the wave-flume set-up with the different instruments is given in Figure 3-9. The operation with LabVIEW with the different submenus is shortly summarized and is mainly based on Van Doorslaer (2008).

- **Calibration wave height meters**: a linear relationship between the surface elevation of the water and the electric resistance of the resistive wave gauges will be determined by measuring the resistance at two locations below the water surface.

- **Time series wave height meters**: the surface elevation of the water level can be visually checked in this submenu.

- **Wave generation**: the time series for the wave train (and the related paddle displacement) conform a certain wave spectrum (JONSWAP) will be generated in this submenu. The following parameters are necessary for the time series; wave peak period $T_p$, significant wave height $H_s$, water depth $d$, peak enhancement factor $\gamma$, and the duration of the test.

- **Wave flume operation**: the effective production of the waves – according to the time series of the previous submenu – will be executed in the flume. The time series for the paddle displacement will be sent to the electro-servo motor with a frequency of 40 positions/ second. This results visually in a fluent motion.

- **Analysis wave data**: the recorded data of the wave gauges can be analysed in time- and frequency domain. This submenu is not used and replaced by an analysis in WaveLab.
Figure 3-9: General wave flume set-up – dimensions in centimetres
Chapter 4  Methodology of experimental model tests

The chapter describes the methodology for the experimental research of the rock berm protection. To develop an experimental test matrix with a scale model, the rock berm dimensions and hydraulic boundary conditions of the tests are based on a prototype structure. This can be done by using an appropriate scaling law and taking into account the physical limits of the available wave flume. The next step contains the explanation of the data processing which is necessary to obtain the next chapter with results.

4.1 Prototype structure

The starting point to define the scale model for the test set-up is using a prototype structure on full scale, a construction in reality. The related geometric and hydraulic boundary conditions of this model are based on suggested dimension by the collaborative company Jan De Nul (JDN).

The geometrical dimensions of the berm – which are commonly used in practice - are summarized in Figure 4-1. The berm width can vary between 2 and 5 metres while the height has an average value of 2 metres. The slope is assumed to be fixed with a ratio of 1:2.5 (vertical to horizontal distance). The rock for the armour layer can have different standard gradings; LMA 4-50, LMA 10-60, and LMA 40-200, with their related median nominal diameters $d_{50}$. The core has a rock grading of 1-3 inches.

![Figure 4-1: Prototype dimensions of a pipeline rock berm protection – proposed by JDN](image)
Hydraulic boundary conditions are obtained by measurements with buoys located in deep (salt) water of the North Sea. From these readings certain hydraulic conditions are obtained, represented in Table 4-1. Due to physical boundaries of the wave flume (section 4.2) these conditions are adapted. The changed wave and current characteristics are given in Table 4-1 for the prototype. Remark that currents are not considered in this manuscript.

<table>
<thead>
<tr>
<th>Water depth</th>
<th>Suggested JDN</th>
<th>Prototype</th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height</td>
<td>6.2 m</td>
<td>3.4 m – 4.5 m</td>
</tr>
<tr>
<td>Peak wave period</td>
<td>11 s</td>
<td>9.65 s – 12.83 s</td>
</tr>
<tr>
<td>Average Current velocity</td>
<td>1.2 m/s</td>
<td>/</td>
</tr>
</tbody>
</table>

### 4.2 Scale model

In order to do experimental tests of the prototype in the wave flume, the dimensions and hydraulic loadings are scaled. The principle of scaling is according to the Froude law; it ensures a correct scaling of the gravity forces. As surface waves are gravity driven, the Froude principle guarantees that wave forces are appropriate scaled in the wave flume. A disadvantage is the incorrect scaling of the viscous forces and the related flow conditions through the stones.

The choice of an appropriate scaling factor for the model is based on the characteristics of the available flume and the execution of the scale model construction. For the available wave flume (Chapter 3), a compromise can be found at a scaling factor $\alpha_L$ of 25. A larger number results in smaller stones in the scale model which is not more executable in the lab. Smaller values result in more limited wave conditions (physical limitation of flume) which produce less damage on the structure.

#### 4.2.1 Stone gradings of rock berm

Rock berm protections are commonly constructed with standardized rock gradings. The following gradings for armour layers are often used in practice for the suggested wave conditions; LMA 5-40, LMA 10-60, and LMA 40-200. These compositions have respectively a median nominal diameter $d_{50}$ of 17 cm, 21 cm, and 32 cm. The core is built of rock stones 1-3” which has a median nominal diameter of 4.4 cm.

To determine the sieve curve of the scaled stone gradings for the wave flume, two extra topics besides the scaling factor $\alpha_L$ should be considered. A certain “transformation” formula is developed. A first point of attention is the change from salt water in the sea to fresh water in the wave flume. This effect can be considered in the formula by using a ratio of relative buoyancies.
The second consideration – to obtain a sieve curve of the grading – is the use of the ratio factor 1.15 (0) between the sieve diameter d and the related nominal diameter \( d_n \).

The scaling formula to transform the nominal diameters of the stones in the prototype structure into stones for the scale model is expressed by Eq. 4-1. This means that the considered armour layers have a scaled median nominal diameters \( d_{n50} \) of respectively; 6.71, 8.05, and 12.40 millimetres. The core has a \( d_{n50} \) of 1.83 millimetres (coarse sand).

When a logarithmic interpolation is used between the scaled sieve diameters of a grading, the sieve curve of the scaled rock gradings for the wave flume can be determined. The three curves are represented in annex B.

\[
d_m = 1.15 \frac{1}{\alpha} \Delta \Delta d_{np}
\]

Eq. 4-1

<table>
<thead>
<tr>
<th>With:</th>
<th>( \alpha )</th>
<th>Scale factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_m )</td>
<td>[m]</td>
<td>Sieve diameter of stone grading – scale model</td>
</tr>
<tr>
<td>( d_{np} )</td>
<td>[m]</td>
<td>Nominal diameter of stone grading – prototype</td>
</tr>
<tr>
<td>( \Delta_m )</td>
<td>[-]</td>
<td>Relative buoyancy in model</td>
</tr>
<tr>
<td>( \Delta_p )</td>
<td>[-]</td>
<td>Relative buoyancy in prototype</td>
</tr>
</tbody>
</table>

To study the influence of the rock grading on the damage levels, the dimensional stone size \( D^* \) of section 2.6.1 are used in the analysis of the results. By considering a constant kinematic viscosity of the fresh water (\( \nu \)) and a ratio of densities of water and stones (\( s \)), an overview of \( D^* \) in function of the three stone gradings is given in Table 4-2.

Table 4-2: Stone gradings and dimensionless grain diameter of scale models

<table>
<thead>
<tr>
<th>Scaled grading</th>
<th>Median nominal stone diameter ( d_{n50} ) [mm]</th>
<th>Dimensionless stone diameter ( D^* ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>LMA 5-40</td>
<td>6.71</td>
<td>158</td>
</tr>
<tr>
<td>LMA 10-60</td>
<td>8.05</td>
<td>190</td>
</tr>
<tr>
<td>LMA 40-200</td>
<td>12.40</td>
<td>293</td>
</tr>
</tbody>
</table>

### 4.2.2 Configuration of rock berm

In practice a wide arrangement of geometrical dimensions for rock berms is possible. However, the experimental tests have limited variations in terms of berm dimensions. The choice is made to fix the height of the berm or crest height \( h_{crest} \) (8 cm) and the slope angle \( \alpha \) (cotangent of 2.5) during all the test set-ups.

The width of the berm \( B \) is investigated mainly for a size of 8 centimetres. The stones with a median nominal diameter of 8.05 mm (LMA 10-60) are placed in an extra test series with a larger berm-width of 20 centimetres. The thickness of each armour layer \( t \) is set equal to the double of the nominal diameter of the relevant grading. This is a common configuration of armour layers in coastal structures which are composed of rocks. Thus the different test series have a mathematical thickness \( t \) of two times \( d_{n50} \).
Rock berm models

A limitation is the availability of stones with a sieve diameter smaller than 2 millimetres (coarse sand). Practical problems appear in terms of reconstruction operations when smaller stones are used, concluding that the core is built by stones with a sieve diameter of about 2 millimetres.

Four different cross-section models are constructed in the experimental research, according to the test matrix that is discussed in chapter 4.3. Models 1, 2, and 3 have the same configuration in terms of the berm-width B but with a different rock grading for the armour layer (and a different layer thickness). Figure 4-2 shows a cross-section of test model 2 with a scaled grading LMA 5-40 as armour layer. Figure 4-4 shows the cross-section of test model 4. The berm width is extended to 20 centimetres and the armour layer has a rock grading with median nominal diameter \( d_{50} \) of 8.05 millimetres (scaled LMA 10-60).

Notional permeability of rock berm models

An important time consumption in the construction of the scale models for every test set-up is the practical handling of the relative small stones. Rebuilding the structure saves a lot of time (couple of hours) and energy when a mop is installed between the core and armour layer (Figure 4-4). The choice of using a mop have an impact on the notional permeability of the structure (chapter 2.2.2). It is possible that the impact is limited because the difference between having a core of sand or using a geotextile on an oscillatory flow in a scale model can be questioned. On the other hand, using a geotextile will block the water flow more which can result in larger uplift forces on the
stones. To have an initial indication on the hydraulic stability, a small case study about applying a mop is performed in chapter 4.4.7.

![Figure 4-4: Left: berm construction with mop - Right: berm construction without mop](image)

### 4.2.3 Hydraulic boundary conditions

#### Water depth and significant wave height

Testing of water depths between 20 and 30 metres in prototype conditions result in water depths of 0.80 until 1.20 metres in the wave flume for a scale factor of 25. As mentioned in Chapter 3 the depth of the wave flume is limited to 1.20 metres. Besides this, a certain space from the top level of the basin should be provided to have an optimal operation of the measuring equipment. The resulting maximum feasible water depth equals 0.90 metres (SWL + H/2 + return flow channel). To prevent the use of a smaller scale (large scaling factor) and a significant decrease of the bed shear stresses, the water depth is adapted to a lower value.

Keeping in mind the criterion that is mentioned in chapter 2.3.1 for breaking waves, a maximum significant wave height of 18 centimetres can be used in combination with a water depth that changes between 45 and 50 centimetres. In prototype this represents a significant wave height of 4.5 metres with a water depth that changes between 11.25 and 12.50 metres. Besides these settings, the maximum wave height in the wave flume is approximately 33.5 centimetres ($H_{\text{max}} = 1.86 H_{1/3} \approx 1.86 H_s$) or 8.37 metres in prototype (irregular waves). This means that a certain percentage of the waves breaks.

To investigate the influence of changing wave heights on the hydraulic instability, an extra input significant wave height at 80% of the maximum is considered in a test series. This concludes that every berm construction is tested by two different significant wave heights.
Wave steepness and wave period

A possible specification of irregular waves is the peak wave steepness $s_p$ which is defined by the division of the significant wave height $H_s$ by the peak wave length $L_p$ (Eq. 4-2). For the suggested hydraulic boundary conditions by JDN, this parameter has values between 0.040 and 0.045. By changing the water depth and significant wave height in previous paragraph, the wave steepness will also change.

To respect the peak wave period $T_p$ of 11 seconds in the suggested hydraulic conditions (2.2 s in scale model) and limit the amount of breaking waves in the wave flume, the peak wave steepness is changed and fixed to a lower value of 0.035. This adjustment is still appropriate to the North Sea conditions which have a general range of peak wave steepness’s between 0.02 and 0.04 (Van Tongeren & Dewitte, 2016). However, in terms of damage it is expected to have more damage with less steeper waves (larger wave period) conform the results of Vidal et al. (1998).

$$s_p = \frac{H_s}{L_p} \quad \text{Eq. 4-2}$$

With:
- $H_s$ [m] Significant wave height
- $L_p$ [m] Peak wave length
- $s_p$ [-] Peak wave steepness

Due to the fixed value of 0.035 for the steepness, there is a clear correlation between the significant wave height $H_s$ and peak wave length $L_p$ in all the tests. As the related peak wave period $T_p$ is an input parameter for the wave generation, it can be estimated by using Eq. 2-9 of the airy wave theory in 0. This formula is applicable in deep, shallow, and transitional water.

