Faculty of Civil Engineering and Geosciences

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Stability of Icelandic type Berm Breakwaters

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MSc Thesis

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Preface

This thesis project represents the final work of my M.Sc. study at Delft University of Technology, the faculty of Civil Engineering. My studies were focused on Coastal Engineering and the subject of this final project is "Stability of Icelandic type Berm Breakwaters". The project is carried out in cooperation with Witteveen+Bos Consulting Engineers.

This project consists of model test experiments as well as general literature study. The scale model experiments were carried out in the long wave flume of the Laboratory of Fluid Mechanics at Delft University of Technology. I would like to thank the members of the graduation committee as well as co-workers in the Laboratory for their assistant during this project. I would also like to thank Mr. Arnar B. Sigurdsson for proofreading the document.

The work was supervised by a graduation committee with members from TU Delft, Witteveen+Bos Consulting Engineers and the Icelandic Maritime Administration:

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Summary

A major part of the breakwaters constructed in the world are the so-called conventional rouble mound breakwaters, Figure 1a), that consist of a core, a filter layer and a heavy armour layer. An alternative to the conventional rouble mound breakwater is a berm breakwater. Berm breakwaters have mainly developed in two directions over the last couple of decades. On the one hand a dynamically stable structure, where reshaping is allowed, Figure 1b). And on the other hand a more stable multi layered structure often referred to as Icelandic type berm breakwater, Figure 1c).

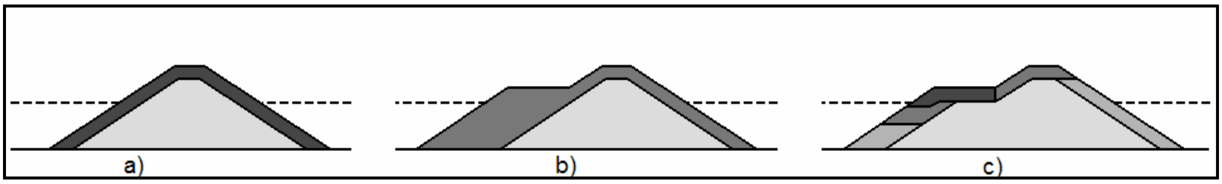

Figure 1. Example of different types of breakwaters, a) Conventional rubble mound breakwater, b) dynamically stable berm breakwater and c) Icelandic type berm breakwater

When there is a quarry, relatively close to the construction site, which is dedicated to the breakwater project, the Icelandic type has proven to be very attractive economically. The basic reason for that is that unlike the other types the Icelandic type utilizes the quarry 100%.

This M.Sc. thesis focuses on the Icelandic type berm breakwater. Before an Icelandic type berm breakwater is constructed the stones are divided into classes depending on their size. The smaller armour stones are then placed rather deep where the influence of the wave attack is less, while the largest stones are placed where the largest wave attack is expected.

Goals of this project:

- Design rules for the transaction of stone classes with depth have not yet evolved and the main goal of this project was to develop a stability criterion for the stones in that area. (Primary goal)
- Stones on berm. Since the total amount of the largest stones (Class I) is usually limited, the combination of the amount of large stones on the berm and down the berm is important. (Secondary goal)
- Recession. Recession will be measured in each test and thereby a large database on the subject will be made available for further research on the subject. (Secondary goal)

- The location of the transition of the original and the reshaped profiles as the berm height changes as well as for different stone setups. This is also closely related to the primary goal of the project. (Secondary goal)

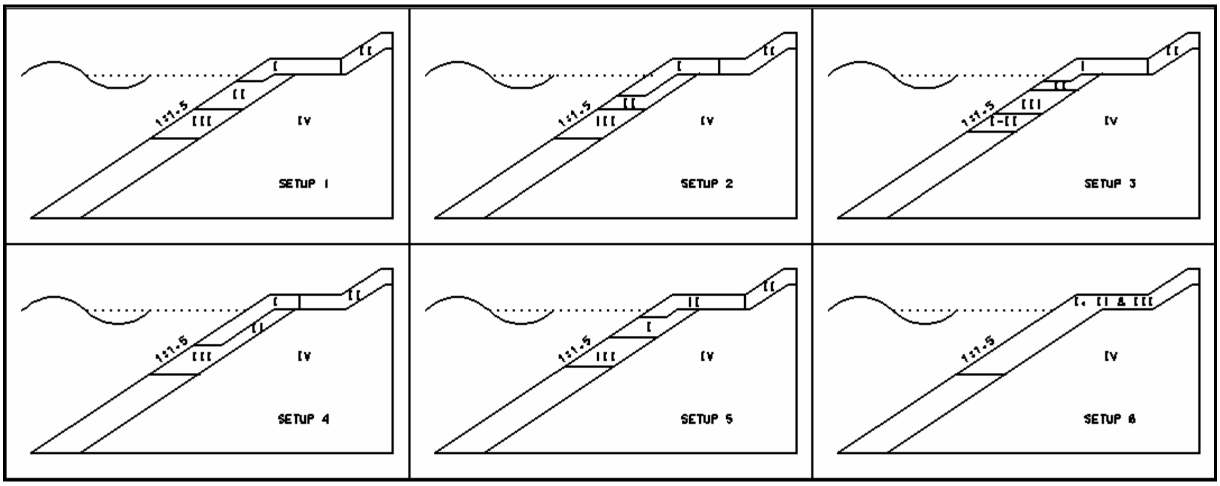

Figure 2. Different model setups used in the model tests

Numbers of model tests were performed in order to reach those goals or a total of 70 tests with 6 different model setups (Figure 2). To follow the influence of different model setups and different berm heights on the behaviour of the structure the following measures were used:

- Visual observation with the help of a camera that was use to take photos regularly during the tests (500, 1000, 2000 and 3000 waves) and a video camera.
- Recession measurements. After each test the recession at berm level was measured.
- General erosion measurements. After each test the cross section profile was measured and from that the total erosion was evaluated.
- To follow the location of the transition of the original and the reshaped profiles after each test, the cross section profile was used.

After analysing the test results the main conclusions were the following:

1. Class I stones on the berm are recommended to reach at least further into the berm than the expected recession from a design storm.
2. Class I stones are recommended to reach as far down as:

$$\mathbf{h_{I\text{-}II} \geq 1.45 \cdot \Delta D_{n50,\,Class\ I}}$$

or

$$\mathbf{h_{I\text{-}II} \geq 1.85 \cdot \Delta D_{n50,\ Class\ II}}$$

The one of the two that gives the larger $h_{I\text{-}II}$ in each case is the recommended choice.

3. If there is more of Class I stones available after meeting the recommendations above, (1 and 2) they should be placed on the berm.
4. Class II stones should in any case reach at least down to the transition of the original and the expected reshaped profile, h_f.

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List of symbols

A_e	Erosion area on rock profile	(m^2)
B_B	Berm width	(m)
D	Diameter of stone	(m)
D_n	Nominal block diameter, $D_n = (M/\rho_s)^{1/3}$	(m)
D_{n50}	Median nominal diameter, $D_{n50} = (M_{50}/\rho_s)^{1/3}$	(m)
D_{n85}	85% value of sieve curve	(m)
D_{n15}	15% value of sieve curve	(m)
f_d	Water depth factor, Tørum	(-)
f_g	Gradation factor, Tørum	(-)
g	Gravitational acceleration	(m/s^2)
H	Wave height, from trough to crest	(m)
H_0	Stability number, $N_s = H_0 = H_s/(\Delta D_{n50})$	(-)
H_0T_0	Period stability number $H_0T_0 = H_s/(\Delta D_{n50})\sqrt{(g/D_{n50})}\cdot T_z$	(-)
H_0T_{0p}	Period stability number with T_p	(-)
$H_{1/10}$	Average of 10% highest waves	(m)
H_{m0}	Significant wave height calculated from the spectrum, $H_{m0} = 4\sqrt{m_0}$	(m)
H_s	Significant wave height	(m)
h	Water depth; water depth at structure toe	(m)
h_f	Distance from water level to profile transitions	(m)
h_B	Distance from berm level to water level	(m)
$h_{I\text{-}II}$	Distance from the water level to the transaction of Class I and Class II stones	(m)
$h_{II\text{-}III}$	Distance from the water level to the transaction of Class II and Class III stones	(m)
K_D	Stability coefficient, Hudson formula	(-)
L	Wavelength, in the direction of propagation	(m)
L_o	Deep-water wavelength, $L_0 = g/T^2/2\pi$	(m)
L_{om}	Deep-water wavelength of mean period, T_m	(m)
L_{op}	Deep-water wavelength of peak period, T_p	(m)
$L_{m\ ,\ Lp}$	Wavelength at structure toe, based on T_m and T_p	(m)
M	Mass of an armour unit	(kg)
M_{50}	Mass of particle for which 50% of the granular material is lighter	(kg)

N	Number of waves over the duration test	(-)
P	Permeability parameter	(-)
Rec	Recession	(m)
S_d	Non-dimensional damage, $S_d = A_e/D^2_{n50}$	(-)
Sc	Scatter in recession measurements	(-)
s	Wave steepness, $s = H/L$	(-)
s_0	Wave steepness, defined as $H_s/L_0 = 2\pi H_s/(gT^2_m)$	(-)
s_{om}	Wave steepness for mean period wave, $s_{om} = 2\pi H_s = (gT^2_m)$	(-)
s_{oz}	Wave steepness for zero up crossing period wave, $s_{om} = 2\pi H_s=(gT^2_z)$	(-)
s_{op}	Wave steepness for peak period wave, $s_{op} = 2\pi H_s = (gT^2_p)$	(-)
s_p	Wave steepness at toe for peak period wave, $s_p = H_s/L_{op}$	(-)
T	Wave period	(s)
T_m	Mean wave period	(s)
T_z	Zero up-crossing wave period	(s)
T_p	Spectral peak period, inverse of peak frequency	(s)
α	Structure slope angle	(°)
Δ	Relative buoyant density of material, ie for stones $\Delta = (\rho_s - \rho_w)/\rho_w$	(-)
ξ	Surf similarity parameter or Iribarren number, $\xi = \tan\alpha/\sqrt{s_o}$	(-)
$\xi_{transition}$	Critical surf similarity parameter, Van der Meer formula	(-)
ξ_m	Surf similarity parameter for mean wave period T_m	(-)
ξ_z	Surf similarity parameter for mean wave period T_z	(-)
ξ_p	Surf similarity parameter for mean wave period T_p	(-)
ρ	Mass density	(kg/m^3)
ρ_s	Mass density of stones	(kg/m^3)
ρ_w	Mass density of water	(kg/m^3)

1 Introduction

1.1 General

This document presents my M.Sc. thesis in Hydraulic Engineering at the faculty of Civil Engineering of Delft University of Technology. This project is carried out in cooperation with Witteveen+Bos Consulting Engineers.

Breakwaters are designed and constructed all over the world. The function of a breakwater is to protect coastal areas from waves and currents. The most common use is to protect the sailing path of vessels when approaching harbours. Furthermore, sometimes breakwaters serve as protection against coastal erosion.

When it comes to designing a breakwater there are many types to choose from, constructed with natural rock or concrete blocks or a combination of the two. In rocky areas rocks are usually a relatively cheaper construction material than concrete blocks, thus the choice in those areas is likely to be between different breakwaters constructed with of rocks or in some cases a combination with concrete blocks might be interesting.

The most common type of breakwaters constructed from rocks is the so called conventional rubble mound breakwater, Figure 1.1a), which is a stable structure consisting of a quarry run core which is protected with one or more filter layers and larger and heavier armour layer. This structure is usually designed as a stable structure and movement of rocks during extreme storm events is limited.

An alternative solution is a so called berm breakwater. Berm breakwaters have basically developed in two directions over the last couple of decades. On the one hand a dynamically stable berm breakwaters, where reshaping is allowed, Figure 1.1b). And on the other hand a more stable structure often referred to as the Icelandic type berm breakwater, Figure 1.1c). The latter one is a multi layer berm breakwater where the stones are divided into classes depending on their size and lined up in such a way that the largest stones are the once that are under the greatest wave attack. This MSc thesis report focuses on the Icelandic type berm breakwater.

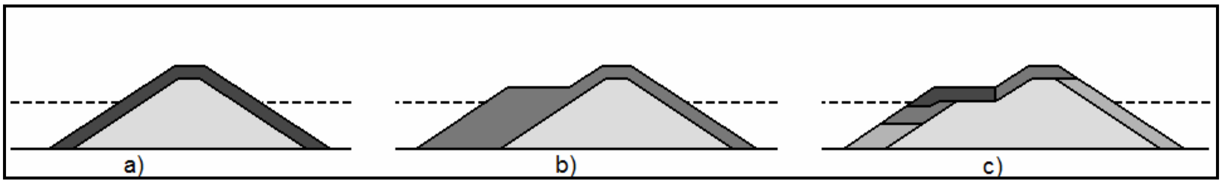


Figure 1.1 Example of different types of breakwaters, a) Conventional rubble mound breakwater, b) dynamically stable berm breakwater and c) Icelandic type berm breakwater

1.2 Historical review

In 1983 a design for a berm breakwater for a tank terminal was accepted in Helguvik, Iceland, where at that time there was an American army base. This project turned out to be a critical point in the design of berm breakwaters. From that moment many berm breakwaters have been designed and constructed in Iceland. The concept of dynamic berm breakwaters did however never establish itself there, where from the beginning the development was in the direction of a more stable structure, later known as Icelandic type berm breakwaters. Whereas elsewhere, where berm breakwaters were designed the dynamically stable form was usually chosen.

In the early stages of the learning curve of the Icelandic type berm breakwaters, one breakwater failed to fulfil its expectation when it experienced a storm resulting in wave conditions close to the design wave height. This breakwater was constructed in Bakkafjordur, a small fishing village in the North-East of Iceland, construction was completed in 1984. The poor quality of rocks is considered the main reasons for the failure. [Sigurdarson et al (1998)]

Figure 1.2 Sirevåg berm breakwater, stock pile of stone classes I and II. Sigurdarson et al 2001

Since then many Icelandic type berm breakwaters have been constructed, mainly in Iceland and Norway. Some of those breakwaters have experienced a storm in the order of magnitude of the design storm without considerable damage. The best known example is the breakwater in Sirevåg, on the west coast of Norway. Where construction was completed in 2001 and during the first winter in use it experienced a storm reaching the design wave height. The breakwater survived the storm without considerable damage, and no repair work was needed. Although it is unusual that a structure experiences a design storm in the first year in service it shows the quality of the design of the Icelandic type berm breakwaters.

Figure 1.3 Inspection of the front slope of the berm of the Sirevåg breakwater, a section with Class I stones. Sigurdarson et al 2001

1.3 Benefits of the Icelandic type berm breakwater

One of the biggest advantages of the Icelandic type berm breakwater is the complete utilization of the quarry. This makes the Icelandic breakwater economically attractive, when there is a quarry dedicated to the project, relatively close to the construction side. The design and construction method focuses on tailor-making the structure around the design wave conditions, possible quarry yield, available construction equipment, transport routs and required functions. The Icelandic method has developed in close cooperation with all parties involved, designers, geologists, supervisors, contractors and local governments.

Another advantage over the dynamic berm breakwater is the placement of the biggest armour stones, from the berm and down the slope. The rocks are carefully placed in such a way that they give good interlocking between them and therefore strengthen the structure.

Compared to the dynamic berm breakwaters the total volume of armour rocks needed in the Icelandic type is less but on the other hand larger rocks are needed. Unlike the dynamic berm breakwaters the Icelandic type has a narrow stone gradation which results in higher permeability, which increases the ability to absorb wave energy. There is also less movement of stones than in the dynamic structure, abrasion and breaking of stones it thereby minimized. This can have effects in the long run resulting in longer service life of the structure.

Compared with the conventional rubble mound breakwater on the other hand the Icelandic type requires less volume of big stones. The reason is that the conventional breakwater is required to be almost statically stable whereas little movement is allowed in the Icelandic type berm breakwaters.

There is a basic difference between the Icelandic type and the other two. That is that more time and therefore cost is spent on design and preparation, starting with the quarry yield prediction. This has proven to be economically attractive since costs are saved in other parts of each project as a result of good planning.

There are a few disadvantages as well compared to the other types mentioned. The Icelandic type is slightly more complicated to construct than the dynamic berm breakwater since the structure is not homogenous and due to the fact that part of the rocks are placed carefully but not dumped randomly. Another disadvantage is that more time is needed for preparation work. That includes more work for the contractors in sorting the stones, since the stones have to be divided in about 5 different groups instead of 1-2 for “normal” projects. But as stated before it has proven to be economical where there is a quarry relatively close to the construction site that is dedicated to the project. If a quarry is dedicated to a project of a dynamic berm breakwater or a conventional breakwater it can however possibly be economically attractive, depending on the design wave height and if some other usage is found for the part of stones that is not used in the breakwater project. That is however beyond the scope of this M.Sc. thesis project.

1.4 Project definition

The subject of this research project concerns the stability of so called Icelandic type berm breakwaters. An Icelandic type berm breakwater is a multilayer berm breakwater, where the stones are divided into classes depending on their size. Researches until now on the Icelandic type berm breakwater have been focused on various aspects on, or closely related to, the subject.

- Recession (Tørum (1998) (2000), Sigurdarson et al (2003))
- Stone breaking strength (Tørum et al (2003))
- Ice ride-up on berm breakwaters. (Myhra (2005))
- Variety of articles on constructed breakwaters and researches as preparation in the design process. Special attention has been on the breakwater constructed in Sirevåg, Norway.

When it comes to design methods for Icelandic type berm breakwaters, a design criterion does exist for determining the stone size needed for the part of the breakwater that is exposed to the biggest wave forces. For this part, where the largest stones are used the stability parameter, H_0, is recommended to be close to 2. This stability parameter is explained in chapter 2.1.1, it has no connection with deep water wave height and should not be confused with that parameter. This part reaches from the berm and at least down to the design water level.

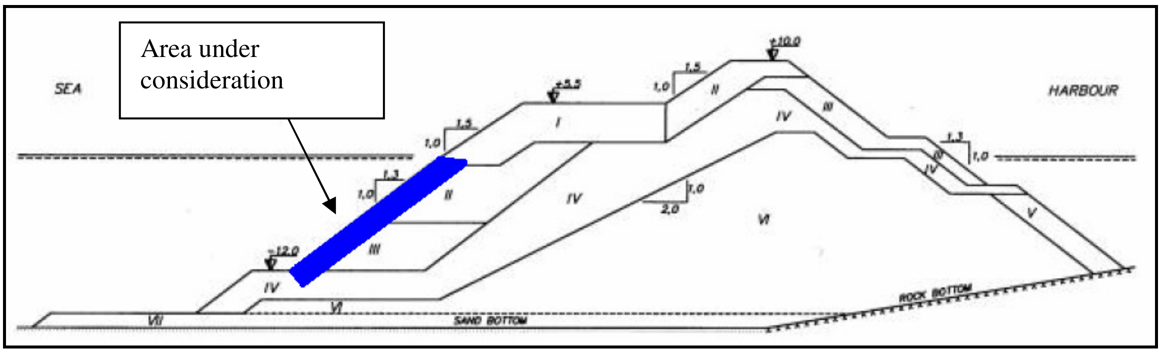


Figure 1.4 Area under consideration

In the view of the above the following research goals were developed for this M.Sc. thesis:

- When looking at the part of the breakwater that is under water the design rules have not fully developed. It is valuable to have a criterion for conditions under water, to develop a better understanding on the structure as a whole. The main goal of this project is therefore to develop design criterion for the location of the transition of the different stone classes in that area. Figure 1.4 explains the part of the breakwater under consideration. (Primary goal)

- Stones on the berm. Since the total amount of the largest stones (Class I) is usually limited, the combination of those large stones on the berm and down the front slope of the berm is important. This secondary goal is thereby closely related to the primary goal of this project. (Secondary goal)
- Recession. Recession is an important subject in berm breakwater design. This subject has been researched already to some degree, but more on structures with a homogenous berm. Further research focused on recession on Icelandic type berm breakwaters is recommended. The data gathered from the number of model tests in this research can be helpful for further study on this subject. (Secondary goal)
- The behaviour of the transition of the original and the reshaped profiles as the berm height changes as well as for different stone setups. This is also closely related to the primary goal of the project. (Secondary goal)

To reach those goals, scale model tests were carried out in a wave flume in the Fluid Mechanics Laboratory of the Faculty of Civil Engineering at TU Delft. The model tests are described in chapter 3.

1.5 Outline

This introduction chapter is followed with a chapter (chapter two) of literature study. Wherein the most important parameter considered in berm breakwater design and supervision are considered as well as the relative wave parameters. The damage/reshape measurements used in this research are then explained while the chapter is finalized with the current design rules of Icelandic type berm breakwaters.

In chapter three the experimental setup is explained throughout. That includes the model setup in the flume and an explanation of the different setups that were tested in this experiment. The material used is analysed, the observation methods are explained as well as the equipments related to the experiment.

The analyses takes place in chapter four where the different measures introduced in chapter two are used to analyse the test results. Those measures are mostly related to the goals of this M.Sc. thesis project.

Chapter five includes general discussion of the model tests. The report is then finalized with chapter six where the conclusions of this research project are introduced and recommendations are made for further work on the subject and related subjects.

2 Literature study of the theory

2.1 Waves

2.1.1 Wave impact

The following parameters are commonly used when considering the stability of berm breakwaters.

When an Icelandic type berm breakwater is designed the most commonly used parameter is the stability parameter Ns or H_0, in this report referred to as H_0. This parameter is dimensionless and gives a relation between the armour layer and the impact of the incoming wave, equation 2.1

$$N_s = H_0 = \frac{H_s}{\Delta D_{n50}} \tag{2.1}$$

where H_s (m) is the significant wave height, Δ (-) the relative buoyant density, and D_{n50} (m) is the median nominal diameter of the armour stones

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{2.2}$$

where ρ_s (kg/m^3) is the density of the stones and ρ_w (kg/m^3) is the density of water, and

$$D_{n50} = \left(\frac{M_{50}}{\rho_s}\right)^{1/3} \tag{2.3}$$

where M_{50} (kg) is the median stone mass, the mass of a theoretical block for which 50 % of the mass of the sample is lighter.

The wave period stability number H_0T_0 is another parameter where the effect of wave period is added to the stability number H_0,

$$H_0T_0 = \frac{H_s}{\Delta D_{n50}}\sqrt{\frac{g}{D_{n50}}}T_z \tag{2.4}$$

where T_z (s) is the mean zero up-crossing period.

Those stability parameters, H_0 and H_0T_0, should not be confused with the parameters used for deep water wave height and deep water wave period. In berm breakwater design, those parameter are widely used and generally accepted in the engineering world.

Lamberti et al (1995), Lamberti and Tomasicchio (1997) and Archetti and Lamberti (1999) performed extensive researches to obtain detailed information on the mobility of armour stones on berm breakwaters. The researches were based on conditions in the range of $1.5 < H_0 < 4.5$. The main conclusions are summarized as follows:

- the stones on a berm breakwater start to move when $H_0 = \sim 1.5 – 2$;
- the mobility is low when $2 < H_0 < 3$;
- when $H_0 > 3$ the mobility increases very rapidly;
- a berm breakwater will reshape into a statically stable profile if $H_0 \leq \sim 2.7$; and
- if $H_0 > \sim 2.7$ the berm breakwater will reshape into a dynamically stable profile.

In summation, Table 2.1 shows stability criterion for the three categories of berm breakwaters for modest wave attack β = ±20°.

Table 2.1 Stability criterion for modest angle of wave attack β = ±20°

	H_0	H_0T_0
Little movement - Statically stable non-reshaped	<1.5 - 2.0	<20 - 40
Limited movement during reshaping - Statically stable reshaped	2.0 - 2.7	40 - 70
Relevant movement, reshaping - Dynamically stable reshaped	> 2.7	> 70

2.1.2 Waves on slope

The wave slope is expressed in the following equation;

$$s_0 = \frac{H}{L_0} = \frac{2\pi H}{gT^2} \tag{2.5}$$

where, H [m] is the wave height, L_0 [m] is the deep water wave length, T [s] is the wave period and g [m/s^2] the acceleration of gravity. This formula represents a state in deep water, where H is the height of a single wave. To make this parameter useful for design purpose the incident wave is usually given as significant wave height. This parameter can be given as a time domain analysis $H_{1/3}$ or as a spectral analysis H_{m0}, in this report it will from this point on be referred to as H_s. For the wave period T, the zero up crossing period, T_z, and wave peak period, T_p, are used and the slope is referred to as s_{0z} and s_{0p}, respectively.

To describe a wave action on a slope the dimensionless Iribarren number ξ (-), also known as the surf similarity parameter, is important. The parameter is defined as:

$$\xi = \frac{\tan\alpha}{\sqrt{s_0}} \tag{2.6}$$

where α is the angle of the slope of the structure while s_0 is a representative for the wave slope. The formula used to calculate the wave length is valid for deep water conditions, L_0, the input in that formula in this case is however the local wave period, T_z or T_p. When s_{0z} and s_{op} are used the surf similarity parameter becomes ξ_z and ξ_p respectively.

Battjes (1974) describes the different shapes of waves breaking, depending on the surf similarity parameter. The transition between the breaker types is gradual and the values of the transition between them are just an indication, Figure 2.1.

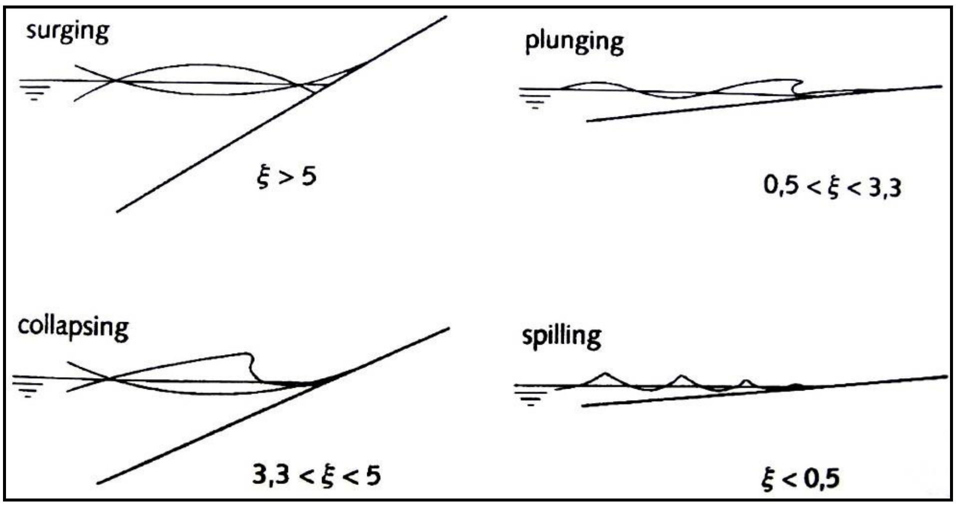


Figure 2.1 Breaker types as a function of the surf similarity parameter

2.2 Reshaping measurements

2.2.1 Recession

Recession is an important parameter when considering berm breakwaters, see Figure 2.2 for explanation. Although the design of Icelandic type berm breakwaters does not really depend on formulas for berm recession, it is an important tool in estimating the damage of the structure.

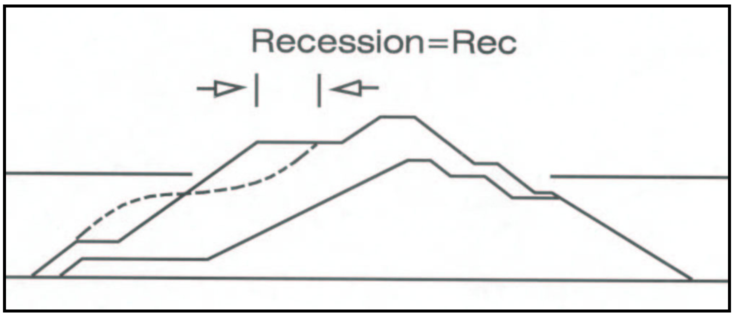


Figure 2.2 Recession of berm breakwaters

Tørum (1998) presented a formula for recession, Rec (m), of the berm as a function of rock diameter Rec/D_{n50} (-) and hydraulic boundary conditions, H_0T_0 (-).

$$Rec/D_{n50} = 0.0000027(H_0T_0)^3 + 0.000009(H_0T_0)^2 + 0.11\ H_0T_0 - 0.8 \qquad (2.7)$$

This formula has since been modified by Menze (2000), adding stone gradation and water depth as factors in recession. The latter formula is to large extent based on test results from multilayer berm breakwaters.

$$Rec/D_{n50} = 0.0000027(H_0T_0)^3 + 0.000009(H_0T_0)^2 + 0.11\ H_0T_0 - (-9.9f_g^2 + 23.9f_g - 10.5) - f_d \qquad (2.8)$$

with

$$f_g = D_{n85}/D_{n15} \text{ and}$$

$$f_d = -0.16\ d/D_{n50} + 4.0$$

where f_g (-) represents a stone gradation factor, D_{n85}, D_{n15} and D_{n50} are the nominal diameters for 85% , 15% and 50% respectively.

The gradation factor is the parameter that takes into account the narrow gradation of Icelandic type berm breakwaters compared to a homogeneous one. It does not, however, include parameters taking into account the effect of the smaller stones, if the Class I stones do not reach far down the slope, and are not the cause of damage. That situation is, on the other hand, quite complicated. Another parameter that is not included is the berm height, h_B (see Figure 2.5). Other parameters such as the shape of stones and different placement methods are among the parameters that influence the large scatter in recession formulas.

Sigurdarson et al (2007) came up with a more simple formula wherein it is assumed that the influence of stone grading and water depth on berm recession is rather small, especially given the large overall scatter.

$$Rec/D_{n50} = 0.037(H_0T_0 - Sc)^{1.34} \tag{2.9}$$

with: $Rec/D_{n50} = 0$ for $H_0T_0 < Sc$

with: $\mu(Sc) = 20$ and $\sigma(Sc) = 20$,.

where, Sc, is the scatter in recession measurements.

2.2.2 Damage definition, S_d

The damage number S_d describes the damage using the area of erosion in the cross sections as the basis. This damage number is a simple dimensionless parameter wherein only the eroded area and the nominal diameter of the armour layer are included. Despite being simple it is a very useful tool in damage estimation and, due to its simplicity, it can be applied to almost any type of structure. Figure 2.3 further explains the formula for the stability number, equation 2.10.

$$S_d = \frac{A_e}{D_{n50}^2} \tag{2.10}$$

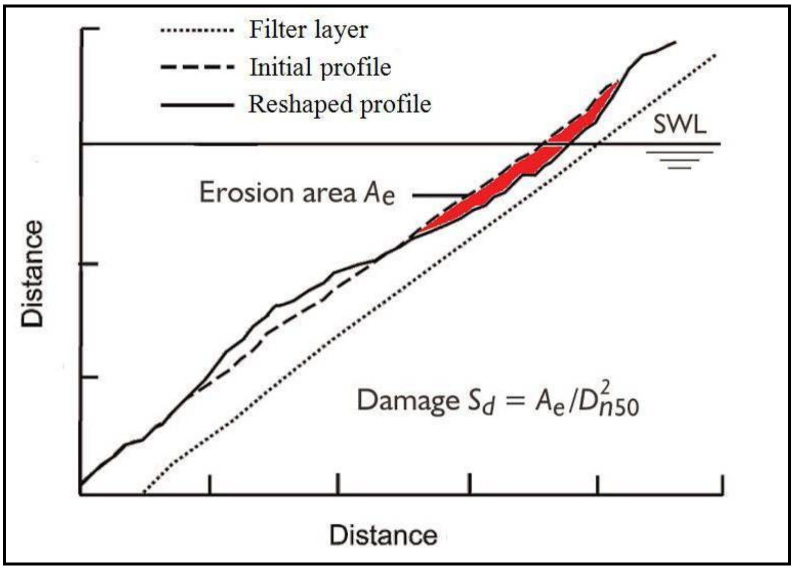


Figure 2.3 Explanation figure for the damage number S_d

Although this damage number was designed for a uniform stable structure where very little reshaping is accepted, it can also be used as a tool to compare the damage between individual tests in this research.

2.3 Design methods for uniform slope

Many research projects have been completed on the subject of stability of stones on uniform slope, starting with a research done by Iribarren in 1938. Hudson (1953 and 1959) did a great deal of research on this subject, resulting in a formula that is still widely used today. Between 1965 and 1970 the first wave generators that could generate irregular waves according to a predefined wave spectrum were introduced. After that, it became possible to replicate a real sea state more accurately than before. Many researchers performed a number of experiments in the following years without coming up with a formula that was accepted over the formula of Hudson. In his PhD project at TU Delft in 1988, however, Van der Meer introduced a formula that has been generally accepted in the engineering world. In the following sections, the methods of Hudson and Van der Meer are briefly explained.

2.3.1 Hudson

Hudson (1953, 1959) introduced the following formula based on researches based on model tests with regular waves on non-overtopped stone structure, with permeable core,

$$W_{50} = \frac{\rho_s g H^3}{K_D \Delta^3 \cot\alpha} \tag{2.11}$$

where K_D is the stability coefficient and represents many different influences. Among those influences is the shape of blocks, placement methods, type of wave attack, and wave period, among others.

The original Hudson formula can be rewritten as a function of the stability number H_0. Originally the formula was used with H = H_s, but later was revised to use H = $H_{1/10}$, since $H_{1/10}$ = 1.27H_s the formula written as a function of the stability parameter becomes

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D \cot\alpha)^{1/3}}{1.27} \tag{2.12}$$

To include the damage level parameter, S_d, in equation 2.12, Van der Meer (1998) proposed the use of equation 2.13 as the function of the stability number $H_{0,}$

$$\frac{H_s}{\Delta D_{n50}} = 0.7(K_D \cot\alpha)^{1/3} S_d^{0.15} \tag{2.13}$$

2.3.2 Van der Meer

Van der Meer conducted tests with relatively deep water at the toe of the structure. A large amount of tests were performed with irregular waves on stone structures with uniform slope. Different levels of permeability were used, resulting in the inclusion of permeability in the formula. This research resulted in two formulas where one representing plunging breakers and the other representing surging breakers. The transition between the two is also explained in a formula.

For plunging breakers ($\xi_m < \xi_{transition}$):

$$\frac{H_s}{\Delta D_{n50}} = 6.2 P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{2.14}$$

For surging breakers ($\xi_m \geq \xi_{transition}$):

$$\frac{H_s}{\Delta D_{n50}} = 1.0P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^P \sqrt{\cot\alpha} \tag{2.15}$$

Where $\xi_{transition}$ represents the transition between the two :

$$\xi_{transition} = \left[6.2P^{0.31}\sqrt{\tan\alpha}\right]^{\left(\frac{1}{P+0.5}\right)} \tag{2.16}$$

Where S (-) is the damage number, explained in chapter 2.2.2, P (-) is a permeability parameter explained in Figure 2.4, while ξ_m is the surf similarity parameter explained in section 2.1.2, where T_m is used.

