Wave overtopping and scale effects

Experimental case study New Afsluitdijk

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TUDelft

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by

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This thesis is confidential and cannot be made public until January 30, 2021.

Preface

My roots lie in the Northern part of the Netherlands, my dad lived for a while on the Island Vlieland and I grew up in the beautiful city of Groningen. Every holiday we went with our family to one of the Wadden Islands, so all the islands and the Waddensea area are familiar to me. During my master thesis I went to Vlieland to volunteer at a festival. I decided to go with my bike to the city of Harlingen and take the bicycle path of the Afsluitdijk. The cover picture of this document is an impression of that day. As a student, I am very pleased with doing my master thesis about the Afsluitdijk. It is an honour to be able to contribute to this unique project, which has protected large parts of the Netherlands against flooding since 1932. After completing the reinforcement of the Afsluitdijk, the Netherlands will once again have an iconic project as a symbol of the innovative nature of Dutch hydraulic engineering.

This thesis is written as a conclusion of the master Hydraulic Engineering with the specialization of Coastal Engineering at the University of Technology in Delft. Many people have contributed to this thesis and I would like to take the opportunity to thank them. Firstly, Bas Reedijk & Michael van den Koppel for their help and my internship place at Delta Marine Consultants. I would like to thank Coen Kuiper who had a double position in my committee, working for the TU Delft and the project team of Levvel. His expertise and commitment to this topic and the Afsluitdijk in general was very important and he also gave valuable critiques and suggestions. Stefan Aarninkhof & Bas Hofland of the TU Delft are important in my committee with their critical view to assumptions and conclusions in this research. Many thanks to my friends Lindert, Mario and Rik for their view on my work and their cooperation. And last but not least my family and my girlfriend for their mental support!

Nico Huppes

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Summary

After more than 85 years, the iconic Afsluitdijk is in need of a reinforcement. During the latest safety assessment of Rijkswaterstaat the dike did not meet the standards at two components; the amount of wave overtopping (>10 l/m/s) and armour stability (Rijkswaterstaat, 2013). This research goes into detail about the overtopping on the Afsluitdijk. In the new design a lot a of different factors influence the amount of wave overtopping: the roughness of several elements and a berm. The combination of these different factors makes it difficult to make a reliable estimation of the amount of wave overtopping. In the design new materials are used of which the effect on wave overtopping is unknown. The band width of the design is very narrow so an accurate prediction of the amount of overtopping in the prototype is necessary. Physical model tests of two different dike section are used in this research. These dike sections are tested on different scales (small to large) to validate the final design on prototype scale. Not much is known about the possible scale effects on wave overtopping so this research goes into more detail on this issue.

Data that is gathered during this research consist of two different dike sections: 8b and 17a. These two dike sections are tested in three different flumes (BAM, Schelde and Delta). This document is divided into two main parts, where the two different dike sections are discussed. In the end, these two parts will give a final conclusion about any possible scale effects by matching two identical set-ups in two different scales with each other. The data of dike section 17a consists of overtopping tests in two different flumes; the BAM flume (1:32.6) and Delta flume (1:2.95). The data of the BAM flume is generated by myself and the data of the Delta Flume is generated by Deltares. The tests in the BAM flume are mainly focused on hydraulic parameters that influence the roughness of the upper slope. The data of dike section 8b consists of overtopping tests in two different flumes; the BAM (1:32.6) and Schelde flume (1:19.8). The data of the BAM flume is generated by Levvel and the data of the Delta Flume is generated by Deltares. The tests in the BAM flume focus on structural parameters that influence the roughness of the upper slope. All the tests in the different flumes are used to look at any possible scale effects between different scales.

The influence of structural parameters is investigated with the data of section 8b in the BAM flume. The Equation of Capel overestimates the amount of overtopping with a larger rib height. This is mainly caused by scale effects on small scale. The amount of overtopping is calculated correctly if approximately the same configurations are used as in tests where the Equation of Capel is based on, otherwise the accuracy decrease. The influence of the Hydraulic parameters; q,H_{m0} and s_0 are tested with section 17a in the BAM flume. With smaller overtopping rates than 3 l/m/s the amount of overtopping is overestimated, the material behaves rougher than the Equation of Capel. With a lower wave steepness than 0.048, the amount of overtopping is underestimated, with a higher steepness the amount of overtopping is overestimated. The influence of the wave height was difficult to discuss during this research, because other parameters were also varying.

According to the EurOtop manual, scale effects could only occur for overtopping discharges that are lower than 1 l/m/s. With the tests of dike section 17a in the Delta flume, it can be noted that if ribs of 0.05 meter are used with overtopping discharges larger than 1 l/m/s, the amount of overtopping is underestimated in the Delta flume and calculated correctly in the BAM flume. The calculation error in the Delta flume reduces if ribs of 0.21 meter are tested. Ribs of 0.21 meter behave rougher than calculated in the BAM flume. So in both situations with ribs of 0.05 and 0.21 meter, scale effects occur. During the tests of dike section 8b, there is more overtopping measured for tests in the Schelde flume compared with the BAM flume for overtopping discharges larger than 1 l/m/s. Scale effect occur in this situation to.

In the BAM flume overtopping discharges lower than 1 l/m/s are measured for dike section 17a. The roughness of the upper slope in these situations is larger than calculated with the Equation of Capel and less overtopping is measured. Because the Equation of Capel is based on experiments of a scale of 1:22 and the experiments in the BAM flume are performed on a scale of 1:36.2, scale effects occur. Less overtopping is measured on small scale than calculated with the theory. The amount of overestimated overtopping is larger than the up-scaled overtopping according to the EurOtop manual. More overtopping is measured in the Schelde and Delta flume than calculated with the theory. The amount of underestimated overtopping is smaller than the up-scaled overtopping according to the EurOtop manual.

To increase the reliability of scale effects on overtopping in future research, it is preferable to continue making comparisons of new large and small scale overtopping tests. In this research cross sections are usually not exactly the same across different flumes. More tests with identical slopes on different scales will provide more knowledge about scale effects. This research notes that, in contrast to the literature, scale effects occur for overtopping discharges larger than 1 l/m/s and scale effects are large than literature for overtopping discharges that are smaller than 1 l/m/s.

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1
INTRODUCTION

This introduction outlines different aspects of this research. At first a small introduction is given about the Afsluitdijk and the new Afsluitdijk. Secondly the problem description, the research objectives and research questions are mentioned.

1.1. General overview

1.1.1. General introduction Afsluitdijk

The Afsluitdijk has been an example of Dutch coastal engineering for the last 100 years. The dam was constructed between 1927 en 1932 between provinces North Holland en Friesland. The dike has a length of 32 kilometers with a width of 90 meters and a initial height of 7.90 meters. The cross section of the dike is uniform over the full length and is given in Figure 1.1. This thesis will focus on the strengthening of this dike section (8b) in combination with another dike section (17a), this will be discussed later on.

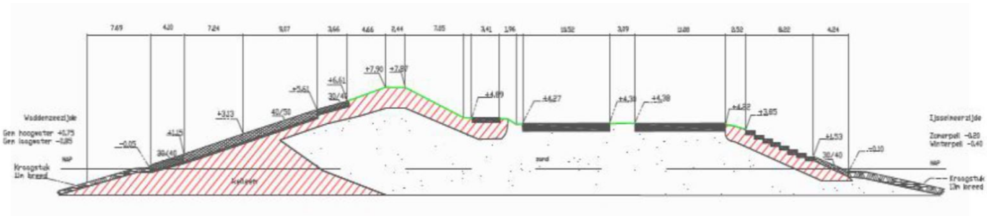

Figure 1.1: Cross section of the Afsluitdijk.

The dam was a fundamental part of the Zuiderzee works, which consist of damming the whole Zuiderzee and create the IJsselmeer. The dam protects the provinces around the Zuiderzee from flooding due to sea level rises and storms. The old Zuiderzee consisted of salt water due to its connection to the Waddensea. Due to the dam, the IJsselmeer is nowadays a fresh water lake and is fed by different Dutch waterways. The dam is besides its hydraulic component also an important connection in the Dutch road network. The dam consist of the highway A7 and a bicycle path which are important in the design later on.