In the calculation of stability parameters, it can be required to use the mean wave period $T_z$ instead of the peak wave period. The mean value can be determined by a time series analysis in WaveLab. An alternative option is to estimate $T_z$ by the relationship with the JONSWAP spectrum which is mentioned in 0. This coefficient of 0.781 is proposed by the USACE, in further analysis a value of 0.769 is used conform the more recent propositions of Roulund et al. (2016).

Hydraulic regime

Scaling of the stones and hydraulic boundary conditions with the law of Froude means that the wave Reynolds number $Re_w$ and relative roughness $r$ changes for the oscillatory flow over the armour layer (chapter 2.3.1). Besides this, the kinematic viscosity differs also slightly because of the change in temperature and density of the water in the scale model. The effects can cause differences in terms of the hydraulic flow regime through the stones of the armour layer. The effective influence in the test set-up through scaling is discussed in chapter 4.4.4.
4.3 Test matrix

4.3.1 Geometric and hydraulic parameters

A general overview of the different parameters which are related to the experiments with the scale model are described in this section. According to the discussed parameters in previous subchapter a test matrix is developed to investigate the impact of the variables on the damage of the rock berm protection.

The test matrix is divided in four main groups which are based on changing the rock grading of the armour layer ($d_{50}$) or the berm width ($B$). A graphical representation is given by Table 4-3 and the following parameters are assumed to be constant in the experiments;

- Relative buoyant density of stones $\Delta$: 1.65 [-]
- Slope angle of front and back slope $\alpha$: 1:2.5 [V:H]
- Height of the rock berm $h_{crest}$: 8.0 cm
- Shape and material of core (rock berm) $d_{n50}$: $\pm$ 2.0 mm

- Peak wave steepness $s_p$: 0.035 [-]
- Number of waves $N_w$: Approximate 1000 waves [-]
- Density of fresh water $\rho_{w,fresh}$: 1000 kg/m$^3$
- Temperature of fresh water: 16 °C
- Kinematic viscosity of fresh water $\nu$: $1.11 \times 10^{-6}$ m$^2$/s

The kinematic viscosity of fresh water is not be measured but determined by measuring the density and temperature of the water in the wave flume.

In each test, a mop is used between the armour layer and the core material (section 4.2.2).

The methodology of the different test series is explained in next paragraphs. Unless the rock gradings are scaled, the names of these gradings are still used in further analysis to make the comparisons easier for understanding.

Test series 1

The first series of tests are performed with the scaled rock grading LMA 10-60 for the armour layer. The purpose is to investigate the damage of the armour layer by applying different hydraulic boundary conditions which induce the highest possible bed shear stress wherefore the amount of breaking waves is limited to 5%. The damage is determined by comparing the initial profile scan that is measured by the laser profiler with the profile scan of the scaled rock berm after the waves have been executed. The scanning area is limited by a length of 15 centimetres (x-direction) of the rock berm length and the total berm width (z-direction: berm width $B$ + slopes). For each test, the berm width $B$ is kept constant.
Methodology of experimental model tests

Table 4-3: Test matrix of experimental study on rock berm protection

<table>
<thead>
<tr>
<th></th>
<th>Rock grading</th>
<th>Significant wave height</th>
<th>Water depth</th>
<th>Peak wave periods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>d_{s50} [mm]</td>
<td>H_s [cm]</td>
<td>d [m]</td>
<td>T_p [s]</td>
</tr>
<tr>
<td><strong>Test series 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Berm width</td>
<td>B = 0.08 m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak wave steepness</td>
<td>S_p = 0.035</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.05</td>
<td>18.0</td>
<td>0.50</td>
<td>2.459</td>
</tr>
<tr>
<td></td>
<td>(LMA 10-60)</td>
<td></td>
<td>0.45</td>
<td>2.566</td>
</tr>
<tr>
<td></td>
<td>14.4</td>
<td></td>
<td>0.50</td>
<td>2.024</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.45</td>
<td>2.103</td>
</tr>
<tr>
<td><strong>Test series 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Berm width</td>
<td>B = 0.08 m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak wave steepness</td>
<td>S_p = 0.035</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.71</td>
<td>17.0</td>
<td>0.50</td>
<td>2.338</td>
</tr>
<tr>
<td></td>
<td>(LMA 5-40)</td>
<td></td>
<td>0.45</td>
<td>2.437</td>
</tr>
<tr>
<td></td>
<td>13.6</td>
<td></td>
<td>0.50</td>
<td>2.001</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.45</td>
<td>1.929</td>
</tr>
<tr>
<td><strong>Test series 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Berm width</td>
<td>B = 0.08 m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak wave steepness</td>
<td>S_p = 0.035</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12.40</td>
<td>18.0</td>
<td>0.50</td>
<td>2.459</td>
</tr>
<tr>
<td></td>
<td>(LMA 40-200)</td>
<td></td>
<td>0.45</td>
<td>2.566</td>
</tr>
<tr>
<td></td>
<td>14.0</td>
<td></td>
<td>0.50</td>
<td>2.024</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.45</td>
<td>2.103</td>
</tr>
<tr>
<td><strong>Test series 4</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Berm width</td>
<td>B = 0.20 m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Peak wave steepness</td>
<td>S_p = 0.035</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.05</td>
<td>18.0</td>
<td>0.50</td>
<td>2.459</td>
</tr>
<tr>
<td></td>
<td>(LMA 10-60)</td>
<td></td>
<td>0.45</td>
<td>2.566</td>
</tr>
<tr>
<td></td>
<td>14.0</td>
<td></td>
<td>0.50</td>
<td>2.024</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.45</td>
<td>2.103</td>
</tr>
</tbody>
</table>

To have an initial visualisation of the stone movements in a test, the rocks are coloured by three different colours. More concrete, the stones of the front slope, crest, and back slope have a different colour. Attention is paid that the colours are suitable for the reflection of incoming light of the laser profiler (yellow, red, and white are used). A downside of using these colours is the time needed for sorting the rocks after each test (± 10 minutes extra).

To research the effect of lowering the water depth d on the damage, two different values of d are considered, namely 50 centimetres and 45 centimetres (a decrease of 10 %).

The influence of changing the significant wave height H_s are investigated by changing this value from 18.0 centimetres to 14.4 centimetres (a decrease of 20 %). During the tests, the wave elevation is measured by three wave gauges in front and three wave gauges after the rock berm structure. By using this set-up of wave gauges, the reflection of the rock berm structure can be compared with the reflection coefficient of the passive wave absorber.

A third variation is the change of the peak wave period T_p because of the constant value for the wave steepness s_p. This means that a range of the peak wave period is tested. The lowest one has a
value of 2.024 seconds while the highest one has one of 2.459 seconds for the largest water depth
(an increase of 18 %). During the tests, this parameter is also measured by the wave gauges.

For the sensitivity study and conclusions later, attention should be paid that the increase of the
wave period is accompanied by an increase of wave heights and vice versa. However, according to
Eq. 2-17, the root-mean-square velocity $u_{\text{rms}}$ is more vulnerable to the wave height than the wave
period. For example the considered increase of 18 % of the wave period at a certain water depth
means only an increase of 8% for $u_{\text{rms}}$. While the increase of the wave horizontal orbital velocity
component is linear with the significant wave height.

Test series 2

This second test series is almost identical to the first one. The main difference is the use of
smaller stones in the armour layer (scaled LMA 5-40). For this reason the core is enlarged to have
the constant berm-height of 8 centimetres.

A second difference is that input significant wave height $H_s$ has a maximum of 17 centimetres
instead of 18 centimetres. This adaptation is made to limit the amount of damage which would
have been unnecessary when checking the results later.

In this test series the coloured stones are not sorted after each test because of relatively small
stone sizes. Sorting these stones is much more time consuming and should take at least 25 minutes
extra.

Test series 3

The third test series has no differences with the first test series in terms of hydraulic boundary
conditions. However, the rock armour layer is changed to a coarser grading (scaled LMA 40-200).
It is expected that the damage levels $S$ are less with a larger coefficient of variation in these tests.

Test series 4

The last test series has the same hydraulic boundary conditions as mentioned in test series 1 and 3.
The main difference is the change in geometry. The berm width $B$ is extended to 20 centimetres
(increase of 250%) while the armour layer is composed of a rock grading with $d_{50}$ which is equal
to 6.71 millimetres (scaled LMA 10-60).

4.3.2 Shakedown test

For the definition of damage it is important that the initial stone settlements due to the presence of
the first waves are not considered in the analysis. To counter this extra level of damage, a
shakedown test is performed before the design wave test.

The test includes the generation of waves with an input significant wave height at 50% of the
design waves (same wave period) and for a period of about 333 waves (33% of the time series
Methodology of experimental model tests

duration). This shakedown test is performed for each grading (at least 3 test series) under the
different hydraulic boundary conditions. After several shakedown tests in a test series, a small
analysis is made in the lab to check if the influence is significant for the remaining tests.

4.3.3 Reflection in wave flume

Using a wave flume for an experimental study on a structure with waves introduce wave reflection
in the flume. In case of a near-bed structure, the reflection is possible from two structures; the end
of the wave flume (concrete wall) and the rock berm structure.

The first one is the partially compensated by the passive wave absorption system at the end of the
wave flume (section 3.1.3). By doing a small optimization study for the positioning of the blocks
and adding the blue absorption matrasses, a minimum and maximum reflection coefficient of
about respectively 0.19 and 0.25 is obtained. It can be seen in chapter 4.4.1 that the energy and
wave height of the reflected waves have no influence on damage level of the rock berm armour
layer.

The second source of reflection is the rock berm structure under water. The related reflection
coefficient can be estimated by compensating the reflection coefficient which is measured with
wave gauges 4, 5, and 6 by the reflection coefficient which can be determined by wave gauges 1,
2, and 3.

4.3.4 Reliability of experimental tests

An experimental research is subjected to uncertainties of different origin. To make analyses later,
these uncertainties should be quantified by doing test not once but several times. For this reason,
all the experimental tests of each test series are performed at least twice. Additional tests are
executed when unexpected phenomena arise during the tests or initial results would show strange
values.

The tests are performed in the time span of 10/02/2017 until 10/03/2017 in the wave flume of the
UGhent. In total 70 tests are investigated in the wave flume with respect to the stability of the
armour layer of the rock berm protection. The different optimization and operating tests of
working with the wave flume and measurement devices are together with the case studies included
in the time span. The general flow chart of an experimental test is described in annex D.

4.4 Data analysis from experiments

4.4.1 Wave analysis with WaveLab

The estimation of the bed shear stress to calculate the Shields stability parameter requires data
about the wave horizontal orbital velocities. This parameter can be estimated by a certain wave
theory or a measuring device. When the first approach is used it is important to measure the effective waves in the basin during the tests. Measuring the wave characteristics are performed by the six resistive wave gauges (Chapter 3) which measure the wave elevation. The data is processed by LabVIEW and analysed in the software WaveLab. Besides this data processing, the amount of breaking waves is counted manually in each experiment.

Because of the six wave gauges with a registration frequency of 40 hertz, three in front and three after the rock berm structure, it is the possible to make a reflection analysis of each experimental test per three wave gauges in WaveLab. The analysis separates the incident and reflected waves by using digital filters (Frigaard & Andersen, 2014). From both sets of wave gauges, the wave characteristics can be determined by a frequency domain analysis and a time domain analysis. The calculations in this manuscript are performed with the wave characteristics from the frequency domain analysis. It is common used in coastal engineering research and has a physical correlation to the comprised energy.

**Time domain boundary effects**

Wind waves are generated by wind that blows over the water surface which induces ripples. When further developing these waves become bigger and propagate further in a fully developed sea state. In the wave flume the waves are generated by a paddle that moves back and forth. Equally to an actual sea state the waves need a certain time to develop when the paddle starts moving. At the end of a test series the waves need a certain time to run-out, to get in a state of still water.