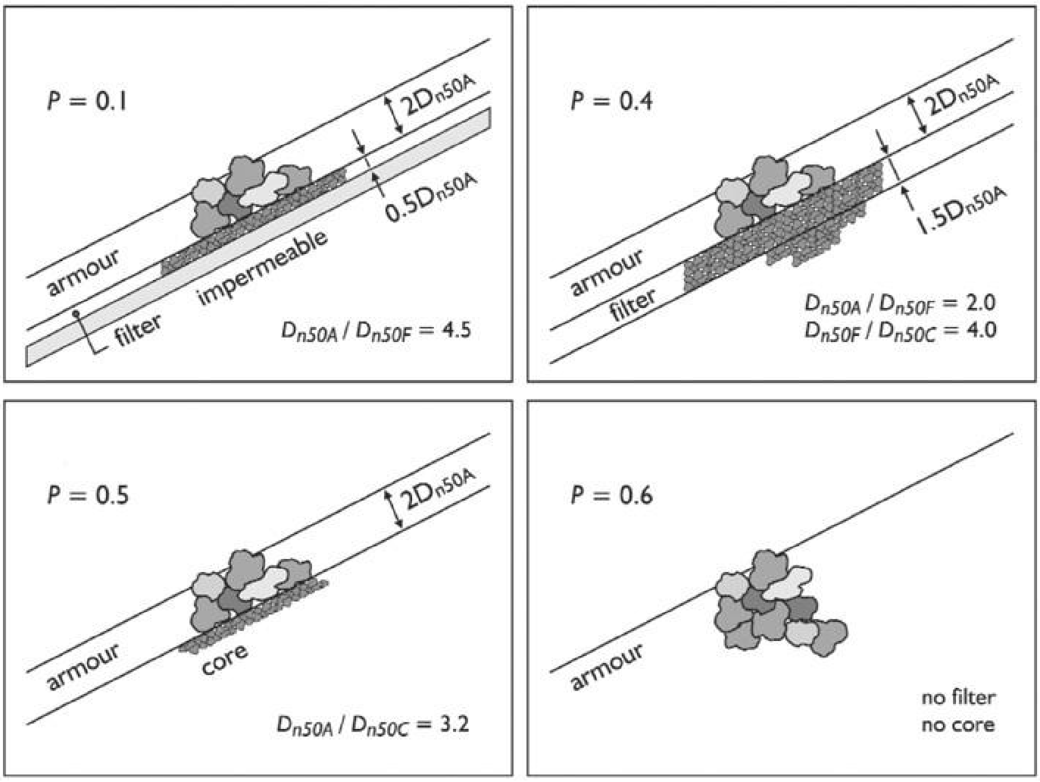


Figure 2.4 Permeability coefficients, P, for various structures for the formula of Van der Meer (1988)

There are a few advantage of the Van der Meer formula over the formula represented by Hudson. The research of Van der Meer was performed with irregular waves while the Hudson formula is derived from regular waves. The Van der Meer formula does also includes the duration of storm, the wave period, the permeability of structure, and has a clearly defined damage level.

2.4 Design methods for Icelandic type berm breakwaters

Strict rules for the main design parameters of Icelandic type berm breakwaters have not evolved. There are, however, design guidelines that have been developed and are constantly being reviewed. Those guidelines, as represented in Sigurdarson, et al (2007), are the following.

Design rules:

- The upper layer of the berm consists of two layers of rock and extents on the down slope at least to mean sea level;
- The rock size of this layer is determined by $H_0 = 2.0$. Larger rock may be used too;
- Slopes below and above the berm are 1:1.5;
- The berm width is 2.5 - 3.0 H_S;
- The berm level is 0.65 H_S above design water level;
- The crest height is given by $R_C/H_S{*}s_{op}^{1/3} = 0.35$;

According to the Rock Manual (2007), the Class I stones should preferably reach down to the point where the reshaped profile crosses the original profile. Where the vertical distance from the water level down to the transition of the two profiles is referred to as h_f, see Figure 2.5. This indicates that putting the transition between Class I and Class II stones higher would decrease the stability of the structure.

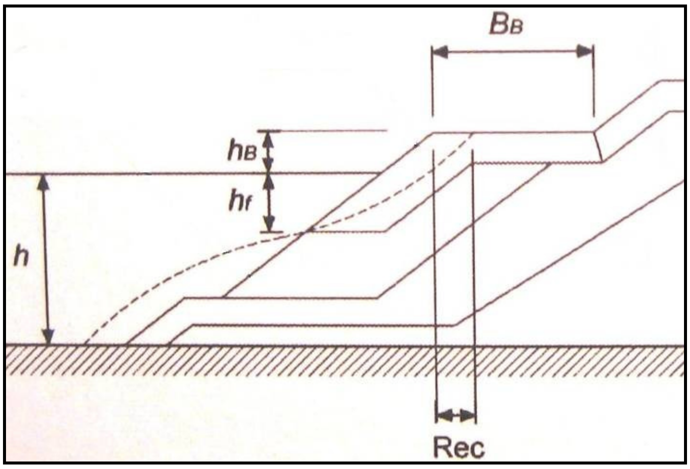


Figure 2.5 Key parameters in berm breakwater design

The point of transition of the original and the reshaped profile, h_f, is according Tørum, et al (2003), defined as

$$\frac{h_f}{D_{n50}} = 0.2\frac{h}{D_{n50}} + 0.5 \qquad (2.17)$$

within the range

$$12.5 < \frac{h}{D_{n50}} < 25$$

3 Experimental setup

3.1 The Wave Flume

The scale model tests on the Icelandic type berm breakwaters were executed in a wave flume in the Fluid Mechanics Laboratory of the Faculty of Civil Engineering at Delft University of Technology. The flume was 42m long, 80cm wide and had a maximum height of 90cm, the flume has a flat bottom profile. During the experiment the flume was divided into two parts, the length of the part used in these model tests was 25m, see overview photo of the flume, Figure 3.1.

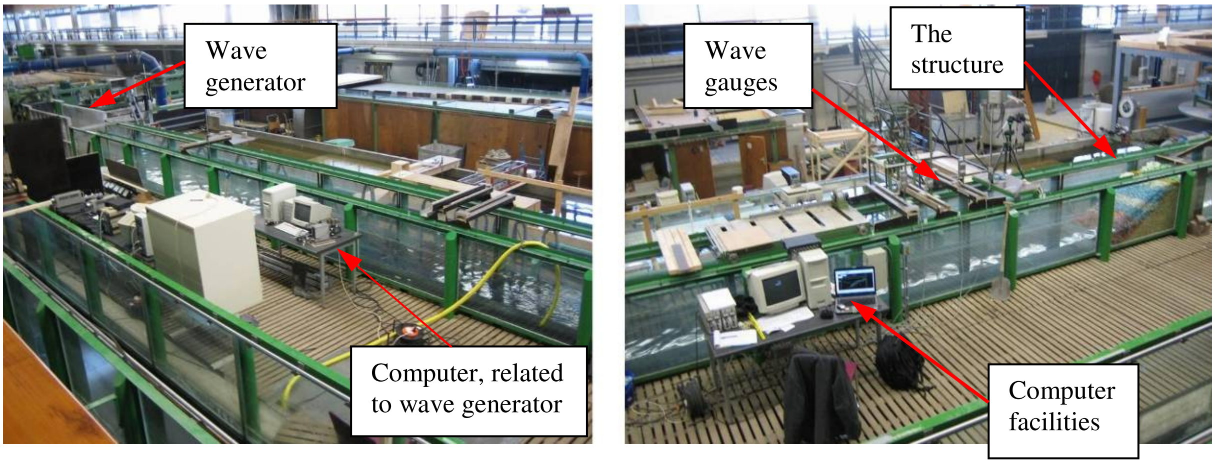


Figure 3.1 Overview of the wave flume

On one end of the flume there was a wave generator which can generate regular or irregular waves. The wave generator was equipped with an active reflection compensation system to minimize wave reflection back from the wave board. The motion of the wave board compensates with the reflected waves, preventing them from reflecting off the wave board and back toward the model and thereby affecting the measurements.

3.2 Observation equipments

The observation equipments used in the experiments included, three wave gauges, a camera, a video camera and profile measurement equipment. An overview of the setup of those equipments is shown in Figure 3.2.

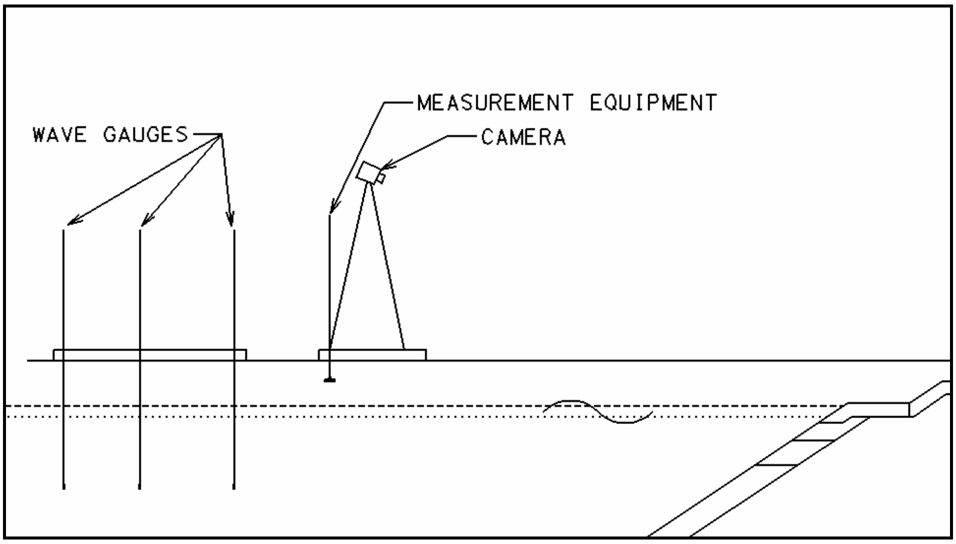


Figure 3.2 Overview of the observation equipments in the wave flume

The wave data was recorded with three wave gauges that were located in front of the breakwater, 2m from the toe. The distance between the gauges was 40.6cm and 37.2cm. These gauges gathered data from which the wave properties could be calculated from. The measured waves represented the situation at the toe of the structure, excluding reflected waves.

A camera was used to capture photos at different stages of the tests. Photos were taken after 500, 1000, 2000 and 3000 waves in each test in order to follow damage development. The camera was located 140cm from the toe of the structure and 178cm above the bottom of the flume. The camera was located on top of a movable plate but was however always located at the same place when photos were taken.

A video camera was also used, to follow the motion of the stones during wave attack. The video camera was not located at a fixed point but was generally placed in such a way that it captured the profile of the structure. A few minutes of each test was recorded with the video camera. A view of the capture from the video camera can be seen in Figure 3.3.

Figure 3.3 A view from the video camera, setup S03TM, H_s = 0.12m

To measure the profile, measurement equipment was used that consisted of a narrow stick with a flexible plate at the end. The bottom area of the plate was approximately 2.5cm x 2.5cm. Figure 3.4 shows the equipment in use. The profiles were measured at the end of each wave series, which was after 3000 waves.

Figure 3.4 Equipment used for profile measurements

3.3 Model setup

The first breakwater model setup tested was similar to the Sirevåg breakwater. The main difference, however, was that the water depth in the model was relatively deeper. Then depending on the damage development and failure of earlier tests, the setup of the structure was changed and a total of six setups were tested. The first five setups are explained in Figure 3.5, Table 3.1, and Table 3.2. For setup number six which was the last one, all the stones were mixed and, that setup is therefore not included in the explanation Figure/table. The cross sections for each test can be viewed in Figure 3.6 and also along with the reshaping development profiles in appendix A.4.

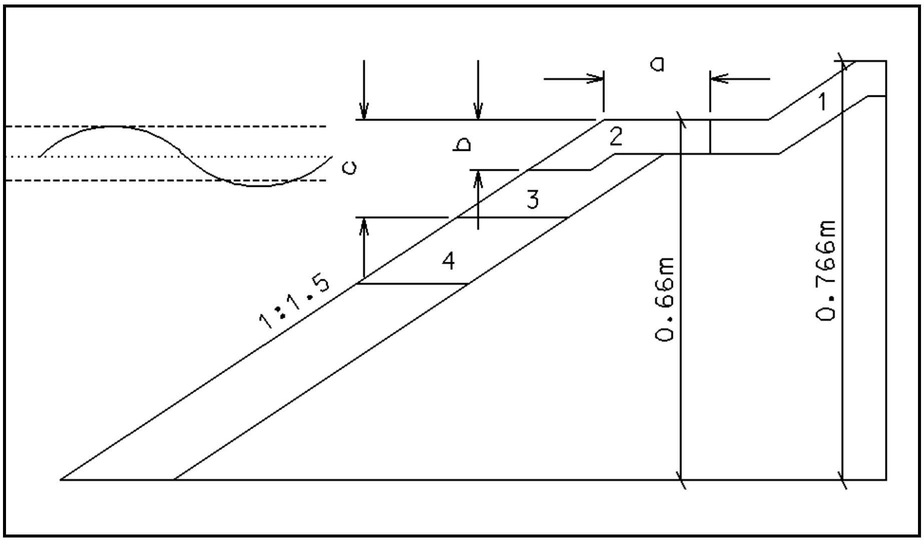


Figure 3.5 Explanation figure of the model setup

Table 3.1 Explanation table for test setups, parameters from Figure 3.4

				Class in area #			
Setup	a [m]	b [m]	c [m]	1	2	3	4
1	0.300	0.093	0.209	II	I	II	III
2	0.195	0.151	0.209	II	I	II	III
3	0.300	0.093	0.128	II	I	II	III
4	0.128	-	0.209	II	I	I	III
5	0.300	0.093	0.179	II	II	I	III

In Table 3.2 the setups are further explained. In this table, h_B is the vertical distance from the berm level to the water level, Figure 2.5, while b-h_B and c-h_B represent the vertical distance from the water level to the transitions between classes.

Table 3.2 Further explanation for test setups, parameters from Figure 3.5 and Figure 2.5

	Water level 0.645m				Water level 0.590m				Water level 0.545m			
Setup	h_B [m]	b-h_B [m]	c-h_B [m]	h [m]	h_B [m]	b-h_B [m]	c-h_B [m]	h [m]	h_B [m]	b-h_B [m]	c-h_B [m]	h [m]
1	0.015	0.078	0.194	0.645	0.07	0.023	0.139	0.590	0.115	- 0.022	0.094	0.545
2	0.015	0.136	0.194	0.645	0.07	0.081	0.139	0.590				
3	0.015	0.078	0.113	0.645	0.07	0.023	0.058	0.590				
4	0.015	-	0.194	0.645	0.07	-	0.139	0.590				
5	0.015	0.078	0.164	0.645	0.07	0.023	0.109	0.590				

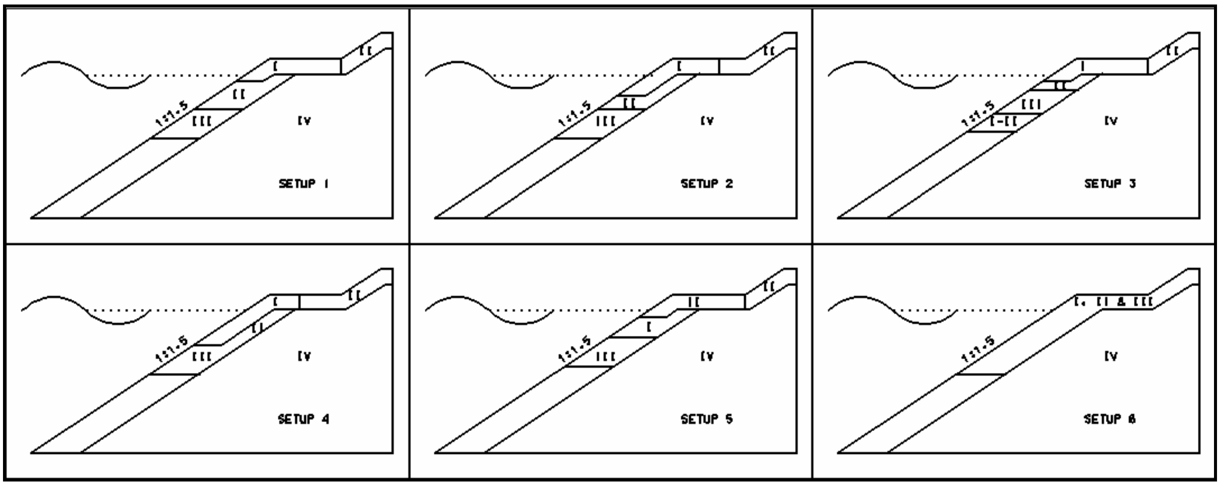


Figure 3.6 Overview of the different setups

For every setup, the geometry of the structure was the same. Specifically the berm width, B_B (m), the berm height, the crest height and the slopes α, were not changed between tests. The parameters that changed between setups were the water level and the location of different stone classes. For most setups, two water levels, 0.645m and 0.590m, were tested. A third level, 0.545m, was tested for the first setup only. Other relevant parameters were:

B_B = 0.30m (berm width)

α = 1:1.5 for all slopes

In appendix A.1 those dimensions are given as a function of the different wave heights, H_s, as well as a function of the median nominal diameters, D_{n50}, of the different stone classes.

A total of thirteen setups were tested. For each setup the number of tests was based on the moment of failure. This resulted in four to six tests for each setup or a total of 62 tests in the wave flume. In addition to those tests, a total of eight tests for two setups were repeated. Those tests were used to estimate the uncertainties of the tests and increased the total number of tests to 70.

3.3.1 Wave conditions

For each setup, tests were carried out for up to six different wave conditions, see Table 3.3. These tests started with the lowest waves (test 1) and then continued with higher waves until there was a complete failure of the structure. A complete failure was defined as the start of core erosion. The wave steps were based on the stone sizes as such that the stability number, H_0, was between 1.5 and 3.0 for Class I stones. The chosen wave conditions can be viewed in Table 3.3.

Table 3.3 Wave series planned for the tests

	Test #	H_S [m]	s_{op}	Tp [s]	W-S	D_{n50}	H_0	H_0T_{0p}
	1	0.085	0.04	1.17	Jonswap			
Class I						0.032	**1.50**	31
Class II						0.025	1.92	44
Class III						0.020	2.45	64
	2	0.100	0.04	1.27	Jonswap			
Class I						0.032	**1.76**	39
Class II						0.025	2.26	57
Class III						0.020	2.88	82
	3	0.120	0.04	1.39	Jonswap			
Class I						0.032	**2.12**	51
Class II						0.025	2.71	74
Class III						0.020	3.46	107
	4	0.135	0.04	1.47	Jonswap			
Class I						0.032	**2.38**	61
Class II						0.025	3.05	89
Class III						0.020	3.89	128
	5	0.155	0.04	1.58	Jonswap			
Class I						0.032	**2.73**	75
Class II						0.025	3.50	109
Class III						0.020	4.47	157
	6	0.170	0.04	1.65	Jonswap			
Class I						0.032	**2.99**	86
Class II						0.025	3.84	126
Class III						0.020	4.90	181

For all tests the target wave steepness was chosen to be fixed at $s_{0p} = 0.04$.

Measured wave conditions during the model tests were not exactly the same as the target values in Table 3.3 but were generally closed to those values. The measured values can be viewed in Appendix A.2.

3.3.2 Wave spectra

In order to simulate the waves in front of the breakwater as realistically as possible, irregular waves were used for the execution of all tests. When choosing wave spectra for irregular waves the wave flume offers two possibilities apart from making new wave spectra. These possibilities are the Pierson-Moskowitz spectrum (1964) and the Jonswap spectrum (1973).

The Pierson-Moskowitz spectrum represents a fully developed sea and was developed by offshore industry. It assumes deep water conditions with unlimited fetch and was developed using North-Atlantic data.

The Jonswap spectrum on the other hand represents a sea at young state and was also developed by offshore industry. It represents conditions with limited fetch and was developed using North Sea data. Since the wave spectrum is hardly ever fully developed in nature, the Jonswap spectrum is often used. In this experiment the Jonswap spectrum was used with the peak enhancement factor, $\gamma = 3.3$.

3.3.3 Water depth

Three water depths were used in the experiments, 0.645m and 0.590m. For one test, a water depth of 0.545m was also tested. Waves can be classified as deep water waves, intermediate water waves and shallow water waves according to the following definitions:

$$\frac{h}{L} > \frac{1}{2} \Rightarrow \text{deep water waves}$$

$$\frac{1}{20} < \frac{h}{L} < \frac{1}{2} \Rightarrow \text{intermediate water waves}$$

$$\frac{h}{L} < \frac{1}{20} \Rightarrow \text{shallow water waves}$$

Table 3.4 h/L ratio for all water levels used in the experiments

H_s [m]	T_p [s]	h [m]	L[m]	h/L[-]	h [m]	L[m]	h/L[-]	h [m]	L[m]	h/L[-]
0.085	**1.17**	**0.645**	2.05	0.31	**0.590**	2.03	0.29	**0.545**	2.00	0.27
0.100	**1.27**	**0.645**	2.36	0.27	**0.590**	2.32	0.25	**0.545**	2.28	0.24
0.120	**1.39**	**0.645**	2.73	0.24	**0.590**	2.67	0.22	**0.545**	2.62	0.21
0.135	**1.47**	**0.645**	2.98	0.22	**0.590**	2.91	0.20	**0.545**	2.85	0.19
0.155	**1.58**	**0.645**	3.30	0.20	**0.590**	3.21	0.18	**0.545**	3.13	0.17
0.170	**1.65**	**0.645**	3.52	0.18	**0.590**	3.42	0.17	**0.545**	3.33	0.16

For all the test series the combination of waves and water depth represents waves in intermediate water depth.

3.4 Material

3.4.1 Shape of stones

Small stones, like the ones used for model tests, have a tendency to have a different shape than the big rocks used for the armour layer of a breakwater. Therefore, the stones for the model test have to be carefully chosen. Shape is particularly important for the Class I stones, that are carefully placed, since it can significantly affect the armour layer stability. There are a few useful measures concerning the shape of stones, including the length-to-thickness ratio (LT), blockiness (BLc), roundness and angularity. In this study, the length-to thickness ratio was measured and used as a criterion for armour stone selection of the model. The other aspects were kept in mind when choosing the stones but not applied directly. This is especially important when choosing the Class I stones since interlocking between them is an important factor in the design of the breakwater and is very dependent on the shape of the stones.

Length-to-thickness ratio (LT)

Length-to thickness ratio, LT (-), is defined as the maximum length, l (m), divided by the minimum distance, d (m), between parallel lines through which the particle would just pass, as is explained in Figure 3.7.

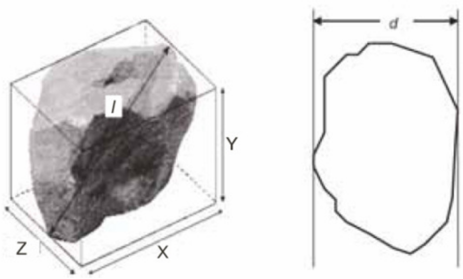


Figure 3.7 Illustration of the measurements of the LT ratio

For heavy armour stones in cover layers, the number of stones with LT ratio greater than 3:1 is recommended to be lower than 5%. While for light armour stones in cover layer, the same ratio is recommended to be lower than 20%.
[CUR/CIRIA (2007)]

A sample of stones from each class was chosen randomly and the LT ratio measured, the results are the following:

Class I stones with **LT ratio > 2:1 = 30%**
Class I stones with **LT ratio > 3:1 = 0%**
The average LT ratio of Class I stones is: 1:1.9

Class II stones with **LT ratio > 2:1 = 60%**
Class II stones with **LT ratio > 3:1 = 3%**
The average LT ratio of Class II stones is: 1:2.1

Class III stones with **LT ratio > 2:1 = 50%**
Class III stones with **LT ratio > 3:1 = 0%**
The average LT ratio of Class III stones is: 1:2.2

For all stone classes the samples meet the recommendations for heavy armour stones. The stones are therefore considered a good representative for armour rocks in a real project.

3.4.2 Characteristic of stones

The density of the armour stones (Class I, Class II and Class III) is ρ_s = 2770 [kg/m^3] while the density of the core material is ρ_s = 2620 [kg/m^3]. The nominal diameter, D_{nXX} for different percentages, XX, is widely used to estimate various aspects, including stability and filter requirements. The XX is replaced by a percentage representing a diameter of a theoretical block for which XX% of the mass of the sample is lighter.

One of the biggest different between an Icelandic type berm breakwater and a homogeneous berm breakwater or a conventional breakwater, concerning stones is the gradation of the stones. Since in the Icelandic type the gradation in each stone class is narrower due to the sorting of stones by size into different classes. The gradation is grouped depending on the D_{n85}/D_{n15} ratio:

Narrow gradation if	$\mathbf{D_{n85}/D_{n15} > 1.5}$
Wide gradation if	$\mathbf{1.5 < D_{n85}/D_{n15} < 2.5}$
Very wide gradation or quarry run if	$\mathbf{D_{n85}/D_{n15} > 2.5\text{-}5.0+}$

Measurements of the D_{n85}/D_{n15} ratio for the three stone classes as well as for the core material fell under the category of ‘Narrow gradation’. For comparison, toward the end of the experiment all the stones were mixed together. That resulted in homogenous armour layer with wide gradation. The sieve curve of this experiment becomes close to represent a sieve curve of the situation before the stones are sorted. However, since all the cross sections did not have exactly the same amount of stones from each class and the back side of the breakwater was also not considered, this mixed sieve curve is more helpful as a comparison of different setups and is only an approximation for a possible quarry result. Having one sample with wide gradation eases comparison with earlier research, for example recession.

The sieve curves and additional basic information about the material can be found in appendix A.3.

3.4.3 Law of scaling

Before a physical model test can be performed, the prototype must be scaled down to a size which can be handled by the available test facility. Although the purpose of this research project is not to look at a specific prototype, the breakwater in Sirevåg, Norway, was used as a semi-prototype when deciding on stone diameter and other relevant parameters. One of the problems with scaling is that certain parameters cannot be or are difficult to scale. Those parameters include gravity and the characteristics of fluid. Scaling of the hypothetical prototype is done by dividing the physical parameters or quantities of the prototype by a scale factor:

$$N_x = \frac{x_p}{x_m} \tag{3.1}$$

where N_x is the scale factor of the physical parameter or quantity x, subscript p stands for prototype and subscript m stands for model

- **The Froude number** is a parameter that expresses the relative influence of inertial and gravity forces in a hydraulic flow. It is given as the square root of the ratio between inertial and gravity forces:

$$Fr = \sqrt{\frac{inertial\_force}{gravety\_force}} = \sqrt{\frac{\rho L^2 U^2}{\rho L^3 g}} = \frac{U}{\sqrt{gL}} \tag{3.2}$$

The Froude number is required to be the same in the model as in the prototype

$$\left(\frac{U}{\sqrt{gL}}\right)_p = \left(\frac{U}{\sqrt{gL}}\right)_m \tag{3.3}$$

Expressed in scale ratios

$$\frac{N_U}{\sqrt{N_g N_L}} = 1 \text{ or } N_{Fr} = 1 \tag{3.4}$$

Where

U - Velocity

L – Length

t - Time

g – Gravitational acceleration

Substituting $N_U = \frac{N_L}{N_t}$ into last equation gives the Froude time scale

$$N_t = \sqrt{\frac{N_L}{N_g}} \tag{3.5}$$

For practical purpose gravity is not scaled, thus $N_g = 1$. The Froude time scale is thereby simplified to

$$N_t = \sqrt{N_L} \tag{3.6}$$

The majority of hydraulic models in coastal engineering are scaled according to the Froude criterion. Consequently, this is usually the most important criterion to be studied when designing a coastal scale model. Since in this experiment the most important parameter is the wave attack, the Froude criterion will be used.

[Hughes, 1995]

4 Analysis

For analysis of this research, the profile measurements from the tests in the flume were used along with photos and videos recorded during the tests. Visual observations during the tests are also influential in the analysis. In appendix A.4, the profile developments of the tests can be followed.

4.1 Damage development

Test results showed that the start of damage and the damage development changes for different test setups. In this section the damage development and the failure mechanisms for each test are briefly explained.

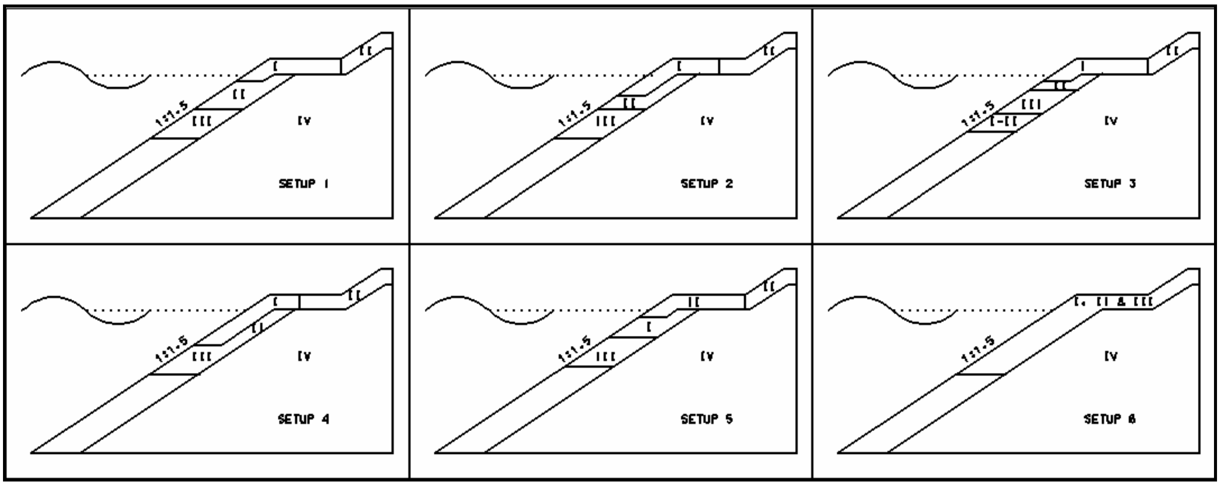


Figure 4.1 Overview of the different setups

4.1.1 Setup 1

Water level 0.545m

Setup 1 was the only setup where water level 0.545 was used. The water level was located below the transition of Class I and Class II. The water level was also closer to the transition of Class II and Class III than in the other setups, apart from setup 3.

Start of damage: The damage starts clearly in the part where the Class II stones are located, where stones start moving with relatively small wave height. The reshaping that had already taken place after the first wave set (H_s = 0.085m) is not acceptable for a stable structure.

Failure mechanism: The cause of the early failure of this setup is the low location of the water level. The low location both minimized the effects of the berm in energy dissipation, and caused the smaller stones to be located closer to the water level and therefore under greater wave attack. Although the water level is lower than in the other tests, the Class III stones were located deep enough not to be a part of the failure process.

Water level 0.590m

Start of damage: The structure is stable and without significant reshaping for the first two wave sets. Lack of stability and sliding in Class II stones causes the start of severe movement of Class I stones during the third wave set.

Failure mechanism: The relatively close position of Class II stones to the water level influences the failure process from the third wave set, until there was a complete failure of the structure.

Water level 0.645m

Class I and Class II stones started moving at similar time. The real reshaping started in Class I, then the Class II stones started to give in as well and accelerate the damage process. The lower parts were the Class II stones were located would therefore have been stronger if the Class I stones had reached further down.

Start of damage: The Class I stones roll down due to the wave attack on the edge of the berm. No influence on the Class II stones early on in the process. The behaviour is similar to when the structure had a homogenous armour layer.

Failure mechanism: The Class II stones possibly play some part in the failure process at latter stages.

4.1.2 Setup 2

Water level 0.590m

Start of damage: In Class I stones.

Failure mechanism: In this setup the failure mechanism is similar to if the structure had a homogeneous armour layer. Specifically, all the real damage takes place in the Class I stones.

Water level 0.645m

When the recession reached the transition of Class I and Class II stones on the berm, it increased. Apart from that it is similar to setup 1.

Start of damage: In Class I stones.

Failure mechanism: In this setup the failure mechanism is similar to if the structure had a homogeneous armour layer. Specifically, all the real damage takes place in the Class I stones.

4.1.3 Setup 3

Water level 0.590m:

In this setup, the Class III stones are located closer to the water level than for the other setups. The reason for this is to better understand the ideal location for the transition of Class II and Class III stones.

Start of damage: The damage clearly starts in the location of the Class III stones, where stones start moving with relatively small wave height. The reshaping that had already taken place after the first wave set ($H_s = 0.085$m) is not acceptable for a stable structure.

Failure mechanism: The cause of the early failure of this setup is the low location of the transition of Class II and Class III stones. The failure mechanism is the same as for setup 1 with water level 0.545m. The only difference in this case is that the Class III stones are the main cause of this behaviour.

Water level 0.645m: The failure process is similar to the setup 1, which indicates that the location of Class III stones is low enough not to affect the failure process.

Start of damage: In Class I stones.

Failure mechanism: It is possible that the Class II stones and even Class III stones play some part in the failure process at latter stages.

4.1.4 Setup 4

Water level 0.590m

Setup 4 shows exactly the same damage development as setup 2 for this water level.

Start of damage: In Class I stones.

Failure mechanism: In this setup the failure mechanism is similar to if the structure had a homogeneous armour layer. Specifically, all the real damage takes place in the Class I stones.

Water level 0.645m

When the recession reached the transition of Class I and Class II stones on the berm, it increased. Apart from that, this setup behaves similar to setup 1. For this setup this influence is clearer than for setup 2 with the same water level because the transition is closer to the edge of the berm.

Start of damage: In Class I stones.

Failure mechanism: In this setup the failure mechanism is similar to the structure had a homogeneous armour layer. Specifically, all the real damage takes place in the Class I stones.

4.1.5 Setup 5

Water level 0.590m

In this setup, the Class II stones are put as the top layer and the Class I stones below them. This is the exact opposite of setup 1 when concerning those two classes.

Start of damage: In Class II stones that are located on the berm. The recession on the edge of the berm early on in the process is more than in previous setups.

Failure mechanism: The failure process starts earlier then for setups 1, 2 and 4 but slows down because of the location of Class I stones lower on the berm. In the end, the duration is similar to those setups. The damage develops step by step throughout all the wave series.

Water level 0.645m

The recession of the berm starts earlier than in most of the other setups. However, the existence of the thick layer of Class I stones below slows that process down. Towards the end the recession is becomes less than for setup 2 and 4, but slightly more than in setup 1. This shows that having the Class I stones in this position, influences the recession development in a positive way, at the latter stages.

Start of damage: In Class II stones that are located on the berm. The recession on the edge of the berm at the early stage of the process is more than before.

Failure mechanism: Damage starts from the edge of the berm as expected with the Class II stones located there. The damage develops step by step throughout all the wave series.

4.1.6 Setup 6

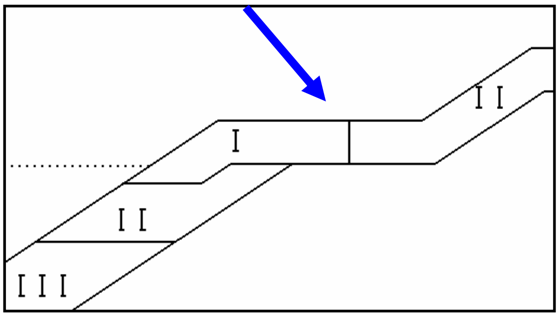
The armour layer in setup 6 is homogeneous, where the failure mechanism is as expected for such a structure. The reason for this setup is, however, to compare the damage level with the other setups in order to assess the benefits of sorting stones into classes.