1.1.2. The New Afsluitdijk

After more than 85 years, the Afsluitdijk is in need of a reinforcement. During the latest safety assessment of Rijkswaterstaat the dike fails at two components; the amount of wave overtopping (>10 l/m/s) and armour stability (Rijkswaterstaat, 2013). Due to these two failure mechanisms, the Afsluitdijk needs to be strengthened. Rijkswaterstaat wrote out the tender to reinforce the dike in 2017. On behalf of local governments, working together under the name "De Nieuwe Afsluitdijk", regional ambitions are also being implemented. These regional ambitions consist of a fish migration river and a bicycle path on the Wadden Sea side. This last ambition is important in the design of the new dike.

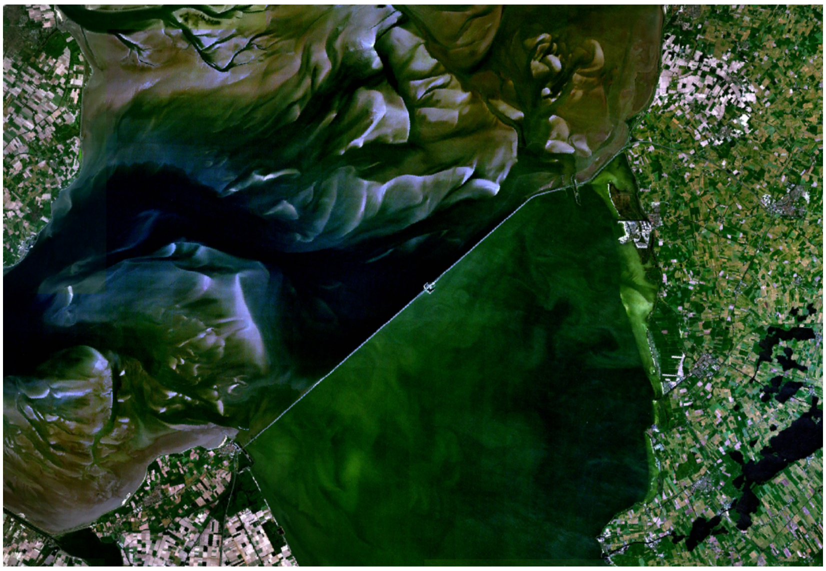

Figure 1.2: Satellite image of Afsluitdijk.

Rijkswaterstaat has awarded the Afsluitdijk project to the Levvel consortium. The Levvel consortium consists of Van Oord (with Aberdeen/APG), BAM (with PGGM) and Rebel (BAM, 2018). The name Levvel is based on the Dutch abbreviation for "Lely's Heritage Secured". In addition, the name refers to the water level management of the IJsselmeer and the turning of high water, two important functions of the Afsluitdijk.

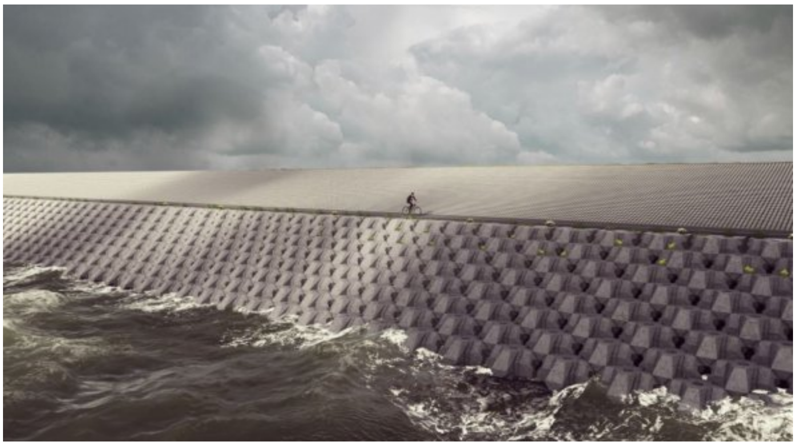

Figure 1.3: Design new Afsluitdijk.

The Afsluitdijk needs to offer protection even during extreme weather conditions, the coincidence of a spring tide and an extreme north-westerly storm with a risk of occurring once in ten thousand years. These storm conditions will give an exceedance probability of 1:10.000 for the stability of the dike for 2024. These conditions give an exceedance probability of 1:43.500 for the overtopping in 2024. For this reason, the Afsluitdijk will be replaced with a cladding that is of sufficient strength. This cladding should also slow down the water that is flowing over the dam, so the amount of water that is going over the structure is limited. A little bit of water flowing over the structure is not a problem, but high amounts will give significant problems. This document is going into more detail about this problem of overtopping.

In this research two dike sections are used to elaborate their test results on wave overtopping. The final design of the main part of the new Afsluitdijk is given in Figure 1.3, this dike section is called 8b. This dike is a composite slope with a lower slope, upper slope and a berm. The lower slope of this composite slope consist of Levvelblocs. These armour units are originally called $Xbloc^{+}$, but are named Levvelblocs in the Levvel consortium. The berm consists of asphalt and is a cycling path during calm weather conditions. The upper slope consists of Quattro-blocs that are placed in a specific pattern to create extra roughness. The other researched dike section is 17a. This dike section consists of a lower slope with blocs, an asphalt berm and an upper slope with Quattro-blocs. The location of the two sections is given in Figure 1.4.

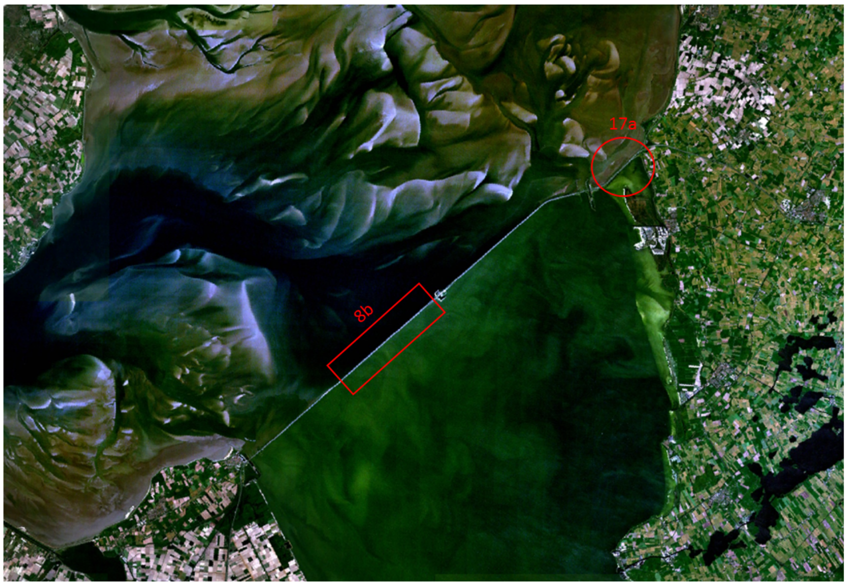

Figure 1.4: Section 8b and 17a of Afsluitdijk.

1.2. Problem description

With the choice of Rijkswaterstaat to set a strict value on the wave overtopping for the Afsluitdijk of 10 l/m/s, new challenges to predict this amount of wave overtopping are introduced. In the new design a lot a of different factors influence the amount of wave overtopping such as the roughness of several elements and a berm. The combination of these different factors makes it difficult to make a reliable estimation of the amount of wave overtopping. In the design new materials are used of which the effect on wave overtopping is unknown. The band width of the design is very narrow so an accurate prediction of the amount of overtopping in the prototype is necessary. The two different dike sections will be tested on different scales (small to large) to validate the final design on prototype level. There is not much knowledge on the possible scale effects on wave overtopping.