In order to deal with this type of behaviour a moving window analyses is performed. The sequence is based on the thesis of Van Tongeren and Dewitte (2016):

1. Analysis of the full data set to determine the mean wave period $T_m$ and the length of the full time signal $T_{total}$.
2. Generate the window for the analyses. To get statistically representative data, approximately 1000 waves need to be examined. Than the window is fixed to 1000 times $T_m$. To make each window unique the starting point need to be defined, indicated as $T_{start}$. In WaveLab not the time but the number of data points is insert which is related by the sampling frequency of the wave gauges (40Hz).
3. The first window is placed at the start of the signal. With a spectral analysis the spectral domain significant wave height $H_{m0}$ and the peak wave period $T_p$ are determined.
4. In the next step the window is shifted with 10 seconds and again $H_{m0}$ and $T_p$ are determined.
5. This procedure is repeated until the end of the signal is reached.

This procedure is applied towards several test series and a straight line with small deviations (<1%) is found with respect to spectral domain significant wave height $H_{m0}$. An example of one test (data set of test 102) is given in Figure 4-6. Comparing this figure with the results of the dissertation of Van Tongeren and Dewitte (2016), different results are obtained.
The reason for this difference is that the main wave period of the considered test series is larger than the main wave period in their thesis. It means that their signal is analysed with a shorter window which results in bigger influence of the start and end of the signal.

The generation and run-out time are visible by looking visually to the time signal of a certain experimental test. It is decided to cut-off 1800 data points (= 45 s) at both the start and the end of the time signal of each test. This procedure is illustrated for one wave signal in Figure 4-6, the signals of wave gauges 1, 2, and 3 are represented.
Frequency domain analysis

The next step after the filtering of the surface elevation is the actual wave analysis in the frequency domain. The signal is considered as stochastic and stationary with a Normal distribution. By the application of a Discrete Fourier Transformation (DFT) via a Fast Fourier Transformation (FFT) for a digital aperiodic signal, an estimation of the variance-density spectrum (in WaveLab: spectral density) can be determined of each wave experiment.

Settings or influence parameters for this analysis are:

- filter,
- the frequency resolution (data points in FFT block),
- overlap of the subseries,
- taper width.

The filter ensures that the total spectral energy and $H_{1/3}$ are determined by an integration procedure over a finite frequency bandwidth. It means that very long and short waves are filtered from the data set. The limitations are spectrum dependent and set to $0.33f_p$ and $3f_p$ for the low pass and high pass frequency respectively.

To determine the frequency resolution, it is chosen to use the tool of the software which determines automatically the frequency resolution to get a balance between a high resolution and an acceptable coefficient of variation.

There is no guideline regarding the optimal number of data points in the FFT blocks. An overlap ratio of the subseries is set to 20 % for each test. The advantage is the reduction of the variance and the related smoothing on the spectrum.

To prevent an uncertainty and a bias of the spectral estimate, data tapering is applied. It means that the time series is multiplied by a window function which has a unitary value within a time
interval and drops to zero at the end of the subseries. The length of the taper width is set to 20% of the subseries and equals the subseries overlap.

With the spectral analysis the measured spectra can be compared with the intended JONSWAP spectra which has a peak shape parameter $\gamma = 3.3$. The spectral density of a certain test is show in Figure 4-7 where the expected spectra (parametric expression with input parameters for wave generation) is represented by a black curve.

In the figure it is visible that the spectrum of the data-set fits globally with the intended spectral density curve (the blocks are situated within the curve), except for the range with a frequency between 0.7 and 0.9 hertz. A mean comment is that the expected spectra is more energetic compared with the measured one. This explains also the smaller values of the significant wave height $H_s (= H_{m0})$ towards the input values, indicated in Figure 4-8. The difference can be related to the presence of breaking waves in the wave flume – before the wave gauges – during each test. It should also be noticed that the calculated spectrum is an estimation and that the paddle actuator needs a certain time to respond to new instructions which can result in a difference between intended and actual paddle displacement.

By doing the spectral analysis for each experimental test, the phenomena of the second energy accumulation for waves with a shorter wave period returns in each test. A possible explanation is the presence of breaking waves some metres in front of the structure which can cause the generation of waves with a smaller wave period. However, near bed structures are mostly harmed by smaller wave frequencies and higher energy levels. Concluding that the lower energy levels at higher frequency are neglected in further analysis. The topic of less energy in the density spectra and thus a smaller spectral domain significant wave height $H_{m0}$ is still usable and can be countered by using the measured wave characteristics in the further damage analysis.

**Difference between input and output**

The generated waves have generally a smaller spectral domain significant wave height $H_{m0}$ than initially intended, smaller than the input significant height. Figure 4-8 represents this comparison and it can be seen that for each test $H_{m0}$ is lower (with average of 4%) than the input value for $H_s$. This is the conclusions of previous paragraph. Adapting the wave heights to a larger value in the experiments would give a larger ratio than 5% of breaking waves, with respect to the total number of waves.

Figure 4-9 indicates that the input values of the peak wave period are close to the measured values as a directional coefficient of 1.00 is found. The values show a certain spread which is also acceptable as the correlation coefficient of the plot is 0.97.
Influence of the rock berm

The reflection by the structure – determined by the six wave gauges according to subchapter 4.3.3 – is about 0.5% ± 0.4%. This is a relative small value and according to the propositions of the Rock Manual (CIRIA, CUR, CETMEF, 2007). reflection slightly affects the incident wave parameters.

The analysis with the six wave gauges with respect to the wave transformation over the rock berm gives over all test series a decrease of the incident significant wave height with a maximum of 3.6% by passing over the structure. When the wave period is calculated before (gauges 1, 2 and 3) and after (gauges 4, 5 and 6) the structure, the peak wave period indicates an increases up to 5%.
4.4.2 Verification wave theory and breaking waves

In the tests relative large wave heights are present with respect to the water depth. It is clear that the larger waves form the spectrum behave as nonlinear waves. To have an idea about the present waves in the wave flume, some general data points of the tests are plotted on Figure 2-8 of Le Méhauté (1976), the result is shown in Figure 4-10. Remark that the data points in the figure are determined by the use of the significant wave height $H_s$ and the peak wave period $T_p$ of the relevant test.

The figure explains that a higher order theory of Stokes can give a better mathematical expression for the waves that are present over the structure in transitional water. As mentioned in chapter 2.3.1, an alternative approach is the use of the Fenton’s Fourier approximation. The linear wave theory is compared with the Fourier approximation in next chapter 4.4.3 in terms of the measured wave horizontal orbital velocity in the experiments of the test matrix.

A second conclusion is that waves with a higher wave height and smaller wave period can reach the breaking limit boundary. During the tests it is clearly visible that some solitary waves appear and even breaking waves occur. Attention was paid to limit the breaking waves below a percentage of 5%.

*Figure 4-10: Application range of various waves with data points of the test matrix*
4.4.3 Horizontal orbital velocity analysis

The determination of the horizontal orbital velocities related to the waves, in a point just located above the armour layer, are difficult to quantify. A first possibility is making an estimation of the velocities by a certain wave theory and state-of-the-art papers. The second option, doing a point measurement of the orbital velocities by an electromagnetic velocity recorder. The installation of this device is explained in (chapter 3.2).

The analysis with the measured data from a point velocity recorder is investigated in this paragraph. The velocity components in a point (y and z direction) are recorded with the open-source software PuTTY by a sampling frequency of 16 hertz.

Reliability of electromagnetic velocity recorder

Before using the data from the velocity recorder in analysis, a reliability check with regular waves is performed. The test conditions are monochromatic waves with a wave height of 0.18 metres and a wave period of 1.635 seconds at a water depth of 0.55 metres (in the zone above the return channel), these waves where applied for 300 seconds which is approximately 180 waves.

By doing a point measurement of the horizontal orbital velocity component at a certain water depth, the measured data can be compared with the theoretical velocities conform a certain wave theory. Figure 4-11 gives two plots of the theoretical velocity distributions according to the Airy theory and Fenton’s Fourier approximation with respect to the tested hydraulic boundary conditions. There are two curves presented for the Fenton’s Fourier approximation, due to the non-linearity of the waves the velocity under the crest (Fenton+) is not equal as under the through (Fenton-). The data points of the measurement horizontal orbital velocity \( u_m \) – velocity definition of irregular waves – are located in between these two curves. It is a result that should be expected from nonlinear waves in transitional water where the airy wave theory is not applicable. Out of Figure 4-11 it could be concluded that linear wave theory is applicable for the velocity at the level of the crest.

The velocity definition \( u_m \) of irregular waves is used because the wave velocities do not indicate the same peak values for each individual wave. The measured values for \( u_m \) are in both cases 0.271 and 0.272 m/s which lies close to the velocity expectation of the airy wave theory and the velocity under the through according to Fenton’s theory. It confirms that the use of the airy wave theory is a good approach for the estimation of the horizontal wave orbital velocity, conform the recommendations of Roulund et al. (2016).

Data processing with irregular waves

The electromagnetic velocity recorder is used in almost each experiment to have an estimation of the horizontal orbital velocity component at the crest level. Similarly, to the analyses of wave heights, the measured values at the start and the run-out of an experiment is filtered from the time series. Besides the filtering, it is chosen to analyse only 30 minutes (28 800 data points) from a time series which reduces the time of data processing.
From the velocity measurements different velocity quantifications can be determined. The following definitions from a data set are compared:

- The maximum horizontal orbital velocity,
- The minimum horizontal orbital velocity,
- The root-mean-square value of the horizontal velocity $u_{\text{rms}}$ (positive and negative),
- Significant horizontal orbital velocity $u_{1/3}$ (positive and negative),
- $u_{1/10}$ (positive and negative),
- Absolute values of above mentioned definitions.

The velocity definitions $U_{1/3}$ and $U_{1/10}$ means that the absolute values of the data set are sorted to the number from high to low. Than the average is taken value from the respectively 1/3 or 1/10 of the highest absolute horizontal orbital velocities, conform the definition “significant wave height”. A representation of the different velocity definitions is shown in Figure 4-12.

The value of the average of the one third highest waves $U_{1/3}$ is almost similar to $U_{\text{m}}$. Analysing several measured velocity tests, an average variation between two equal test series of 1% is found, which is equal to the precision of the electromagnetic velocity recorder (chapter 3.2).

**Horizontal orbital velocity in experiments**

A comparison can be made between the velocity $u_{\text{m}}$ of a test and the calculated velocity $u_{\text{m, theory}}$ conform the paper of Roulund et al. (2016), based on experiments of Soulsby (2006). Figure 4-13 represents this comparison for 70 tests and shows that the theoretical values are conservative with an average overestimation of 8.4 %.
4.4.4 Verification of hydraulic regime in armour layer

For the verification of the hydraulic regime in the armour layer, the presence of the rock berm slopes on the bottom is neglected. This allows a possible comparison between the flow conditions in the granular material between the scale model and prototype. It can be expected that due to the slopes more turbulence is present in the flow regime.

In an oscillatory flow the wave friction coefficient with the structure is different in the scale model compared with the prototype. Scaling of a water flow by the law of Froude can change the hydraulic regime of this flow. It is obvious that for example a change of a turbulent flow towards a laminar regime has an effect on the hydraulic stability. The related boundaries between the different flow regimes are explained in chapter 2.3.1. The first purpose is to compare the
hydraulic regime of the scale model with the prototype. A next step is investigating the impact on the wave friction coefficient and the related bed shear stress at the crest due to scaling.

Figure 4-14 is a representation of Figure 2-10 with the data points of experimental tests and the estimated values of the prototype. It is visualized that the flow regime in prototype is “rough turbulent”. The flow regime in the experiments is estimated by the calculation with the measured wave orbital velocity at the crest (conform Eq. 2-16), wave peak period, and water depth at the crest. From the figure with the data points of the scale model it is found that the regime is at the boundary between transition and rough turbulent flow (curves from Jonson).