4.1.7 Compilation

Berm height

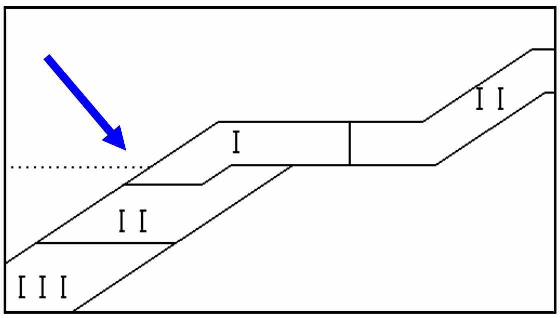
Comparing the water level 0.590m in setup 1 to the water level 0.645m in setup 2, where the depth down to the transition of Class I and Class II is similar, it can be concluded that the Class II stones are more affected when the water level is lower on the berm. This shows that, for the different water levels 0.590m and 0.645m there is a basic difference. There is more energy dissipated when the water level is closer to the berm. Therefore, the effects do not reach as deep when the water level is close to the berm. This can be seen in the damage figures where the transitions between Class I and Class II stones are at similar depth. While on the other hand, when the water level is high, there is more movement of stones up the berm.

On berm



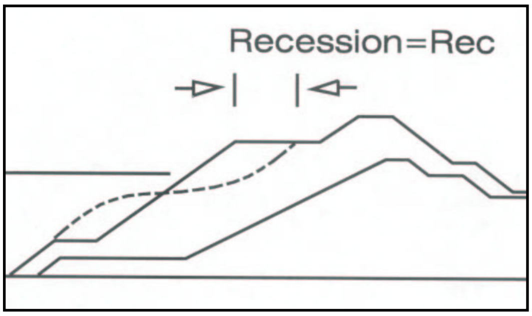
From setups 1, 2 and 4, the effect of the transition between Class I and Class II stones on the berm can be seen. As soon as the recession reaches this point of transition (in setup 2 and 4), the recession increases quickly almost to the end of the berm. While in setup 1, where there were Class I stones on the whole berm the recession stabilized earlier. However, when Class I and Class II stones were switched in setup 5, it did not have the same effect. The conclusion of that is that by putting a thicker layer of Class I stones lower on the berm, although the recession on the berm starts faster (due to the Class II stones on the berm), it slows down the recession development at the later stages.

On slope



The point of transition between Class I and Class II stones on the berm does not seem to affect what happens on the slope of the berm. The location of the transitions between classes on the slope is therefore the only parameter that is looked at concerning this. For each of those transitions there is an optimum level, to locate this optimum level both for transition of Class I and Class II stones and the transition of Class II and Class III is the primary goal of this research project.

4.2 Recession



Recession of the berm is an important part of the damage process. In this chapter, the test results will first be compared to existing recession formulas. Then, the test results for different setups will be analysed. In order to ease comparison of different tests, the stone parameters from the sample when the stones were mixed together was used. This provides a good ground for comparison since the mixed sample represents the same material before being sorted into classes. In appendix A.5, a variety of recession graphs are shown.

4.2.1 Comparison with recession formulas

In Figure 4.2 and Figure 4.3, the results of recession measurements are compared with two formulas, the modified formula of Tørum (2.8) and formula 2.9. For the test results points in the first graph (Figure 4.2), the stone parameters of the Class I stones are used when comparing setups 1-5 with existing formulas. For the second graph (Figure 4.3) the stone parameters of the mixed stone sample were used when comparing setup 6 to existing formulas.

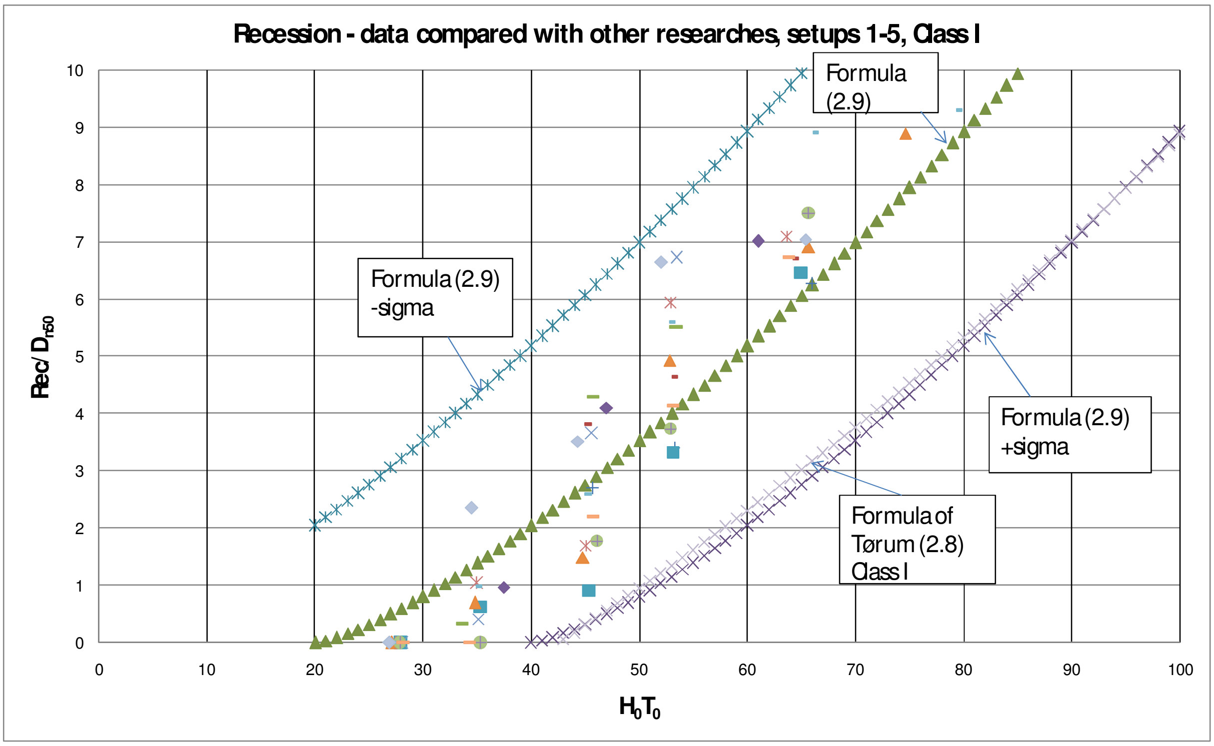


Figure 4.2 Recession, comparison of test results with recession formulas, setups 1-5, Class I stones used as reference.

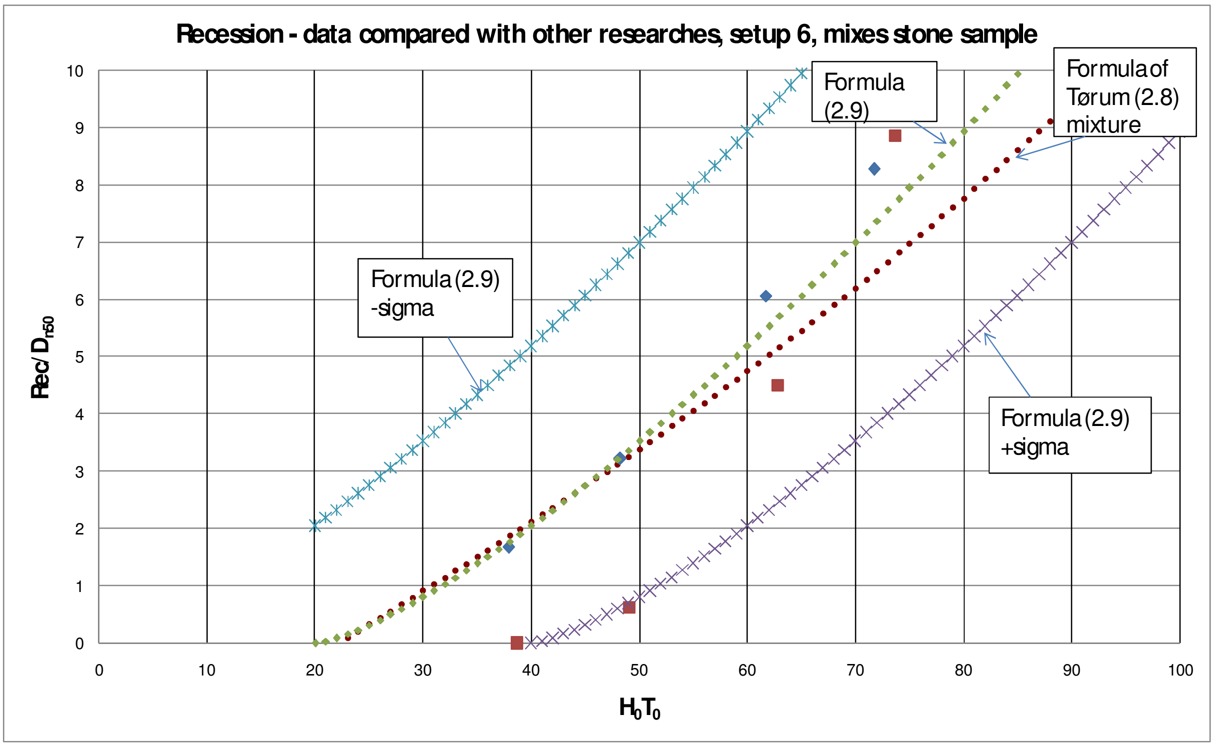


Figure 4.3 Recession, comparison of test results with recession formulas, setups 1-5, Class I stones used as reference.

As can be seen from Figure 4.3, the results for the mixed stones fit reasonably well with the formula of Tørum (2.8). However when looking at Figure 4.2, the comparison of setups 1-5 with the formula of Tørum, with stone parameters from Class I, the results do not fit well. The test results give more recession than the formula predicts, some of the following factors might be the cause of that.

- The narrow stone gradation; the recession might not be as sensitive to stone gradation as formula 2.8 indicates.
- The placement method; what placement methods were used when the formulas were derived, is unknown.
- The effect of the berm level; the berm level is not included in the formulas but has effects on the recession.
- The influence of the different location of the transition of stone classes; for some of the tests the Class I stones are not the only stones involved in the recession development.

Formula 2.9 however is not sensitive to the gradation of stones as compared to the formula of Tørum (2.8). For formula 2.9, the majority of the tests are within the limitation of one standard deviation from the mean values. The general test results show that the average recession from the tests is a little higher than the mean value. The reasons for this might be the same as some of the ones mentioned above.

The results from this research indicate that the influence of stone gradation is overrated in formula 2.8. The only setup that gave results that fit reasonably well to this formula was the setup with a homogenous berm. This indicates that the formula is possibly not valid for the narrow gradations used in other tests.

4.2.2 Influence of berm height on recession

For comparison figures of the recession with different berm height for each setup, appendix A.5.2 can be viewed. Figure 4.4 shows, however, recession for setup 4 with different water levels and thereby different berm heights. In the figure, S04TH stands for setup 4 with high water level (0.645m), while the others are for the medium water level of the three that were used in the research (0.590m).

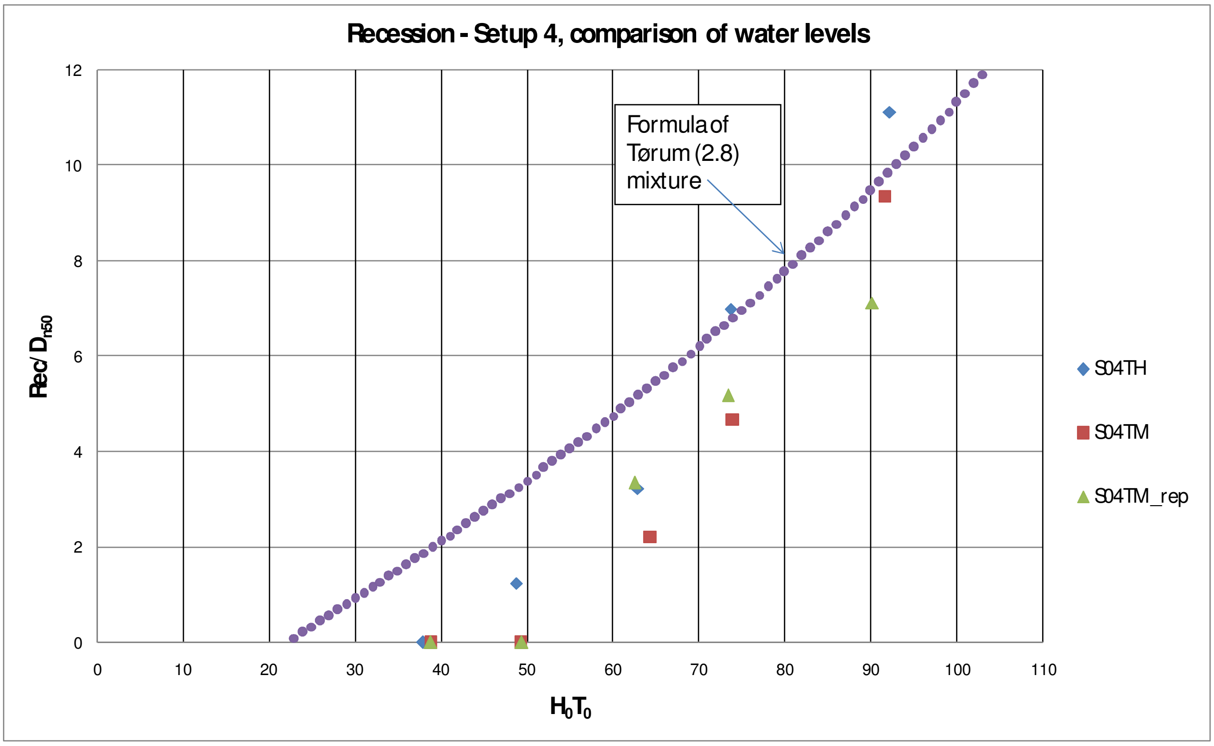
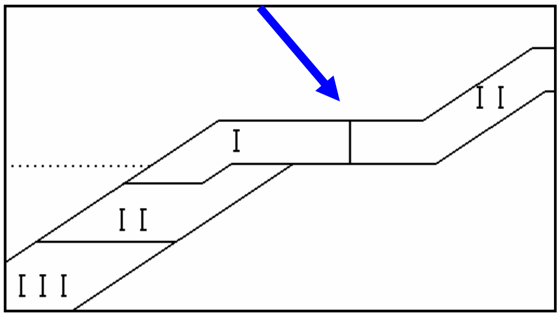


Figure 4.4 Recession, comparison of different water levels, setup 4

The general results show that the recession on the edge of the berm is usually more with a higher water level. That is as expected, since wave attack is more intense on the berm when the water level is closer to the berm. Setup 3 this turned out differently because the damage process started early, since the water level was lower and thereby the distance down to the Class III stones was shorter. Then, as mentioned before the failure mechanism is different and the leading cause of recession is not the attack on the edge of the berm but the attack on the smaller stones located further down the slope.

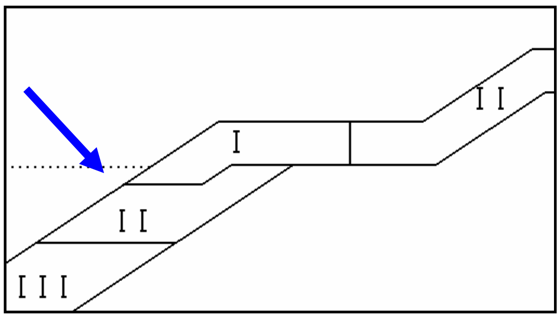
4.2.3 Influence of stones on berm, on berm recession



The effect of the stones that were placed on the berm and the location of the transition of the stone classes on the berm are briefly discussed here.

Both for setup 2 and setup 4 for high water level (0.645m) where the Class I stones do not reach all the way to the end of the berm, this research shows that when the recession reaches the transition of Class I and Class II stones the recession increases. This is especially notable for setup 4, where this transition is closer to the berm than in setup 2, therein setup 4, the recession increases almost to the end of the berm soon after it reaches this transition which is located $4D_{n50}$ from the edge of the berm. For setup 2, on the other hand, the recession does not reach this transition until the last wave series, where the total damage of the structure is already close to failure. It can be concluded that this transition should reach at least long enough into the berm, that the expected total recession does wound not reach that point, because then the runs the risk of rapid increasing of recession.

4.2.4 Influence of stones on the front slope, on berm recession



The stones on the berm are usually the ones that cause the start of recession, although in some cases it might be small. There are always a few stones on the slope that role down early in the process. In case of the weaker structures, that is structures where the transition between Class I and Class II stones is relatively close to the water level, erosion in the part with Class II stones causes a large part of the stones above to roll down. This causes a quick increase in the recession of the berm. This recession only occurs when the larger stones do not reach far enough down the berm. This behaviour is not attractive for such a structure because it causes too much damage relatively early on in the process, when the structure should still be stable.

4.2.5 Comparison of setups

In Figure 4.5 and Figure 4.6, the different recession development for the different setups, water levels 0.590 and 0.645 respectively, is compared. It should be noted that for the last values of each setup, where the H_0T_0 value is the highest, the test series was not completed for all the tests. The number of waves in the last measurements was therefore different for the last point and was never 3000 waves like for the other points. In chapter 4.4 the duration until the failure of each setup is explained.

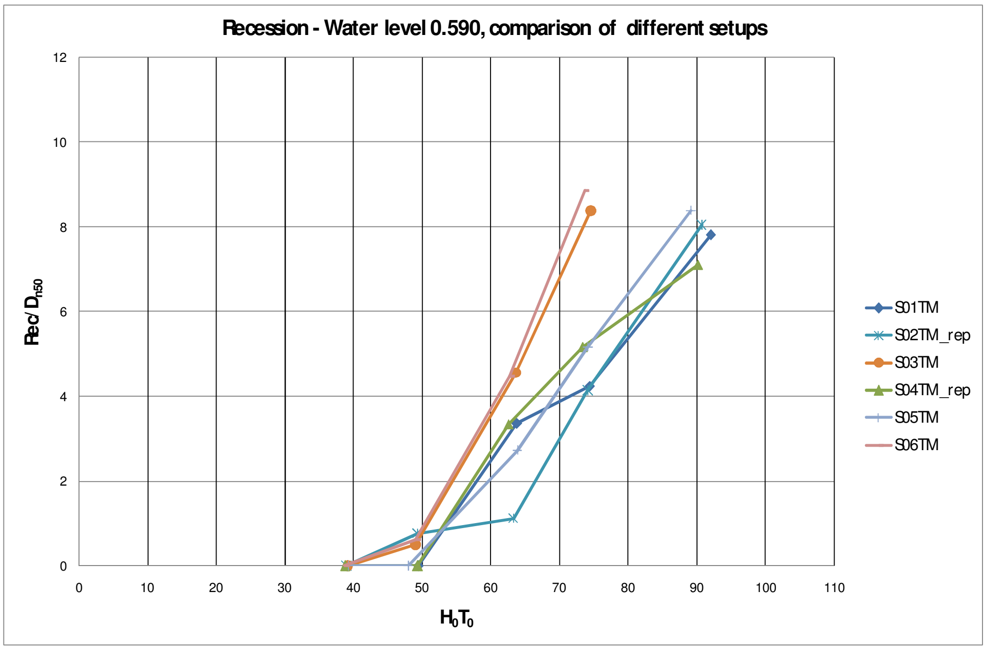


Figure 4.5 Recession, comparison of different setups, water level 0.590m

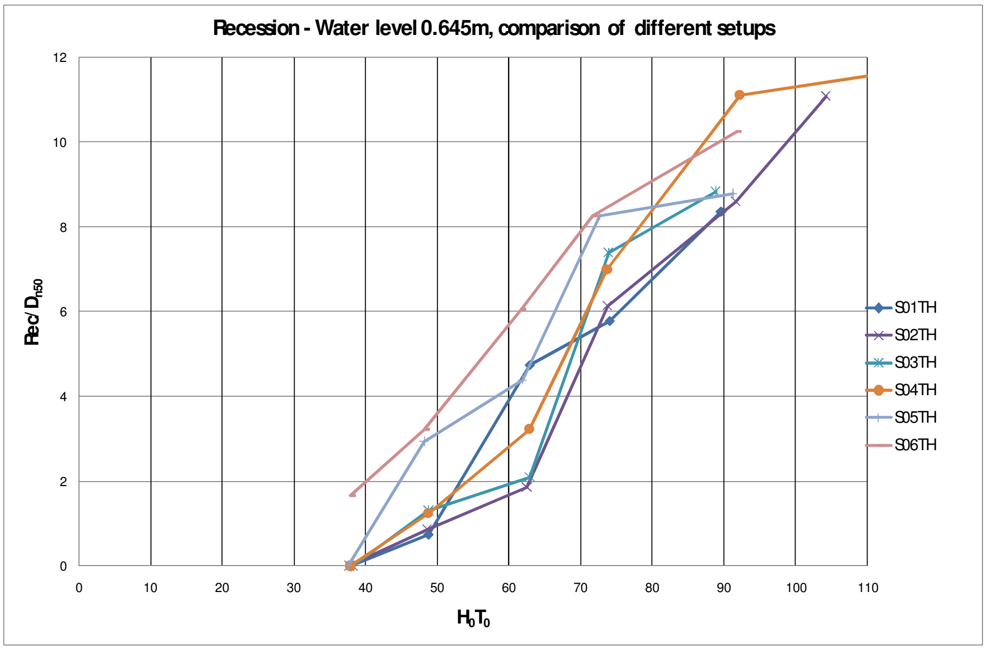


Figure 4.6 Recession, comparison of different setups, water level 0.645m

4.3 Damage number S_d

The damage number S_d is, as explained before (in chapter 2.2.2), a good tool to compare total damage of different setups. This measure is, however, not as useful when comparing with different structures such as conventional two layered breakwaters, where much less reshaping is accepted and the S_d value therefore lower.

For comparison, the damage number is calculated by using the nominal diameter, D_{n50}, from the sample of mixed stones. This is done for ease of comparison of different setups, while at the same time create a dimensionless parameter to work with that represents the damage development. In appendix A.6, a variety of damage development graphs are shown.

4.3.1 Influence of berm height on the damage number

In appendix A.6.1, the graphs from all setups, where the different effect of berm height on the total damage/erosion is explained. From the results of setup 2, Figure 4.7, it can be seen that the total amount of damage/erosion is more as the water level becomes lower. This turned out to be the case for all setups, within the limitations of the three water levels tested.

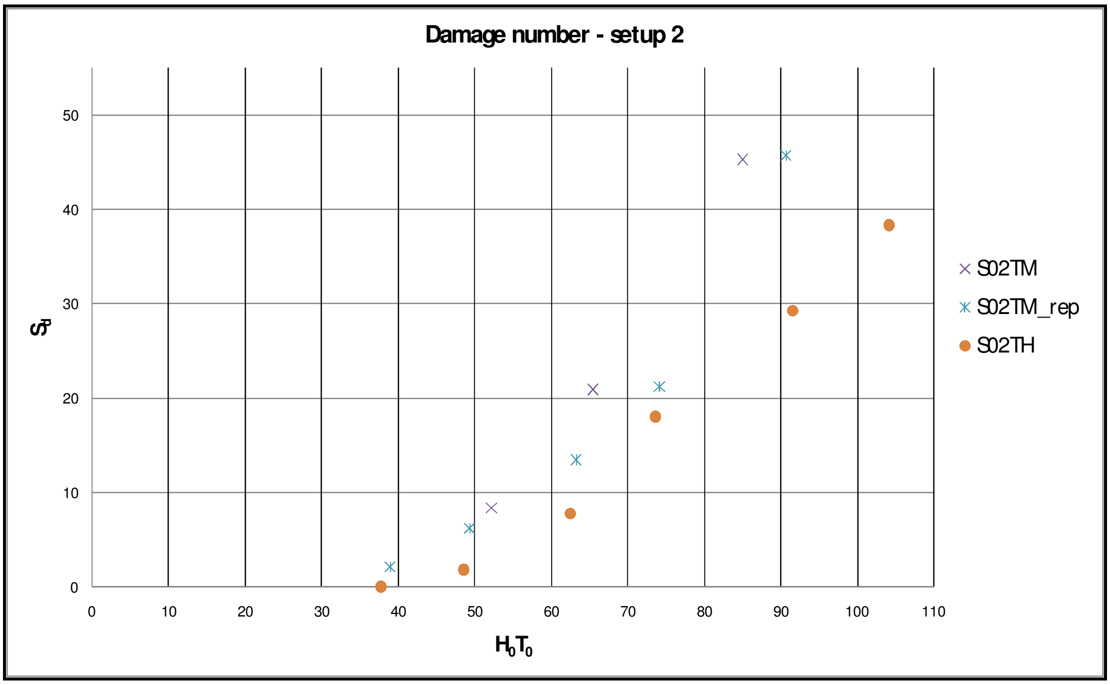


Figure 4.7 Damage number, comparison of different water levels for setup 2

This difference is significant for all setups, there can be a few explanations for that.

- As the water level is changed, the distance from the water level down to the transition of different stone classes changes as well. Therefore, for many of the tests, this can explain more erosion as the distance down to the smaller stones is less. This does, however, not explain why the same behaviour occurs with other setups as well, both for setup 5 where Class I and Class II are switched and for the mixed stones, setup 6.
- More energy dissipation due to the berm occurs when the water level is closer to the berm level. There is more overtopping and attack on stones located further up the berm. Individual stones are even, in some cases, moved far up the berm.
- With the water level closer to the berm the breakwater is closer to its natural reshaped profile. Therefore less reshaping and, consequently, less erosion is required to reach a stable reshaped profile.

4.3.2 Comparison of setups

In Figure 4.8 and Figure 4.9, the different erosion development for the different setups is compared for the different water levels of 0.590m and 0.645m. It should be noted that for the last values of each setup, where the H_0T_0 value is the highest, the test series was not completed for all the tests. Consequently, the number of waves in the last measurements was different between setups and never 3000 waves like for the other points. In chapter 4.4, the duration until failure of each setup is explained.

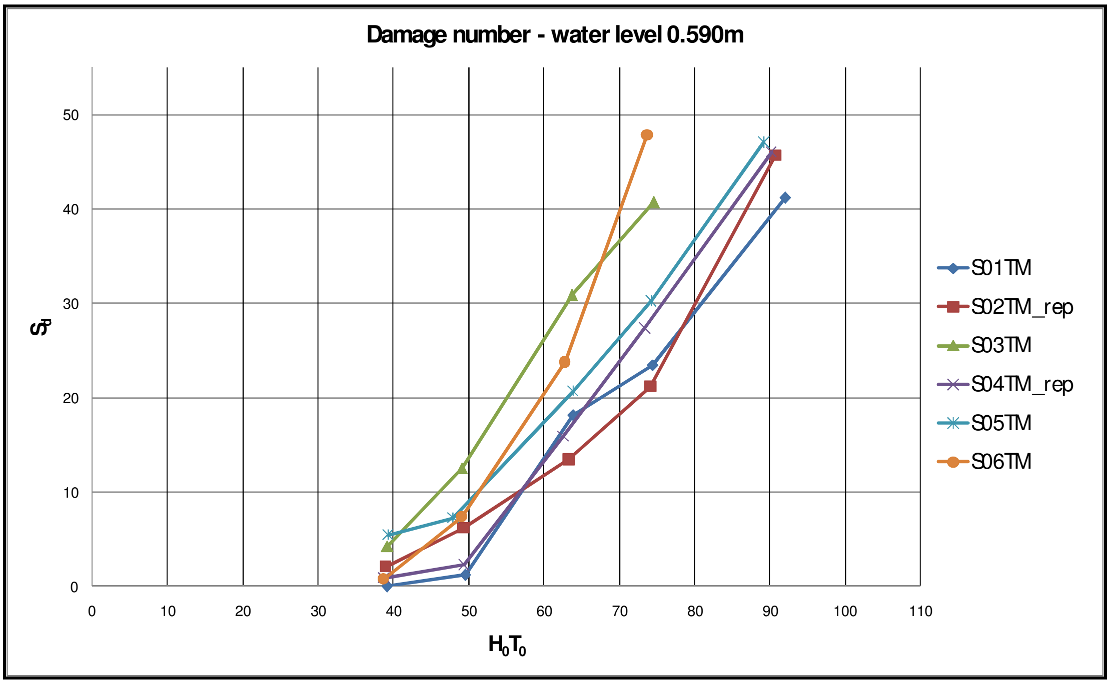


Figure 4.8 Damage number, comparison of different setups with water level 0.590m

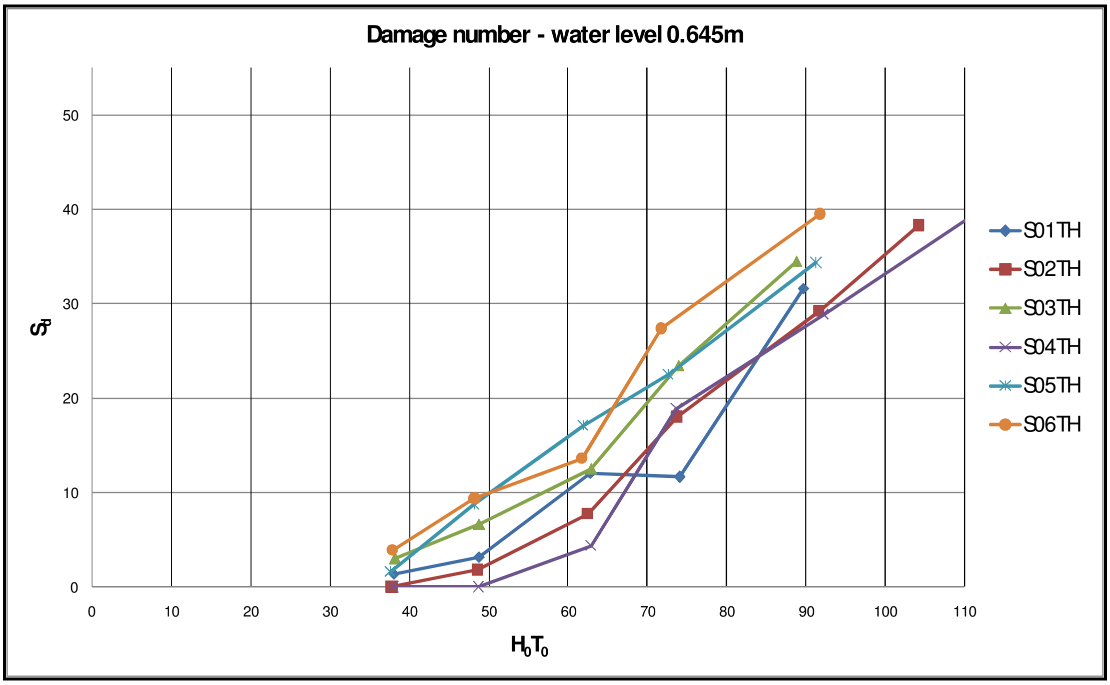


Figure 4.9 Damage number, comparison of different setups with water level 0.645m

For both water levels (0.590m and 0.645m), as expected, the most damage occurred for the matching wave period stability number in setups 3 and 6. Setup 5 also had high level of erosion when the water level was at 0.645m.

When looking into the situation where the water level is 0.590m, setups 1, 2 and 4 shows the least damage for the matching wave period stability number. Setup 5 for this water level caused more damage throughout the tests but the difference decreases with higher wave stability number, H_0T_0.

Where the water level is at 0.645m, setups 2 and 4 show the best results. Setup 1 shows similar total erosion development for the first four wave sets but then the damage increases in the fifth wave set. The reason for this can possibly be traced to the fact that the distance down to the transition of Class I and Class II stones is shorter for that setup than the other two and, as a result, it starts to affect the damage development severely when the waves get higher.

4.4 Duration until failure/core erosion

The point of failure was defined as the point where core erosion started. For each test this point of failure was noted. In most cases the test was also stopped around that moment but in some cases later. The last points in the concerning recession and general erosion do therefore not always represent the situation after 3000 waves. In the following tables, the duration until core erosion for each test can be viewed. The planned wave properties for each wave series can be viewed in Table 3.3, while the measured wave properties in each test can be viewed in appendix A.2.

Table 4.1 Duration of tests, water level 0.545m

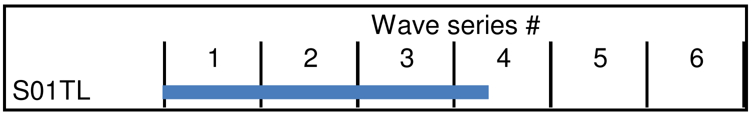


Table 4.2 Duration of tests, water level 0.590m

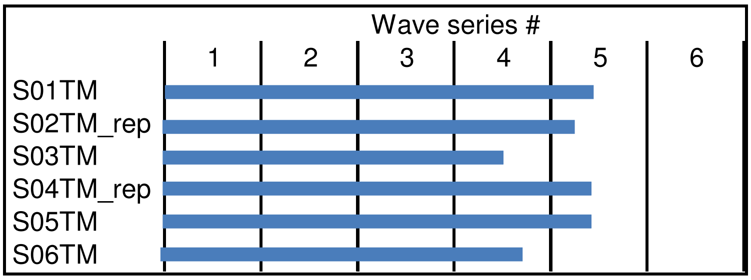
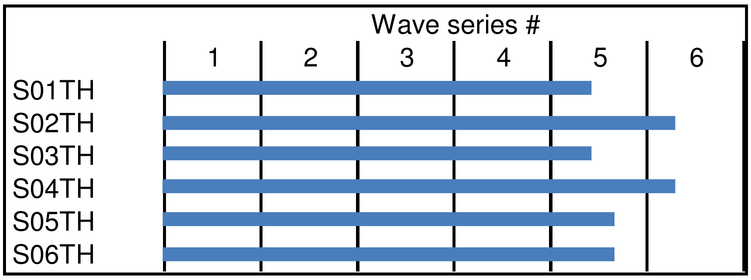


Table 4.3 Duration of tests, water level 0.645m



Generally the duration of tests until failure, turned out to be longer with the water level close to the berm than with lower water levels. For water level 0.590m, there is not much difference in the duration between setups 1, 2, 4 and 5. While for the higher water level, 0.645m, setups 2 and 4 were the ones with the longest duration.

4.5 Transition of original and reshaped profile, h_f

Expected values for the distance from the water level down to the point of transition of the original and the reshaped profile are expressed in Tables 4.4 and 4.5. Where in Table 4.4 the D_{n50} from Class I stones are used while in Table 4.5 the D_{n50} from the sample of mixed stone is used.