1.3. Research objectives and questions

The solution space to build the Afsluitdijk to create less wave overtopping than 10 l/m/s is narrow. The main objective of this research is therefore to give an accurate estimation of the amount of wave overtopping on prototype. In order to realize this, the roughness of different elements of the structure needs to be analyzed. The common approach on wave overtopping for a combination of different influence factors needs to be analyzed. Due to lack of knowledge on scale effects it is unknown if these will occur and under which circumstances.

Research questions

In order to answer to the research objectives given above, research questions have been composed. The main research question has been subdivided into several sub questions. The main question follows from the problem definition and research objective and can be stated as:

> What is the combined effect of roughness and composite slopes on wave overtopping and are there any scale effects on wave overtopping for the new Afsluitdijk?

Sub questions are:

- What are the different influence factors on the roughness coefficient of the lower slope for the Levvel-blocs?

- What are the different influence factors on the estimation of the upper slope for the rib pattern and what is the reliability of these results?
- Is the estimation of the amount of overtopping reliable for the combination of different roughness factors with a berm?
- Are there any scale effects on wave overtopping and if so, under which circumstances do these scale effects occur?

1.4. Reading guide

In this first Chapter the introduction and research questions are mentioned. In Chapter 2 the literature study about the important theory is given. In Chapter 3 the methodology is described, this methodology is the systematic, theoretical analysis of the methods applied on the data that is generated from the physical model tests that will be described in Chapter 4. In Chapter 5 & 6 the data of the different physical model test are discussed. The results of these two chapters are discussed in Chapter 7. Finally the conclusion and recommendations are given in Chapter 8.

2
LITERATURE STUDY

2.1. Wave overtopping

2.1.1. Definition wave overtopping

In most situations, it is not economically feasible to construct coastal structures high enough to prevent wave overtopping, as a consequence most coastal structures suffer a amount of overtopping. The mean wave overtopping discharge q (l/m/s) is a key design parameter for many coastal structures that are designed to limit q below a selected admissible discharge. It is not surprising, that wave overtopping has been the study of many academic research studies and the topic of engineering design guidance manuals, the latest manual is called EurOtop 2016.

Wave overtopping is a complex phenomenon and dependent on many different parameters. The hydraulic conditions, geometry and materials of the structure will influence the amount of overtopping. To understand better the amount of overtopping the main factors of overtopping should be determined. The following parameters are influencing the amount of overtopping;

- Hydraulic conditions; wave height, wave period, angle of attack, wave steepness, water level and layer thickness at the structure.
- Geometry; crest height, berm width and slope angle of the structure.
- Materials; roughness, armour layer configuration, crest configuration and permeability of the structure

2.1.2. Maximum overtopping Afsluitdijk

There are made a lot of recommendations of tolerable overtopping in different circumstances. The tolerable overtopping can be mainly classified in three categories (Allsop et al., 2008);

- Overtopping for structural design
- Overtopping for property and operation
- Overtopping for people and vehicles

The first category is the most important for the design of the Afsluitdijk. There is no specific property at the backside of the dike and the highway will close during storm conditions. The backside of the dike consist of a grass slope, the overtopping criteria for the design of the Afsluitdijk is restricted because of the stability of the grass slope. Rijkswatersaat has made a strict restriction of 10 l/s/m in the design for the maximum overtopping. This limit should not be exceeded in the final design of the two different cross sections.

2.2. Structural and hydraulic parameters

2.2.1. Structural parameters

Crest height

The crest height of the structure is influencing the crest freeboard R_c. The basis definition of the crest freeboard for a dike is the distance between the still water level (SWL) and the top of the structure (Schiereck, 2003). The top of the structure of the Afsluitdijk and the still water level are measured relative to Nieuw Amsterdamse Pijl (NAP). The difference between SWL and the crest height is important for the amount of overtopping.

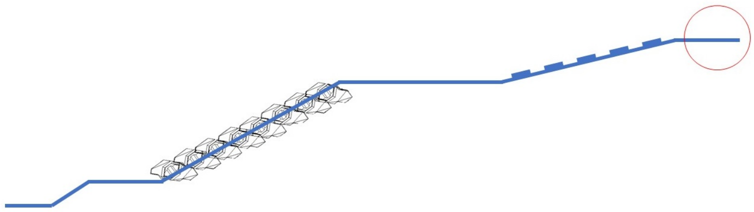

Figure 2.1: Crest height.

Slope

A part of the structure is defined as a slope if the slope op a part lies between 1:1 and 1:8 (Van der Meer et al., 2016). The Afsluitdijk is a composite profile with two different slopes and a berm in the middle. A characteristic slope is required to calculate the surf similarity parameter ($\xi_{m-1,0}$) which is discussed later on. It is recommended to calculate the characteristic slope from the point of wave breaking to the maximum run-up height. This approach need some calculation time due to iterative solution of the wave run-up height $R_{u2\%}$. According to the EurOtop manual is mentioned to make a first estimation with the $1,5 \cdot H_{m0}$ above the water. The second estimation is done with the wave run-up height from the first estimation. This steps are given in Equation 2.1 & 2.1. In Figure 2.2 the second step to determine the length of the slope is given, in this report 10 steps with new wave run-up heights are calculated iterative.

1^{st} estimation:

$$\tan\alpha = \frac{3\,H_{m0}}{L_{slope} - B} \tag{2.1}$$

2^{nd} estimation:

$$\tan\alpha = \frac{1.5\,H_{m0} + R_{u2\%(1^{st}estimation)}}{L_{slope} - B} \tag{2.2}$$

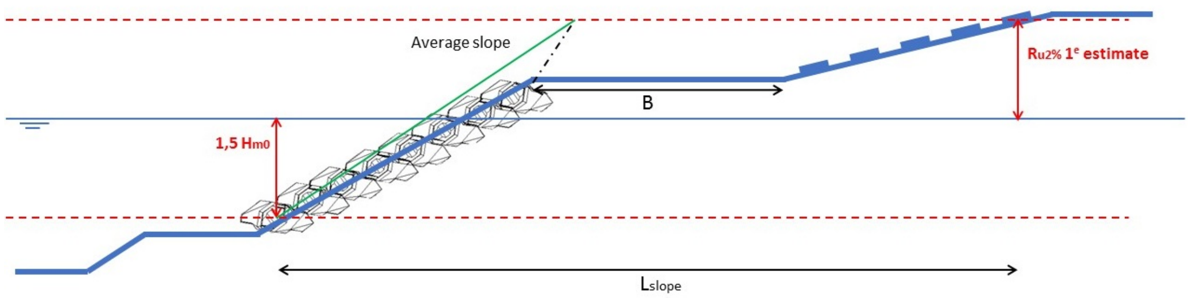

Figure 2.2: Determination of the average slope (2nd estimate).

According to the EurOtop manual it is recommended to calculate the characteristic slope from the point of wave breaking to the maximum run-up height. The lowest point of the slope of the Afsluitdijk is in some cases higher than $1,5 \cdot H_{m0}$ under the still water level, due to the toe. This limitation should be added in Equation

2.2, so that the characteristic slope for wave breaking is smaller in these cases. If $R_{u2\%}$ is higher than the crest freeboard described earlier, the crest freeboard is dominant for the length of the slope. The characteristic length of the slope is lower due to these two effects so the 2^{nd} estimation of $\tan\alpha$ is changed to Equation 2.3.

$$\tan\alpha = \frac{\min(1.5\,H_{m0}, SWL - h_{toe}) + \min(R_{u2\%}, R_c)}{L_{slope} - B} \tag{2.3}$$

Foreshore

The foreshore, the depth limiting seabed in front of a structure, is defined as having a minimum length of one wavelength $L_{m-1,0}$. At a shallow foreshore the waves are breaking and the wave height decreases in front of the structure. In case of the Afsluitdijk the foreshore is given in Figure 2.3. There are two specific location at this foreshore that are relevant in this report; deep water point and Hydra NL point. Later on, different hydraulic conditions of the two locations are discussed for the discussion of the results. The slope between Deep water and Hydra NL is 1:10, this will influence the wave height and wave period of the incoming waves significant due to shoaling and wave breaking.