Considering the wave friction factor $f_w$ from Eq. 2-31, the factor is not constant but slightly different because of the difference in fresh and salt water in the scaling. It can be estimated that the wave friction factor itself is 3% larger during the tests in the wave flume, compared with the prototype conditions. This has influence on the design bed shear stress which is on the other hand partially compensated by the difference in water densities.

A comment is that the scale models are constructed on the concrete floor of the wave flume. In reality the underground is soil which can also interact with the armour layer. For example, stones at the toe can penetrate in the soil. Thus in the basin a transition appears from a rather smooth surface (concrete) towards a rougher one of the armour layer. For these reasons the damage profile at the toes is not considered and the first 5 centimetres of the cross-section are neglected in the analysis.
4.4.5 Rock berm damage analysis

The measuring of the cross-section profile before and after the wave test are done by a non-contact laser profiler (chapter 3.2.3). The software EPro can calculate the erosion area $A_e$ to come up with a damage level $S$ of the rock berm. EPro provides also the option to divide the scanned surface in different zones such that the front slope, back slope and crest can be investigated separately. An additional output value of the software is the 3D damage level which is based upon the eroded volume.

Scan operation and data processing

The scanning area of the profiler depends on the width of the scale model, while the scan length is limited to a distance of 15 centimetres. The related coordination system with definitions is explained in chapter 3.2.3. For the test series with a berm width $B$ of 8 centimetres a total scan width of 50 centimetres is performed. The test series 4 with the berm width of 20 centimetres have a scan width of 60 centimetres.

In each scan it is important that a part of the concrete floor (around the toe) is scanned to calibrate the scanned profiles later. The scan operation is performed by a grid of 5 by 5 millimetres, which is slightly smaller than the $D_{50}$ values of the rocks gradings.

<table>
<thead>
<tr>
<th>Section</th>
<th>$B = 8$ cm</th>
<th>$B = 20$ cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Front slope</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Crest</td>
<td>8</td>
<td>20</td>
</tr>
<tr>
<td>Back slope</td>
<td>15</td>
<td>15</td>
</tr>
</tbody>
</table>

In the evaluation of the erosion area, the cross-section of the berm is divided in three parts: front slope, crest and back slope. The stability of the toe sections are not studied because of the interaction with the concrete floor. This means that a cut-off of 5 centimetres is applied in both the front and back slope. Table 4-4 gives a final overview of the section widths which are considered for analysis.

Before the quantification of the erosion area for the damage analysis, an appropriate filter based on recommendations of previous research in the UGhent (De Vos, 2008) is used. The filter takes into account a nine-point grid, to the centre point of the grid a fictive weight of 15 is given while for the surrounding points a weight of 1 is designated. A remarkable setting in EPro is to apply the filter once, it corresponds with a number of passes which is equal to two. This is a bug in EPro and is unfortunately noticed after the data processing, this “mistake” is quantified and led to an average underestimation of the mean damage levels by 2.4%. It should also be noted that the standard deviations of the profiles improved as the filtering was passed twice. Eventually its concluded that this has no significant influence upon the test results.
Mean damage levels 2D and 3D

With the filtered profiles a damage level can be determined of the erosion area $A_e$ between the two measured cross-section profiles (before and after a wave test). The quantification of the damage level can be defined in two ways.

The first option in EPro is the averaged damage level $S$ between two laser profiles in 2D, conform the damage parameter for breakwaters. The level is determined by first averaging each contour plot of the cross-section profile to one averaged cross-section. Next step is the calculation of the eroded area $A_e$ between two averaged contour plots. The averaged damage level can then be determined finally by dividing the eroded area by the square of the mean nominal diameter squared $d_{n50}$ (Eq. 2-50).

A second method with EPro is the calculation of the 2D mean damage level out of the 3D damage value or the eroded volume. Considering the scanned length of 15 centimetres, it is possible to define an averaged damage level in 2D (Eq. 4-3) without averaging the laser profiles to one averaged cross-section. This method gives larger absolute mean damage levels, compared with the first method due to the difference in averaging. The procedure compensates less the eroded area differences along the length of the profile ($x$-axis).

The software EPro is developed by the university of Aalborg and from their experience it is decided to use the first method. The rock berm structure in the flume is considered as a 2D structure and is analysed by the 2D output value of EPro.

$$S_{3D} = \frac{V_e}{D_{n50}^3} = \frac{A_e \cdot L_{scan}}{D_{n50}^2 \cdot D_{n50}} = \frac{L_{scan}}{D_{n50}}$$  \text{Eq. 4-3}

With:

- $A_e$ [m²] Eroded area
- $D_{n50}$ [m] Average nominal grain diameter
- $L_{scan}$ [m] Scanning length of rock berm ($x$-direction rock berm)
- $S$ [-] 2D mean damage level
- $S_{3D}$ [-] 3D based mean damage level
- $V_e$ [m³] Eroded volume

An example of the result from a damage calculation in EPro is shown in Figure 4-15. It can be seen that a mean value of the damage level $S_{mean}$ is determined with its standard variation of the averaged cross-section profile.

Attention should be paid that the scaling factor of 0.764 needs to be taken into account for the damage determination – explained in chapter 3.2.3 – because of scan operations under water. The value is considered in the quantity of the eroded area. The area is correctly measured in the $x$- and $z$-direction because of the profiler grid steps and should thus be scaled only once ($y$-direction).
Damage accuracy in measurements

The 2D damage calculation is performed for each test of the test series, resulting in a mean level with a standard deviation. From these values a coefficient of variation (COV) can be determined in function of the mean damage level, the result is graphically represented in Figure 4-16. The trend in the graph shows clearly that larger mean damage levels are accompanied with a smaller COV. A lot of experiments have a relative small mean damage level with a very large COV until 240% which has a significant impact on the accuracy of their damage analysis.

Besides the influence of the mean damage level, the accuracy of laser profiling in combination with the motion vehicle can also be questioned. To obtain an indication of the accuracy, a test is performed with scanning in dry conditions of the cross-section initially once (profile 1) and three times after the wave test is applied (profile 2, 3, and 4). The wave test is executed on a scale model of test series 2 with an input significant wave height of 16 centimetres, peak wave period of 2.653 seconds, and at a local water depth of 0.45 metres (same as test 030 but with different peak wave period). The results of the small comparison are shown in Table 4-5. The COV of the test has a value of 10.7 % which indicates that the measurements itself also have a certain uncertainty. More research is necessary to check different influences on the measurement accuracy; it is obvious that absolute values of tests with a relative low mean damage level (0 – 10) give no much information. For this reason, the results in chapter 5 are mostly discussed with the help of large damage-tests.
Influence scanning length

In the experimental tests a constant scanning length of 15 centimetres is applied to reduce the measurement time of each test. To obtain an indication of the influence, three test are performed with scanning in dry conditions of a larger length over a distance of 30 centimetres (initially and after wave test). The longer scans are executed on the random tests 2, 11, and 30. The analysis with EPro can done by comparing the 3D damage levels of the two different scanning lengths.

From the results it can be mentioned again that a larger mean damage level results in better estimations, summarized in Table 4-6. Nevertheless, a larger scanning area can influence the damage levels with some percentages. However, the scanning length is not enlarged in the tests as the time increases significant for a larger area and the obtained measurements are not worse than accuracy of the laser profiler itself.

<table>
<thead>
<tr>
<th>Test</th>
<th>Area</th>
<th>Scan length</th>
<th>( S_{3D} )</th>
<th>Deviation from mean ( S_{3D} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Zone 1</td>
<td>15</td>
<td>9.5</td>
<td>6.0 %</td>
</tr>
<tr>
<td></td>
<td>Zone 2</td>
<td>15</td>
<td>10.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total area</td>
<td>30</td>
<td>10.1</td>
<td></td>
</tr>
</tbody>
</table>
4.4.6 Case study: influence of shakedown test

The influence of the shake down test (explained in chapter 4.3.2) on the final mean damage level $S_{\text{mean}}$ is evaluated for each test series (each rock grading) with a total of 26 shake down tests. Table 4-7 shows the mean damage of multiple test that were performed in one test series. In the table only the results of the highest wave heights are plotted as these are most relevant.

The tests with shakedown give results with a larger or equal mean damage level than the tests without shakedown. This is remarkable and means that the shakedown test induces a certain additional loading upon the berm which means that the placement of the stones has not a noticeable influence on the mean damage levels. For test series 1 and 2 increases between 20% and 30% are observed, test series 3 is less relevant due to the rather low damage levels (high COV). For these reasons shakedown tests are not implemented in further experiments.

<table>
<thead>
<tr>
<th>Series</th>
<th>Test</th>
<th>Shakedown</th>
<th>$S_{\text{mean}}$</th>
<th>Average $S_{\text{mean}}$</th>
<th>Deviation shakedown</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$[-]\times 10^{-3}$</td>
<td>$[-]\times 10^{-3}$</td>
<td>$[-]\times 10^{-3}$</td>
</tr>
<tr>
<td>Test series 1 - H+</td>
<td>2</td>
<td>no</td>
<td>6.70</td>
<td>5.78</td>
<td>33.4%</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>no</td>
<td>4.86</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>yes</td>
<td>6.39</td>
<td>8.69</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>yes</td>
<td>10.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>no</td>
<td>5.75</td>
<td>8.23</td>
<td>20.1%</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>no</td>
<td>11.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>no</td>
<td>7.63</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>yes</td>
<td>10.29</td>
<td>10.29</td>
<td></td>
</tr>
<tr>
<td>Test series 2 - H+</td>
<td>21</td>
<td>no</td>
<td>10.72</td>
<td>10.72</td>
<td>20.1%</td>
</tr>
<tr>
<td></td>
<td>21</td>
<td>yes</td>
<td>10.01</td>
<td>13.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td>21</td>
<td>yes</td>
<td>16.81</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>no</td>
<td>9.36</td>
<td>10.35</td>
<td>30%</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>no</td>
<td>11.34</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>yes</td>
<td>13.16</td>
<td>14.77</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>yes</td>
<td>12.48</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>yes</td>
<td>18.67</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Methodology of experimental model tests

<table>
<thead>
<tr>
<th>Test series 3</th>
<th>Use of mop</th>
<th>S_{mean}</th>
<th>Average S_{mean}</th>
</tr>
</thead>
<tbody>
<tr>
<td>38 no</td>
<td>2.65</td>
<td>2.65</td>
<td>-61.8%</td>
</tr>
<tr>
<td>38 yes</td>
<td>1.64</td>
<td>1.64</td>
<td>29.5%</td>
</tr>
<tr>
<td>47 no</td>
<td>1.30</td>
<td>1.30</td>
<td></td>
</tr>
<tr>
<td>47 yes</td>
<td>1.85</td>
<td>1.85</td>
<td></td>
</tr>
</tbody>
</table>

4.4.7 Case study: influence of mop in rock berm

The purpose of this small case study is to investigate if a significant trend is visible on the mean damage level S_d when the mop in the rock is not applied. The core material is not correctly scaled (to fine material) thus the notional permeability is also without the mop changed from of the rock berm in prototype. Without the mop under the armour layer, the water particles are less blocked to flow through the rock berm core when orbital motions are present. It should be expected that the uplift forces become lower and result in less mean damage levels.

The influence is researched by performing four tests with a relative large mean damage level (test series 1 and 2). An overview of the tests with their results are summarized in Table 4-8. Remark that only three tests are shown in the table because the fourth test reached only mean damage levels of 4 until 7. From the three tests the conclusions are not straightforward. Two tests series 1 and 2 show that the mean damage level decrease when no mop is used. While one test series 3, with a large mean damage level indicates that there is an increase of damage. It should be noticed that the tests without the mop of each series are applied only once.

A main remark is that unfortunately more tests are required to make clear statements about the influence of the mop. Unless no significant trend is possible to find with this small case study, it is assumed that the tests with the use of the mop have a conservative approach, compared with the prototype model. However, the tests of the different test series with a mop are comparable from which its chosen to conduct the test with a mop for practical ease.