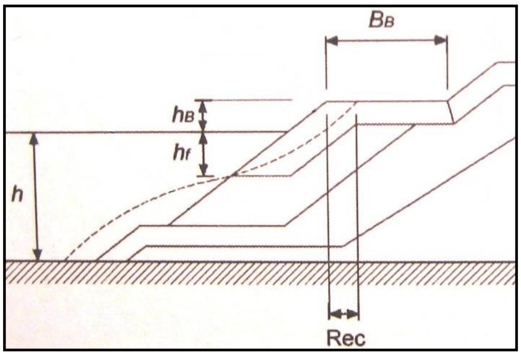


Table 4.4 Expected transition points of original and reshaped profiles, Class I

	L	M	H
h	0.545	0.59	0.645
D_{n50}	0.0320	0.0320	0.0320
h/D_{n50}	17.0	18.4	20.2
h_f/D_{n50}	3.91	4.19	4.53
h_f	0.125	0.134	0.145
h_B	0.115	0.07	0.015
h_B/D_{n50}	3.59	2.19	0.47

Table 4.5 Expected transition points of original and reshaped profiles, mixed stone sample

	L	M	H
h	0.545	0.59	0.645
D_{n50}	0.0257	0.0257	0.0257
h/D_{n50}	21.2	23.0	25.1
h_f/D_{n50}	4.74	5.09	5.52
h_f	0.122	0.131	0.142
h_B	0.115	0.07	0.015
h_B/D_{n50}	4.47	2.72	0.58

As can be seen from the tables, the distance from the water level to the expected point of transition of the profiles does not differ greatly whether D_{n50} is used for Class I stones or for stones of the mixed sample. The difference is well within the uncertainties of the tests, because in relatively deep water, the influence of D_{n50} is less and the difference of D_{n50} of the two samples is rather small. For setups 1-5, the D_{n50} from Class I stones are used as a measure of the relative parameters (h_f/D_{n50}, h_B/D_{n50} and h/D_{n50}), while for setup 6 the D_{n50} from the sample of mixed stones was used. It should be noted that Tables 4.4 and 4.5 includes the values for h_B/D_{n50} which is the distance from the berm down to water level as a function of stone diameter. This parameter is the most likely one, apart from the once included in formula 2.17, to have influence on h_f/D_{n50}.

4.5.1 Interpretation of test results

The results of the measurements are explained in graphs in appendix A.7. First, the graphs compare different berm levels, h_B/D_{n50}, in Figures A.44 – A.47, then compare the different setups with the berm level fixed in Figures 4.10, A.48 and A.49.

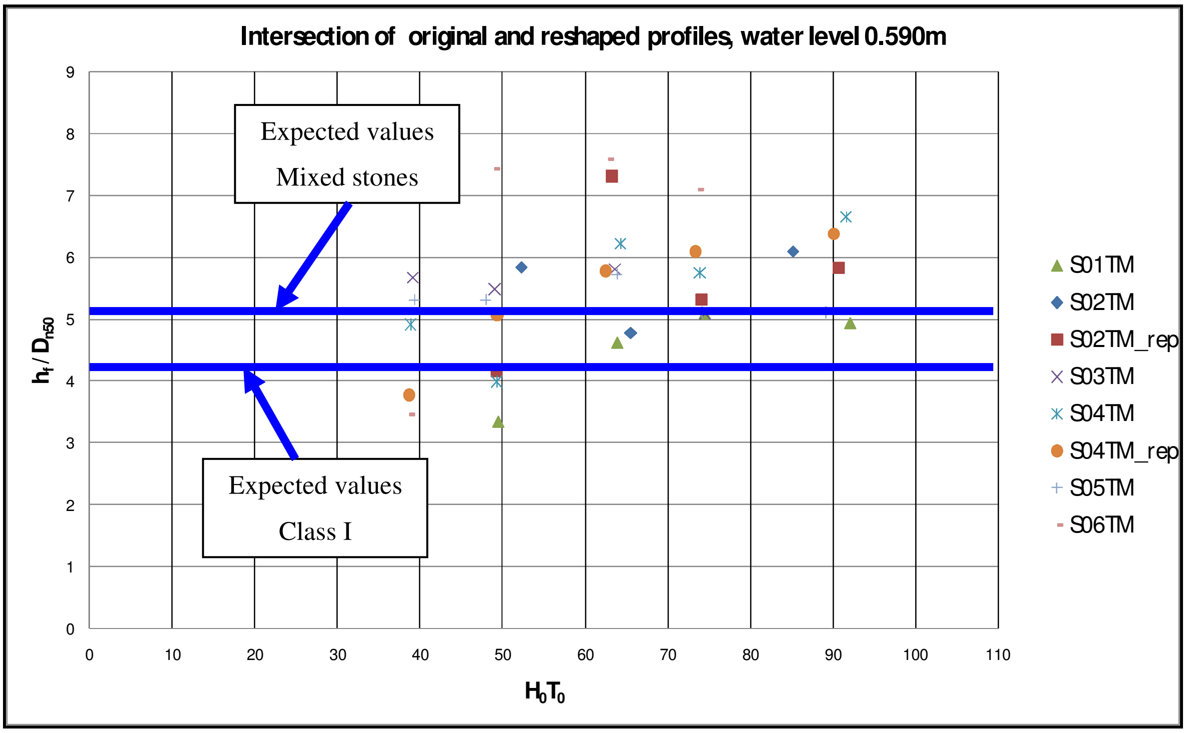


Figure 4.10 Transition of original and reshaped profile, hf, water level 0.590m

The results of the measurements were derived from those graphs and are the following:

For Class I stones:

h_B/D_{n50} = 3.59 and h/D_{n50} = 17.0 → **h_f/D_{n50} = 6.0** (only one sample with water level, 0.545m)

h_B/D_{n50} = 2.19 and h/D_{n50} = 18.4 → **h_f/D_{n50} = 5.1 - 5.6** (water level 0.590m)

h_B/D_{n50} = 0.47 and h/D_{n50} = 20.2 → **h_f/D_{n50} = 4.0 - 4.5** (water level 0.645m)

For stones from mixed sample (only one measurement for each):

h_B/D_{n50} = 2.72 and h/D_{n50} = 23.0 → **h_f/D_{n50} = 7.3** (water level 0.590m)

h_B/D_{n50} = 0.58 and h/D_{n50} = 25.1 → **h_f/D_{n50} = 5.7** (water level 0.645m)

The results show that the value of h_f/D_{n50} gets larger as the berm height, h_B/D_{n50}, gets larger. That means that as the water level is lower on the berm, the point of transition gets not only lower compared to the berm but also compared to the water level. The influence of h_B/D_{n50} is opposite to, and for these water depths, larger than the influence of water depth, h/D_{n50}. This proved to be the case for every setup, which shows an influence of berm height, h_B/D_{n50}, on h_f/D_{n50}. The influence can be explained by the fact that the lower the water level was compared to the berm. This results in larger forces acting lower on the structure.

With the water level relatively close to the berm, h_B/D_{n50} relatively low, formula 2.17 gives results close to the ones measured in the tests. However when that distance, h_B/D_{n50}, increases the test results give higher values compared to the formula. This suggests that to make the formula valid for situations where the water level is lower on the berm, the inclusion of berm height, h_B/D_{n50}, would be needed.

An addition of $0.6h_B/D_{n50}$ to formula 2.17 gives a formula that better fits the results of the experiment, both for the different berm heights. That is, with water level at 0.645m, 0.590m and for the only test that was performed with the water level at 0.545m. It also fits reasonably well for the different stone diameters, that is both where Class I was used and where the mixed sample was used.

$$\frac{h_f}{D_{n50}} = 0.2\frac{h}{D_{n50}} + 0.6\frac{h_B}{D_{n50}} + 0.5 \qquad (4.1)$$

4.6 Uncertainties

In this section the uncertainties of the experiment are discussed. The results are discussed in terms of results of the tests that were repeated and possible uncertainties of the model tests from real situation.

4.6.1 Uncertainties of tests

To realize the uncertainties of the model tests, the tests for two setups that were repeated will be discussed. Those tests are with setup 2 and 4, in both cases for water level at 0.590m. For setup 2, two different wave series were used, that is the wave conditions were different between the first and the repeated tests. While for setup 4, the same tests were repeated again since in the first test more stones were moved early on than expected.

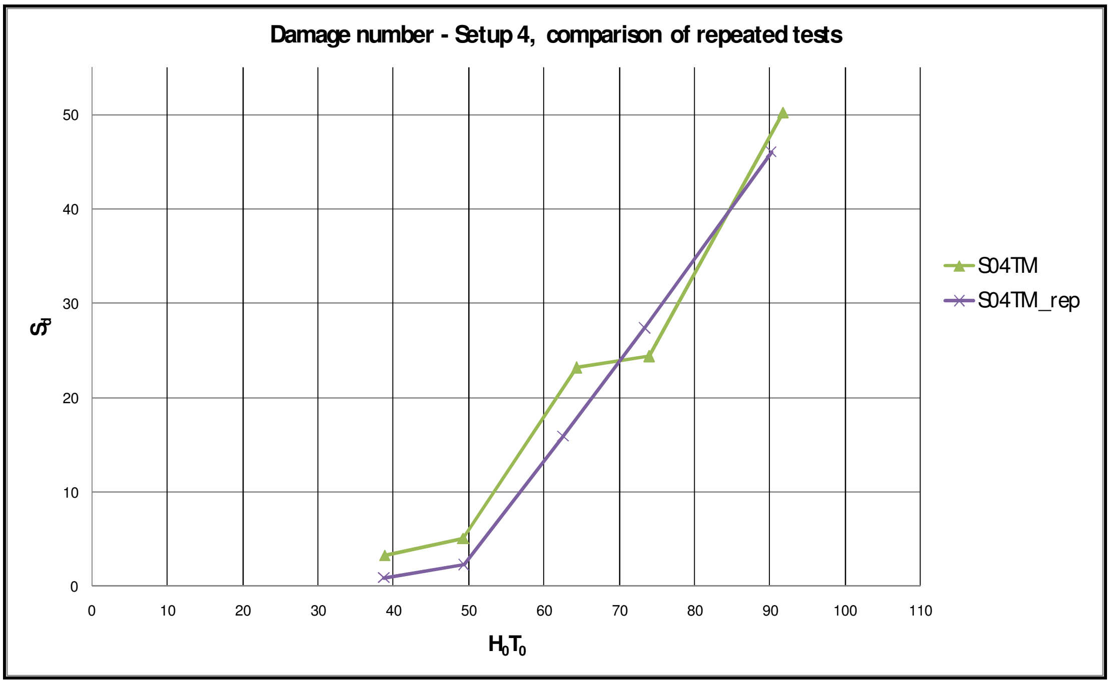


Figure 4.11 Damage number, comparison of original and repeated test of setup 4, with water level 0.590m

First, the results will be reviewed from setup 4, where the same wave series were used for both the original and the repeated tests. It has to be noted that for the original test, S04TM, the wave generator was not stopped at the point of core erosion, due to unavoidable disturbance in the laboratory. This explains that, for both the recession graph, Figure A.32, and the damage number graph, Figure 4.11 and Figure A.43, the point for that test gave more damage. This is especially notable in the recession graph. However, the general results however both for the recession and the damage number graphs, give results that are very similar between the two tests, see Figure 4.11. This indicates, although it has to be kept in mind that this assumption is only based on one repeated test, that the tests are reliable at least when compared individually. Other general uncertainties are discussed in chapter 4.6.2.

For setup 2, the situation is different since in the original test, S02TM, the wave steepness was different than for any other test. The comparison between the original and the repeated tests for this setup did not give as convincing results as for setup 4. When the recession was looked at as a function of H_0T_0, Figure A.30, as was done for every test, there was a large and constant difference of more than $2Rec/D_{n50}$ throughout the tests. This difference does, however, decrease if the wave period part, T_0, is neglected, Figure A.31, and the recession is measured as a function of H_0. For the total erosion of the structure that is measured with the damage number, S_d, the difference is not as large. There is a notable difference in the original graph, Figure A.41. However when looked at as a function of H_0, Figure A.42, it fits well.

4.6.2 General uncertainties

One of the most important aspect when an Icelandic type berm breakwater is constructed is the placement of the Class I stones on top of the berm and down to the water level. In reality the stones are carefully placed one by one, in such a way that they give good interlocking between them. This part is difficult to replicate in a model test. In this research the stones were placed in a more stable way than just random dumping but most likely in a less stable way than in a full size project.

The flume is rather narrow, 90cm ($28D_{n50}$ for Class I stones), and therefore it does not offer the possibility of measuring many cross sections at different locations. There are also clear effects from the sides of the flume that reach 10-15cm into the structure from each side. The active width of the flume in this experiment was therefore 60-70cm. The profiles were measured at the middle of the structure, with measurement equipment as is explained in chapter 3.2 and with Figure 3.4. It would have been preferable to have some kind of scanning equipment to measure the whole area and then afterwards the parts that are affected by the sides of the flume and other abnormal behaviour could be excluded.

5 Discussion of model tests

In order to get the desired wave heights in the flume the water level was deeper than originally planned. With the deep water condition, the biggest waves do not break like they would in some cases due to the water depth in front of the structure. That means that almost all the big waves hit the structure in the model, only the very biggest waves broke, but in most cases where a breakwater is constructed the waves would break when they are relatively smaller. The big waves are the ones that make the most damage and the top waves are not cut off as in situations where they break due to water depth.

In the tests all waves were perpendicular to the structure, longshore transport is therefore not taken into consideration in the research. When waves approach the structure at an angle the stones do, however, not only move up and down in the same cross section but some stones are also moved along the structure. This can affect the stability of the structure, but this is not considered to be a problem for a rather stable structure such as the Icelandic type berm breakwater, and will not be further discussed in this report.

Stone breaking strength is another factor that is not considered in this research. For the stable Icelandic type berm breakwater there is generally not much movement of stones, except in the largest storms. The stone breaking strength is therefore not as big an issue as when dynamic berm breakwaters are considered. The quality of stones can however not be neglected, when working with lower quality stones it is better to be more strict on the stability criteria.

Another idea would be to design some kind of a mix between the Icelandic type and the dynamic type berm breakwaters. Where the structure would be allowed to reshape like a dynamic berm breakwater but it would, however, be multi layered. I would reshape in such a way that less amount of Class I stones would be lost in the process. For that kind of a structure, which might be able to withstand bigger waves than the others, the stone breaking strength would become more important.

When a breakwater is designed, it is usually at locations where tidal influences occur. The magnitude of those tidal influences is, however, different for different locations. The fact that the Icelandic type berm breakwater has smaller stones lower on the berm makes it a bit sensitive when there is a large tidal difference. It is therefore important to look into both the largest expected storms with a high water level and with a low water level. Although the largest expected storms are likely to be smaller with a low water level, due to breakage of wave as a result of water depth, it has to be kept in mind that with the water level lower on the structure it takes smaller waves to cause damage.

When the water level was close to the berm there were some stones that moved up the berm.A little movement up the berm is not a problem, if however there are stones that move over the berm that might cause damage on vessels located on the other side of the structure. This should not be a problem, but it needs to be considered.

5.1 Discussion of current design rules

The upper layer of the berm consists of two layers of rock and extents on the down slope at least to mean sea level.

In this research the thickness of the layer on the berm consisted of two layers of Class I stones or that thickness where other stone classes were used. Between setups the amount of Class I stones on the berm was changed, in order to realize whether the Class I stones far up the berm are important and to find an optimal usage of Class I stones.

The rock size of this layer is determined by $H_0 = 2.0$. Larger rock may be used too.

Each setup was tested until failure, therefore different H_0 are tested, from 1.5 to 3.0. It was not one of this project's goals to question this stability criterion.

Slopes below and above the berm are 1:1.5

The slope in this research was fixed at 1:1.5 and therefore this was not tested. It is known that with a gentler slope the structure becomes stronger but on the other hand the total amount of material increases, especially with relatively deep water.

The berm width is 2.5 - 3.0 H_S

The berm width was constant throughout this research; this design rule is therefore outside the scope of this research.

The berm level is 0.65 H_S above design water level.

The berm level was played with in this research where three different berm levels were tested.

The crest height is given by $R_C/H_S{*}s_{op}^{1/3} = 0.35$

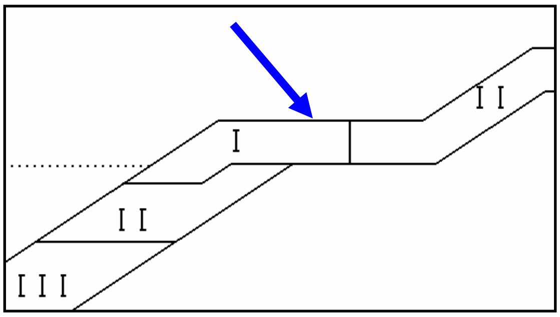
The crest height was constant throughout this research; this design rule is therefore outside the scope of this research.

5.2 Discussion of test results

Although all tests were held out until a complete failure of the structure, it is interesting to take a look at the situation after a design storm according to the current design rules of Icelandic type berm breakwaters. The value of the stability number, H_0, after the third wave series is 2.1 and that can be viewed as a design storm. This represents a storm that is a bit larger than is suggested in the current design rules, but in the same order of magnitude. The situation at a design storm can be read from the graphs in the analysis chapter, where H_0T_{0Z} is close to 45 where Class I is used but close to 63 where the mixed sample is used, which is the case for most comparative graphs.

5.2.1 Stones on the berm

The Class I stones on the berm are recommended to reach at least further than the expected recession, during a design storm. Depending on the availability of Class I stones in each project, the combination of the amount of Class I stones on the berm, on one hand, and down the slope on the other hand, has to be decided.



5.2.2 Berm level

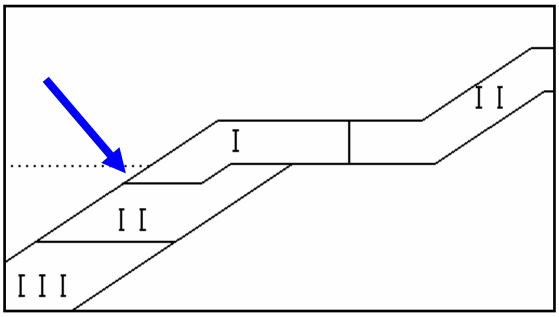
The results of this research indicate that it is feasible to design berm breakwaters with the water level close to the berm level. Some relevant aspects that influence the design of a breakwater are however not taken into account, the most relevant in this case is overtopping. It is usually one of the design requirements to keep the overtopping limited, other measures to limit overtopping are for example to increase the berm width, to increase the crest height or to add a structure at the top of the crest with the sole purpose of limiting overtopping. Decisions whether to stick to the former method or to take any of the mentioned action would depend on cost-benefit analysis. The results of those analyses might differ between projects and is beyond the scope of this research.

Wave reflection from a structure is another important factor of the function of a breakwater. To avoid wave reflection, from breakwater structures, causing a lot of unrest in the harbour basin it is favourable to keep it reasonably low. By placing the berm level closer to the water level the wave reflection from the structure is reduced considerably.

A downside to designing with the water level close to the berm is that there is more recession of the edge of the berm. That makes it less attractive when designing a stable structure.

5.2.3 Transition of Class I and Class II stones

In order to explain the recommended location of the transition of Class I and Class II stones, a new parameter, $h_{I\text{-}II}$, is introduced. This parameter represents the vertical distance from the water level down to the transaction of Class I and Class II stones.



For berm height $h_B/D_{n50,Class\ I}$ = 2.19 (water level 0.590m):

For this water level the strength of the structure improved when the transition of Class I and Class II was moved further down the slope, from setup 1 to setup 2. However, the same did not occur when the transition of Class I and Class II stones was moved further down the slope, from setup 2 to setup 4. This indicates that putting Class I stones further down than in setup 2 ($2.53\cdot h_{I\text{-}II}/D_{n50,Class\ I}$) does not improve the structure significantly. It is therefore recommended that the location of this transition should be as follows:

$$h_{I\text{-}II} \geq 2.55 D_{n50,\ Class\ I} \rightarrow \mathbf{h_{I\text{-}II} \geq 1.45 \cdot \Delta D_{n50,\ Class\ I}}$$

or

$$h_{I\text{-}II} \geq 3.25 D_{n50,\ Class\ II} \rightarrow \mathbf{h_{I\text{-}II} \geq 1.85 \cdot \Delta D_{n50,\ Class\ II}}$$

The one of the two that gives the larger $h_{I\text{-}II}$ in each case recommended to be chosen.

For berm height $h_B/D_{n50,Class\ I}$ = 0.47 (water level 0.645m):

For this water level the strength of the structure did not improve significantly when the transition of Class I and Class II stones was moved further down the slope, from setup 1 to setup 2. This indicates that by locating Class I stones further down than in setup 1 ($2.44\cdot h_{I-II}/D_{n50,Class\ I}$) does not improve the structure significantly. It is therefore recommended that the location of this transition should be as follows:

$$h_{I-II} \geq 2.45 D_{n50,\ Class\ I} \rightarrow \mathbf{h_{I-II} \geq 1.40 \cdot \Delta D_{n50,\ Class\ I}}$$

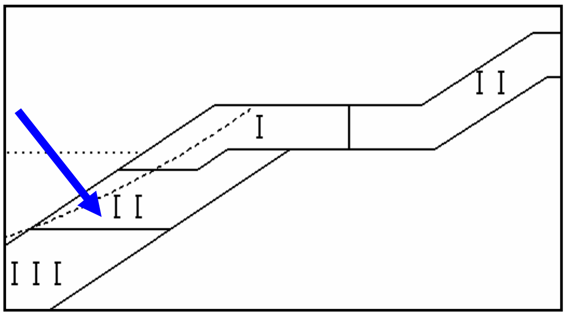
or

$$h_{I-II} \geq 3.15 D_{n50,\ Class\ II} \rightarrow \mathbf{h_{I-II} \geq 1.80 \cdot \Delta D_{n50,\ Class\ II}}$$

The one of the two that gives the larger h_{I-II} in each case is the recommended choice.

5.2.4 Transition of Class II and Class III stones

In order to explain the recommended location of the transition of Class II and Class III stones, a new parameter, h_{II-III}, is introduced. This parameter represents the distance from the water level down to the transition of Class II and Class III stones.



Class II stones should reach at least down to the expected transition of the original and the reshaped profile, h_f. For the transition of the Class II and Class III stones it turned out to influence the damage progress severely the only time this transition was above this level while it did not have an effect in the other tests. It is therefore recommended that:

$$\mathbf{h_{II-III} > h_f}$$

The distance from the water level down to the transition of the original and the reshaped profile, h_f, in the model tests did not turn out to be the same as for formula 2.17 when the water level was not close to the berm level. The formula is missing a parameter representing the berm height, h_B, which turned out to be very effective in this experiment.

For the situation where the water level is close to the berm level, $h_B/D_{n50} \approx 0.50$, the results of the experiment fit well with formula 2.17. However with the water level lower on the berm this is not the case. An addition of $0.6h_B/D_{n50}$ to formula 2.17 gives a formula that better fits the results of the experiment, both for the different berm heights, that is with water level at 0.645m, 0.590m and for the only test that was performed with the water level at 0.545m. It also fits reasonably well for the different stone diameters that is both where Class I was used and for the mixed sample as well.

$$\frac{h_f}{D_{n50}} = 0.2\frac{h}{D_{n50}} + 0.6\frac{h_B}{D_{n50}} + 0.5 \tag{6.1}$$

It has to be kept in mind that the water level is relatively deep in this experiment. This formula is not based on any tests in more shallow water where the influence of the water level on h_f/D_{n50} is more. The formula is therefore not necessarily valid for that situation. It is, however, clear that if different berm heights are to be considered, the inclusion of the berm height parameter, h_B/D_{n50}, is needed.

6 Conclusions and suggestions for further work

6.1 Conclusions

The conclusions of this M.Sc. thesis project are derived from a limited number of model tests of Icelandic type berm breakwaters. The results should therefore be approached with care and further research is recommended on the subject.

The conclusions are based on the different aspects discussed in chapter four and chapter five. That is recession and total erosion of different tests, as well as general interpretation of the reshaping profiles and visual observations.

6.1.1 Suggested additions to current design rules

The following is suggested as an addition to the current design rules, based on some of the current design rules being unchanged. Most importantly that the slope should be kept at 1:1.5 and the size of the largest stones should be determined by $H_0 = 2.0$. These are four steps that should give guidance of how to allocate the stones depending on the availability of different stone classes in each case.

1. Class I stones on the berm are recommended to reach at least further into the berm than the expected recession from a design storm.
2. Class I stones are recommended to reach as far down as:

   $$\mathbf{h_{I\text{-}II} \geq 1.45 \cdot \Delta D_{n50,\, Class\ I}}$$

   or

   $$\mathbf{h_{I\text{-}II} \geq 1.85 \cdot \Delta D_{n50,\, Class\ II}}$$

   The one of the two that gives the larger $h_{I\text{-}II}$ in each case are recommended to be chosen. The strength of the structure did not seem to improve when the Class I stones reached further down the berm.
3. If there is more of Class I stones available after meeting the recommendations above, (1 and 2) they should be placed further into the berm.
4. Class II stones should in any case reach at least down to the transition of the original and the expected reshaped profile, h_f. In the only setup that it was not the case it weakened the structure significantly while in the other setups it did not have any effect on the damage development.

6.2 Suggested further research

There are still a few aspects of the Icelandic type berm breakwaters, like for most coastal structures, that would gain from being further researched. During the work on this M.Sc. thesis project some ideas for topics that would be interesting to further investigate, came to light.

1. One of the important factors of the design of an Icelandic type berm breakwater is the careful placement of the largest stones. This part turned out to be a bit difficult to imitate since in reality the stones are carefully placed one by one. To have some kind of a method to imitate the situation in a relatively easy way would be helpful. It should be noted that this is a delicate subject and it is important also not to overdue the interlocking between the stones.

2. Similar research as the one executed in this project. But use the results from the model tests from this project to choose appropriate wave heights with the goal of approaching more accurately the critical points in the failure process. Using more accurate cross section measurements would also be helpful. To be able to measure the cross sections at different location along the structure as well as measuring the cross sections more often, for example for every 500 waves. That would also open the possibility of investigating the influence of storm duration of the damage development as has proven to be an important factor for conventional breakwaters, van der Meer (1988).

3. To combine the benefits of on one hand, the Icelandic type berm breakwater and on the other hand, a dynamic berm breakwater would be an interesting subject. The idea is that the structure would be like the Icelandic type, multi layered, but as for the dynamic type it would be allowed to reshape. The setup should preferably be done in a manner that makes the structure stronger after the main reshaping and where most of the Class I stones will still be in the upper part. For the current setup of the Icelandic type there are always some Class I stones that are lost (roll down the slope) early in the process. Test setup number 5 in this research, where Class I and Class II stones switched places, might be a possible starting point for such research.

4. To use the data gathered in this research to further develop the understanding of the Icelandic type berm breakwater. That might be on the subject of recession where the data gathered in this research might be compared with data from other research studies on that subject, and possibly a new formula might be developed where the scatter would be lower.

5. The influence of wave period or the wave steepness is another issue that was not investigated in this research since the target wave steepness was the same throughout the research. The results from the only test where accidentally different wave steepness was used, indicates that it would be interesting to look further into that subject. Research focusing on the wave steepness as well as the influence of storm duration might be an interesting combination.

6. The influence of overtopping and different measures to decrease overtopping when the water level is kept close to the berm level. It might also be interesting to measure the difference in overtopping with different berm height while the crest height remains constant. Such research might include playing with the berm width, the crest height or by including of some structure with the purpose of decreasing overtopping.

7 References

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A Appendices

A.1 Model setup, key parameters expressed dimensionless

In this section the parameters expressed in chapter 3.3, as a function of first the different wave heights used in the tests and then the different median nominal diameters, D_{n50}, of the three different stone classes.

A.1.1 Key parameters expressed as a function of wave height

Table A.1 Key parameters expressed as a function of wave height 0.085m

H_S [m]=	0.085												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.18	0.92	2.28	7.59	0.82	0.27	1.64	6.94	1.35	-0.26	1.11	6.41	3.53
2	0.18	1.60	2.28	7.59	0.82	0.95	1.64	6.94					
3	0.18	0.92	1.33	7.59	0.82	0.27	0.68	6.94					
4	0.18	-	2.28	7.59	0.82	-	1.64	6.94					
5	0.18	0.92	1.93	7.59	0.82	0.27	1.28	6.94					

Table A.2 Key parameters expressed as a function of wave height 0.100m

H_S [m]=	0.100												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.15	0.78	1.94	6.45	0.70	0.23	1.39	5.90	1.15	-0.22	0.94	5.45	3.00
2	0.15	1.36	1.94	6.45	0.70	0.81	1.39	5.90					
3	0.15	0.78	1.13	6.45	0.70	0.23	0.58	5.90					
4	0.15	-	1.94	6.45	0.70	-	1.39	5.90					
5	0.15	0.78	1.64	6.45	0.70	0.23	1.09	5.90					

Table A.3 Key parameters expressed as a function of wave height 0.120m

H_S [m]=	0.120												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.13	0.65	1.62	5.38	0.58	0.19	1.16	4.92	0.96	-0.18	0.78	4.54	2.50
2	0.13	1.13	1.62	5.38	0.58	0.67	1.16	4.92					
3	0.13	0.65	0.94	5.38	0.58	0.19	0.48	4.92					
4	0.13	-	1.62	5.38	0.58	-	1.16	4.92					
5	0.13	0.65	1.37	5.38	0.58	0.19	0.91	4.92					

Table A.4 Key parameters expressed as a function of wave height 0.135m

H_S [m]=	0.135												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.11	0.58	1.44	4.78	0.52	0.17	1.03	4.37	0.85	-0.16	0.70	4.04	2.22
2	0.11	1.01	1.44	4.78	0.52	0.60	1.03	4.37					
3	0.11	0.58	0.84	4.78	0.52	0.17	0.43	4.37					
4	0.11	-	1.44	4.78	0.52	-	1.03	4.37					
5	0.11	0.58	1.21	4.78	0.52	0.17	0.81	4.37					

Table A.5 Key parameters expressed as a function of wave height 0.155m

H_S [m]=	0.155												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.10	0.50	1.25	4.16	0.45	0.15	0.90	3.81	0.74	-0.14	0.61	3.52	1.94
2	0.10	0.88	1.25	4.16	0.45	0.52	0.90	3.81					
3	0.10	0.50	0.73	4.16	0.45	0.15	0.37	3.81					
4	0.10	-	1.25	4.16	0.45	-	0.90	3.81					
5	0.10	0.50	1.06	4.16	0.45	0.15	0.70	3.81					

Table A.6 Key parameters expressed as a function of wave height 0.170m

H_S [m]=	0.170												
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	h_B/H_s	$(b-h_B)/H_s$	$(c-h_B)/H_s$	h/H_s	B_B/H_s
1	0.09	0.46	1.14	3.79	0.41	0.14	0.82	3.47	0.68	-0.13	0.55	3.21	1.76
2	0.09	0.80	1.14	3.79	0.41	0.48	0.82	3.47					
3	0.09	0.46	0.66	3.79	0.41	0.14	0.34	3.47					
4	0.09	-	1.14	3.79	0.41	-	0.82	3.47					
5	0.09	0.46	0.96	3.79	0.41	0.14	0.64	3.47					

A.1.2 Key parameters as a function of median nominal diameter, D_{n50}, of different stone classes

Table A.7 Key parameters expressed as a function of D_{n50}=0.032m

D_{n50}[m]=	0.032	Class I											
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	B_B/D_{n50}
1	0.47	2.44	6.06	20.16	2.19	0.72	4.34	18.44	3.59	-0.69	2.94	17.03	9.38
2	0.47	4.25	6.06	20.16	2.19	2.53	4.34	18.44					
3	0.47	2.44	3.53	20.16	2.19	0.72	1.81	18.44					
4	0.47	-	6.06	20.16	2.19	-	4.34	18.44					
5	0.47	2.44	5.13	20.16	2.19	0.72	3.41	18.44					

Table A.8 Key parameters expressed as a function of D_{n50}=0.025m

D_{n50}[m]=	0.025	Class II											
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	B_B/D_{n50}
1	0.60	3.12	7.76	25.80	2.80	0.92	5.56	23.60	4.60	-0.88	3.76	21.80	12.00
2	0.60	5.44	7.76	25.80	2.80	3.24	5.56	23.60					
3	0.60	3.12	4.52	25.80	2.80	0.92	2.32	23.60					
4	0.60	-	7.76	25.80	2.80	-	5.56	23.60					
5	0.60	3.12	6.56	25.80	2.80	0.92	4.36	23.60					

Table A.9 Key parameters expressed as a function of D_{n50}=0.020m

D_{n50}[m]=	0.020	Class III											
	Water level 0.645m				Water level 0.590m				Water level 0.545m				
Setup	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	h_B/D_{n50}	$(b-h_B)/D_{n50}$	$(c-h_B)/D_{n50}$	h/D_{n50}	B_B/D_{n50}
1	0.76	3.94	9.80	32.58	3.54	1.16	7.02	29.80	5.81	-1.11	4.75	27.53	15.15
2	0.76	6.87	9.80	32.58	3.54	4.09	7.02	29.80					
3	0.76	3.94	5.71	32.58	3.54	1.16	2.93	29.80					
4	0.76	-	9.80	32.58	3.54	-	7.02	29.80					
5	0.76	3.94	8.28	32.58	3.54	1.16	5.51	29.80					

A.2 Wave measurements

In this section there are tables that show the results of the wave measurements from the tests. These are output numbers from each test along with some key numbers, among those numbers are the stability number, H_0 and the wave period stability number, H_0T_0, both for D_{n50} from Class I stones and from the mixture of the stones.