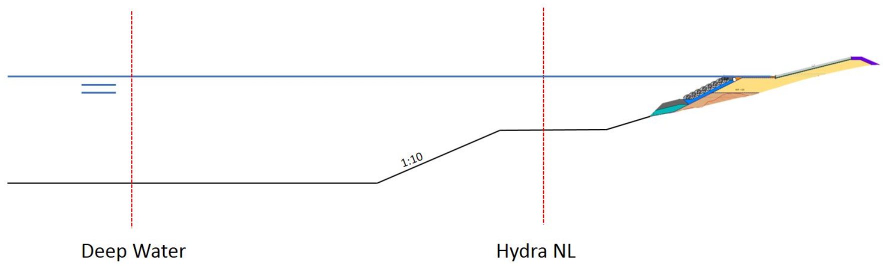

Figure 2.3: Foreshore Afsluitdijk.

Permeability and porosity

Porosity is defined of voids between units or particles. This parameter is mainly influenced by the shape, grading an method of placement of the armour units and the material of the core of the dike. The porosity and permeability of the armour layer will be discussed later in the roughness section. Difference is rubble mound structures made between “impermeable under layers or core” and a “permeable core”. In both situation the same armour unit is used, but the structure and under layers differ.

A rubble mound breakwater often has an under layer of large rock, a second under layer of smaller rock and a core with even smaller material. Waves that run up the structure can penetrate into the armour layer and will then sink into the under layers and core. This is a structure with a permeable core where water can penetrate intro the structure.

An embankment can also be covered by an armour layer of rock, such as the Afsluitdijk. The under layer is often small and placed on a geotextile or an impermeable material. Underneath the armour units the structure is blocking the up-rushing waves. Such an embankment with armour units has an “impermeable core”. Run-up and wave overtopping are dependent on the permeability of the core. A rubble mound slope dissipates significantly more wave energy than an impermeable smooth slope with the same characteristics (Molines and Medina, 2015). This is caused by the roughness and porosity of the armour layer , but also the permeability of the under layer and core of the structure contribute to this. Also the the Run-up form an impermeable core is significant for an impermeable core than a permeable core. In some of the cross section later on, a parts of the Afsluitdijk are more permeability than others. It’s good to notice this effect in combination with the roughness of the slope which is discussed below.

Roughness Armour unit

The roughness coefficient (γ_f) is one of the most important parameters regarding overtopping. Roughness is the irregular surface which created more resistance than a smooth slope (Molines and Medina, 2015). In a rubble mound slope the blocks or rocks forming the armour layer, the irregularity creates a higher roughness and thereby a lower overtopping. A smooth slope had a roughness factor of 1.0 (Van der Meer et al., 2016). With armour units the roughness factor is usually considered a parameter associated with the armour unit geometrical shape, however γ_f is not dependent only on the geometry, but also on the packing density, number of layers and other structural parameters are important (Molines and Medina, 2015).

The reduction factor, γ_f, is an empirical parameter that is generally estimated from the mean overtopping discharge and hence depends on the chosen wave overtopping predictor (van der Meer, 2002). As the knowledge on wave overtopping increases and the formulae is modified, the roughness coefficient will also change. It becomes clear that the roughness factor is dependent on the specific overtopping formula selected by the corresponding author and the dataset used.

None of the overtopping formulas in the literature includes armor porosity or packing density. Test data reported in the clash database gives insight in the relation between packing density and armor roughness(Molines and Medina, 2015). The γ_{f50}-calibrated roughness factors from from the CLNN is used to study the influence of packing density (ϕ) and armor roughness (γ_f) on overtopping (Molines and Medina, 2015). Figure 2.4 shows the influence of armour porosity (p=1-ϕ/n) on the roughness factor, the armor porosity used here as a recommend value for the type of armour (Molines and Medina, 2015). In general, armour units placed with a higher porosity tend to have a lower roughness factor. The line $\gamma_f = 1 - 1.25p$ shown in Figure 2.4 fits the data, except for the double rock layer. Armour porosity is important for the hydraulic stability of the armor unit, recommend packing density must be followed to avoid changes during the lifetime of the structure. Designing an armor with a high porosity decrease the roughness, amount of overtopping and the economic cost. When there is a high porosity there will be significant settlement, this will lead to increasing porosity and affects the armour stability even more (Molines and Medina, 2015).

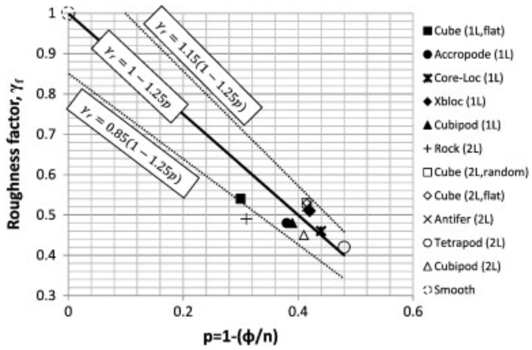

Figure 2.4: Packing density to roughness.

In this report, the roughness coefficients recommended in the EurOtop manual are taken as reference since these are the last ones that are calculated with the new formula. If there are roughness coefficients that are unknown, a prediction will made for that specific armour unit. As can be seen in Figure 2.5, the lower slope of the Afsluitdijk consist of Levvel-blocs. From previous research the roughness coefficient of Levvel-blocs is γ_f = 0,55 (Jiménez Moreno, 2017). The result of this study are questionable, due to other design parameters. In the design of the Afsluitdijk the Levvel-blocs are under the waterline and the Afsluitdijk consists of an impermeable core. The roughness coefficient of the Levvel-blocs will be different due to these effects. In this report a new fit of γ_f for the Levvel-blocs for the case Afsluitdijk will be made.

Roughness Block Revetment

Block revetments are used on dikes to protect the structure against the waves. In the design of the Afsluitdijk a block revetment is used on the upper slope, given in Figure 2.5. In general the revetments form a smooth slope by having each set of blocs leveled at the top. In order to reduce overtopping, blocs can be placed in a special pattern. Different pattern introduce open space between blocks, this will reduce the amount of wave run-up and wave overtopping. A study from Deltares (Capel, 2015) showed that the roughness coefficient

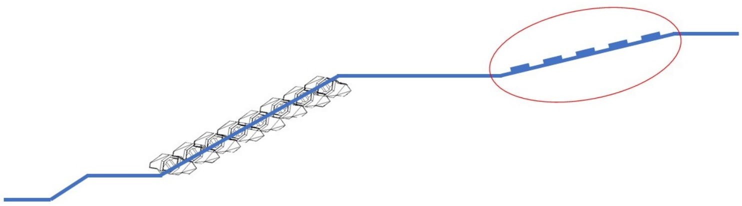

Figure 2.5: Roughness elements.

could be determined with the following formula;

$$\gamma_f = 1 - 0.585 \cdot \sqrt{0.075 - s'_{m-1,0}} \cdot \rho_{\gamma_f}{}^{0.5} \cdot \left(ln\left(\frac{q}{\sqrt{g \cdot H_s^3}} \right) \right) \tag{2.4}$$

The first parameters that is influencing the roughness is the dimensionless mean overtopping discharge expressed by $\ln(\frac{q}{\sqrt{g \cdot H_s^3}})$. The roughness effect due to the pattern reduces when the mean overtopping discharge increases. This can be explained by the fact that the flow depth of the overtopping water is larger in case of larger overtopping discharges. The roughness of the pattern is therefore less experienced by the run up of the water.

Next to the fact that overtopping discharge is influencing the roughness coefficient, The local shallow water wave steepness plays a roll. By increasing the steepness of the waves, the roughness of the pattern reduce. The value of 0.075 is seen as the theoretical maximum of wave steepness. Therefore the relation of the wave steepness is given by $\sqrt{0.075 - s'_{m-1,0}}$.