<table>
<thead>
<tr>
<th>Test</th>
<th>Use of mop</th>
<th>S_{mean}</th>
<th>Average S_{mean}</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Yes</td>
<td>5.8</td>
<td>9.1</td>
</tr>
<tr>
<td>11</td>
<td>Yes</td>
<td>10.3</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Yes</td>
<td>11.3</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>No</td>
<td>7.6</td>
<td>7.6</td>
</tr>
<tr>
<td>21</td>
<td>Yes</td>
<td>10.7</td>
<td>13.8</td>
</tr>
<tr>
<td>21</td>
<td>Yes</td>
<td>16.8</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>No</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>30</td>
<td>Yes</td>
<td>11.3</td>
<td>12.3</td>
</tr>
<tr>
<td>30</td>
<td>Yes</td>
<td>13.2</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Yes</td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>No</td>
<td>18.7</td>
<td>18.7</td>
</tr>
</tbody>
</table>
Chapter 5  Results

This chapter of results starts with the description of different sensitivity studies about the investigated parameters, with the obtained data set from the test matrix. A second part compares the hydraulic stability of the different sections of the rock berm cross-section; front slope, crest, and back slope. This part is then continued with an investigation of the Shields stability parameter versus with respect to the related damage levels. Finally, a research is made with available data sets of comparable tests in literature.

In the upcoming figures and explanations, the largest significant wave heights coming from the input $H_s$ (18 and 17 cm) are symbolized with $H^+$. The smaller ones from the input $H_s$ (14 and 13.6 cm) are characterized by $H^-$. Thus by using $H^+$, no distinction is made between 18 and 17 centimetres.

5.1 Sensitivity study

The sensitivity study includes the investigation on the 2D damage level $S$ when a geometric or hydraulic parameter during the experiments is changed. This not always straightforward because of the possible large standard deviation on the damage levels, explained in chapter 4.4.5. For this reason, the extreme values of the data set are filtered to enhance the accuracy of the data. More concrete, the mean damage levels which differ more than one standard deviation are neglected from the relevant data set. This is also why test series 3 with the largest stones are rarely used in this sensitivity studies because of the relatively small damage levels.

5.1.1 Influence of the water depth

Each test series have been tested with two different water depths $d$ for a certain significant wave height $H_s$ and wave period $T_p$. When the water depth is adapted to a lower value, the bed shear stress $\theta$ increases because of the larger horizontal orbital velocity $u_m$ at the crest level. Finally, it should result in an increase of the damage level $S$. To verify this theoretical statement, a data set is taken from the experiments with two different rock gradings and water depths (test series 1 and 2).
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for the largest significant wave heights (H+). The berm width has a constant value of 8 centimetres.

The increase of the bed shear stress $\theta$ is indicated by the horizontal orbital velocity measurement $u_m$ of the water particles at the crest level. A lower water depth $d$ should result in an increase of this velocity component. The velocity parameter $u_m$ of the measured velocities at the crest level in function of the water depth is graphically shown in Figure 5-1 for both test series. The figure shows clearly the trend that the horizontal orbital velocity increases when the water depth lowers. It should be remarked that the test series 2 with the smaller grading gives smaller values of $u_m$. This is due to the smaller input significant wave height, 17 instead of 18 centimetres.

The analysis of the mean damage levels $S_{\text{mean}}$ of each test (with repetitions) and the comparison between both water depths is graphically shown in Figure 5-2. A first impression of the graph is the large standard deviation on the mean damage level $S_{\text{mean}}$ of a test. However, both grading’s have the trend that the mean damage levels decrease when the tests are applied in larger water depth $d$. The trend is more pronounced for the smaller stone grading as the related shear stress is closer to the “critical shear stress” for those stones and thus more movement of the stones or damage occurs. More movement of the stones gives a better indication of the parameter influence.

![Figure 5-1: Influence of water depth on horizontal orbital velocity – test series 1, 2, and 3](image-url)
5.1.2 Influence of the significant wave height

Besides the change in water depth $d$, the input significant wave height $H_s$ is also changed in each test series. From the principles in Chapter 2 it should be expected that a change in wave height has an influence on the horizontal orbital velocity of the water particles and on the related hydraulic stability of the armour stones. An increase in wave height leads towards a raise of the bed shear stresses. A similar study as for the water depth is done.

A first investigation is to check whether the horizontal orbital velocity $u_m$ near the crest is changed by a different wave height $H_s$. Test series 1 and 2 with their different water depths $d$ and significant wave heights $H_s$ are used for this comparison. Figure 5-3 represents $u_m$ in function of the measured significant wave height $H_s$ and it gives the indication that the velocity increases linear with the wave height, as expected from Eq. 2-17.

Figure 5-4 represents the data with the measured mean damage level $S_{\text{mean}}$ in function of the measured significant wave height $H_s$. The damage levels for both grading’s – for a constant water depth $d$ – increase with the significant wave height $H_s$, as expected. Again the smaller grading results in larger values of the mean damage level $S_{\text{mean}}$. It can be motivated visually in the graph that the experiments of test series 2 are executed with a lower significant wave height $H_s$ than the ones of test series 1.
5.1.3 Influence of the peak wave period

The different test series are executed with a constant input peak wave steepness $s_p$ of 0.035. When the significant wave height $H_s$ is a variable parameter, the peak wave length $L_p$ varies. This means that the input peak wave period $T_p$ is a variable during the tests. As explained in the test matrix of Chapter 4 and in the chapter of waves (chapter 2.3.1), for irregular waves it is expected that for the considered test matrix the influence of the peak wave period $T_p$ is limited compared with the changing input of the significant wave height $H_s$.

To represent the theoretical findings with the performed experimental test matrix, Figure 5-5 is plotted which shows the mean damage level $S_{\text{mean}}$ in function of the measured peak wave period $T_p$ with a constant water depth $d$ for test series 1 and 2. The first impression of the graph confirms...
the expectation of a larger mean damage level $S_{\text{mean}}$ when the waves become longer. But the trend can also be a reason because of the related increase of the significant wave height $H_s$. Which indicates no general trend of this parameter can be concluded with the performed experiments.

Figure 5-5: Influence of peak wave period on mean damage level – test series 1 and 2

5.1.4 Influence of the mean nominal stone diameter

The damage of a rock berm armour layer is influenced by the mean nominal stone diameter $d_{50}$ of the related rock grading. Following the literature as mentioned in chapter 2.6.1, the critical bed shear stress to have sediment transport becomes lower for the considered test matrix when smaller stones are installed.

Test series 1, 2, and 3 can be compared for the same hydraulic boundary conditions with respect to the mean damage level of each experimental test. Considering the largest significant waves $H+$ with a constant water level $d$ of 45 centimetres a comparison between the three stone grading’s can be analysed, Figure 5-6. The same analysis is done for the water depth of 50 centimetres with the results shown in Figure 5-7.

The graphical representations show clearly that the mean damage levels $S_{\text{mean}}$ become lower with increasing stone sizes. The trend is not linear but rather a function with a decreasing power regression. It seems that the standard variation of the damage level on a test becomes significant larger at certain damage level. The type and amount of damage depends not only on the geometric and hydraulic parameters but also on other effects such as; placement accuracy, initial settlements of the stones, accuracy of the cross-section profiling with the laser profiler, etc. Nevertheless, an estimation of the expected damages can be made but with a certain attention towards the standard deviation.

Comparing the mean damage levels $S_{\text{mean}}$ of both water depths $d$, the averages give the indication that a lower damage level is obtained by a larger water depth, conform the findings in paragraph
Results

5.1.1. It should be remarked that the evolution of the mean damage levels between both figures lies in the uncertainties (one standard deviation) of the tests. However, it is quite logic that smaller stones and larger water depths induce a larger hydraulic instability of the stones.

![Figure 5-6: Influence of d_{50} on S_{mean} – water depth d = 0.45 m – test series 1, 2, and 3](image)

![Figure 5-7: Influence of d_{50} on S_{mean} – water depth d = 0.50 m – test series 1, 2, and 3](image)

5.1.5 Influence of the berm width

The berm width B at the crest level is changed in test series 4. As the same stone grading is only applied in test series 1, this test series can be used for the investigation of a different berm width. Figure 5-9 represents the data results of the mean damage level S_{mean} in function of the berm width B. Only the test with the largest significant wave heights H+ are used for the comparison.

A larger width of the berm results in a larger mean damage level S_{mean} (as defined in paragraph 2.5.2) because of the larger possible erosion area (van den Bos, 2006). This principle is graphically shown in Figure 5-8. Assume two rock berms that exist of the same; stone grading, berm height, and slope but with a different berm width. If one layer of stones would be removed theoretically, according to the current definition of a damage level S, the widest berm have a larger damage level.

![Figure 5-8: Situation sketch of eroded area for two different berm widths](image)
To counter this theoretical bias for comparisons with variable berm widths, a new formula is introduced from literature. Van den Bos (2006) proposed an expression for the adapted damage level $S^*$ which includes the berm width $B$, Eq. 5-1. This gives approximately an evaluation of how many stones are removed in one layer. Figure 5-10 shows graphically the new defined mean damage level $S^*_{\text{mean}}$ in function of the tested berm widths.

$$S^* = \frac{A_e}{B d_{n50}} = S \frac{d_{n50}}{B}$$  \hspace{1cm} \text{Eq. 5-1}

With:

- $S$ [-] Damage level
- $S^*$ [-] Damage level with berm width (eroded thickness)
- $A_e$ [m$^2$] Eroded area
- $d_{n50}$ [m] Median nominal diameter of stone grading
- $B$ [m] Berm width
The figure with the new definition $S^*$ shows a different trend compared with the previous case. It can be concluded that the eroded thickness of the total rock berm is smaller for a larger crest width $B$, unless a larger damage level $S$ is present. Before trying to find some explanations, the damage levels of the different cross-sections are investigated in next chapter 5.2.

5.2 Hydraulic stability of rock berm

5.2.1 General damage development

The general damage development is described in this paragraph. This is done for a test of test series 2 with the largest significant wave height and a water depth $d$ of 0.45 metres. Relatively large damage values which results in clear pictures.

Figure 5-11 is the initial situation of test 11, the coloured stones of the three different stones are separated in red, yellow, and white for the respectively crest, front slope, and back slope section.

Next Figure 5-12 describes the situation after the shakedown test. A few stones are moved but the difference with previous figure is not pronounced. The mean damage level is very limited with a value of 0.7 and the berm height indicates a maximum decrease of 1 millimetre which is due to settlement of stones.

Last Figure 5-13 is the situation after the design wave test with a significant wave height of 17 centimetres. The damage is clearly visible and corresponds with a mean damage level of 10.7. The back slope of the berm is accreted and the front slope and crest are eroded. Stones from the crest are mainly transported to the back slope. The average settling of the crest level is equal to 4 millimetres.

Figure 5-11: Initial configuration – Left: top view – Right: averaged cross-section – Test 11
Results

5.2.2 General sectional damage development

For the evaluation of the damage levels from the different rock berm sections, the cross-section is divided into three sections in the analysis with EPro, Figure 5-14. The standard definition of the rock berm width is defined as the width of the crest-zone.

Figure 5-14: Definition sketch - rock berm sections
Results

During the tests, visual inspection notes that the main damage occurs at the crest where stones start to roll oscillatory or in the wave propagation direction. For the larger waves, transport of stones happens also at the front and back slope. However, stones move more in the wave propagation direction than vice versa. A possible reason can be a net mass transport of the fluid (Stokes drift).

An indication of the average percentage distribution of the mean damage level of the rock berm is graphically shown in Figure 5-15. The damage of each section is plotted towards the total damage, with a linear regression through the data points. It can be seen that the mean damage level is mainly represented by the damage at the crest (45 %), followed by the front slope (35 %) and less by the back slope (20 %). The coefficient of determination is significant lower for the back slope. On average less damage occurred in this zone with a large spreading on the measured values. A possible reason is the presence of more turbulence at the back of the structure. Besides this, it means also that the wave propagation has an influence on the sectional damage of the rock berm for transitional water.