A.2.1 Setup 1

Table A.10 Wave properties of setup 1, water level 0.545m

S1TL									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1														
2	0.102	0.051	0.057	1.13	1.07	2.9	2.8	0.27	**1.80**	36	34	**2.25**	50	47
3	0.122	0.042	0.053	1.36	1.21	3.2	2.9	0.31	**2.16**	51	46	**2.69**	72	64
4	0.138	0.044	0.056	1.42	1.26	3.2	2.8	0.32	**2.43**	60	53	**3.03**	84	75
5														
6														

Table A.11 Wave properties of setup 1, water level 0.590m

S1TM									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.088	0.045	0.052	1.12	1.04	3.2	2.9	0.28	**1.54**	30	28	**1.93**	42	39
2	0.103	0.040	0.053	1.29	1.12	3.4	2.9	0.29	**1.81**	41	35	**2.26**	57	49
3	0.122	0.042	0.053	1.37	1.21	3.3	2.9	0.31	**2.15**	52	46	**2.69**	72	64
4	0.137	0.044	0.055	1.41	1.27	3.2	2.9	0.29	**2.41**	59	53	**3.01**	83	74
5	0.158	0.045	0.055	1.50	1.36	3.1	2.8	0.29	**2.78**	73	66	**3.47**	102	92
6														

Table A.12 Wave properties of setup 1, water level 0.645m

S1TH									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.041	0.049	1.15	1.05	3.3	3.0	0.20	**1.48**	30	27	**1.85**	42	38
2	0.101	0.039	0.052	1.29	1.12	3.4	2.9	0.22	**1.78**	40	35	**2.23**	56	49
3	0.121	0.041	0.053	1.37	1.21	3.3	2.9	0.26	**2.13**	51	45	**2.66**	71	63
4	0.136	0.043	0.055	1.42	1.26	3.2	2.8	0.26	**2.40**	60	53	**3.00**	83	74
5	0.155	0.042	0.054	1.54	1.35	3.3	2.9	0.25	**2.72**	73	64	**3.40**	102	90

A.2.2 Setup 2

Table A.13 Wave properties of setup 2, water level 0.590m, first test

S2TM									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1														
2	0.109	0.051	0.055	1.17	1.12	3.0	2.8	0.26	**1.91**	39	37	**2.39**	55	52
3														
4	0.130	0.047	0.060	1.33	1.18	3.1	2.7	0.29	**2.29**	53	47	**2.86**	74	66
5	0.156	0.051	0.062	1.40	1.27	3.0	2.7	0.33	**2.75**	67	61	**3.43**	94	85
6														

Table A.14 Wave properties of setup 2, water level 0.590m, repeated test

S2TM_rep									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.085	0.036	0.048	1.23	1.07	3.5	3.0	0.26	**1.50**	32	28	**1.88**	45	39
2	0.103	0.039	0.052	1.29	1.12	3.4	2.9	0.27	**1.80**	41	35	**2.25**	57	49
3	0.122	0.042	0.053	1.37	1.21	3.3	2.9	0.31	**2.14**	51	45	**2.68**	72	63
4	0.137	0.044	0.055	1.41	1.27	3.2	2.9	0.30	**2.40**	59	53	**3.00**	83	74
5	0.155	0.042	0.054	1.54	1.36	3.3	2.9	0.28	**2.73**	74	65	**3.41**	103	91
6														

Table A.15 Wave properties of setup 2, water level 0.645m

S2TH										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.039	0.049	1.18	1.05	3.4	3.0	0.17	**1.48**	30	27	**1.84**	42	38
2	0.102	0.039	0.052	1.29	1.12	3.4	2.9	0.19	**1.79**	40	35	**2.23**	56	49
3	0.120	0.041	0.053	1.37	1.21	3.3	2.9	0.24	**2.12**	51	45	**2.64**	71	63
4	0.136	0.043	0.055	1.42	1.26	3.2	2.8	0.25	**2.40**	60	53	**2.99**	83	74
5	0.158	0.043	0.056	1.53	1.35	3.2	2.8	0.24	**2.78**	74	66	**3.47**	104	92
6	0.170	0.041	0.053	1.62	1.43	3.3	2.9	0.23	**2.98**	85	75	**3.73**	118	104

A.2.3 Setup 3

Table A.16 Wave properties of setup 3, water level 0.590m

S3TM										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.086	0.040	0.048	1.17	1.07	3.3	3.0	0.28	**1.51**	31	28	**1.89**	43	39
2	0.103	0.039	0.053	1.30	1.12	3.4	2.9	0.28	**1.80**	41	35	**2.25**	57	49
3	0.122	0.042	0.053	1.37	1.21	3.3	2.9	0.30	**2.15**	52	46	**2.69**	72	64
4	0.137	0.044	0.055	1.42	1.27	3.2	2.8	0.32	**2.42**	60	53	**3.02**	84	75
5														
6														

Table A.17 Wave properties of setup 3, water level 0.645m

S3TH										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.040	0.048	1.16	1.06	3.3	3.0	0.20	**1.48**	30	27	**1.85**	42	38
2	0.101	0.039	0.052	1.29	1.12	3.4	2.9	0.22	**1.79**	40	35	**2.23**	56	49
3	0.121	0.041	0.053	1.37	1.21	3.3	2.9	0.26	**2.13**	51	45	**2.66**	71	63
4	0.136	0.043	0.054	1.42	1.27	3.2	2.9	0.27	**2.39**	59	53	**2.99**	83	74
5	0.153	0.042	0.054	1.53	1.35	3.3	2.9	0.27	**2.70**	72	64	**3.37**	101	89
6														

A.2.4 Setup 4

Table A.18 Wave properties of setup 4, water level 0.590m, first test

S4TM										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.085	0.033	0.048	1.28	1.07	3.7	3.0	0.26	**1.50**	33	28	**1.87**	47	39
2	0.103	0.040	0.053	1.29	1.12	3.3	2.9	0.28	**1.81**	41	35	**2.26**	57	49
3	0.123	0.042	0.053	1.37	1.22	3.3	2.9	0.31	**2.17**	52	46	**2.71**	73	64
4	0.137	0.044	0.055	1.41	1.26	3.2	2.8	0.31	**2.40**	59	53	**3.00**	83	74
5	0.158	0.046	0.056	1.49	1.35	3.1	2.8	0.30	**2.78**	72	66	**3.47**	101	92
6														

Table A.19 Wave properties of setup 4, water level 0.590m, repeated test

S4TM_rep										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.036	0.047	1.23	1.07	3.5	3.1	0.26	**1.48**	32	28	**1.85**	45	39
2	0.102	0.039	0.051	1.29	1.13	3.4	3.0	0.27	**1.79**	40	35	**2.24**	56	49
3	0.119	0.041	0.051	1.37	1.22	3.3	2.9	0.31	**2.10**	50	45	**2.62**	70	63
4	0.135	0.045	0.053	1.39	1.27	3.2	2.9	0.30	**2.37**	58	53	**2.96**	80	73
5	0.155	0.043	0.055	1.53	1.35	3.2	2.9	0.30	**2.74**	73	65	**3.42**	102	90
6														

Table A.20 Wave properties of setup 4, water level 0.645m

S4TH										Class I			Mixed	
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.040	0.049	1.16	1.05	3.3	3.0	0.21	**1.48**	30	27	**1.85**	42	38
2	0.102	0.039	0.052	1.29	1.12	3.4	2.9	0.23	**1.79**	40	35	**2.23**	56	49
3	0.121	0.042	0.053	1.36	1.21	3.3	2.9	0.26	**2.13**	51	45	**2.66**	71	63
4	0.136	0.044	0.055	1.41	1.26	3.2	2.8	0.26	**2.40**	59	53	**2.99**	82	74
5	0.159	0.042	0.056	1.55	1.35	3.2	2.8	0.23	**2.80**	76	66	**3.49**	106	92
6	0.178	0.043	0.054	1.62	1.45	3.2	2.9	0.21	**3.13**	89	79	**3.91**	124	111

A.2.5 Setup 5

Table A.21 Wave properties of setup 5, water level 0.590m

S5TM									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.086	0.035	0.048	1.26	1.07	3.6	3.0	0.27	**1.51**	33	28	**1.88**	46	39
2	0.100	0.037	0.051	1.31	1.12	3.5	3.0	0.20	**1.75**	40	34	**2.19**	56	48
3	0.122	0.042	0.053	1.37	1.22	3.3	2.9	0.30	**2.15**	52	46	**2.69**	72	64
4	0.137	0.044	0.055	1.41	1.27	3.2	2.9	0.30	**2.40**	59	53	**3.00**	83	74
5	0.155	0.044	0.055	1.50	1.34	3.2	2.8	0.29	**2.73**	72	64	**3.41**	100	89
6														

Table A.22 Wave properties of setup 3, water level 0.645m

S5TH									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.083	0.042	0.048	1.13	1.05	3.3	3.0	0.20	**1.47**	29	27	**1.83**	40	38
2	0.100	0.038	0.050	1.29	1.13	3.4	3.0	0.20	**1.75**	40	35	**2.19**	55	48
3	0.119	0.041	0.052	1.37	1.21	3.3	2.9	0.24	**2.09**	50	44	**2.62**	70	62
4	0.134	0.043	0.054	1.42	1.26	3.2	2.9	0.24	**2.36**	59	52	**2.95**	82	73
5	0.156	0.041	0.054	1.56	1.36	3.3	2.9	0.24	**2.75**	75	65	**3.43**	105	91
6														

A.2.6 Setup 6

Table A.23 Wave properties of setup 6, water level 0.590m

S6TM									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.084	0.040	0.047	1.16	1.07	3.3	3.1	0.24	**1.48**	30	28	**1.85**	42	39
2	0.102	0.039	0.052	1.29	1.12	3.4	2.9	0.24	**1.80**	40	35	**2.24**	57	49
3	0.120	0.041	0.052	1.37	1.22	3.3	2.9	0.28	**2.11**	51	45	**2.63**	71	63
4	0.135	0.044	0.054	1.41	1.27	3.2	2.9	0.29	**2.38**	59	53	**2.97**	82	74
5														
6														

Table A.24 Table A.16 Wave properties of setup 6, water level 0.645m

S6TH									Class I			Mixed		
Test #	H_S [m]	s_{0p}	s_{0z}	T_p [s]	T_Z [s]	ξ_p	ξ_Z	Refl	H_0	H_0T_{0p}	H_0T_{0z}	H_0	H_0T_{0p}	H_0T_{0z}
1	0.083	0.041	0.047	1.14	1.06	3.3	3.1	0.18	**1.47**	29	27	**1.83**	41	38
2	0.100	0.039	0.051	1.29	1.12	3.4	2.9	0.17	**1.76**	40	35	**2.20**	56	48
3	0.119	0.041	0.052	1.37	1.21	3.3	2.9	0.21	**2.09**	50	44	**2.61**	70	62
4	0.134	0.042	0.055	1.43	1.25	3.3	2.8	0.27	**2.35**	59	51	**2.94**	82	72
5	0.157	0.042	0.054	1.55	1.36	3.3	2.9	0.27	**2.77**	75	66	**3.45**	105	92
6														

A.3 Stone measurements

A.3.1 Class I

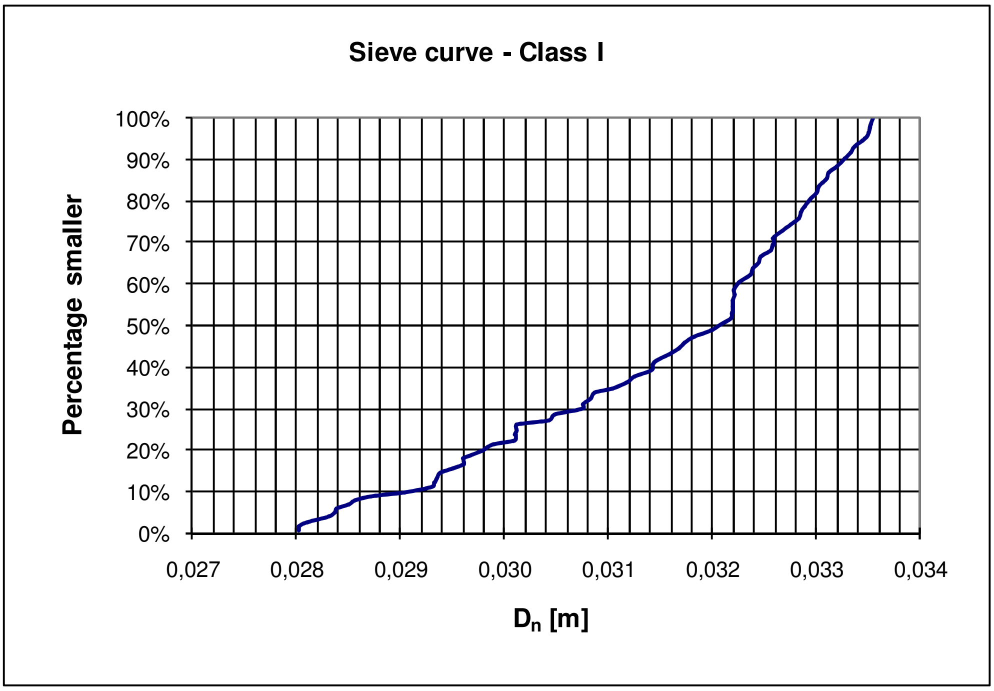

Figure A.1 Sieve curve for Class I stones

Table A.25 Class I stones, key nominal diameters

$D_{n,min}$ [m]	0.0244
$D_{n,max}$ [m]	0.0485
D_{n50} [m]	0.0320
D_{n15} [m]	0.0294
D_{n85} [m]	0.0331
D_{n10} [m]	0.0290
D_{n60} [m]	0.0322

$$\frac{D_{n85}}{D_{n15}} = 1{,}13 < 1{,}5 \Rightarrow Narrow\_grading$$

A.3.2 Class II

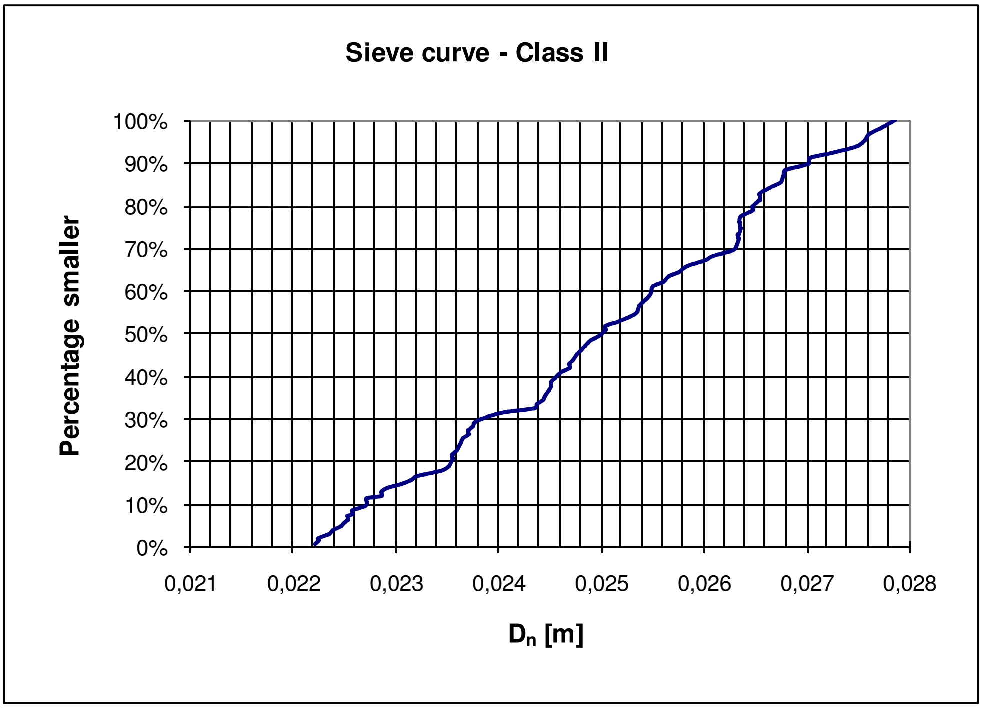

Figure A.2 Sieve curve for Class II stones

Table A.26 Class II stones, key nominal diameters.

$D_{n,min}$ [m]	0.0156
$D_{n,max}$ [m]	0.0288
D_{n50} [m]	0.0250
D_{n15} [m]	0.0231
D_{n85} [m]	0.0267
D_{n10} [m]	0.0227
D_{n60} [m]	0.0255

$$\frac{D_{n85}}{D_{n15}} = 1{,}16 < 1{,}5 \Rightarrow Narrow\_grading$$

A.3.3 Class III

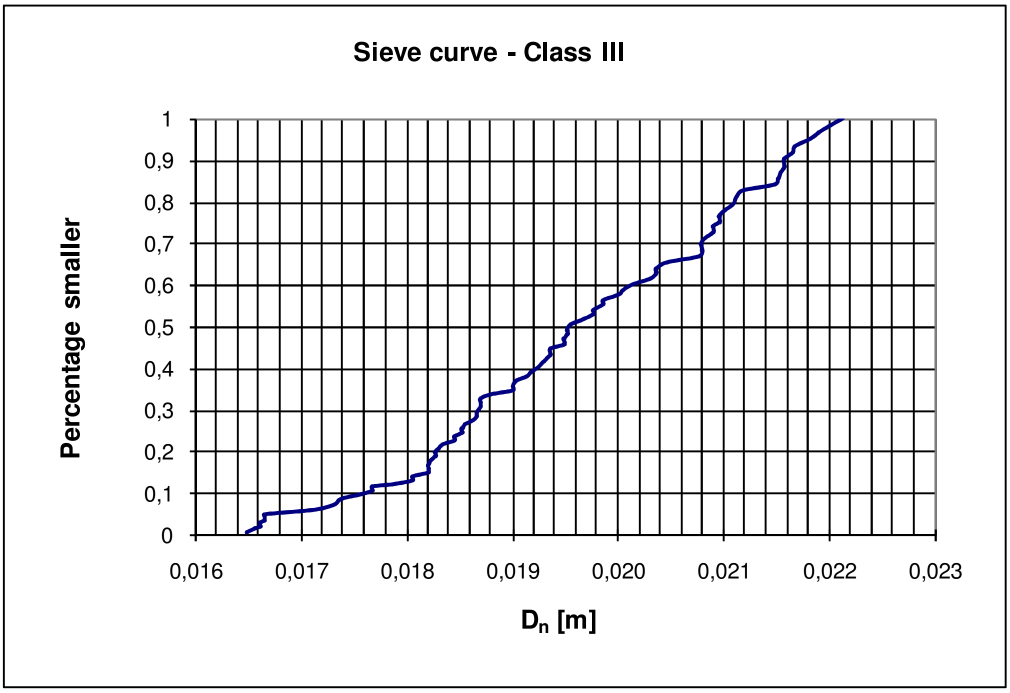

Figure A.3 Sieve curve for Class III stones

Table A.27 Class III stones, key nominal diameters.

$D_{n,min}$ [m]	0.0156
$D_{n,max}$ [m]	0.0288
D_{n50} [m]	0.0198
D_{n15} [m]	0.0170
D_{n85} [m]	0.0215
D_{n10} [m]	0.0176
D_{n60} [m]	0.0201

$$\frac{D_{n85}}{D_{n15}} = 1{,}26 < 1{,}5 \Rightarrow Narrow\_grading$$

A.3.4 Core

The stones used for the core are not the same kind of material as used for the armour layer. Although that is the case when an Icelandic type berm breakwater is constructed, the core is however not being considered in this project. This does not influence the behaviour of the armour layer. The density of the core material is ρ_s = 2620 [kg/m3].

Table A.28 Core material, key nominal diameters.

$D_{n,min}$ [m]	0,0067
$D_{n,max}$ [m]	0,0136
D_{n50} [m]	0,0099
D_{n15} [m]	0,0087
D_{n85} [m]	0,0116

$$\frac{D_{n85}}{D_{n15}} = 1{,}33 < 1{,}5 \Rightarrow Narrow\_grading$$

A.3.5 Mixture of all stones

For comparison all the stones that were used in the experiment were mixed. The reason was to make a homogenous berm breakwater, setup 6, with the same profile as in the other setups. The reason for that setup was to be able to compare the results both with earlier researches, for example recession, and also with the other setups.

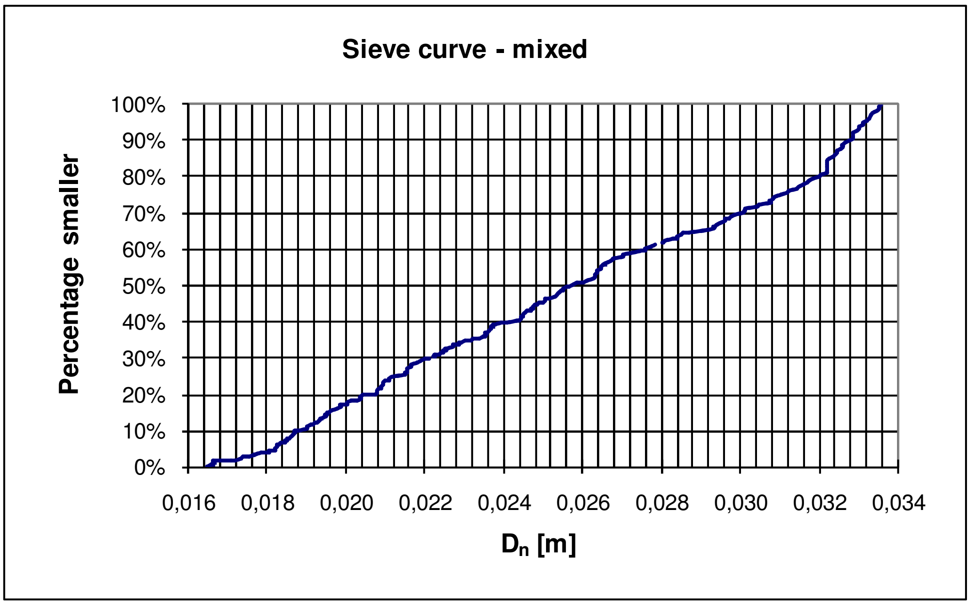

Figure A.4 Sieve curve for the mixture of all stones

Table A.29 Mixture of all stones, key nominal diameters.

$D_{n,min}$ [m]	0.0156
$D_{n,max}$ [m]	0.0485
D_{n50} [m]	0.0257
D_{n15} [m]	0.0198
D_{n85} [m]	0.0323
D_{n10} [m]	0.0188
D_{n60} [m]	0.0276

$$\frac{D_{n85}}{D_{n15}} = 1{,}63 \Rightarrow 1{,}5 < 1{,}63 > 2{,}5 \Rightarrow Wide\_grading$$

A.4 Cross sections and reshaping process

Total of 70 tests were made, where 6 different cross sections or setups. Three different water levels were tested where two of them were tested for every setup while the lowest water level was only tested for setup 1. The key dimensions from every setup can be viewed in Figure 3.5 and are further explained in Table 3.1. As well as in appendix A.1 where it is explained as a function of different the different wave heights used as well as the different median nominal diameters, D_{n50}, of the different stone classes.

In Figure A.5 the colours used to explain the damage development are explained. The number 0 represents the original situation, before any wave action. While the numbers 1-6 each represent a wave test as explained in Table 3.3, number 1 being the smallest waves while number 6 are the largest.

Figure A.5 Explanation figure for the damage development.

A.4.1 Setup 1

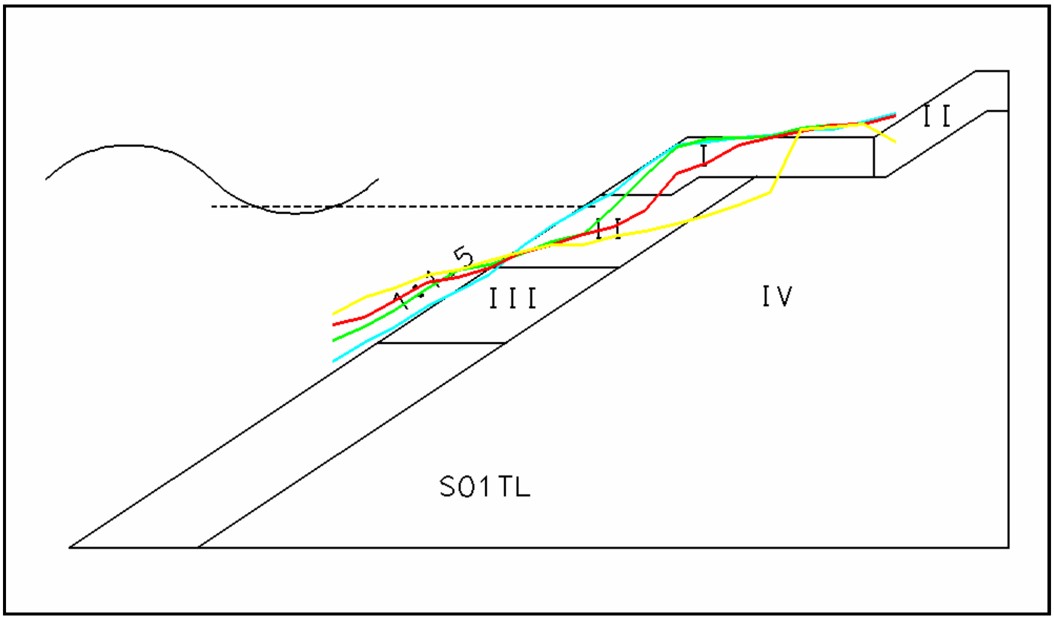

Figure A.6 Damage development of setup 1, water level 0.545m.

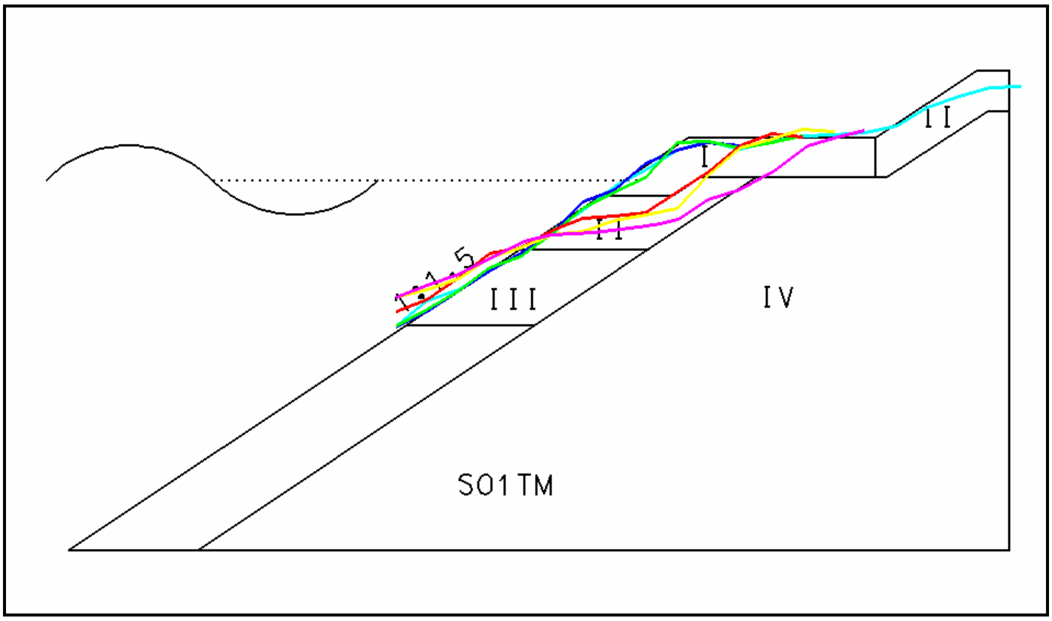

Figure A.7 Damage development of setup 1, water level 0.590m.

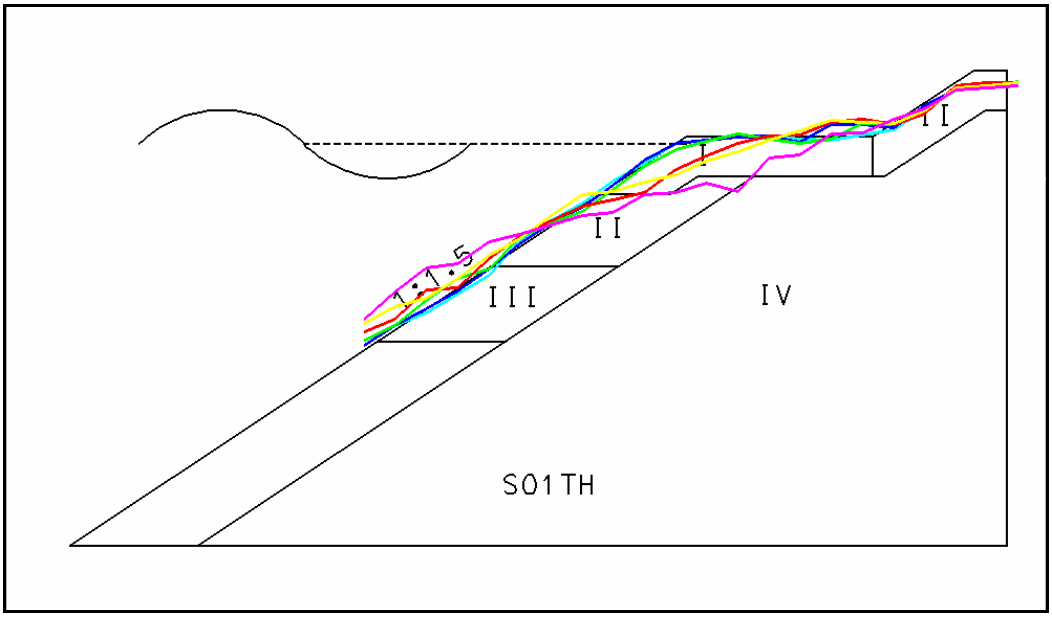

Figure A.8 Damage development of setup 1, water level 0.645m.

A.4.2 Setup 2

For setup 2, two tests were performed with the water level at 0.590m. The reason for this is that in the first tests there was a little problem with the flume that resulted in different wave conditions than originally planned.

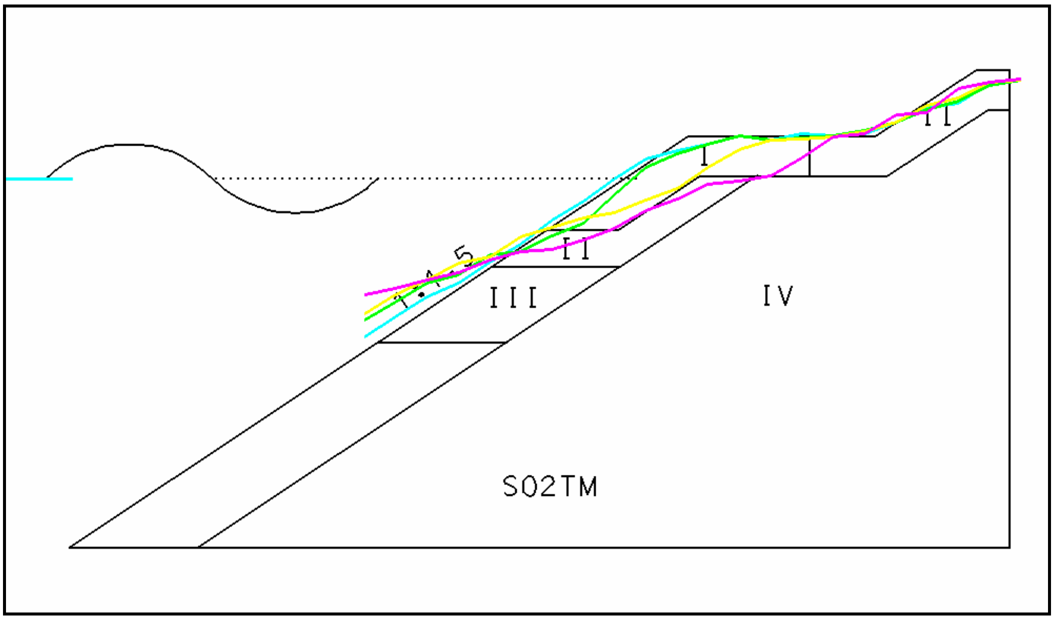

Figure A.9 Damage development of setup 2, water level 0.590m, first test.

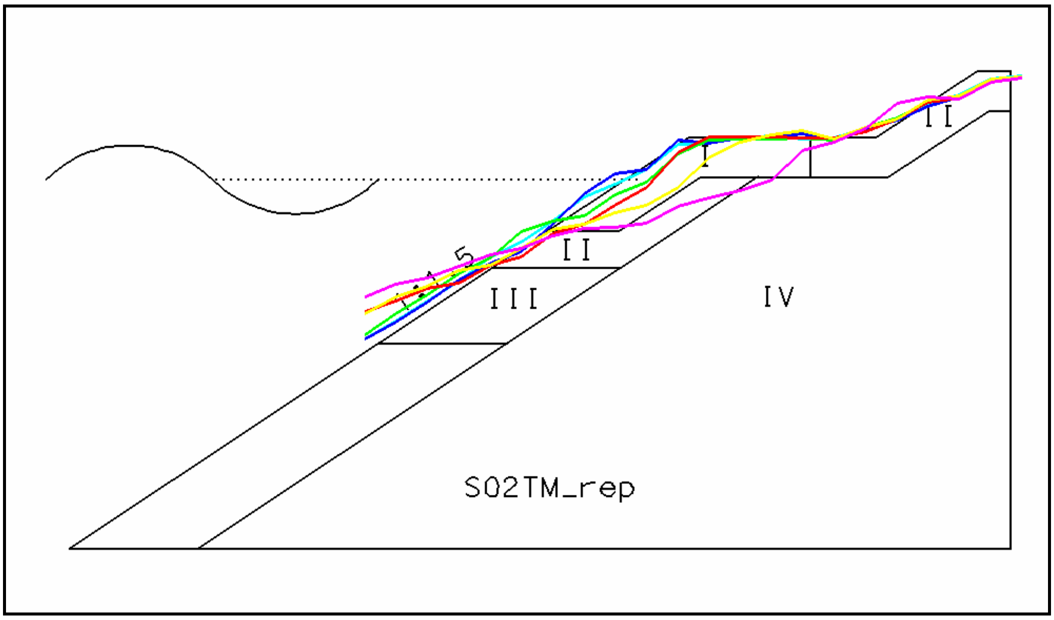

Figure A.10 Damage development of setup 2, water level 0.590m, repeated test.

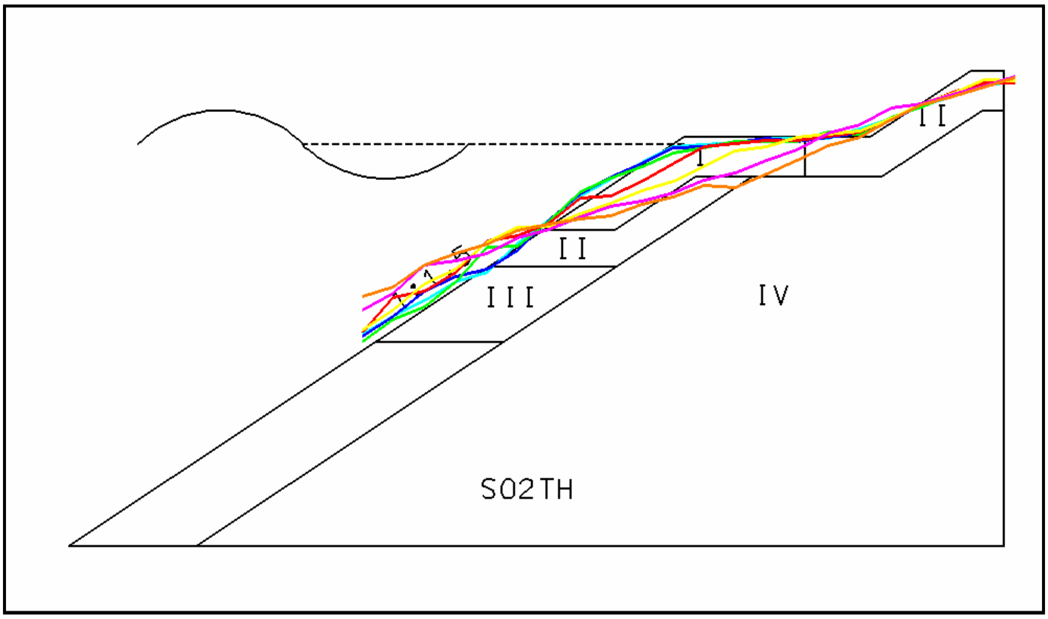

Figure A.11 Damage development of setup 2, water level 0.645m.