The roughness exposed area of the front face can be characterized by a roughness density parameter. The roughness density is based on the 'effective front face area', divided by 'the area of the slope between the crest level and the water line' (Capel, 2015). In case the roughness elements are located under the water line, the water between the water through and waterline should be taken into account (approximately 1/3 of the significant wave height) (Capel, 2015). In order to determine the roughness density parameter, first the dimensionless roughness width $\gamma_{f,w}$ is determined. This is the total width of the exposed elements per meter dike for all the rows (Capel, 2015). This parameter is multiplied with the protrusion height of the elements (h_{prot}) and divided by the length of the slope ($R_c / \sin\alpha$). These parameters are leading to the following roughness density parameter Equation 2.5, for the roughness coefficient this density parameter is corrected by the power of 0.5.

$$\rho_{\gamma_f} = \frac{\gamma_{f,w} \cdot \sin\alpha \cdot h_{prot}}{R_c} \tag{2.5}$$

Berm

A berm is part of a structure profile in which the slope varies between horizontal and a slope of 1:15. The position of the berm in relation to the still water line is determined by the depth (d_b), the vertical distance between the middle of the berm and the still water line (Van der Meer et al., 2016). A berm is defined by the width of the berm B and by the vertical difference between the middle of the berm and the still water level. The width of the berm should me maximum $0.25 \cdot L_{m-1,0}$. The benefit of a berm is that the equivalent slope angle becomes smaller. This will reduce overtopping and leads to a lower required crest level. The best location for a berm is around the design water level due to is maximized influence. The influence factor γ_b for a berm consists of two parts, given by r_B and r_{db} (Van der Meer et al., 2016).

$$\gamma_b = 1 - r_B(1 - r_{db}) \quad \text{for} : 0.6 < \gamma_b < 1.0 \tag{2.6}$$

In the equation (r_B) stands for the influence of the width of the berm L_{Berm} and becomes zero if there is no berm, this will lead to a an berm factor of 1. In the equation (r_{db}) stands for the difference d_b between the still water level and the middle of the berm. This becomes zero if the berm is situated exact on the still water line. The reduction of wave run-up or wave overtopping is maximum for a berm on the still water line, this is also the case in the design of the Afsluitdijk 2.5. If the berm is lying below $2 \cdot H_{m0}$ or above $R_{u2\%}$, the berm has no influence on wave run-up and wave overtopping and r_{db} becomes one.

$$r_B = \frac{B}{L_{berm}} \tag{2.7}$$

$$\begin{aligned} r_{db} &= 0.5 - 0.5 \cdot \cos\left(\pi \frac{d_b}{R_{u2\%}}\right) \quad \text{for a berm above still water line} \\ r_{db} &= 0.5 - 0.5 \cdot \cos\left(\pi \frac{d_b}{2 \cdot H_{m0}}\right) \quad \text{for a berm below still water line} \end{aligned} \tag{2.8}$$

Weighted roughness

Roughness elements are mostly applied in different parts of the slope. Therefore, a reduction factor is needed to take only that part which is relevant. Roughness elements have little to no effect 0.25 $R_{u2\%,smooth}$ below the still water line and 0.5 $R_{u2\%,smooth}$ above the still water line (Van der Meer et al., 2016). The average influence factor for roughness (γ_f) is calculated by weighting the different elements between the two lines where it is the most influenced. The following equation will calculated the resulting influence factor for roughness:

$$\gamma_f = \frac{\sum_{i=1}^{n} \gamma_{f,i} L_i}{\sum_{i=1}^{n} L_i} \tag{2.9}$$

Roughness elements that are completely under the water line have no effect on the amount of overtopping. Its is recommended to place roughness elements on the most influenced area. The construction cost will be lower than placing roughness elements on the entire slope. The area of influence for the weighted roughness for the Afsluitdijk is given in Figure 2.6. It can be seen that the roughness of the lower slope has a lower influence than the top slope of the structure. In each test the area of influence of the roughness is calculated for the specific test.

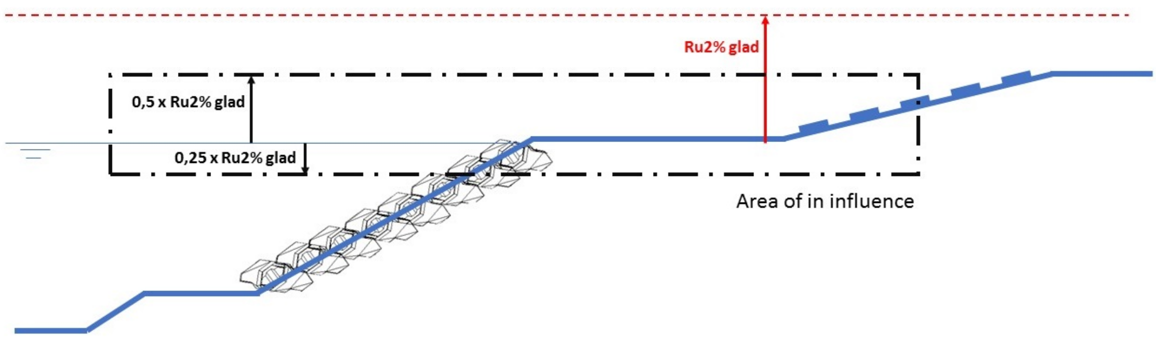

Figure 2.6: Weighted roughness.

2.2.2. Hydraulic parameters

Wave height

The wave height in the overtopping formulas it the incident significant wave height H_{m0} at the toe of the structure. This incident significant wave height is the spectral wave height $H_{m0} = 4(m_0^{1/2})$ (Holthuijsen, 2010). As mentioned earlier, there is a foreshore in front of the structure which influence the wave height due to depth limitation. The wave height that is used for calculating the amount of overtopping is the wave height at the Hydra NL point, in Figure 2.3. Due to the small distance between the Hydra NL point and the toe of the structure, their is assumed that the spectral wave height is approximately the same at these two locations.

Wave period

From a wave spectrum or wave record, various wave heights can be defined. Conventional wave periods are the peak period T_p, the period that gives the peak of the spectrum, the average period T_m, calculated from the wave record (Van der Meer et al., 2016). The relationship between T_p/T_m lies between 1.1 and 1.25

The wave period used for the overtopping formulae is the spectral period $T_{m-1,0} = m_{-1}/m_0$ (Van der Meer et al., 2016). This period gives more weight to the longer periods in the spectrum than the average period. Independent of the type of the spectrum, this spectral period gives similar amount of overtopping for the same values of $T_{m-1,0}$ and the same wave heights. In this way, wave overtopping can be predicted for double-peaked and "flattened" spectra, without the need for the other difficult procedures (Van der Meer et al., 2016). In the case of the Afsluitdijk there is a uniform spectrum and a fixed relationship between the spectral period $T_{m-1,0}$ and the peak period. Therefore in any case where the peak period is known, but not the spectral period the relation of $T_p = 1.1\,T_{m-1,0}$ can be used in this document.

Wave steepness

Wave steepness (s_0) is defined as the ratio between the wave height and wave length ($L_0 = gT^2/2\pi$) (Holthuijsen, 2010). Generally a steepness of $s_0 = 0.01$ refers to typical swell sea, a steepness of $s_0 = 0.04$ to 0.06 refers to typical wind sea. Swells sea is associated with long periods and therefore low wave steepness, where the period is the main parameter that effects overtopping. Wave breaking doesn't occur in these low steepness situation and water is surging over the structure.

The wave steepness at the Afsluitdijk is mainly around 0.04 to 0.05. These wind sea conditions can also lead to low steepness waves if the there is wave breaking on a gentle foreshore. When wave breaking occurs, the period does not change significantly but the wave height decreases resulting in lower steepness. Wave breaking on the shallow foreshore of the Afsluitdijk is not significant, so the wave steepness do not decrease significantly.