To have a visual indication of the main damage levels, Figure 5-16 and Figure 5-17 present top views of the initial rock berm configuration and the profile after the design wave test. The first figure represents a mean damage level of 2.5 while the second shows a level of 11.0. The crest, front slope, and back slope are respectively represented by the red, yellow, and white coloured stones in both figures/tests. It is clear that motion from the back slope towards the front slope is rather limited. The stones of the crest are shifted more to the back slope than vice versa.

![Figure 5-15: Sectional mean damage levels - test series 1, 2, and 3](image)
5.2.3 Influence berm width on sectional damage

The discussion at the sensitivity study of a changing berm width $B$ is explained more detailed in this paragraph, with respect to the different zones in the cross-section. It is analysed with the adapted mean damage level $S^*$ or the eroded thickness definition. Chapter 5.1.5 concluded already that the eroded thickness of the rock berm becomes smaller when a larger berm width $B$ is used.

The crest-zone is the main reason of a lower eroded thickness $S^*$ when the width is enlarged. It shows the same trend as the eroded thickness of the total rock berm, Figure 5-18. A possible reason is that the present definition of the damage level with the parameter “eroded area” not takes into account a possible refill of a scour hole. In a larger berm, the scour holes have more chance to become refilled.
The length of the front and back slope section is not changed and the influence on the eroded thickness $S^*$ is less pronounced. The front slope section follows the same trend as the crest. The back slope section contrary results in a larger mean damage level $S^*_{\text{mean, section}}$ when the berm width is increased. A possible reason is an increase of the turbulence at the back. This can be investigated by research the development of the boundary layer with a particle image velocimetry (PIV) measurement. It should also be noted that the damage levels at the back slope section is significant lower and has a larger coefficient of variation.
5.3 Shields stability parameter

5.3.1 Damage profile method

The mean damage levels $S_{\text{mean}}$ are plotted against the Shields stability parameter $\theta$, Figure 5-21. The parameter is based on the estimated bed-shear stress at the crest level and the median grain diameter $d_{50}$ of the stone grading, Eq. 5-2.

$$\theta = \frac{\tau}{g (\rho_s - \rho_w) d_{50}}$$  \hspace{1cm} \text{Eq. 5-2}

With:
- $\theta$ [-] Shields stability parameter
- $\rho_s$ [kg/m$^3$] Density of stone – equal to $\rho_{\text{app}}$
- $\tau$ [N/m$^2$] Wave total bed shear stress
- $\rho_w$ [kg/m$^3$] Density of water
- $d_{50}$ [m] Median stone diameter of grading
- $g$ [m/s$^2$] Gravity constant

The shear stress $\tau$ is determined by using the horizontal orbital motion velocity measurements ($U_m$) (chapter 4.4.3) and the theoretical values of the friction factors ($f_w$) (chapter 2.3.1).

$$\tau = \frac{1}{2} \rho_w f_w U_m^2$$  \hspace{1cm} \text{Eq. 5-3}

With:
- $\tau$ [N/m$^2$] Wave total bed shear stress
- $\rho_w$ [kg/m$^3$] Density of water
- $f_w$ [-] Wave friction factor
- $U_m$ [m] Orbital velocity quantification of irregular waves

An increasing Shields numbers means that the bed shear stresses becomes larger, which should result in more movements of stones. This is clearly confirmed in Figure 5-21 with an increase of the mean damage level $S_{\text{mean}}$ in function of the Shields stability parameter $\theta$. The trend shows an exponential behaviour which is comparable with other experimental tests about damage analysis in function of the bed shear stress (chapter 5.4).

A prediction formula of mean damage level $S_{\text{mean}}$ with respect to the Shields stability parameter $\theta$ can be developed. This is done by performing a power regression in Figure 5-21 with a coefficient of determination ($R^2$) which is equal to 0.55. It results in an expression which is given by Eq. 5-4 and is accompanied by a significant standard deviation and thus a relative large 90% confidence interval. The standard deviation upon the estimated Shields parameter cannot be determined as it depends on the friction coefficient and the horizontal orbital velocity component.

A detailed overview about the calculated uncertainties of each test is summarized in annex E.
\[ S_{\text{mean}} = 8 \cdot 10^6 \theta^{3.7} \{ R^2 = 0.55 \} \quad \text{Eq. 5-4} \]

With:

\[
\begin{align*}
\theta & \quad \text{[-]} \quad \text{Shields stability parameter} \\
S_{\text{mean}} & \quad \text{[-]} \quad \text{Mean damage level}
\end{align*}
\]

Sectional damage

The discussion of the sectional damage \( S_{\text{mean, section}} \) in chapter 5.2 can be applied with the Shields stability parameter \( \theta \). More concrete, the mean damage levels of each section \( S_{\text{mean, section}} \) is plotted towards the stability parameter, Figure 5-22. Again, a prediction formula of the mean damage levels for each section can be developed. The expressions of the crest, front slope, and back slope sections are respectively given by Eq. 5-5 until Eq. 5-7. Note that these formulas have coefficients of determination \( (R^2) \) with relatively low values.

In chapter 5.1.5 the method with an adapted damage level \( S^* \) is purposed. When a plot is made of the adapted damage levels towards the Shield stability parameter, the damage at the front and back slope tend more to each other. Due to rather low \( R^2 \) the plots are not visualized in this paragraph.

\[ S_{\text{mean, crest}} = 1 \cdot 10^0 \theta^{4.7} \{ R^2 = 0.45 \} \quad \text{Eq. 5-5} \]

\[ S_{\text{mean, front slope}} = 2 \cdot 10^6 \theta^{3.7} \{ R^2 = 0.37 \} \quad \text{Eq. 5-6} \]

\[ S_{\text{mean, back slope}} = 5 \cdot 10^5 \theta^{3.5} \{ R^2 = 0.31 \} \quad \text{Eq. 5-7} \]

With:

\[
\begin{align*}
\theta & \quad \text{[-]} \quad \text{Shields stability parameter} \\
S_{\text{mean}} & \quad \text{[-]} \quad \text{Mean damage level}
\end{align*}
\]
5.3.2 Critical stability method

Van Rijn (1993) defined a threshold of motion boundary-line of the Shields parameter or called the critical Shields parameter $\theta_{cr}$ in function of the dimensionless grain diameter (chapter 2.6.2). The initiation of motion is based upon a percentage of moving stones, with a limit of 10% of the bed material.

For the 2D damage levels no clear boundary or critical Shield stress is proposed in literature. However, considering the different parameters of previous paragraph 5.3.1, in Eq. 5-2, the maximum Shields parameters in the experiments is about 0.028 which corresponds with a mean damage level of about 19. The obtained Shields parameter is significant lower than the critical value of 0.055 (van Rijn, 1993).

An alternative option is defining the critical Shields parameter $\theta_{cr}$ for the damage level where the structures does not fulfil its purpose in protecting the core material. This can be done by using the adapted damage level definition $S^*$ of chapter 5.1.5. This level should be smaller than the number of armour layers placed upon the core ($=2$). This criterion is expressed by Eq. 5-8. As most damage occurs at the crest of the structure, it would be a save assumption to use the berm width.

$$S^* \leq n$$

$$A_e \leq n D_{n50} B$$

Eq. 5-8

<table>
<thead>
<tr>
<th>With:</th>
<th>( B )</th>
<th>[m]</th>
<th>Berm width</th>
</tr>
</thead>
<tbody>
<tr>
<td>( D_{n50} )</td>
<td>[m]</td>
<td>Median nominal grain diameter</td>
<td></td>
</tr>
<tr>
<td>( n )</td>
<td>[-]</td>
<td>Number of layers placed upon the core</td>
<td></td>
</tr>
<tr>
<td>( S^* )</td>
<td>[-]</td>
<td>Adapted damage level (=dimensionless eroded</td>
<td></td>
</tr>
</tbody>
</table>
The preformed experimental tests with an armour layer with two times the average nominal thickness have not reached the critical adapted damage because of relative low Shields numbers. However, a certain damage occurs during the experiments as graphically represented on Figure 2-17. Three intervals are defined with a certain percentage damage of the critical adapted damage level, defined by Eq. 5-8. It is clear that larger values of the adapted damage level (and larger Shields numbers) are closer to the critical Shield stability parameter of van Rijn (1993). Further, the data of the smallest percentage damage levels follows the same trend as the critical curve.

![Graph](image)

*Figure 5-23: Initiation of motion according to critical damage criterion – all test series*

### 5.4 Damage assessment with existing data

This chapter analyses the performed experimental tests with 4 existing research papers. The first three reviews compare the data with other hydraulic model tests. The comparison is done for all the data points and the calculations are according to their purposes. The last review compares the data with a detailed literature study.

#### 5.4.1 Lomonaco and Klomp (1997)

The main investigation purpose in this research is to express a relation between the wave induced parameters with the specific damage parameters for pipeline protection. It is based on knowledge about near bed structures and submerged breakwater design. Experimental tests are performed in the “Scheldt fume” which is a facility owned by Delft Hydraulics.

**Hydraulic conditions**

In the test setup 3 significant wave heights $H_s$, 3 wave steepness $s_{0p}$ and three water depths $d$ are chosen, with the possibility to generate currents. These hydraulic boundary conditions are compared in Table 5-1. The introduced waves are irregular and according to the JONSWAP
Results

spectrum, for each test a series of at least 1000 waves is produced where the wave Reynolds number \( R_{ew} \) is kept larger than 20,000. To measure the incident waves, wave gauges are provided for each berm.

Table 5-1: Hydraulic boundary conditions - Lomonaco and Klomp (1997)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height  ( H_s ) [m]</td>
<td>0.15 – 0.20 – 0.25</td>
<td>0.136 – 0.144 – 0.17 – 0.18</td>
</tr>
<tr>
<td>Wave steepness ( s_{np} / s_p ) [-]</td>
<td>0.02 – 0.03 – 0.04</td>
<td>0.035</td>
</tr>
<tr>
<td>Water depth ( d ) [m]</td>
<td>0.5 – 0.7 – 0.9</td>
<td>0.45 – 0.50</td>
</tr>
<tr>
<td>Current [m/s]</td>
<td>0.00 – 0.10 – 0.13 – 0.21</td>
<td>/</td>
</tr>
</tbody>
</table>

Geometrical berm configurations and damage measurements

In Table 5-2 the geometrical configuration of the rock berm is represented. The damage assessment for small displacements and deformations is done by counting and weighing the stones. For more severe conditions an electronic measurement device PROVO is used. After each series of 1000 waves the profile is measured to get an initial and damaged profile. In order to determine the mean diameter of the grading, a sieve test is executed upon each grading.

Table 5-2: Geometric configuration rock berm - Lomonaco and Klomp (1997)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Berm height ( h_{crest} ) [m]</td>
<td>0.000 – 0.030 – 0.060 – 0.125 – 0.250</td>
<td>0.08</td>
</tr>
<tr>
<td>Berm width ( B ) [m]</td>
<td>0.06 – 0.12 – 0.25 – 0.50</td>
<td>0.08 – 0.20</td>
</tr>
<tr>
<td>Slope ( \cotg(\alpha) ) [-]</td>
<td>Flat – 1 – 2 – 3 – 5</td>
<td>2.5</td>
</tr>
<tr>
<td>Particle size ( d_{50} ) [mm]</td>
<td>2 – 4 (rounded stones)</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td>4 – 8 (rounded stones)</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>3 – 7 (sharp stones)</td>
<td>14.3</td>
</tr>
<tr>
<td></td>
<td>5 – 10 (sharp stones)</td>
<td></td>
</tr>
</tbody>
</table>

Test results

The initiation of movement starts at the crest of the structure, where the particles move towards the slopes. This movement is always a rolling motion which means the stones keep contact with the structure which indicates a relatively large weight of the stones. By analysing the stiffness of the structure, it leads to the conclusion that the stones are compacted after each test. In a pure oscillatory flow, the possibility exist that stones move from the back slope towards the front slope or in the opposite direction. Finally, the stones reach a state where no transport or damage occurs, this happens before 1000 waves. They have also noticed that in the presence of a current the damage was always lower because the measured waves were less steep and smaller.