A.4.3 Setup 3

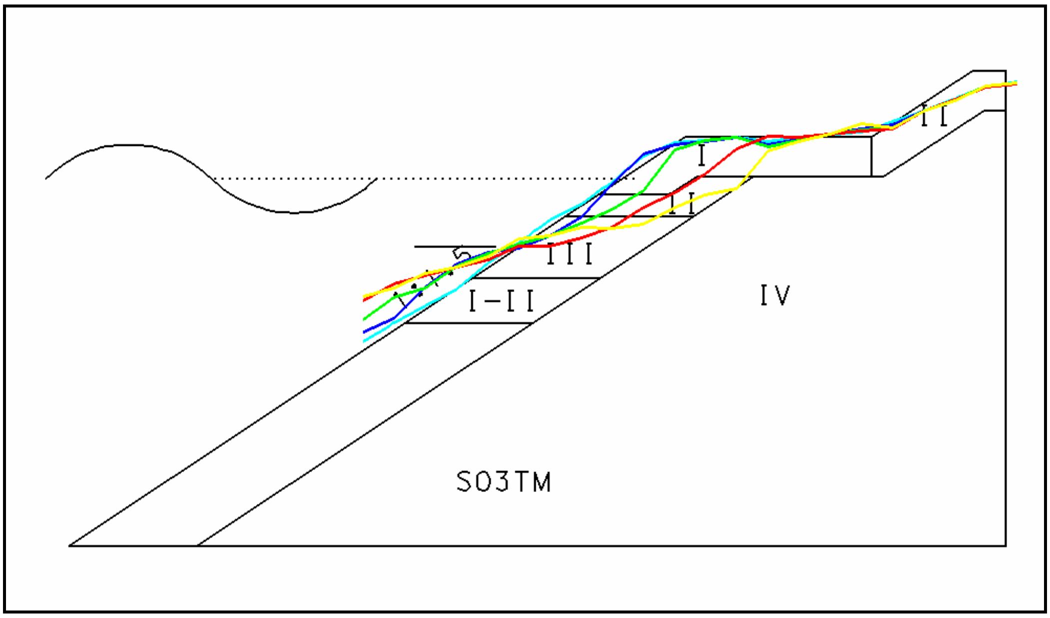

Figure A.12 Damage development of setup 3, water level 0.590m.

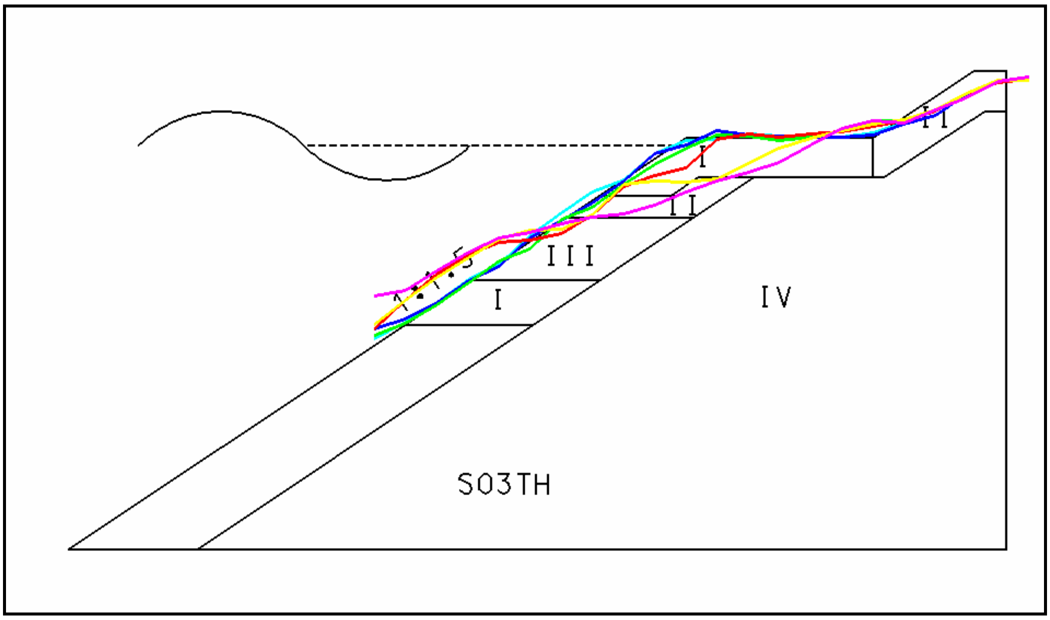

Figure A.13 Damage development of setup 3, water level 0.645m.

A.4.4 Setup 4

For setup 4 as for setup 2, two tests were performed with the water level at 0.590m. The reason in this case was that many stones were moved in early in the test which was not in context with other similar tests performed earlier.

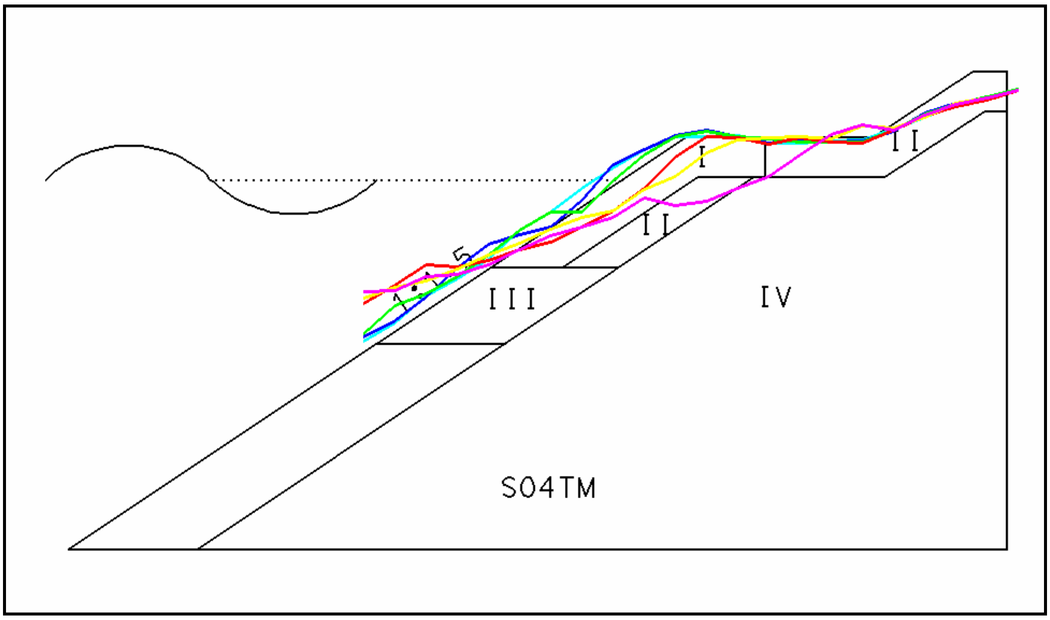

Figure A.14 Damage development of setup 4, water level 0.590m, first test.

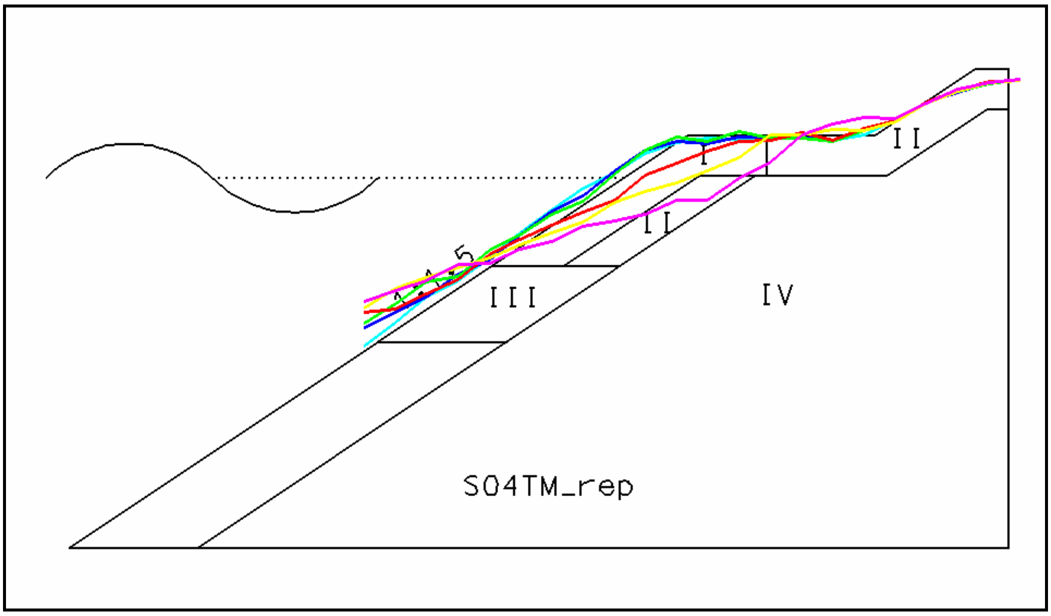

Figure A.15 Damage development of setup 4, water level 0.590m, repeated test.

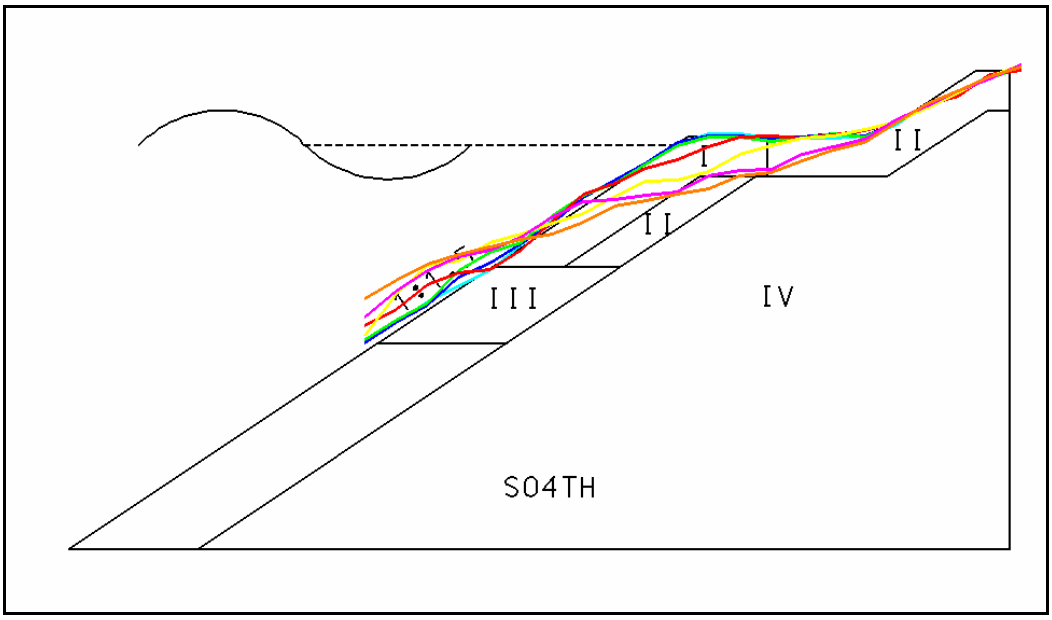

Figure A.16 Damage development of setup 4, water level 0.590m.

A.4.5 Setup 5

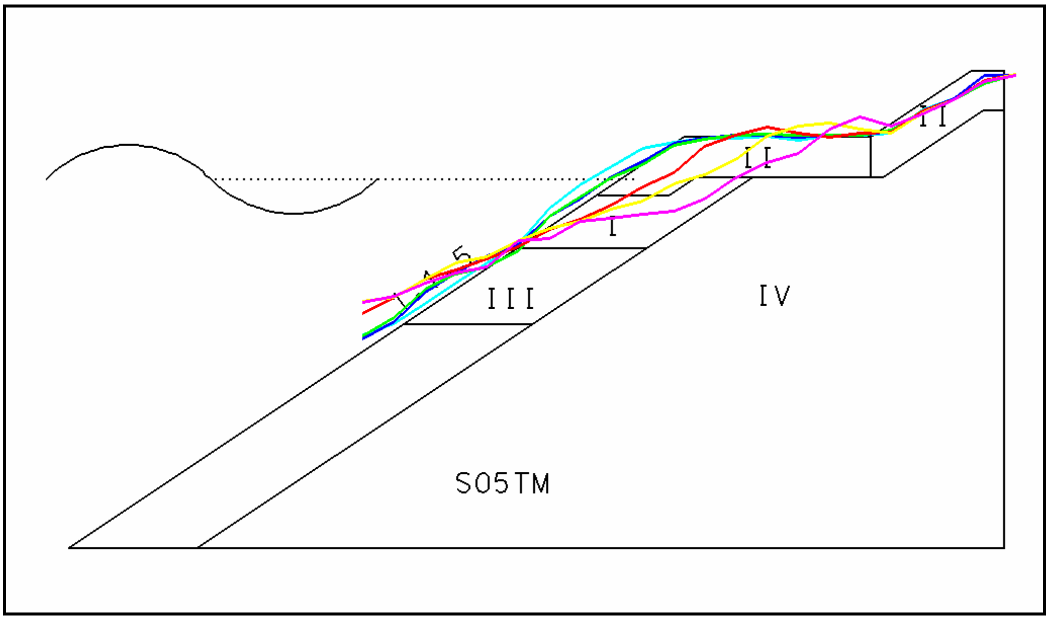

Figure A.17 Damage development of setup 5, water level 0.590m.

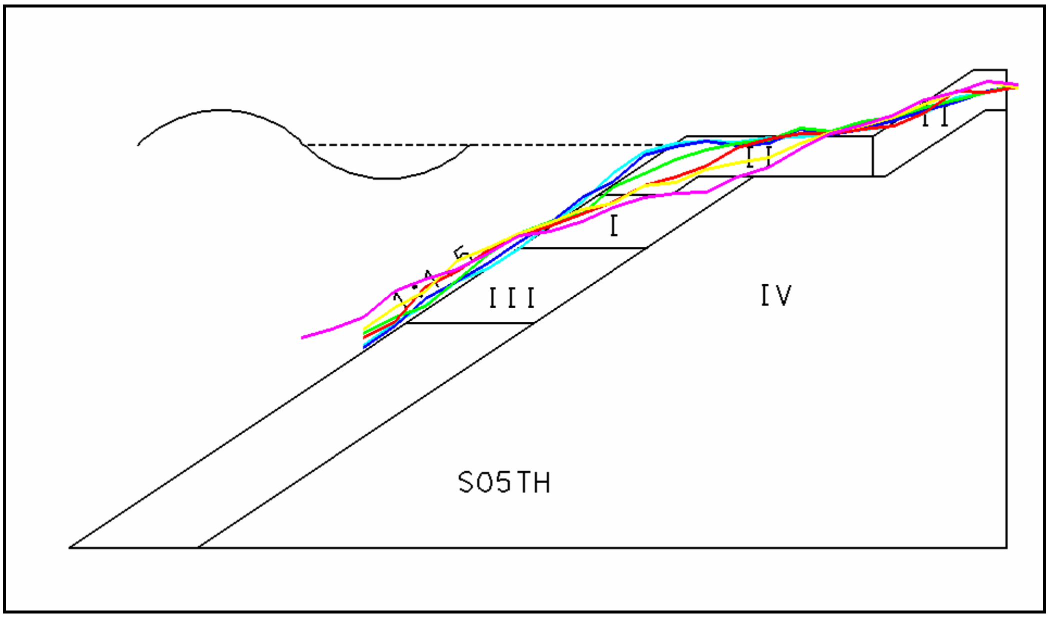

Figure A.18 Damage development of setup 5, water level 0.645m.

A.4.6 Setup 6

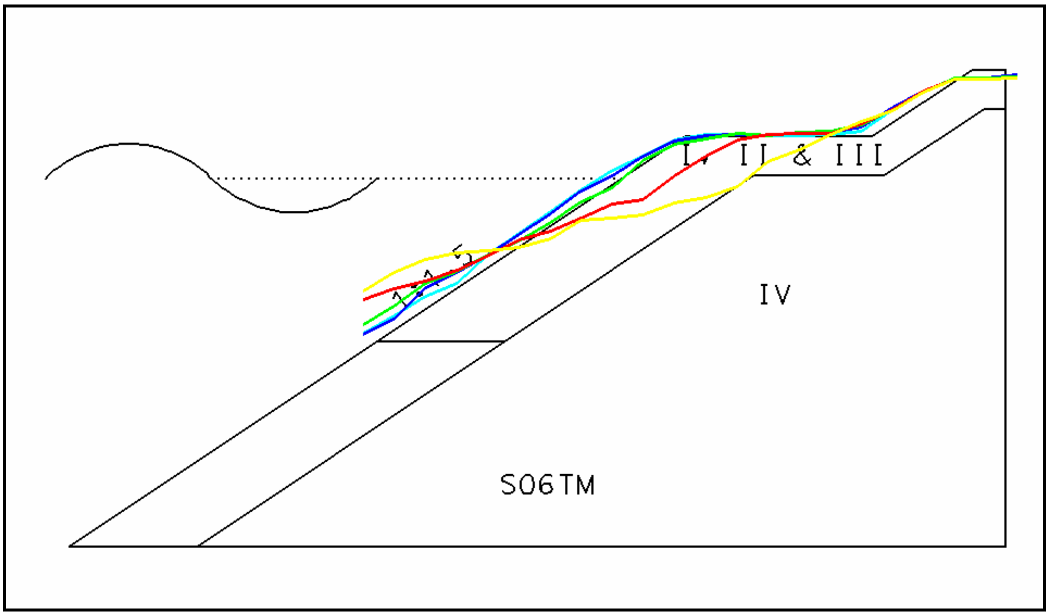

Figure A.19 Damage development of setup 6, water level 0.590m.

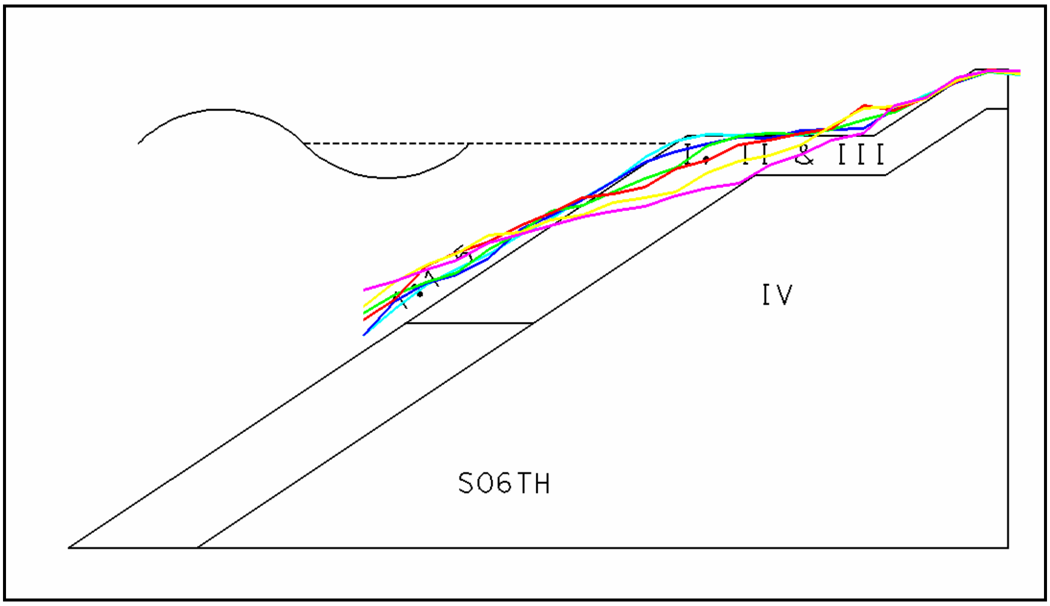

Figure A.20 Damage development of setup 6, water level 0.645m.

A.5 Recession graphs

For all recession graphs TH stands for the highest water level, 0.645m, TM stands for the medium water level, 0.590m, while TL stands for the lowest water level tested, 0.545m, that was only used for one test with setup 1.

A.5.1 Comparison with Tørum

In the following two figures, Figure A.21 and Figure A.22 the results from the recession measurements of the tests are compared to the formula of Tørum. Figure A.21 compares the results of setup 6, which is with the mixture of all stone classes and the formula of Tørum for the same material, values of D_{n50} both in Rec/D_{n50} and H_0T_0 are from the mixture. Figure A.22 however compares the results of setup 4, which should be close to being a representative for Class I, with the formula of Tørum for the same material, values D_{n50} both in Rec/D_{n50} and H_0T_0 are from the Class I stones.

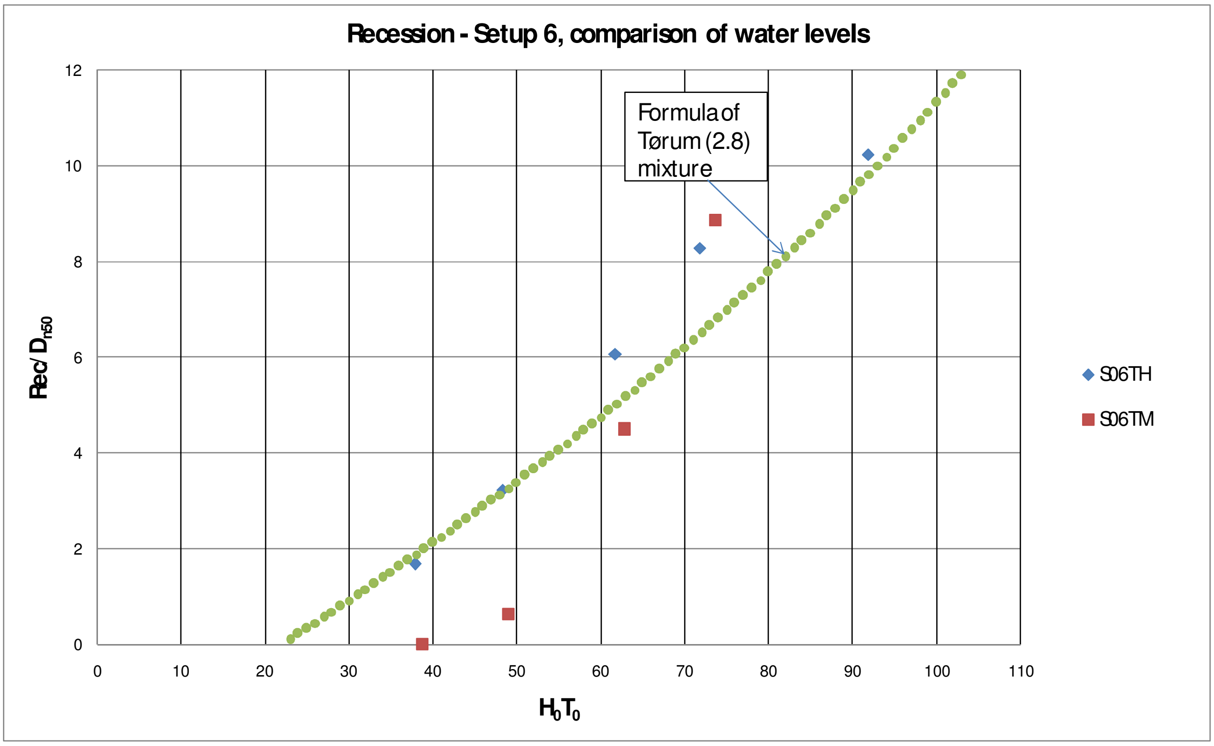

Figure A.21 Recession, setup 6 compared to the modified formula of Tørum for mixed stones

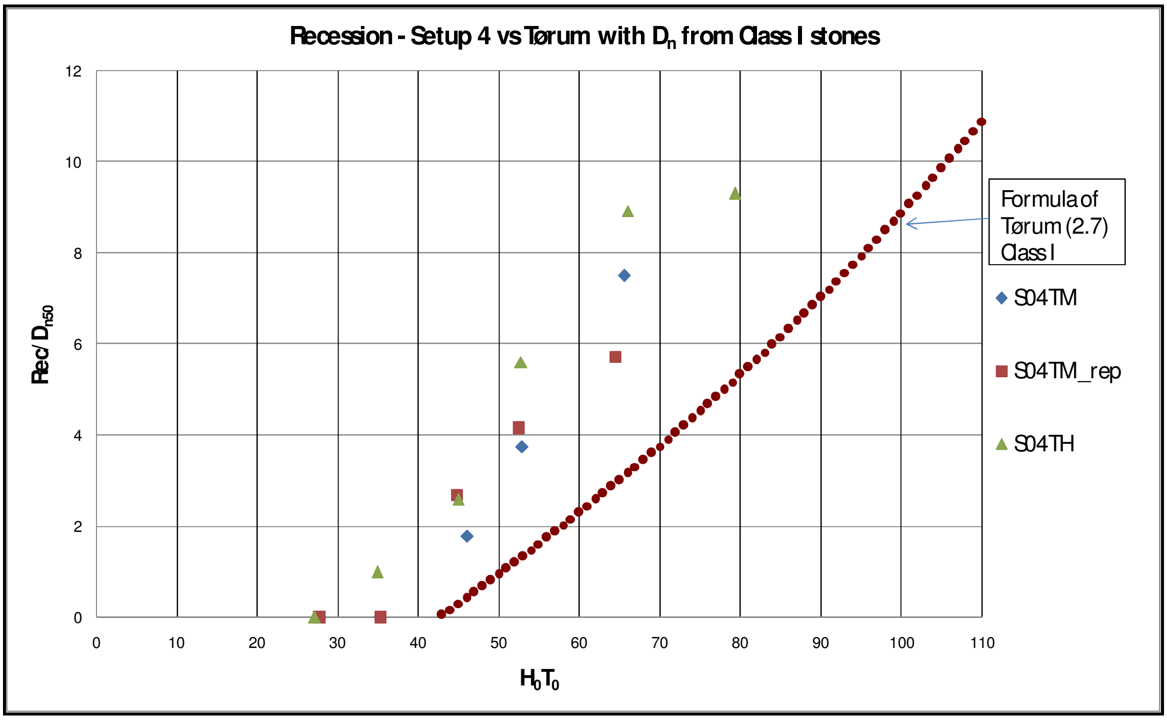

Figure A.22 Recession, setup 4 compared to the modified formula of Tørum for Class I stones

A.5.2 Influence of berm height on recession on the edge of berm

The following graphs are recession measurements for different setups where the water levels are compared. All stone parameters for all setups are taken from the mixed stone sample.