Surf similarity parameter

The surf similarity parameter (ξ) or Iribarren number is defined as $\xi_{m-1,0} = \frac{tan(\alpha)}{H_{m0}/L_{m-1,0}}$, where α is the slope of the front face of the structure and ($L_0 = gT^2/2\pi$) the deep water wave length (Bosboom and Stive, 2013). The calculated wave steepness, is a notional wave steepness and is used to calculate a dimensionless wave period, rather than the actual wave steepness. The combination of the average slope and wave steepness gives a certain type of wave breaking. For $\xi_{m-1,0}$> 2 waves are considered to be surging and not to be breaking, although there may still be some breaking, for $\xi_{m-1,0} < 2$ waves are breaking. For wave run-up on slopes the transition from plunging to surging is given at $\xi_{m-1,0} = 1.8$ (Van der Meer et al., 2016). Waves on a gentle foreshore break as spilling waves and more breaker lines can be found. Plunging waves break with steep and overhanging fronts and the wave tongue will hit the structure or back washing water. The transition between plunging waves and surging waves is known as collapsing. In the design of the Afsluitdijk the surf similarity parameter is around 1.7, so waves can be considered as plunging on the structure. In the beginning of this chapter there is discussed if the Afsluitdijk can be schematizes as a dike (breaking) or a breakwater (non-breaking). With a surf similarity parameter of 1.7 waves are breaking, so this confirmed the assumption that the Afsluitdijk should be considered as a dike regards overtopping.

Wave spectrum

The wave spectrum is the most characteristic aspect of a sea state. A common type of spectrum is JONSWAP (JOint North Sea WAve Project) which is an empirical relationship that defines the distribution of wave energy with frequency for a growing sea state (Hasselmann et al., 1973). JONSWAP-spectrum is characterized by three shape parameters: γ , σ_a and σ_b. For a standard JONSWAP-spectrum, the mean values of these shape parameters are $\gamma = 3.3$, $\sigma_a = 0.07$ and $\sigma_b = 0.09$ (Holthuijsen, 2010). In all the tests for the Afsluitdijk that are discussed later on, the JONSWAP spectrum is used to characterize the wave spectrum.

2.3. Prediction of overtopping

Overtopping of coastal structures can be predicted by a number of different methods, each with their advances (Allsop et al., 2008). Analytic methods can be used to predict structure responses through equations based on the physics of the process. Waves and the overtopping process can not be all well-controlled, so that an analytic method on its own give not reliable predictions. The primary prediction methods are the empirical methods. These methods relate the response of wave with structure parameters using data to determine empirical equations.

Two other methods were derived during the CLASH project. These methods are based on a lot of measured overtopping test from model test in different scales and field measurements (Allsop et al., 2008). The first one uses the CLASH database, to describe overtopping in 31 parameter. Using the database needs some knowledge about the results from the different tests. A more simple method is the Neural Network tool from the database. The Neutral Network tool can be run on its own, or in combinations with other simulation methods (Allsop et al., 2008) and will be discussed later on.

There are cases where empirical data do not exists, or where other methods do not give any reliable results, two different methods can be used in such a case. First numerical methods can simulate the process of overtopping in more detail. Such models involve some simplification of the overtopping process although the most complex CFD tools can simulate all process of relevance. However a complex simulation process cost more computer time and code complexity. The final method to predict the overtopping is physical modelling where a scale model is tested on overtopping. A wave spectrum is generated in a flume which attack the structure and creates an amount of overtopping.

In this section first a few empirical methods will be discussed. These empirical methods will be compared with a Neural Network tool. The method which gives the best match between physical model test will be used in this document.

2.3.1. Empirical methods

Empirical methods are regression models that are based on available overtopping data obtained from physical model experiments. Generally, these methods establish an exponential relationship between the mean discharge and the crest freeboard.

Owen's formula, 1980

Owen's method calculates the overtopping discharge for impermeable smooth sloping structures. Based on the work of Owen (1980) it is long established that wave overtopping generally decrease if the crest freeboard increases (Owen, 1982). This relation is given in formula 2.12.

$$Q^* = a\, exp\left(\frac{-b\, R^*}{\gamma_f}\right) \tag{2.10}$$

$$R^* = \frac{R_c}{T_m\sqrt{g\, H_s}} = \frac{R_c}{H_s\sqrt{s_{0m}/2\pi}} \tag{2.11}$$

$$Q^* = \left(\frac{Q}{T_m\, g\, H_s}\right) \tag{2.12}$$

In these equation is $H_m o$ the spectral significant wave height; and 'a' and 'b' are empirical coefficients that depend on the cross-section and γ_f the correction factor that accounts for the roughness of the slope. According to Owen this coefficient is the ratio between the run-up of a given wave on a rough slope and the run-up of the same wave on a smooth slope. He recommended γ_f 0.5-0.6 for rock slopes and γ_f 1.00 for smooth slopes (Owen, 1982). For the Afsluitdijk it is difficult to use this formula, due to composite slope. The berm γ_b is for example not included in this formula which is relevant for the Afsluitdijk. The method of Owen is better for impermeable breakwater or dikes without a berm and would not be discussed later on.

EurOtop Manual, 2007

Their is a clear distinction between breaking and non breaking conditions (Franco et al., 1995). Therefore, two different formulae for mean wave overtopping discharge have been defined for irregular waves and slopping structures. Equation 2.13 represents the basic overtopping expression in its dimensionless form where empirical coefficients 'a' and 'b' depends on the concerned method and accounts for the wave conditions, the reduction factors and the structure dimensions.

$$\frac{q}{\sqrt{gH_{m0}^3}} = a \cdot exp\left(-b\frac{R_c}{H_{m0}}\right) \tag{2.13}$$

For breaking waves on a sloping structures, the process of overtopping could described by the following Equations:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{A}{\sqrt{tan\alpha}}\gamma_b \xi_{m-1,0}\, exp\left[-\left(B\frac{R_c}{\xi_{m-1,0}\, H_{m0}\,\gamma_f\,\gamma_b\,\gamma_\beta\,\gamma_v}\right)\right] \tag{2.14}$$

With a maximum for non-breaking waves:

$$\frac{q}{\sqrt{gH_{m0}^3}} = C\, exp\left[-\left(D\frac{R_c}{H_{m0}\,\gamma_f\,\gamma_\beta}\right)\right] \tag{2.15}$$

The equation show quite a number of influence factors. γ_b is the influence factor for a berm, γ_f is the influence factor for roughness elements on the slope, γ_β is the influence factor for oblique wave attack and γ_v is the influence factor for a wall at the end of a slope. All these influence factors have been described in (Van der Meer et al., 2016). The roughness- and berm coefficient are important for the design of the Afsluitdijk, the γ_β and γ_v are not important in this case and are equal to 1.

The value of the empirical coefficients A, B, C and D vary depending on the undertaken approach (deterministic or probabilistic design). According to the EurOtop 2007 a more conservative approach for the deterministic design and one standard deviation has been recommended to be added to the mean overtopping discharge (Van der Meer et al., 2007).

Table 2.1: Values for the different empirical coefficients.

	Deterministic Design	Probabilistic Design
A	0.067	0.067
B	4.30	4.75
C	0.2	0.2
D	2.3	2.6

EurOtop Manual, 2016

Research in CLASH resulted in a lot of new data and in prediction formulas equation 2.14 and equation 2.15 for breaking and non breaking waves (van der Meer and Bruce, 2013). Both formulas over predict the amount of wave overtopping for very low and zero freeboard. A fit as in the previous equation describes the data but it is not easy to use two different formulas. It is possible to go the a Weibull-type function with a fitted shape factor. The general formulae for the average overtopping discharge on a slop is than given by the following formula:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{A}{\sqrt{tan\alpha}}\gamma_b \xi_{m-1,0}\, exp\left[-\left(B\frac{R_c}{\xi_{m-1,0}\, H_{m0}\,\gamma_f\,\gamma_b\,\gamma_\beta\,\gamma_v}\right)^{1.3}\right] \tag{2.16}$$

With a maximum for non-breaking waves:

$$\frac{q}{\sqrt{gH_{m0}^3}} = C\, exp\left[-\left(D\frac{R_c}{H_{m0}\,\gamma_f\,\gamma_\beta}\right)^{1,3}\right] \tag{2.17}$$

This formula is almost the same as the formulas from the (Manual, 2007) but represent nature better for low and zero freeboards (R_c/H_{m0}<0.5-1.0). In general it is better to replace the old formula from (Manual, 2007) for 2.16 and 2.17. According to Eurotop2016 the new equation gives better insight of in the amount of overtopping over the full range of zero and positive freeboards according (van der Meer and Bruce, 2013).