The prediction formula of Lomonaco and Klomp (Eq. 5-9) is based upon the mobility parameter which is explained in chapter 2.4.1. This mobility parameter is calculated with the peak bottom orbital velocity based upon the linear wave theory. The damage level is based on the number of waves, slope of the rock berm, and the mobility parameter.
\[ S_{1000} = (21.4 \pm 18.4) \tan(\alpha) \Omega^{2.25} \]  
\text{Eq. 5-9}

With:  
\begin{align*}
S_{1000} & \quad [-] \quad \text{Damage level after 1000 waves} \\
\alpha & \quad [\degree] \quad \text{Slope angle of the berm} \\
\Omega & \quad [-] \quad \text{Mobility parameter}
\end{align*}

Figure 5-24 shows the curve of the damage level in function of the mobility parameter, compared with the performed tests at UGhent. An important notice is that the research by Lomonaco and Klomp (1997) obtains mobility parameters in the range between 1 and 3, which are larger than the maximum mobility parameter of 1.9 at UGhent. Further, the measured damage values with lower damage are actual not damage but just some displacement of stones. For larger values of the mobility parameter, the curve overestimates the data values but indicates a similar trend as represented by the estimated function. A possible reason of the overestimation can be the fitting of their data points which are from experiments with larger mobility parameters or different calculations of the considered wave length as currents were considered.

5.4.2 Van Gent and Wallast (2001)

The purpose of this research is to outline the physical insight related to near-bed structures and bed protections by means of physical model test and re-analysis of existing data. The hydraulic model tests are performed in the “Scheldt flume” by Delft Hydraulics.

Hydraulic conditions and wave measurements

The mean wave steepness \( s_m \) is kept constant and waves are generated according towards a JONSWAP spectrum. Three incident environmental conditions are investigated namely the influence of waves or currents separately or the combination of both. Tests with waves are
performed for 1000 and 3000 waves as the test with currents were run for 20 minutes. The waves are measured with a total of 10 wave gauges placed at different positions over the berm, the currents are measured in front of the structure with a velocity current recorder at a height of 33 centimetres above the bed.

Table 5-3: Hydraulic boundary conditions – van Gent and Wallast (2001)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height H_s [m]</td>
<td>0.06 – 0.08 – 0.09 – 0.012 – 0.15 – 0.19 – 0.21</td>
<td>0.136 – 0.144 – 0.17 – 0.18</td>
</tr>
<tr>
<td>Wave steepness s_m / s_p [-]</td>
<td>0.045</td>
<td>0.035</td>
</tr>
<tr>
<td>Water depth d [m]</td>
<td>0.415 – 0.54 – 0.665</td>
<td>0.45 – 0.50</td>
</tr>
<tr>
<td>Current [m/s]</td>
<td>0.00 – 0.010 – 0.20 – 0.34 – 0.45 – 0.74</td>
<td>/</td>
</tr>
</tbody>
</table>

Geometrical berm configurations and damage measurements

The research studies the near-bed structures as well as the influence upon the bed protection behind the structure, for this reasons 2 of the 4 near-bed structures are fixed with mortar. In two test cases a filter layer is applied, while the other two are performed without. The hydraulic boundary conditions are given in Table 5-4. The measurement of the damage levels is the same as represented in the paper of Lomonaco and Klomp (1997).

Table 5-4: Geometric configuration rock berm – van Gent and Wallast (2001)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Berm height h_crest [m]</td>
<td>0.165</td>
<td>0.08</td>
</tr>
<tr>
<td>Berm width B [m]</td>
<td>0.125</td>
<td>0.08 – 0.20</td>
</tr>
<tr>
<td>Slope cotg(α) [-]</td>
<td>3 – 8</td>
<td>2.5</td>
</tr>
<tr>
<td>Particle size d_50 [mm]</td>
<td>Fixed</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td>3.1</td>
<td>9.3</td>
</tr>
<tr>
<td></td>
<td>7.2</td>
<td>14.3</td>
</tr>
</tbody>
</table>

Test results

In the paper four methods are evaluated to assess damage upon of a near-bed structure namely: Stability number, Morison-approach, Shields-parameter and Mobility parameter. In order to make a clear comparison with our test data, only the last two methods are evaluated. All damage analyses are performed by dividing the dimensionless eroded area by the root of the number of waves.

A design formula is developed based upon the Shield parameter, Eq. 5-10. For the bed shear stresses, the peak bottom orbital velocity u_b,crest is used which is calculated upon the significant wave height H_s, mean wave period T_m, and the water level at the height of the crest d_crest. Note that the constants in the formula are based upon actions by waves only and waves combined with currents.
Results

\[ \frac{S}{\sqrt{N}} = 4 \times 10^5 \theta^5 \]  

Eq. 5-10

With:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>[-]</td>
<td>Number of waves</td>
</tr>
<tr>
<td>S</td>
<td>[-]</td>
<td>Damage level</td>
</tr>
<tr>
<td>(\theta)</td>
<td>[-]</td>
<td>Shields stability parameter</td>
</tr>
</tbody>
</table>

Figure 5-25 shows the graphical representation of the formula with the experiments of UGhent. A main conclusion is that the curve underestimates the damage. A possible reason for this relative large difference is again the range within the shields parameter \(\theta\) is tested. In the test setup of van Gent and Wallast (2001), Shields parameters up to 0.12 are developed. According to their tests, the same conclusion can be found, Figure 5-26. Additionally, the berm configuration is significantly different as the study includes the testing of the bed protection.

![Figure 5-25: Damage assessment – Shields parameter – van Gent and Wallast (2001)](image1)

![Figure 5-26: Original plot of all test data by van Gent and Wallast (2001)](image2)
A design formula is developed based upon the mobility parameter $\Omega$, Eq. 5-11. The parameter is calculated with the peak bottom orbital velocity $u_\delta$ at the height of the crest. Figure 5-27 is a representation of the test data (UGhent) towards the equation of van Gent and Wallast. A similar conclusion could be made as mentioned for the Shields parameter, the fit of their data is not appropriate for the performed tests at the UGhent. In the paper of van Gent and Wallast it is concluded that the mobility parameter $\Omega$ is the best representative stability parameter to give an assessment upon the resulting damage.

$$\frac{S}{\sqrt{N}} = 0.2 \, \Omega^3$$

Eq. 5-11

<table>
<thead>
<tr>
<th>With:</th>
<th>Symbol</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N$</td>
<td>[-]</td>
<td>Number of waves</td>
</tr>
<tr>
<td>$S$</td>
<td>[-]</td>
<td>Damage level</td>
</tr>
<tr>
<td>$\Omega$</td>
<td>[-]</td>
<td>Mobility parameter</td>
</tr>
</tbody>
</table>

Figure 5-27: Damage assessment – mobility parameter – van Gent and Wallast (2001)

5.4.3 Sears (2005)

The hydraulic model tests are performed in the fluid mechanic’s laboratory of TU Delft. To deal with the roughness of the bed, small particles are glued to the bottom in the area of the structure.

Hydraulic conditions and measurements

The hydraulic boundary conditions are shown in Table 5-5. The main difference with the test at UGhent is the investigation on the influence of the amount of waves. This is done by testing the rock berm structure with 1000, 3000 and 6000 waves. The measurement of the wave parameters is performed by wave gauges.
Results

Table 5-5: Hydraulic boundary conditions – Saers (2005)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height $H_s$ [m]</td>
<td>0.16 – 0.18 – 0.20 – 0.22</td>
<td>0.136 – 0.144 – 0.17 – 0.18</td>
</tr>
<tr>
<td>Wave steepness $s_p$ [-]</td>
<td>0.03</td>
<td>0.035</td>
</tr>
<tr>
<td>Water depth $d$ [m]</td>
<td>0.40 – 0.45 – 0.50 – 0.55</td>
<td>0.45 – 0.50</td>
</tr>
<tr>
<td>Current [m/s]</td>
<td>/</td>
<td>/</td>
</tr>
</tbody>
</table>

Geometrical configuration and damage measurements

In the test setup, a median stone diameter $d_{50}$ of 4.4 millimetres is used for the armour layer. A difference on the specific density of the coloured stones is noticed ($\rho_s = 2470$ kg/m$^3$). The mean geometrical parameter that is varied during the tests is the crest height $h_{crest}$. The berm width $B$ and cotg ($\alpha$) of the slope are constant with respectively values of 4 centimetres and 2.5. The damage measurement is performed with the PROVO measuring system, as used for the experiments of Lomonaco and Klomp (1997). Table 5-6 compares the berm configurations of both master thesis dissertations.

Table 5-6: Geometrical configuration rock berm – Sears (2005)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Berm height $h_{crest}$ [m]</td>
<td>0.04 – 0.05 – 0.06</td>
<td>0.08</td>
</tr>
<tr>
<td>Berm width $B$ [m]</td>
<td>0.04</td>
<td>0.08 – 0.20</td>
</tr>
<tr>
<td>Slope cotg($\alpha$) [-]</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Particle size $d_{50}$ [mm]</td>
<td>4.4</td>
<td>7.7, 9.3, 14.3</td>
</tr>
</tbody>
</table>

Test results

The initial movement of stones at the crest of the structure are rolling stones, occasionally bouncing. The stones follow an oscillatory movement and a few move from the upstream slope towards the downward slope, the opposite movement is less observed. When the number of waves is increased, the stones are more stable with occasional movement of three to five stones in group.

From the time dependent testing (amount of waves) it is found that the number of waves are not distributed by the square root of the number of waves (indicated by van Gent and Wallast (2001)) but more logarithmic. In order to evaluate the damage of the structure the mobility parameter is expressed by using the peak bottom orbital velocity $u_{\delta,crest}$ according to linear wave theory. The velocity is determined by the significant wave height, peak wave period, and water depth at the crest. The formula of the damage level in function of the mentioned parameters is given by Eq. 5-12.

$$S = 0.8 \Omega^{2.5} \log(N)$$  
Eq. 5-12
Comparing the data of the tests at UGhent again with the prediction curve of Saers (Figure 5-28), a better fit is obtained, compared with the previous experiments. However, it should be noted that this research of Sears has also investigated larger mobility parameters between 2 and 4.

![Figure 5-28: Damage assessment – mobility parameter – Saers (2005)](image)

### 5.4.4 Van Den Bos (2006)

This manuscript is a literature study and a quantitative analysis for expressions between the stability parameters and damage levels. It differs from the previous paragraphs as no actual hydraulic model tests are performed.

**Conclusions**

Van Den Bos (2006) proposes to give a damage evaluation in function of $S^*$ combined with the influence of the slopes of the rock berm, and the number of waves in a test series. For the damage prediction formula, a mobility parameter with the maximum orbital velocity at the crest is used. In order to determine the velocity, he uses $H_{1\%}$ instead of $H_c$ and the peak wave period, both are determined from spectral analysis. By performing data analyses in the software SPSS, he comes up with three prediction formulas that describe a lower, mean and upper value of the damage level in function of the mobility parameter $\Omega_{hc1\%}$ calculated with a wave height of $H_{1\%}$. These three functions are represented in Eq. 5-13 until Eq. 5-15.

\[
\text{Mean value: } \frac{S^*}{N^{0.3}} = 0.048 \left(\Omega_{hc1\%}\right)^{1.6} \tan(\alpha)^{0.6} \quad \text{Eq. 5-13}
\]
Results

Lower bound: \( \frac{S^*}{N^{0.3}} = 0.02 \left( \Omega_{hc1\%} \right)^{1.6} \tan(\alpha)^{0.6} \) \hspace{1cm} \text{Eq. 5-14}

Upper bound: \( \frac{S^*}{N^{0.3}} = 0.12 \left( \Omega_{hc1\%} \right)^{1.6} \tan(\alpha)^{0.6} \) \hspace{1cm} \text{Eq. 5-15}

With:

- \( N \) [\#] Number of waves
- \( S^* \) [-] Adapted damage level
- \( \alpha \) [°] Angle of slope
- \( \Omega_{hc1\%} \) [-] Mobility parameter based on \( H_{1\%} \) and \( T_p \) at the level of the crest

When the boundaries are represented with the data points of the test at UGhent, Figure 5-29 is obtained. It can be seen that none of the data points over exceed the upper limit. Most damage levels are located between the average value and the lower limit. Some damage levels are located below the lower boundary limit. Again the range of the studied mobility parameters is different and the values of the damage levels at the UGhent have a significant uncertainty for low damage numbers.