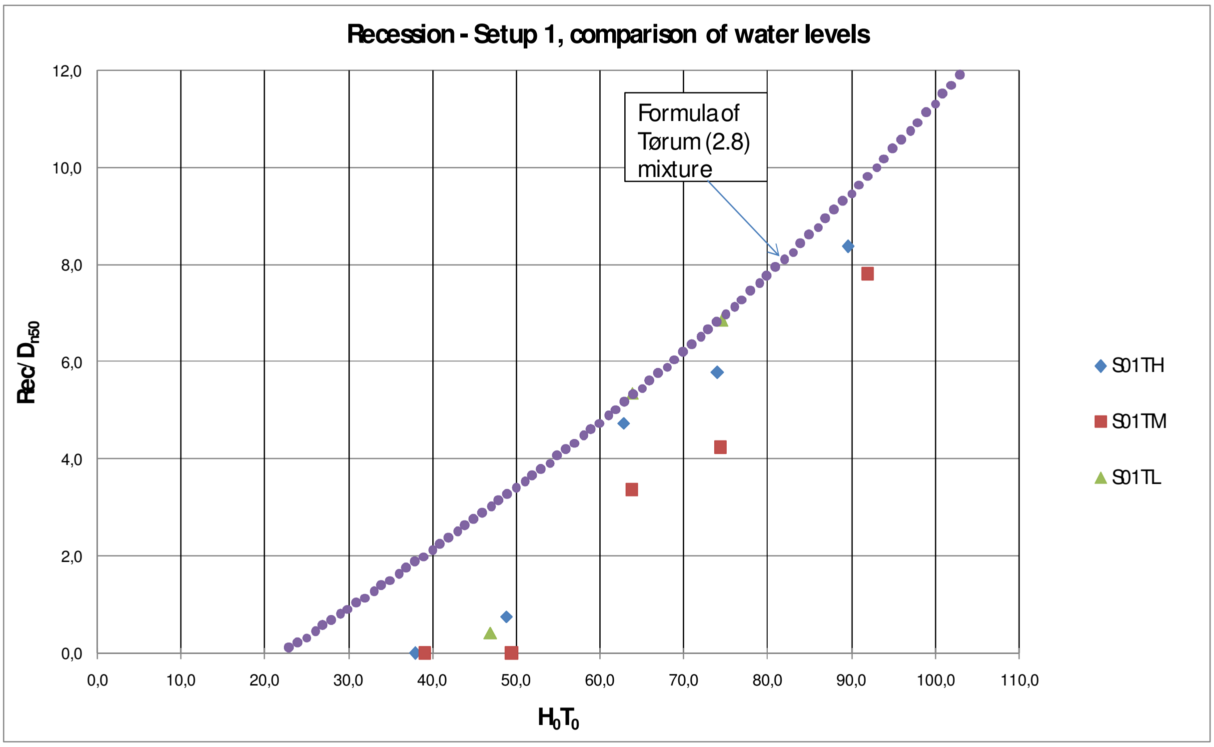

Figure A.23 Recession, comparison of different water levels, setup 1

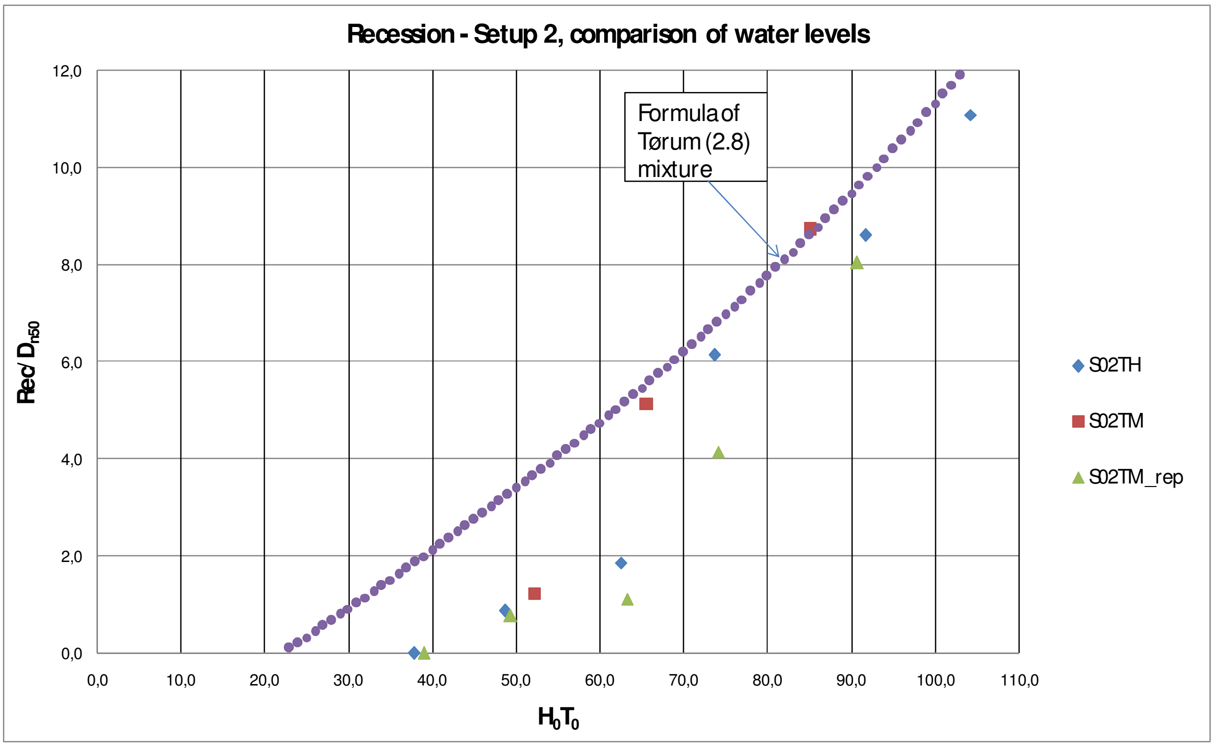

Figure A.24 Recession, comparison of different water levels, setup 2

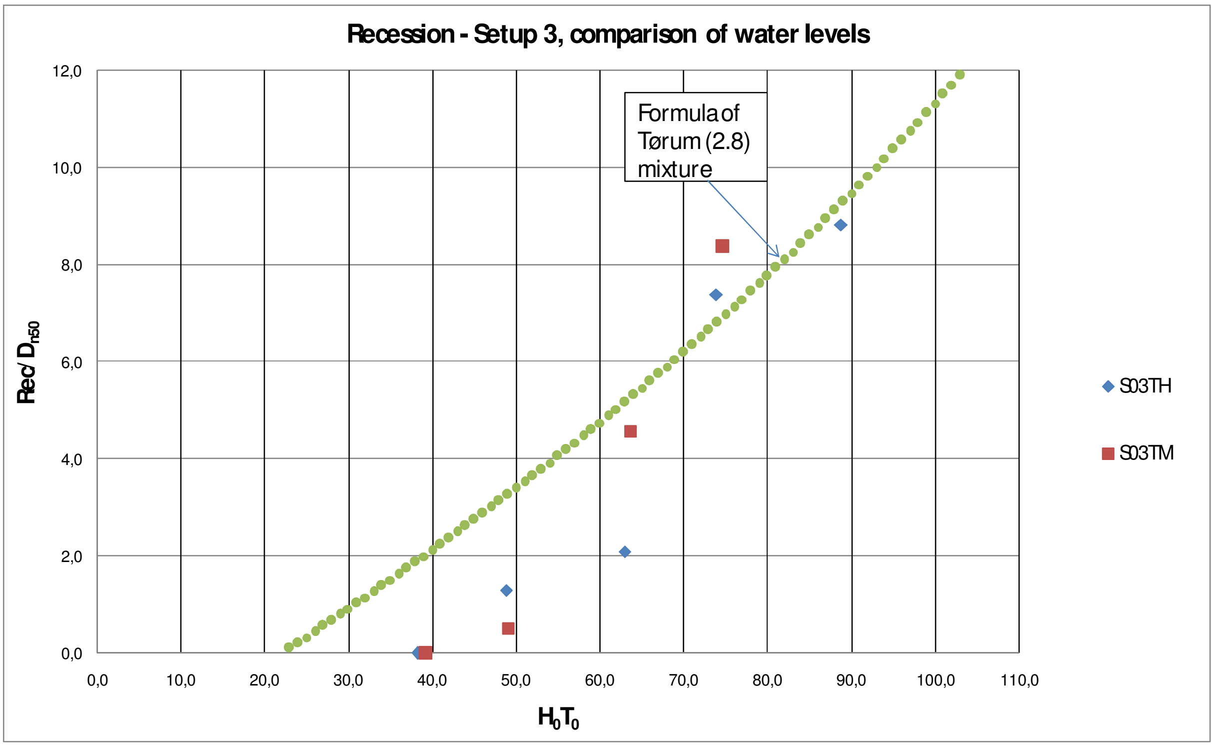

Figure A.25 Recession, comparison of different water levels, setup 3

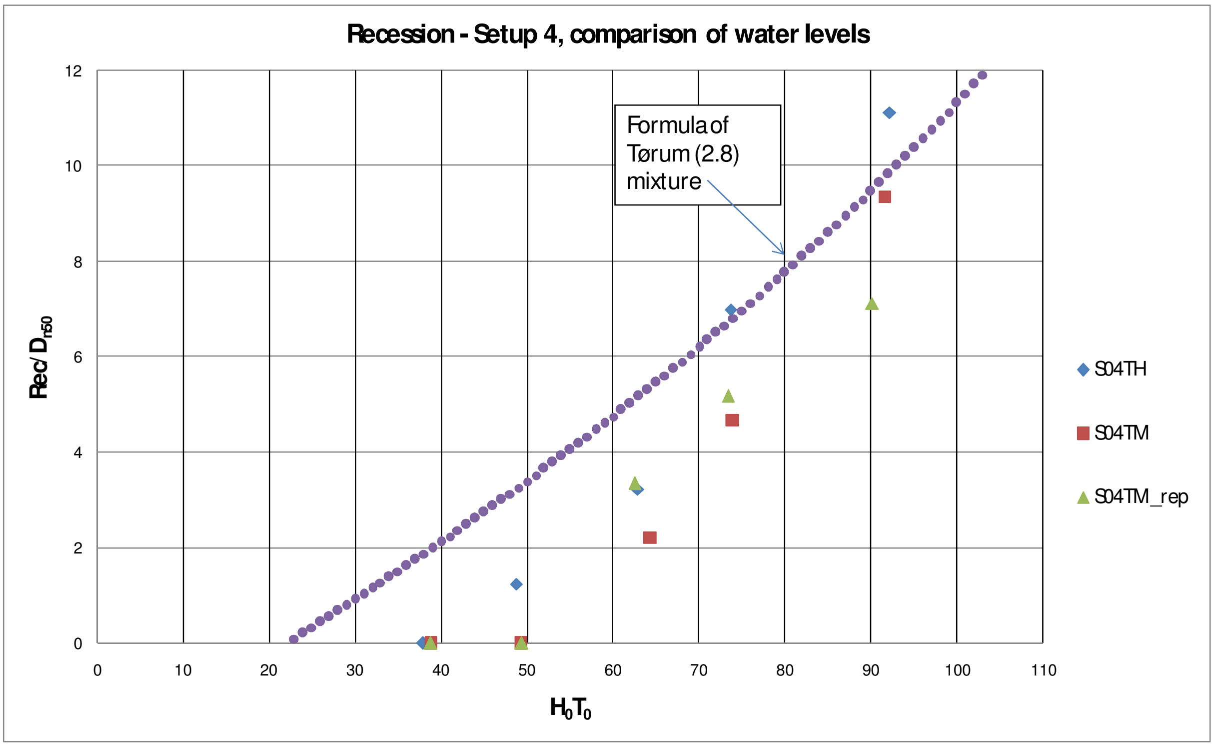

Figure A.26 Recession, comparison of different water levels, setup 4

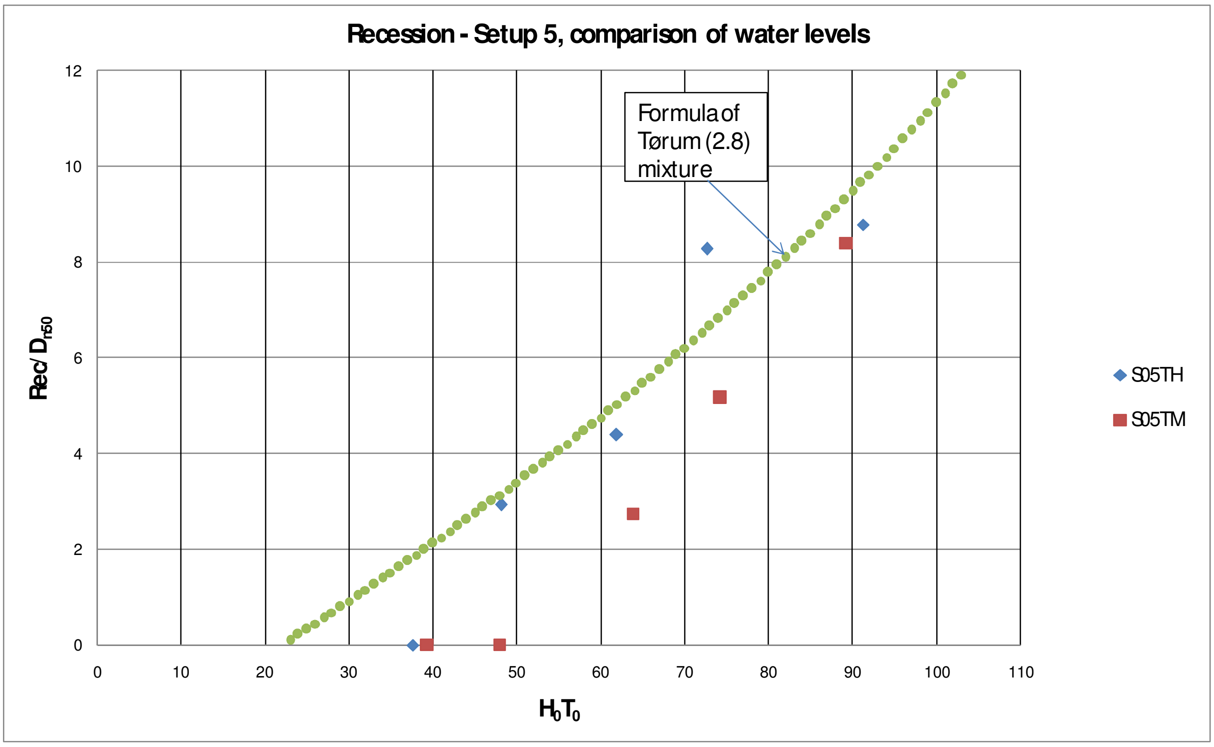

Figure A.27 Recession, comparison of different water levels, setup 5

A.5.3 Comparison of different setups

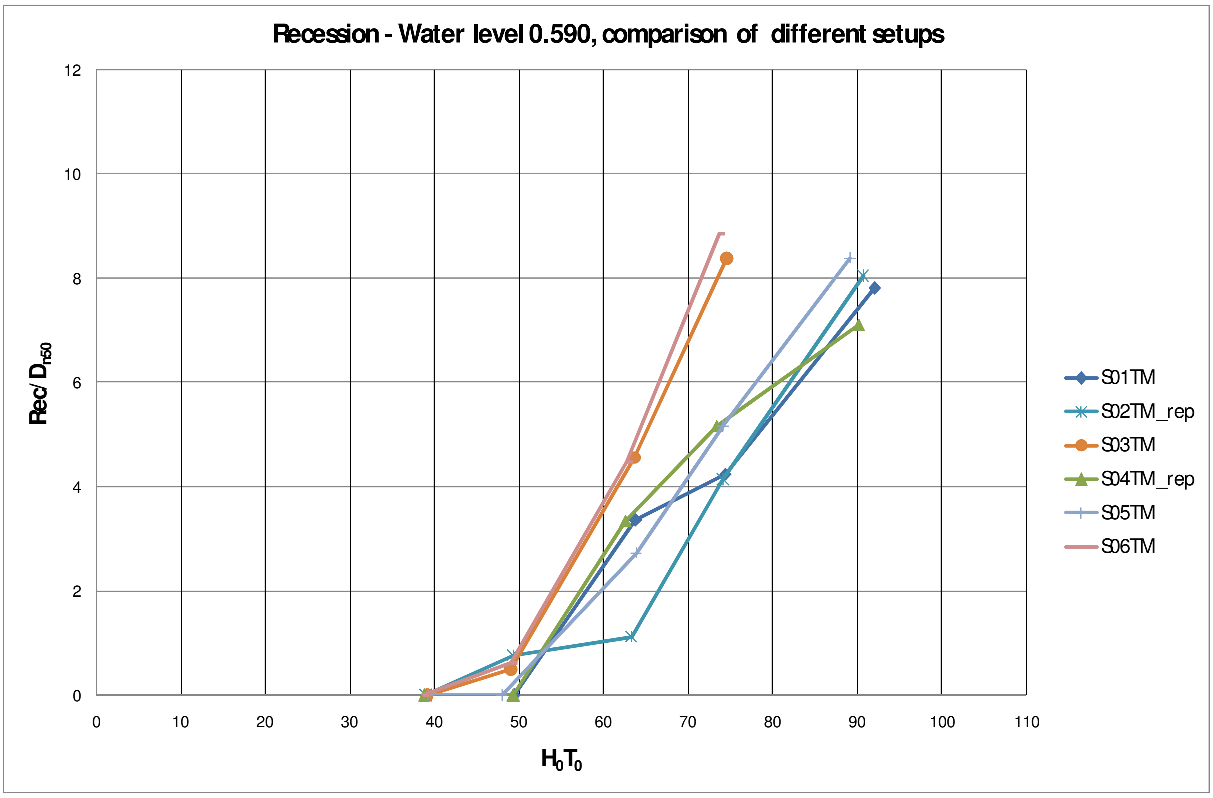

Figure A.28 Recession, comparison of different setups, water level 0.590m

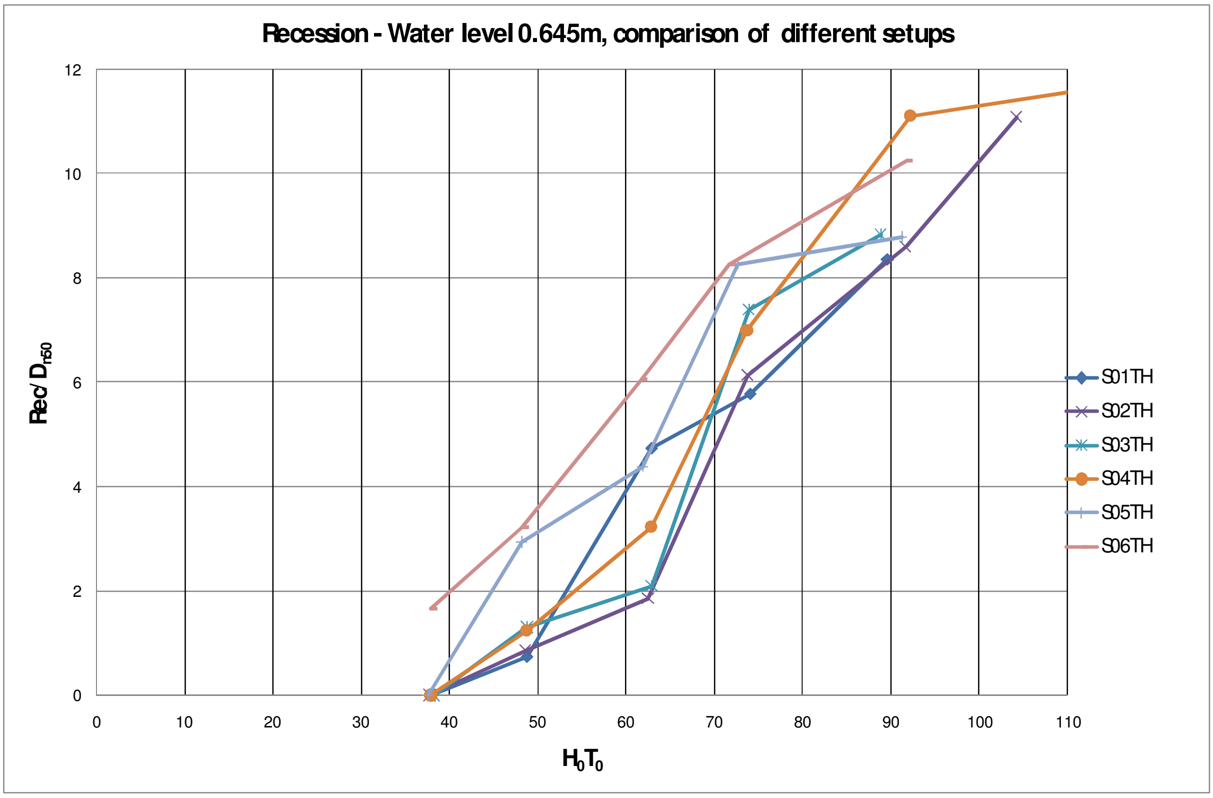

Figure A.29 Recession, comparison of different setups, water level 0.645m

A.5.4 Uncertainties

To estimate uncertainties of tests, the tests that were repeated are looked at.

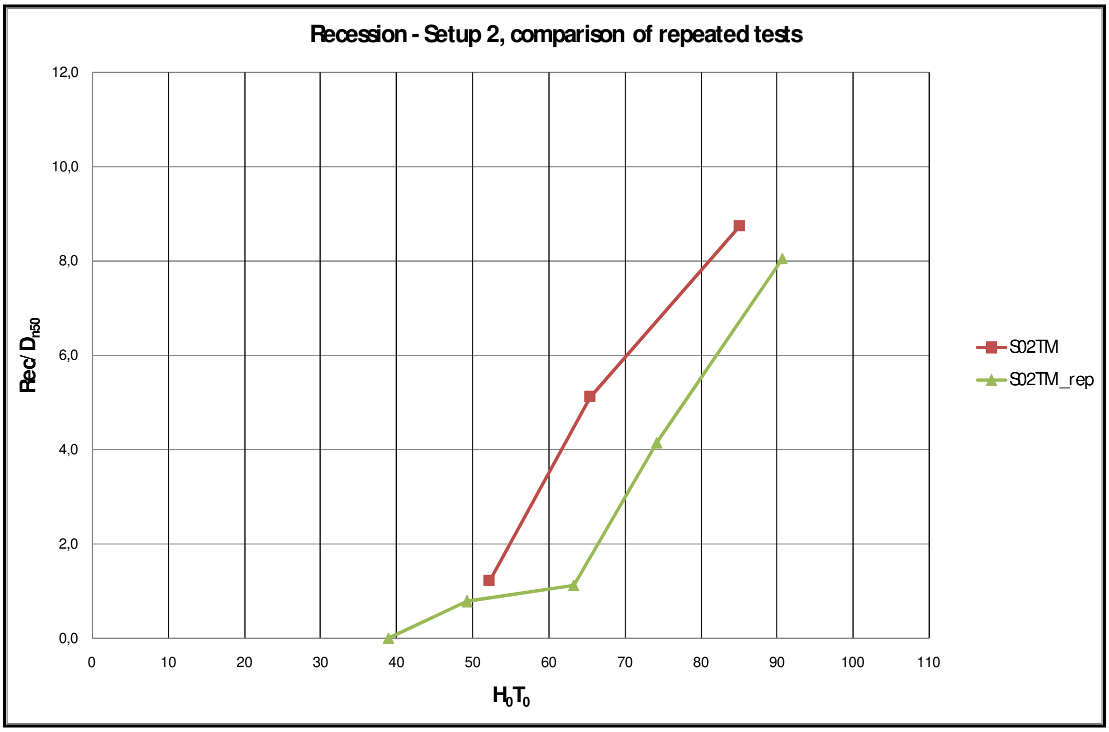

Figure A.30 Recession, comparison of original and repeated test of setup 2, with water level 0.590m

In the following figure, Figure A.31, H_0T_0 is replaced by H_0 as the value on the x-axis.

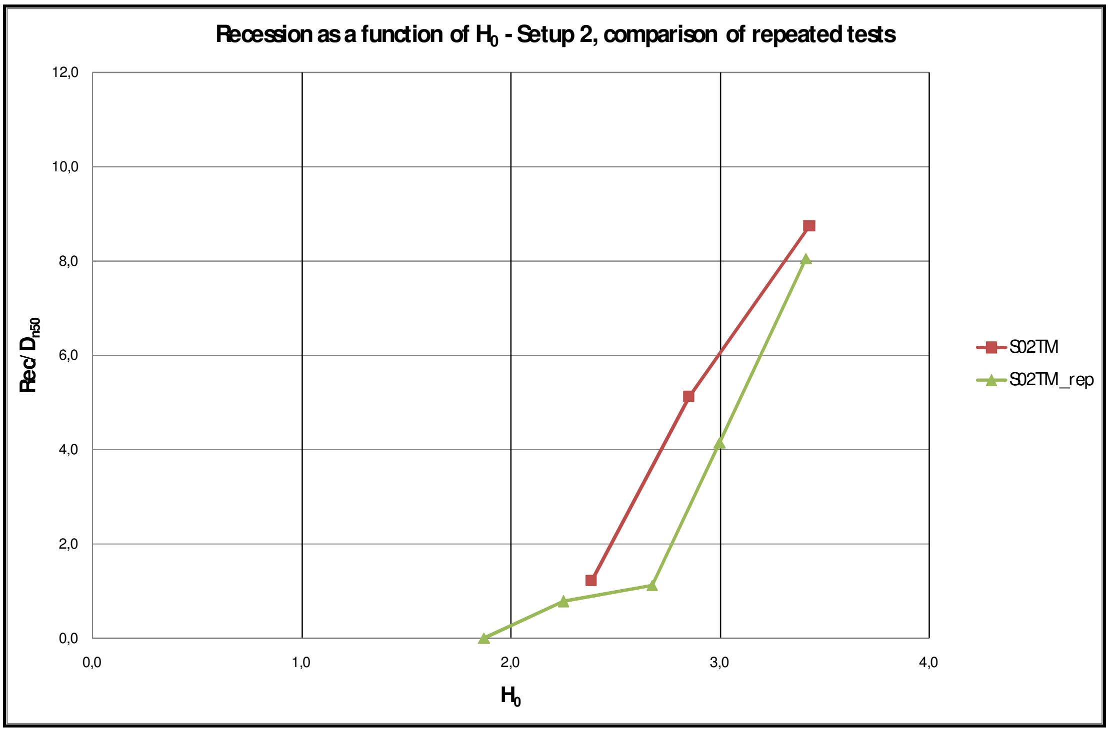

Figure A.31 Recession, comparison of the original and the repeated test of setup 2, with water level 0.590m. H_0 used instead of H_0T_0

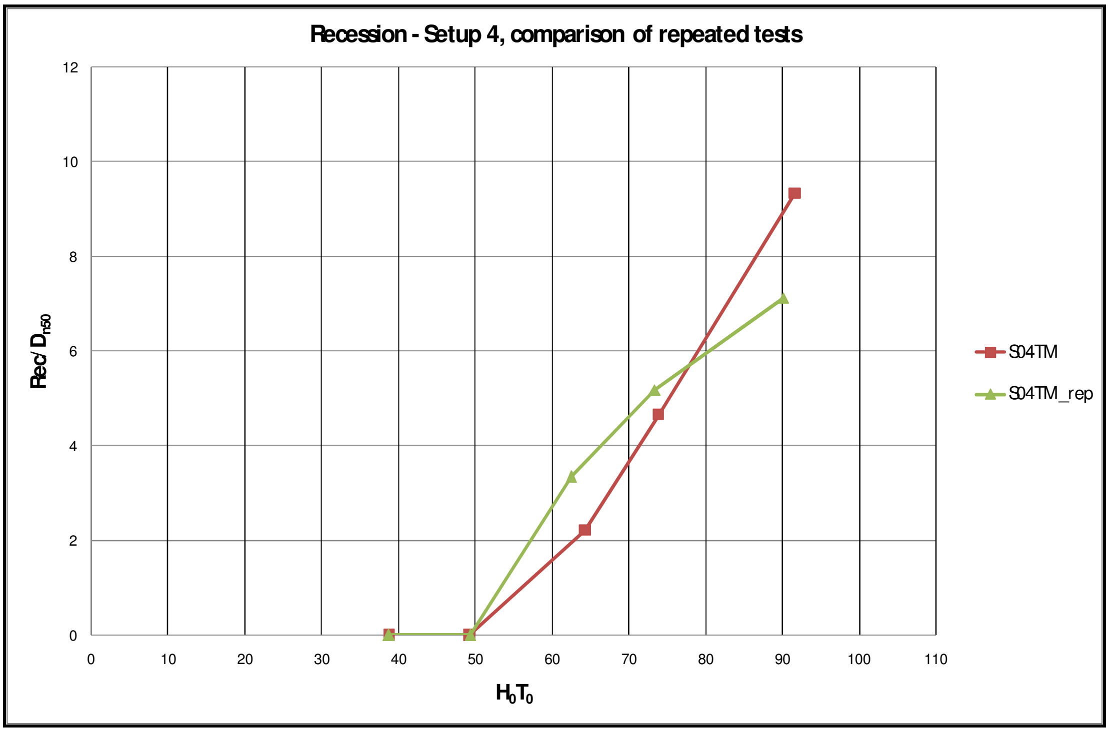

Figure A.32 Recession, comparison of original and repeated test of setup 4, with water level 0.590m

A.6 Damage number – graphs

For all the damage development graphs, the stone diameters are taken from the sample of mixed stones.

A.6.1 Influence of berm height on damage development

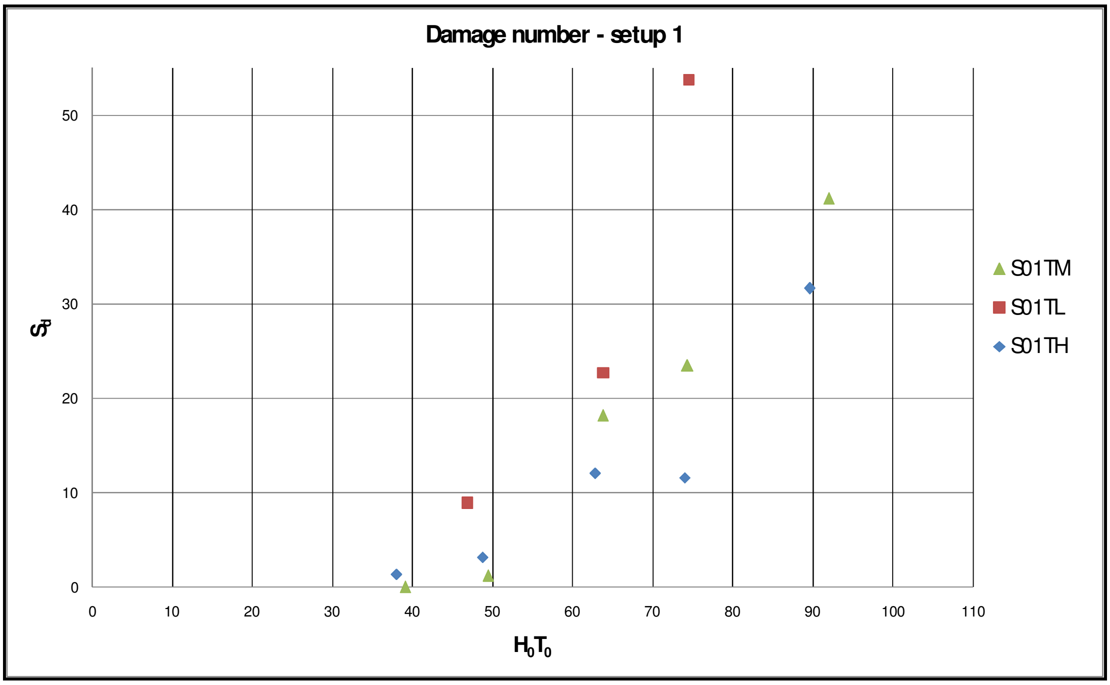

Figure A.33 Damage number, comparison of different water levels for setup 1

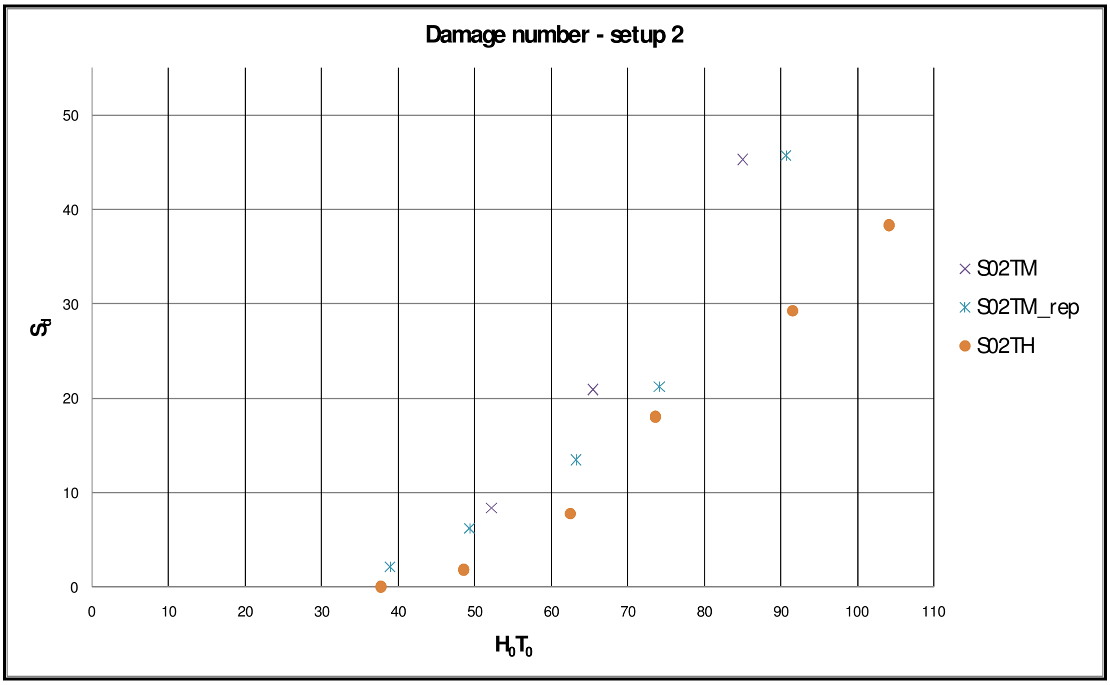

Figure A.34 Damage number, comparison of different water levels for setup 2

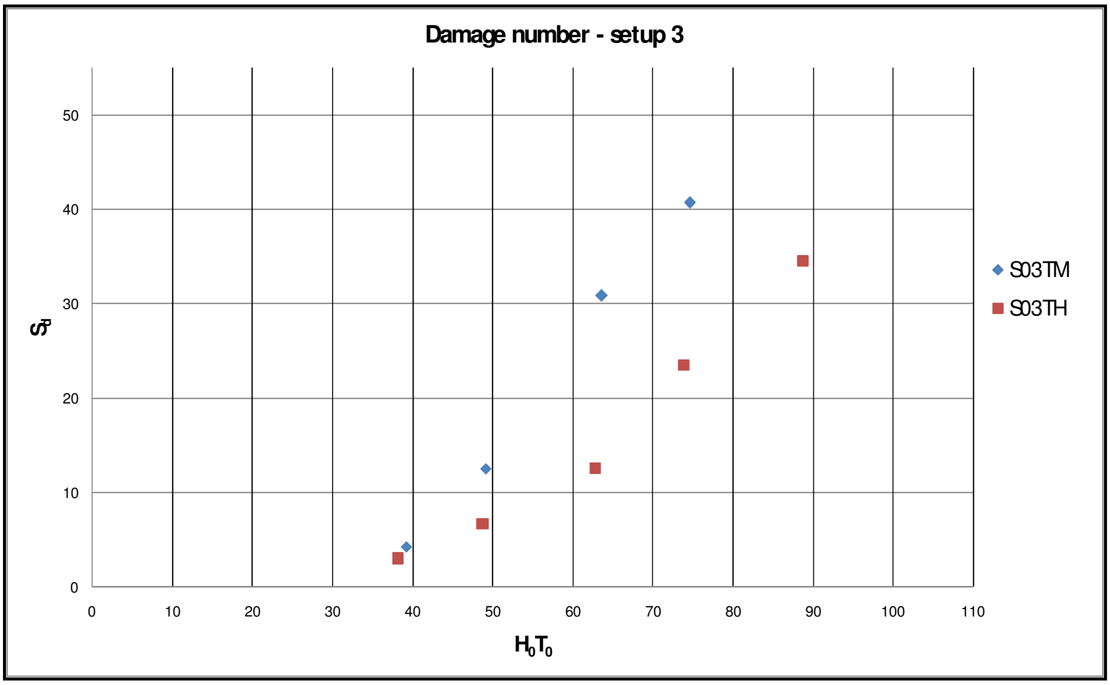

Figure A.35 Damage number, comparison of different water levels for setup 3

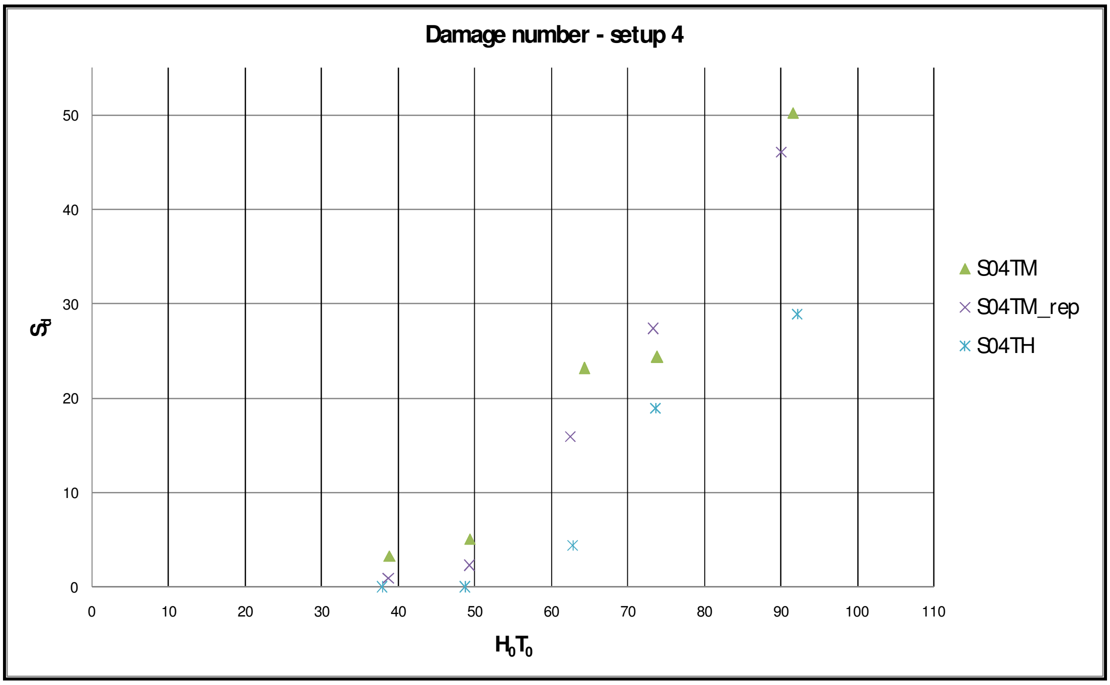

Figure A.36 Damage number, comparison of different water levels for setup 4

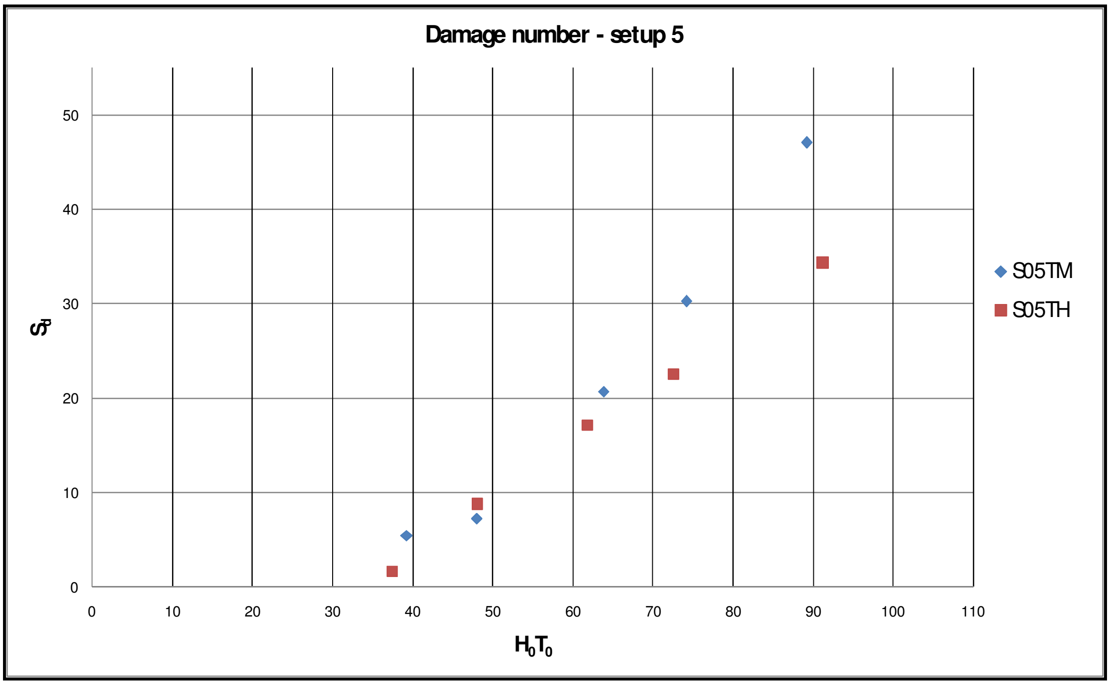

Figure A.37 Damage number, comparison of different water levels for setup 5

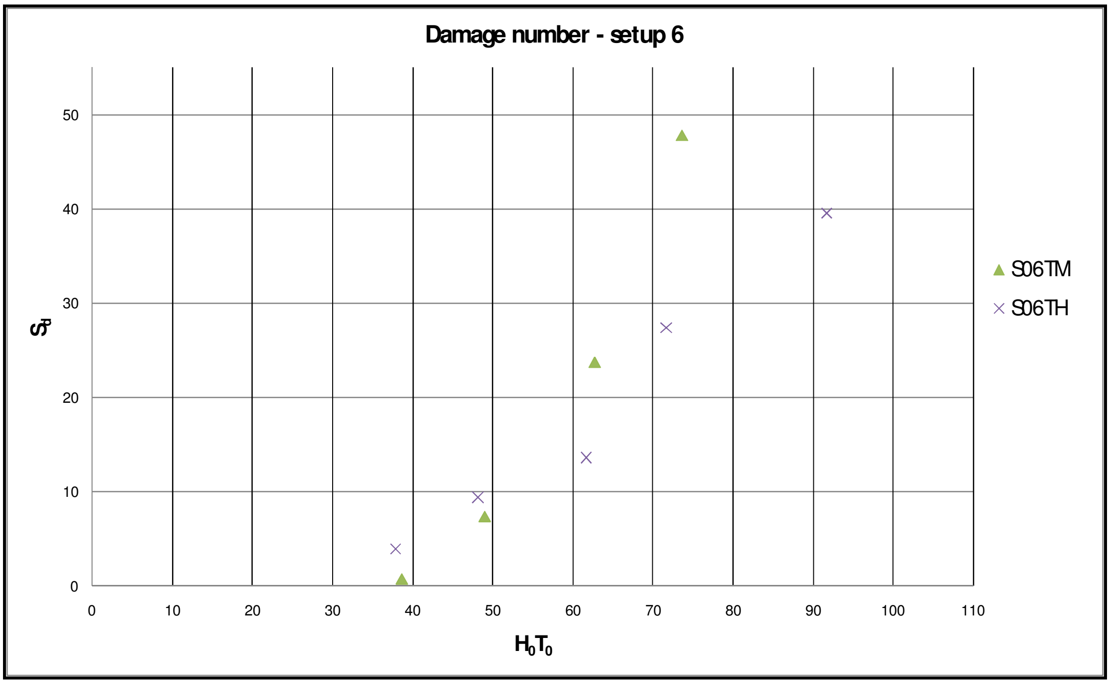

Figure A.38 Damage number, comparison of different water levels for setup 6

A.6.2 Comparison of different setups

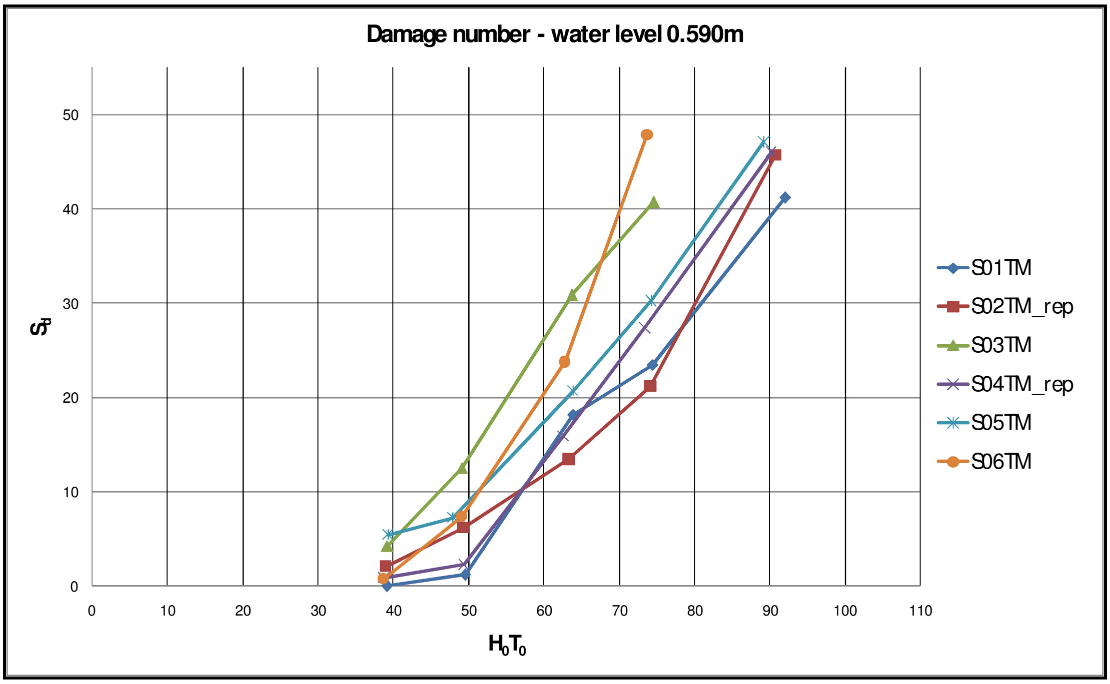

Figure A.39 Damage number, comparison of different setups with water level 0.590m

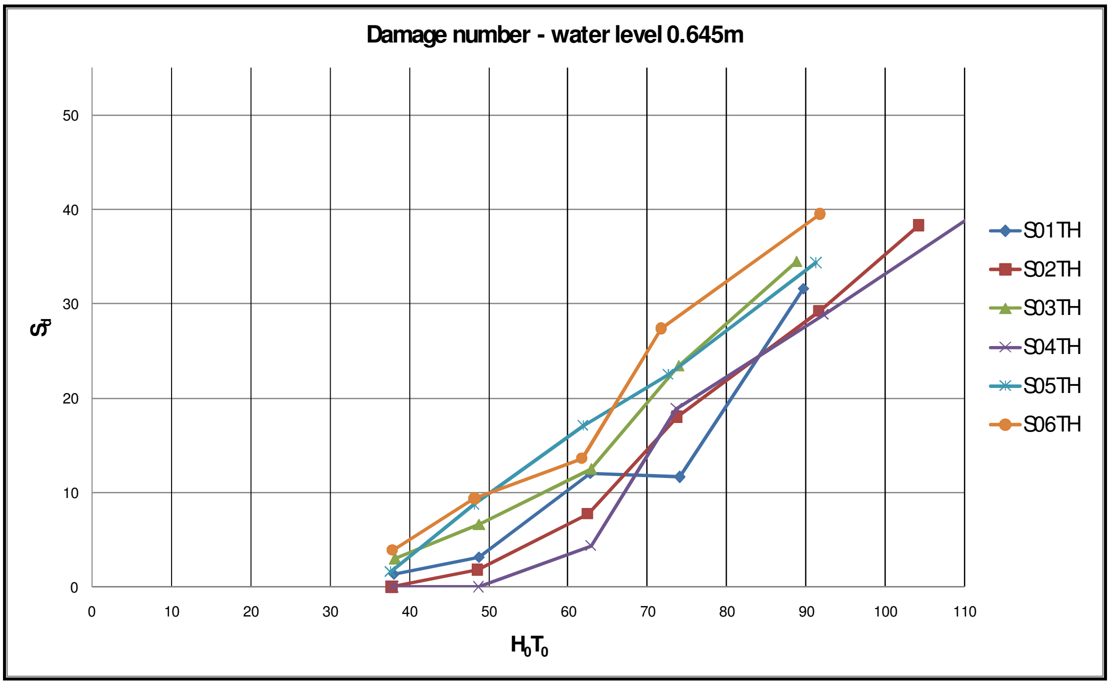

Figure A.40 Damage number, comparison of different setups with water level 0.645m

A.6.3 Uncertainties

To estimate uncertainties of tests, those tests that were repeated are looked at.