The value of the empirical coefficients A, B, C and D vary depending on the undertaken approach (deterministic or probabilistic design). According to the EurOtop 2016 a more conservative approach for the deterministic design and one standard deviation has been recommended to be added to the mean overtopping discharge (Van der Meer et al., 2016).

Table 2.2: Values for the different empirical coefficients.

	Deterministic Design	Probabilistic Design
A	0.026	0.023
B	2.5	2.7
C	0.1035	0.09
D	1.35	1.5

EurOtop Manual, 2018

During this research a new EurOtop Manual is released. This manual is an update of the previous one. The theory that is used during this research is valid for both the EurOtop manual 2016 & EurOtop manual 2018, because these parts are not changed.

2.3.2. Neural Network

The Overtopping Neural Network is a design tool to estimate wave overtopping for different coastal structures. Only one schematizing is used for all types of coastal structures, where not only dikes, rubble- mound breakwaters or vertical breakwaters are defined, but also other non-standard structures are included. Besides the effect of the most common parameters (wave height, wave period and crest freeboard) also the effects of many other wave and structural characteristics are considered.

The prediction method is based on Neural Network modeling. Neural networks have proven to be very useful for solving difficult modelling problems, for the modelling of processes in which the relationship of the individual modelling parameters is unclear while sufficient experimental data is available to identify the relations. The database was created by overtopping tests with 31 different parameters. A total number of more than 10,000 test from 163 independent test series are included. Around 80% of the data is originating from CLASH partners, the other part of the data is from outside the project (STEENDAM et al., 2005).

In CLASH it was sometimes difficult and not always objective to conclude on an average roughness factor, γ_f, for the whole structure, if parts of the structure had different roughness: for example a smooth slope confronted with a large rock berm. In order to be more consistent, the roughness factors for both the upper and down slope have now been gathered in the database (Bruce et al., 2009).

2.3.3. Comparison of methods

In order to make a distinguish between the different methods mentioned above, this section will compare the different methods. Three different methods are compared with each other: EurOtop 2007, EurOtop 2013 and the Neural Network method. The best solution will than be used in order to analyze data from physical model test later on. Owen's method will not be compared, this method is best applicable for breakwaters with a straight slope, the design of the Afsluitdijk do not have a one straight slope. For two random chosen tests, the following parameters are used to calculate the amount of overtopping: All these parameters are used

Table 2.3: Input different methods.

SWL	β	h	H_{m0}	$T_{m-1,0}$	ht	Bt	γ_f	$cot(\alpha_u)$	$cot(\alpha_d)$	R_c	B	hb	$tan(\alpha_B)$	A_c	G_c
5.21	0	11.726	4	7.38	4.71	5	0.84	2	4	4.54	12	0.01	0	4.71	0
5.07	0	11.586	3.99	7.35	4.57	5	0.83	2	4	4.68	12	-0.13	0	4.57	0

in the three different methods for overtopping. In Table 2.5 it can be seen that the results of EuroTop 2007 and EurOtop 2013 matching better than the Neural Network. The two EurOtop methods give also slightly different results, the EurOtop 2007 have a slightly lower error than EurOtop 2013. This comparison is done with only two tests and give not a clear understanding. Therefore another analyses is performed with 9 tests. The average calculation error of the 9 test is for the EurOtop 2013 is 0.65 l/m/s and for the EurOtop 2007 0.59 l/m/s. All these consist of overtopping rates of 5-12 l/m/s. The same procedure is done with tests that include low overtopping rates (1<l/m/s), the average error for EurOtop 2013 is 0.106 l/m/s and for EurOtop 2007 0.224 l/m/s. To conclude both the EurOtop formulas fit better with the physical model test than the

Neural Network. For low overtopping rates it is necessary to use the EurTop 2013 equation, in cases of higher discharge, it is sligthly better to use the EurOtop 2007 equation.

Table 2.4: Overtopping results different methods.

Method	First scenario l/m/s	Second scenario l/m/s
EurOtop 2007	8.44	5.42
EurOtop 2013	8.53	5.31
Neural Network	4.894	4.24
Phyiscal model	8.14	5.10

2.3.4. Difference between a dike & breakwater

Historically, sloping dikes have been the most widely used option for sea defences along coasts around the world. However, coastal structures can also consist of layers of quarried rock fill, protected by concrete armour units or rocks. This type of structures is known as rouble mound breakwater. The armour layer is designed to resist wave action without displacement of the armour units. Under layers of quarry run support the armour and separate it from finer material in the embankment. These porous and sloping layers dissipate a proportion of the incident wave energy in breaking and friction. For this research, the effect of wave overtopping will be tested on different design of the new Afsluitdijk. As the name of "Afsluitdijk" mentioned, the Afsluitdijk is a dike. In the design of the New Afsluitdijk a concrete armour unit is used to reduce the wave energy. These concrete armour units implies that the structure is a rouble mound breakwater. So the Afsluitdijk consist of components of both a breakwater and a dike. In this part the difference between the calculation for a breakwater or dike is discussed.

The latest EurOtop manual specifies that the wave overtopping discharge can be described by two different formulas for coastal dikes and embankments, one for breaking waves on the slope and one for non breaking or surging waves (Van der Meer et al., 2016). These two expressions correspond to Equation 2.18 for non breaking waves and Equation 2.19 for breaking waves respectively. According Jiminez, the amount of overtopping on a breakwater with Levvel-blocs is calculated better with Equation 2.18 (Jiménez Moreno, 2017).

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \cdot \exp\left[-\left(1.5\frac{R_c}{H_{m0}\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_f^*}\right)^{1.3}\right] \tag{2.18}$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}}\gamma_b\,\xi_{m-1,0}\,exp\left[-\left(2.7\frac{R_c}{\xi_{m-1,0}\,H_{m0}\,\gamma_f\,\gamma_b\,\gamma_\beta\,\gamma_v}\right)^{1.3}\right] \tag{2.19}$$

Table 2.5: Different parameters in overtopping equation.

		Unit
q	Overtopping discharge	$[l/m/s]$
g	Acceleration due to gravity (= 9,81)	$[m/s^2]$
H_{m0}	Estimate of significant wave height from spectral analysis	$[m]$
α	Angle between overall structure slope and horizontal	[°]
γ_b	Influence factor for a berm	$[-]$
$\xi_{m-1,0}$	Surf similarity parameter based on $s_{m-1,0}$	$[m]$
R_c	Crest freeboard of structure	$[m]$
γ_f	Influence factor for the permeability and roughness of or on the slope	$[-]$
γ_β	Influence factor for oblique wave attack	$[-]$
γ_v	Influence factor for a vertical wall on the slope	$[-]$

Armoured rubble slopes are characterized with some porosity or permeability, covered by a armour layer consisting of large rock or concrete units. In contrast to dikes and embankment seawalls the permeability of the structure and armour layer plays a role in wave run-up and overtopping. The cross-section of a rubble mound slope, however, may have great similarities with an embankment seawall and may consist of various slopes, but generally they have steeper slopes of around 1:1.5. As rubble mound structures are to some extent similar to dikes and embankment seawalls, the basic wave run-up and overtopping formulae are taken from dikes. The design formulae for dikes will be modified, if necessary, to fit for rubble mound structures. In case of armoured slopes such as rubble mound structures, the EurOtop indicates that mainly Equation 2.18 has to be used since these types of structures often have steep slopes of about 1:1:5, leading to the overtopping equation that gives the maximum with non breaking waves. In this Equation the berm coefficient is not covered, mostly because breakwaters do not consist of a berm. The new Afsluitdijk is a composite slope with a berm, therefore it is difficult to use Equation 2.18 to calculate the amount of overtopping.