![Figure 5-29: Damage assessment – mobility parameter – van den Bos (2006)](image-url)
Chapter 6  Conclusions and recommendations

This final chapter answers the main research objectives which were mentioned in Chapter 1. Additional findings and comments of the experimental tests are also summarized. A more detailed analysis of the methodology and results is given in Chapter 4 and Chapter 5.

6.1 Conclusions

The following conclusions are valid for relative low damage numbers of the rock berm which are loaded under a unidirectional wave direction. No currents or oblique wave attack are considered in the experiments.

- Experimental tests of near bed structures in the large wave flume of UGhent are possible but rather limited until Shields numbers of 0.030. Using larger scaling factors as alternative (instead of 25) lead to smaller grains which are practical difficult for constructing the scale model. The hydraulic flow regime through the stones of the armour layer is more affected towards a laminar flow regime instead of a rough turbulent one (paragraph 4.4.4). By using a smaller scaling factor on the other hand, it results in smaller significant waves heights (physical limitations) which means that smaller Shields numbers or bed shear stresses are achieved and thus lower mean damage levels at the rock berm.

- A scaling factor of 25 is applied considering the physical limits of the wave flume. From this the following hydraulic boundary conditions of the prototype structure that are finally obtained with the experiments are: a significant wave height ranging between 3.4 and 4.5 metres, a water depth of 11.3 or 12.5 metres, and a wave steepness of 0.035. The wave steepness results in peak wave periods between 9.7 and 12.8 seconds. (chapter 4.1)

- The sensitivity study of the hydraulic parameters shows trends which are conform the theory. Larger significant wave heights and smaller water depths results in larger mean damage levels because the executed bed shear stresses become larger. The influence of the
Conclusions and recommendations

- The wave period is less pronounced because of the constant wave steepness that is considered during the experiments. (chapter 5.1)

- It is noticed during the 2D damage analysis that the accuracy of the mean damage levels depends clearly on the amount of damage. However, for larger levels ($S > 10$), a coefficient of variation which is equal to 40 percent is unavoidable because of the significant standard deviations on the damage measurements with the laser profiler or the irregular pattern of damages. For levels lower than 10, the damage is very limited and averaging the cross-section until one 2D-cross-section results in coefficients of variation until 240 percent. The amount of damage is lower than the accuracy of the measurement. (chapter 4.4.5)

- The consideration of shakedown tests results in mean damage levels which are equal or larger than the tests without. Considering also the accuracy of the damage measurements (chapter 4.4.5) it can be concluded that the placement of the stones does not have a significant influence upon the measured mean damage levels. (chapter 4.4.6)

- Placement of a mop in between the core and armour layer has an influence on the damage levels but can be questioned because of the scatter on the damage measurements. A clear trend cannot be defined but it is assumed that the use of the mop gives a conservative approach in terms of stone movements. Without using a mop, the oscillatory water flow is less blocked which can result in less uplift forces, the relative permeability is changed. (chapter 4.4.7)

- Looking to the mean damage levels of the different sections of the rock berm, some regular patterns can be noticed. The main damage occurs at the crest where stones start to roll oscillatory or in the wave propagation direction. For the larger waves, transport of stones happens at the front and back slope. However, stones move more in the wave propagation direction than vice versa. A possible reason can be a net mass transport of the fluid (Stokes drift). Regarding to the total mean damage levels, on average 45 % of the damage occurs at the crest, 35 % at the front slope, and only 20 % damage on the back slope. It means that the wave propagation has an influence on the sectional damage of the rock berm for transitional water. (chapter 5.2)

- The influence of changing the berm width should be accompanied by a new definition of the damage level (van den Bos, 2006). The adapted damage level or eroded thickness includes the berm width because a larger berm will always result in larger damage levels. However, the adapted definition concludes that larger berm-widths result in a lower values of the eroded thickness(paragraph 5.1.5). A possible reason is that the present definition of the damage level with the parameter “eroded area” not takes into account a possible refill of a scour hole. In a larger berm, the scour holes have more chance to become refilled. The front slope and crest have a smaller mean eroded thickness for a wider berm as the back slope indicates a small increase (chapter 5.2.3).
Conclusions and recommendations

- When the mean damage level is plotted towards the Shields stability parameter, the trend is a classic a power function. However, the scatter on the damage values means that the R-squared coefficient (coefficient of determination) gives a relative low value of 0.55. Possible explanations are the inaccuracy of the damage measurements and the irregular pattern of the damage development during the tests (chapter 5.3).

- The Shields stability parameter of the performed test matrix – conform the recent paper of Roulund et al. (2016) – does not exceed the critical Shields curve for waves (van Rijn, 1993). It means that the largest 2D mean damage levels (almost 20) of the tests are still below the maximum tolerated damage. This is due to the maximum tested Shields value of about 0.030 (chapter 5.3.2). A less significant point of attention is the influence of the mop which is assumed to have a positive effect on the damage levels (less damage).

- With the adapted damage level, an alternative definition of the critical Shields is proposed. It is correlated with the moment that the number of scoured armour layers or eroded thickness exceeds the installed armour thickness. With respect to this definition, a maximum eroded thickness of 35 % is obtained in the test set-up. (chapter 5.3.2)

- The data points of the damage levels are compared towards three similar researches and one theoretical review of damage towards rock berms for pipeline protection. The fit through the damage levels of their studies overestimates the damage levels of the tests at UGhent. A possible reason is that the other investigators have tested larger stability parameters which can result in other values of trend line. A second remark in their analyses is that the mobility stability parameter is preferred instead of the Shields stability parameter. (chapter 5.4)

- The non-contact laser profiler developed by the Aalborg University is equipped with the software EPro which is easy in application and operation. The profiler runs sometimes stuck when the measured data point is not measured correctly, this is mostly resolved by using the “repositioning” button in the EPro software. After applying a filter, the software gives values of the averaged 2D and 3D damage levels. By filtering of the cross-section profiles, a bug is discovered (in corporation with the Aalborg University) in the application of the software. (chapter 4.4.5)

- Using the laser profiler is preferred when rather larger damage levels are obtained (> 10) in experimental tests. The case of lower damage levels, an alternative option is counting or weighing the displaced stones.

- The electromagnetic velocity recorder gives quite good measurements with a maximum standard deviation of 4% for the values of the velocity parameter $u_m$ for one test. A downside of the recorder is the equipment size, the device has large dimension. Further, no processing system is available to process the output results, in manuscript it is done by using a spreadsheet program (Excel).
Conclusions and recommendations

6.2 Further research

Further research can be done with respect to the damage development of a rock berm pipeline protection.

- In the wave flume of UGhent, a wider range of hydraulic boundaries can be tested to obtain a larger insight on the influence of the peak wave period and the water depth. For smaller water depths, it should also be studied if breaking waves will have an influence on the damage levels. To investigate a parameter in more detail, tests with regular waves are a good or even a better alternative.

- Geometrical properties of the rock berm have also potential for further investigation. Changing the berm width, crest level, or the slope steepness are some options for a more detailed analysis.

- The influence of the rock grading is not investigated yet. A different grading can result in another flow pattern through the stones or flow regime in a scale model.

- Using a mop in between the core and the armour layer has an influence on the damage levels in experiments. From the small case study, it is assumed that the obtained damage levels are conservative. A larger study should be performed to check the influence more in detail for different hydraulic and geometric conditions.
References


References


References


## Annex

### A. Overview Different Rock Gradings

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<tr>
<th>Class name</th>
<th>range</th>
<th>described in EN13383</th>
<th>range of $M_{50}$ for category &quot;A&quot; (kg)</th>
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<th>$D_{90}/D_{15}$</th>
<th>$D_{90}$ (cm)</th>
<th>Layer thickness 1.5 $D_{90}$ (cm)</th>
<th>Minimal dumping quantity with layer of 1.5 $D_{90}$ (kg/m²)</th>
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B. ROCK GRADINGS

SCALED GRADING OF THE LMA 5-40 ROCK GRADING

SCALED GRADING OF THE LMA 10-60 ROCK GRADING
Scaled grading of the LMA 40-200 rock grading
C. OVERVIEW HYDRAULIC MODEL TESTS

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D. FLOW CHART TEST PROCEDURE

The operation of one hydraulic model test with laser profiling in “wet” conditions includes;

1. Building of the scale model on the concrete floor of the wave flume (± 15 min)
2. Filling the wave basin with fresh water from a water reservoir with two pumps (± 30 min)
3. Laser profiling of armour layer over the cross-section with a width of 150 mm (± 20 min)
4. Calibration of 8 wave gauges in still water (± 2 min)
5. Wave generation with piston-type paddle – displacements of the paddle for irregular waves are produced conform the JONSWAP spectrum with parameters; water depth d, wave peak period $T_p$, significant wave height $H_s$, peak-shape parameter $\gamma$, and duration of the time series $t$ (~ number of waves). (± 40 min)
6. Laser profiling of armour layer over the cross-section with a width of 150 mm (± 20 min)
7. Emptying of the basin to the water reservoir with two pumps (± 20 min)

When a scan happens in “dry” conditions steps 3 and 6 are respectively shifted with steps 2 and 7.

As for some tests shakedown was performed the procedure in “wet” conditions is as follows:

1. Building of the scale model on the concrete floor of the wave flume (± 15 min)
2. Filling the wave basin with fresh water from a water reservoir with two pumps (± 30 min)
3. Laser profiling of armour layer (initial) over the cross-section with a width of 150 mm (± 20 min)
4. Calibration of 8 wave gauges in still water (± 2 min)
5. Wave generation for shakedown test with piston-type paddle – displacements of the paddle for irregular waves are produced conform the JONSWAP spectrum with parameters; water depth d, wave peak period $T_p$, 50% of the significant wave height $H_s$, peak-shape parameter $\gamma$, and $1/3$ of the duration of the time series $t$ (~ number of waves). (± 40 min)
6. Laser profiling of armour layer (after shakedown) over the cross-section with a width of 150 mm (± 20 min)
7. Calibration of 8 wave gauges in still water (± 2 min)
8. Wave generation with piston-type paddle – displacements of the paddle for irregular waves are produced conform the JONSWAP spectrum with parameters; water depth d, wave peak period $T_p$, significant wave height $H_s$, peak-shape parameter $\gamma$, and duration of the time series $t$ (~ number of waves). (± 40 min)
9. Laser profiling of armour layer (final) over the cross-section with a width of 150 mm (± 20 min)
10. Emptying of the basin to the water reservoir with two pumps (± 20 min)

When a test with shake down happens in “dry” conditions steps 3, 6, and 9 are respectively shifted with steps 2,4 and 7. Between these steps a filling and emptying procedure of in total about 50 minutes is necessary. This leads to a total test duration of about 5 hours and a quarter. This is quite inconvenient for practical testing.
## E. AVERAGE AND DEVIATIONS ON TEST RESULTS BY PERFORMING MULTIPLE TESTS

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<th>B [m]</th>
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<th>Hs [m]</th>
<th>Tp [s]</th>
<th>sp [-]</th>
<th>#Nbreak</th>
<th>Um [m/s]</th>
<th>θ [-]</th>
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