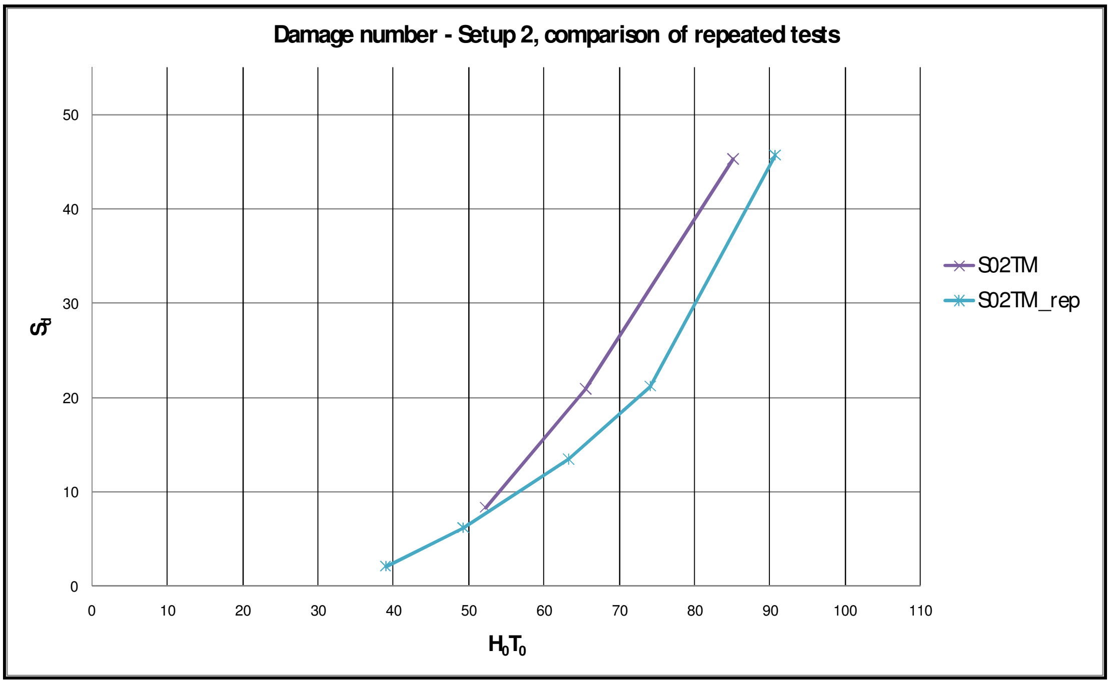

Figure A.41 Damage number, comparison of original and repeated test of setup 2, with water level 0.590m

In the following figure, Figure A.42, H_0T_0 is replaced by H_0 as the value on the x-axis.

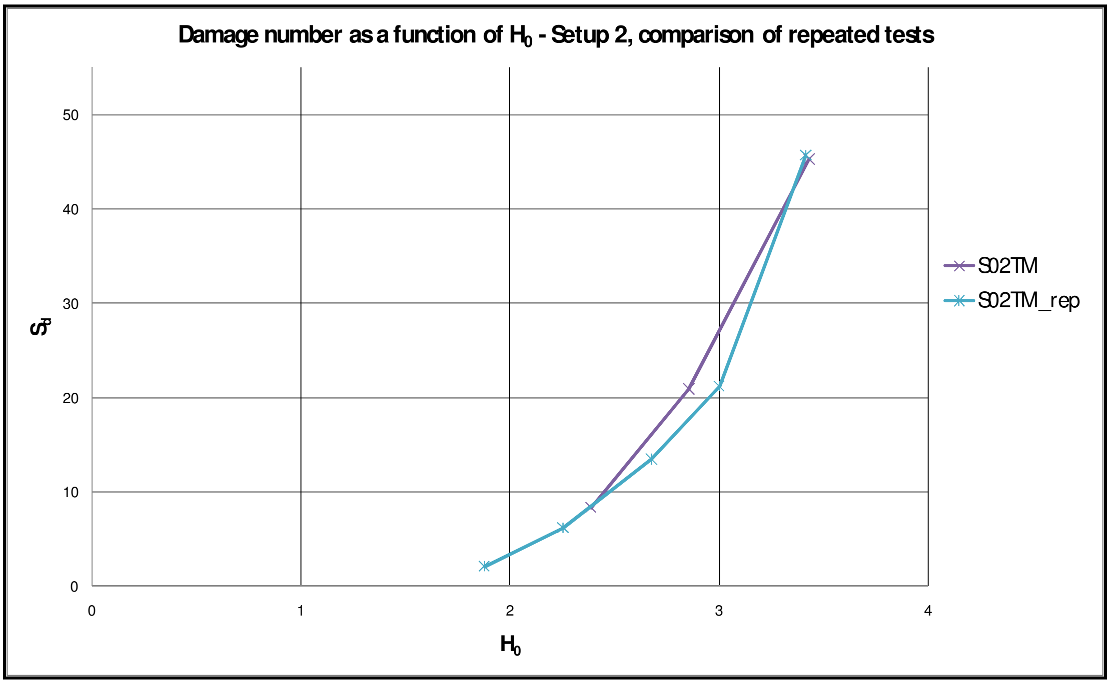

Figure A.42 Damage number, comparison of the original and the repeated test of setup 2, with water level 0.590m. H_0 used instead of H_0T_0

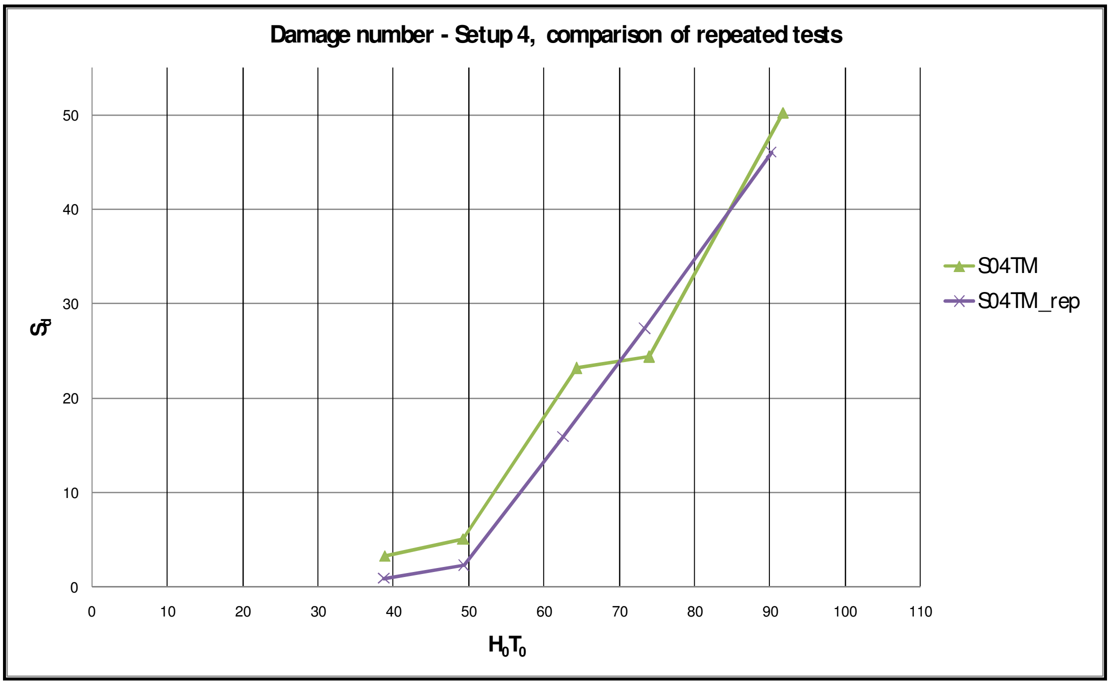

Figure A.43 Damage number, comparison of original and repeated test of setup 4, with water level 0.590m

A.7 Transition of original and reshaped profile, h_f

For test setups 1-5 the median nominal diameter, D_{n50}, of Class I stones was used while for setup 6 the values from the mixed sample was used.

A.7.1 Influence of distance from berm level to water level, h_B, on h_f

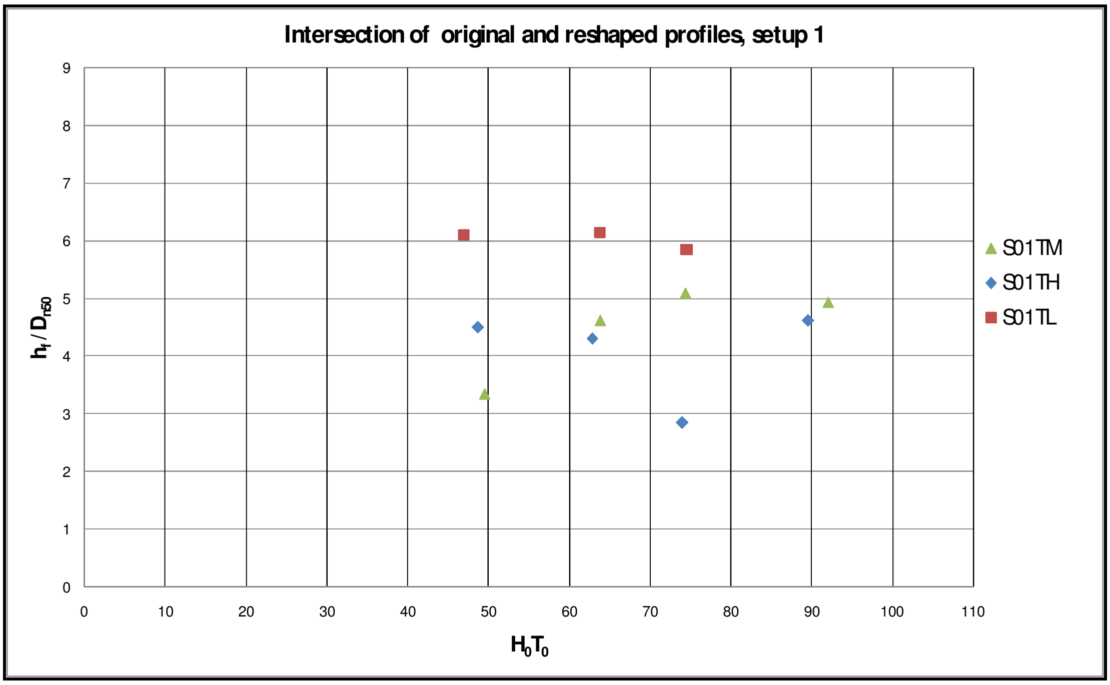

Figure A.44 Transition of original and reshaped profile, h_f, setup 1

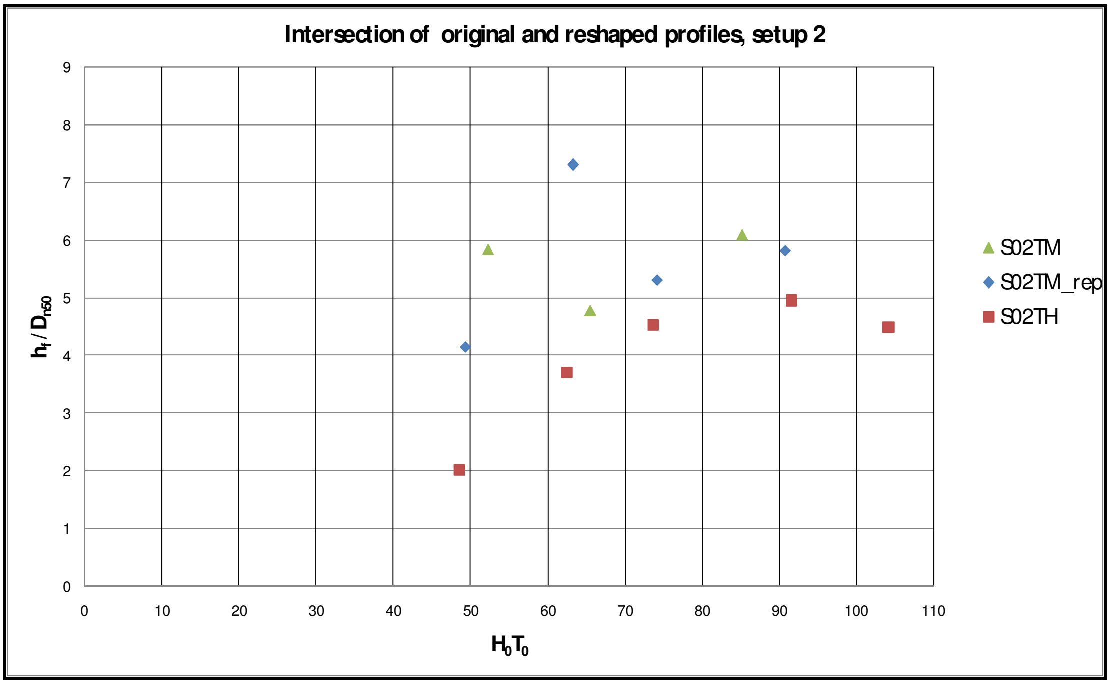

Figure A.45 Transition of original and reshaped profile, h_f, setup 2

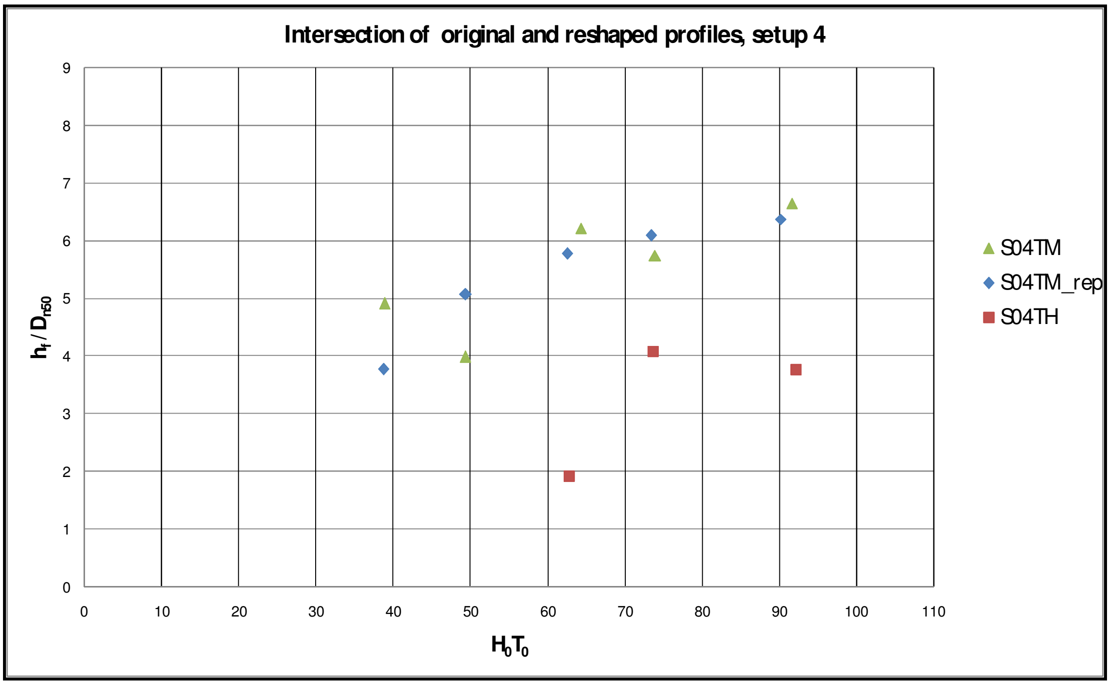

Figure A.46 Transition of original and reshaped profile, h_f, setup 4

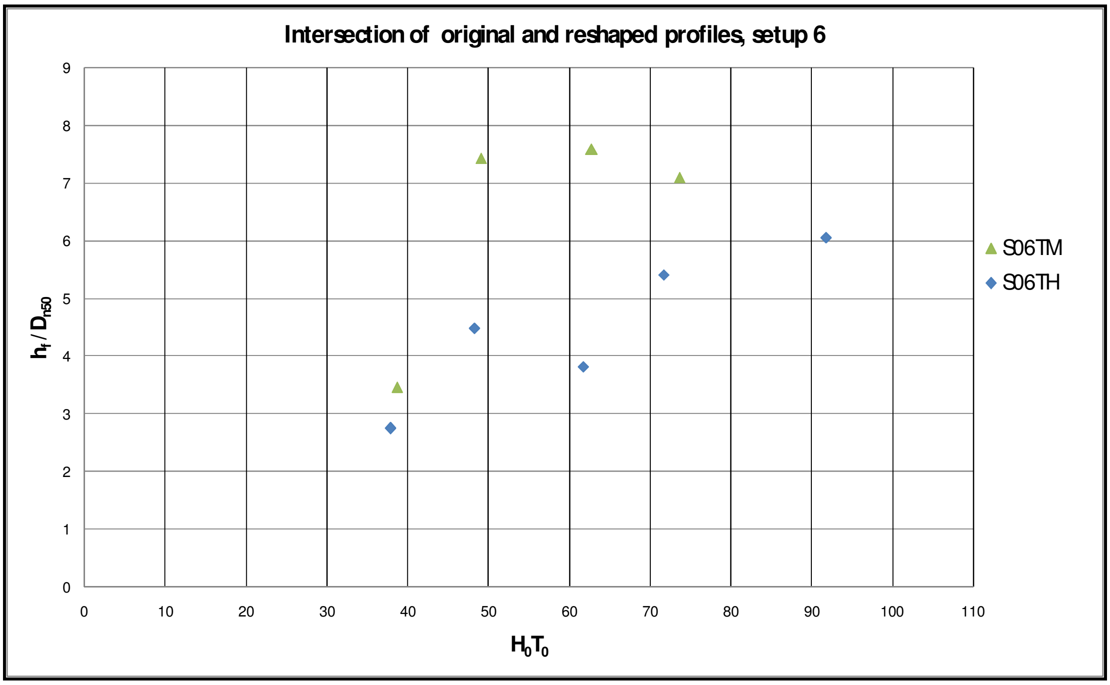

Figure A.47 Transition of original and reshaped profile, h_f, setup 6

A.7.2 Comparison of different setups

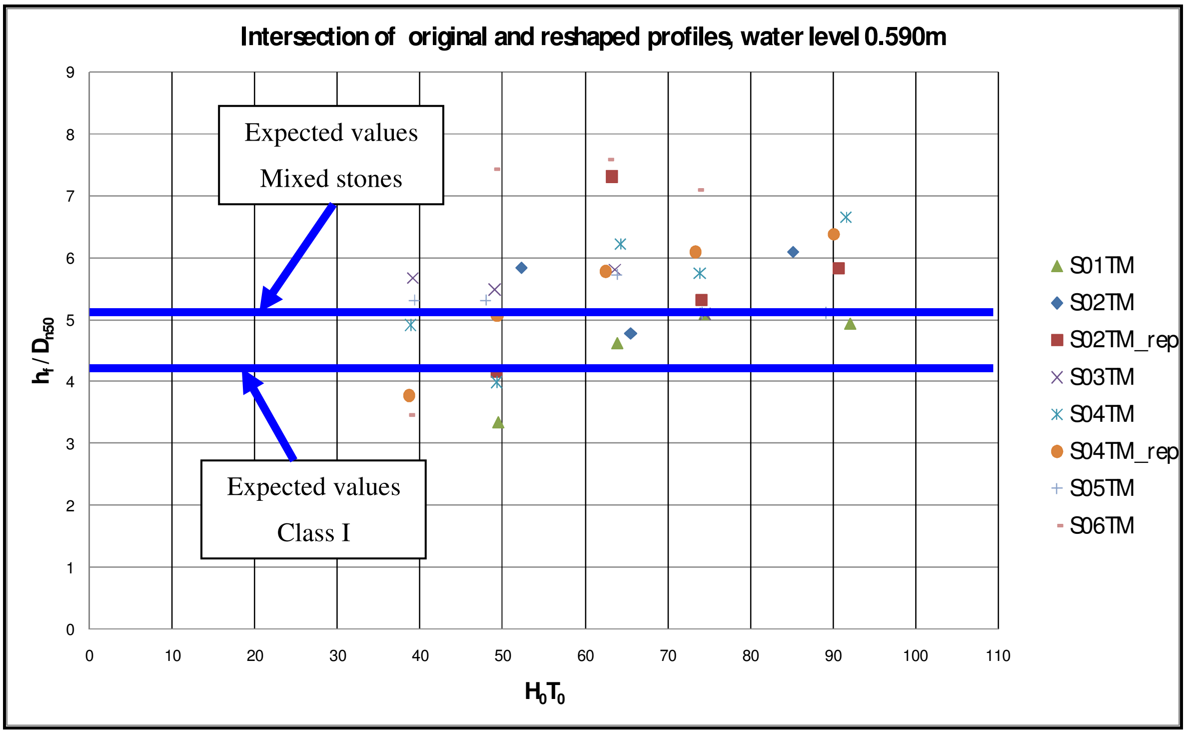

Figure A.48 Transition of original and reshaped profile, hf, water level 0.590m

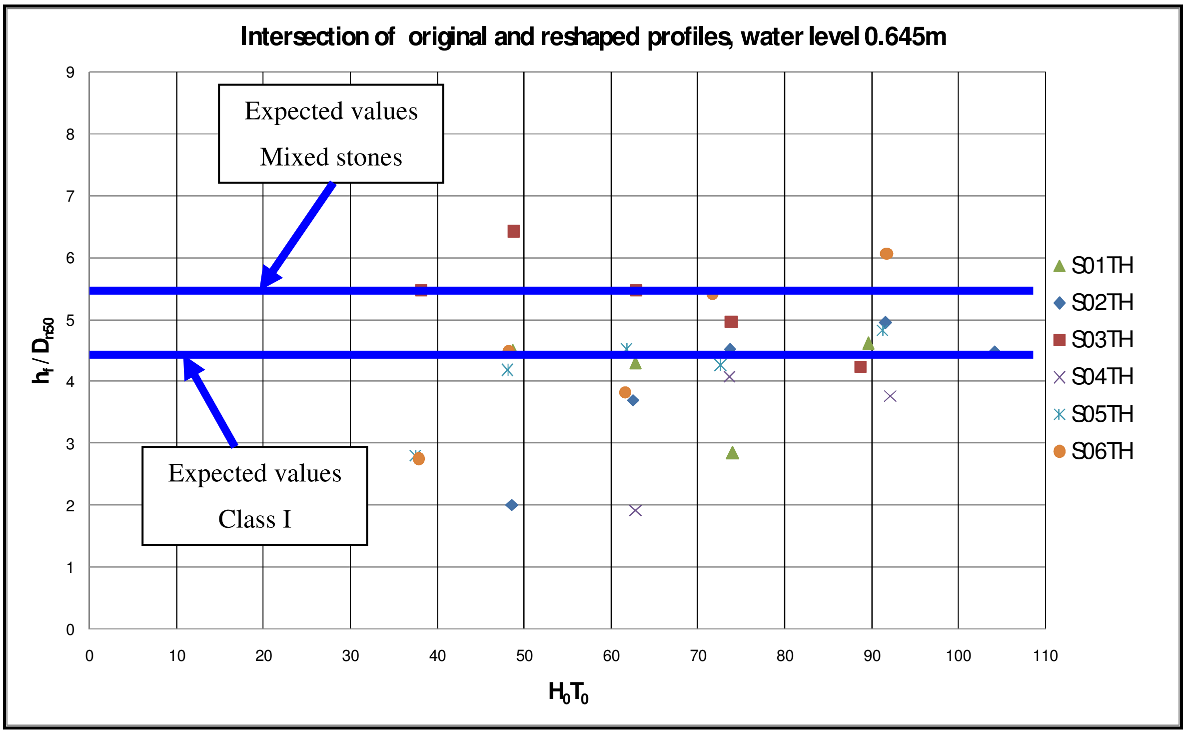

Figure A.49 Transition of original and reshaped profile, h_f, water level 0.645m

A.8 Wave spectra

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

A sea-state can be described with a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency, this is called a wave spectrum. Two of the most widely used spectra are those described by Pierson & Moskowitz (1964) and the JONSWAP spectrum, Hasselmann et al. (1973), shown in Figure A.50. These spectra are formulated using a power function with respect to the frequency containing several scaling parameters and constants.

The Pierson-Moskowitz spectrum represents a fully developed sea and was developed by offshore industry. It assumes deep water conditions with unlimited fetch and was developed using North-Atlantic data. The JOint North Sea WAve Project, that resulted in the JONSWAP spectrum on the other hand represents a sea at young state and was also developed by offshore industry. It represents conditions with limited fetch and was developed using North Sea data. Since the wave spectrum is hardly ever fully developed in nature, the Jonswap spectrum is often used. In this experiment the Jonswap spectrum was used with the peak enhancement factor, γ = 3.3.

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$$E(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp\left[-\frac{5}{4}\left(\frac{f}{f_p}\right)^{-4}\right] \gamma^{\exp\left(-\frac{\left(f/f_p - 1\right)^2}{2\sigma^2}\right)} \tag{8.1}$$

with:

E	spectral energy density	[m²/Hz]
α	scaling parameter (Pierson-Moskowitz)	[-]
f	frequency	[Hz]
fp	peak frequency	[Hz]
γ	peak enhancement factor	[-]
σ	scaling parameter	[-]

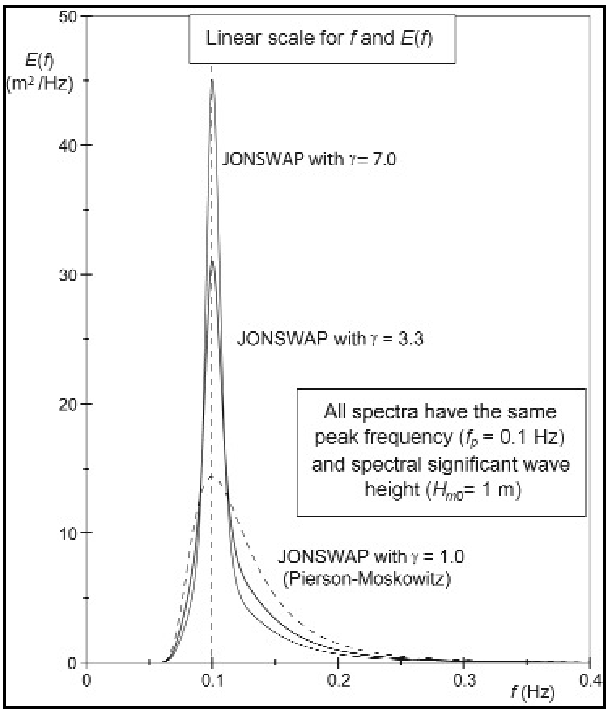

Figure A.50 Pierson-Moskowitz and Jonswap spectrum

A.9 Damage measurement photos

For explanation the damage development from one setup is shown here. The setup chosen is setup 2 with water level 0.590m, since tests for that setup was repeated the repeated tests will be shown here, S02TM_rep.

a)

b)

c)

d)

Figure A.51 Damage measurement photos, S02TM_rep, wave test #2, after a) 500 waves, b) 1000 waves, c) 2000 waves and d) 3000 waves

a)

b)

c)

d)

Figure A.52 Damage measurement photos, S02TM_rep, a) wave test #2 after 3000 waves, b) wave test #3 after 3000 waves, c) wave test #4 after 3000 waves, d) wave test #5 after failure

A.10 Photos from laboratory

Figure A.53 Photo from laboratory, the wave board

Figure A.54 Photo from laboratory, the camera stand and the structure

Figure A.55 Photo from laboratory, taken behind the structure

Figure A.56 Photo from laboratory, measurement equipment

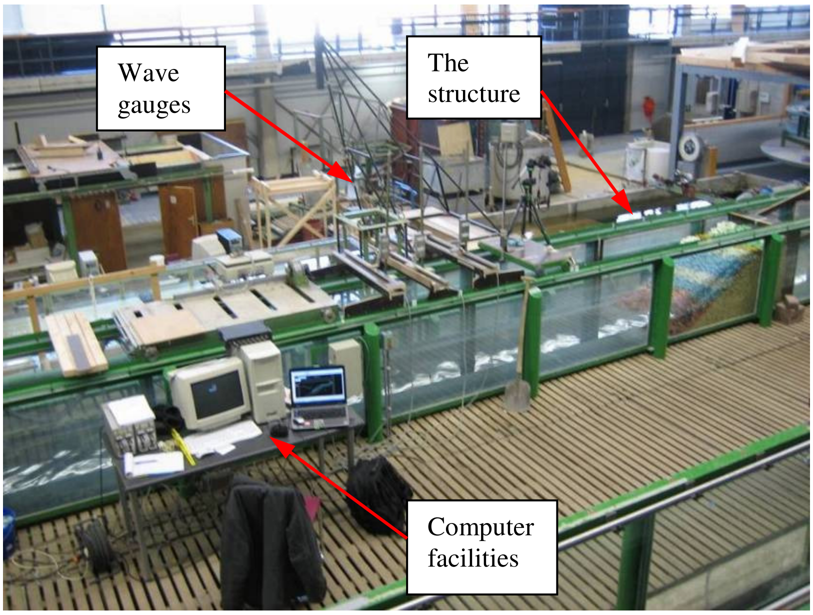

Figure A.57 Photo from laboratory, overview, looking in the direction of the structure

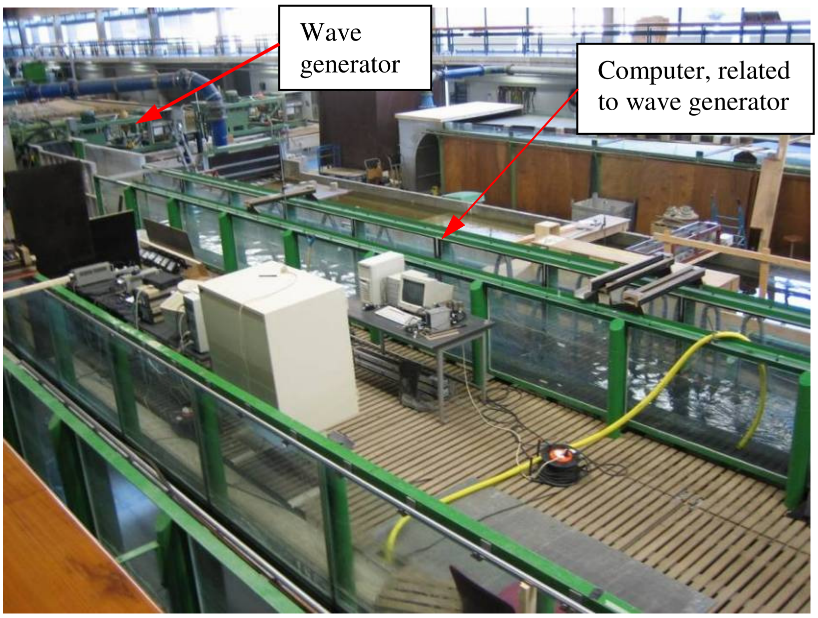

Figure A.58 Photo from laboratory, overview, looking in the direction of the wave generator