2.4. Physical modelling

Physical modelling is commonly used to assess wave overtopping and develop empirical formulae for predicting it. The large number of relevant parameters influencing this phenomenon makes it hard to develop theoretical or numerical approaches that represent the nature of overtopping well. In contrast, experimental tests are an established and reliable method for determining mean wave overtopping discharges for arbitrary coastal structures (EurOtop 2016). However, the actual empirical formulae do not predict wave overtopping discharges and individual volumes very accurately due to scale and model effects present always at some degree in the physical model.

2.4.1. Scale effects overtopping

Model tests with surface waves must be scaled according to the Froude's model law which correctly reproduces the ratio of inertia and gravity force. Froude's model law misscale surface tension, viscous and elastic forces. For scale effects related to rock rubble mound armour stability, it is generally accepted that no detectable scale effect exists if the Reynolds number $Re = \frac{D_{n50}\sqrt{g\,H_s}}{\nu}$ is larger than approximately 3 x 10^4. Although indicating that this Reynolds number should at least be reached in order to avoid scale effects on overtopping, it is unlikely that this definition of a Reynolds number is sufficient related to run-up and overtopping (Andersen et al., 2011). This is due to the differences in the critical stages of the phenomena in the wave cycle. In order to discuss the overtopping scale effects the various flow regimes present during run-up on a rubble mound slopes are given in the Figure 2.7 (Burcharth and Lykke Andersen, 2009).

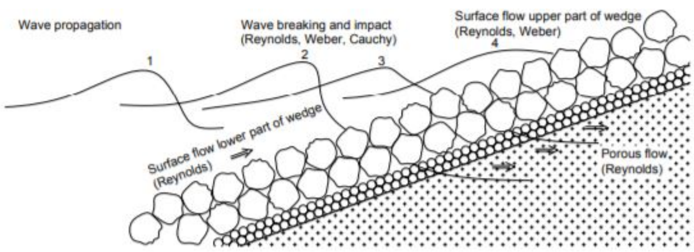

Figure 2.7: Illustration of surface flow and porous flow domains during run-up.

Viscosity scale effects related to the porous flow in the core

Using Froude model length scale to scale the size of the core of the material is laminar flow in the small scale, in prototype this flow is turbulent. It is not possible in the model to create kinematically similar flows in the model and prototype by adjusting the core material because the flow is changing in space and time (Andersen et al., 2011). It is expected that run-up and overtopping will be larger if core material is determined by Froude when the size is not increased in the model.

Viscosity scale effects related to the run-up flow

The run-up surface flow changes in space and time during the run-up. When the thickness of the run-up is greater compared with the roughness of the armour units, the flow is compared with flow over a rough bottom slope. When and where the thickness is less than the roughness, the flow has to flow around the obstacles. The flow resistance in the region of small water depth is dominated by the drag force exerted upon the armour unit, inertia forces are of less important. The effect of the variation in drag coefficient is smaller in the small scale models than in prototypes. Due to decreasing layer thickness and run-up velocities for decreasing overtopping discharges, this scale effect is expected to be much more significant for small overtopping rates than for the larger ones (Andersen et al., 2011).

Scale effects surface tension

In breaking waves the surface tension plays a role causing surface tension scale effects. This is due to the decreased curvature radius in the crest and to air entertainment. Entrained air escapes from the water either as rising bubbles or by dissolving into the water. Wave breaking on a rubble slope and the front wedge flow past the armour units enclose air in the water. This water is transported the dissolved air over the rubble slope. Due to this effect relatively more air will be enclosed in large models than in the small scale models (Burcharth, 2004). Due to differences in relative air contents, the average mass density of the up-rushing water is smaller in large models than in the small scale models.

A reduction in mass density of 5% will cause an approximately 5% higher run-up. Although the overtopping water contains more air in the the volume of solid water will be larger due to the non-linearity between the theoretical run-up level and overtopping volume (Andersen et al., 2011).

Scale effects breakwaters

Results of the CLASH project suggested significant differences between field and model results on wave overtopping. This has been verified for different sloping rubble mound structures. Results of the comparisons in that project have led to a scaling procedure which is mainly dependent on the roughness of the structure γ_f [-]; the seaward slope cotα of the structure [-]; and the mean overtopping discharge, based on small scale tests or predictions, but up-scaled to prototype, q_{us} [m3 /s per m] (Van der Meer et al., 2016). It is proven that wave run up on rough slopes is underestimated in small scale model tests if overtopping values are small (De Rouck et al., 2005). Scale effects are occurring if the mean overtopping rates are lower than about 1.0 l/s per m and may include significant adjustment factors for these rates (De Rouck et al., 2005). In the case of the Afsluitdijk most overtopping values from 1 tot 10 l/s/m should be observed, therefore hardly any scale effects should occur in this final design. Their are measurements which consists of low overtopping discharge, these will be discussed and analyzed if their are any scale effects.

2.4.2. Model effects

Model or laboratory effects originate from the incorrect reproduction of the prototype structure, geometry and waves and currents, or due to the boundary conditions of a wave flume (side walls, wave paddle, etc.). Modelling techniques are most of the time similar, but there are still influences of model effects on hydraulic model results to be expected. Measurement effects result from different measurement equipment used for sampling the data in the prototype and model situations. These effects, which are referred to as “measurement effects” may significantly influence the comparison of results between two identical models. It is therefore essential to quantify the effects and the uncertainty related to different flumes that are tested and refer these effects to model effects.

3 METHODOLOGY

3.1. Method to analyze the data

The methodology is the systematic, theoretical analysis of the methods applied on the data that is generated from the physical model tests that will be described in Chapter 4. It comprises the theoretical analysis of methods associated to answer the research questions mentioned in Chapter 1. Data that is gathered during this research consist of two different dike sections: 8b and 17a. These dike sections are tested in three different flumes (BAM, Schelde and Delta) mentioned later on in more detail. In Figure 3.1 a flow diagram is given in which the data of these different flumes and dike sections are analyzed. In this section the scheme is systematically explained, this systematic approach is the red line in the document. The document is divided into two main parts, where the two different dike sections are discussed. These two will give in the end a final conclusion about any possible scale effects with matching two identical set-ups in two different scales with each other.

Figure 3.1: Flow diagram of the methodology

3.1.1. Section 17a

In Figure 3.1, the left side of the figure represents the way in which the data of dike section 17a is discussed. This data consist of overtopping tests in two different flumes; the BAM flume (1:32.6) and Delta flume (1:2.95).

The data of the BAM flume is generated by myself and the data of the Delta Flume is generated by Deltares. The tests in the BAM flume are mainly focused on hydraulic parameters that are influencing the roughness of the upper slope.

In the first part the accuracy of γ_b in Equation 2.19 is discussed with data of the BAM flume. A totally smooth slope is tested. The tested cross-section is given in Figure 3.2. With this smooth slope all the other influence factors (γ_f, γ_β & γ_v) are equal to 1, so only the influence of γ_b could be determined. The berm coefficient is adjusted to increase the accuracy of the test. This adjusted berm coefficient is the starting point to look at the influence of different hydraulic parameters on the upper slope.

Secondly the reliability of Equation 2.4 to calculate γ_f of the upper slope is discussed with data of the BAM flume. This is done by looking at three different factors; q,H_{m0} and s_0. The tested cross-section consists of a smooth lower slope with a rib pattern on the upper slope, this cross-section is given in Figure 3.3. To look only at the influence of the amount of overtopping (q), the crest height is increased five times, reducing the amount of overtopping with each step. In every step whereby the crest height is increased, a separate test is performed with a larger wave height, to create the same amount of overtopping. With these separate tests, the influence of the wave height (H_{m0}) (with the same amount of overtopping) could be determined. Before the test, the amount of overtopping is estimated according to the theory, the wave height is chosen in a way that the theory gives a overtopping of 10 l/m/s. Lastly, a different wave steepness (s_0) is tested by increasing the wave period with the same cross-section.

In the end the data of the BAM and of the Delta Flume are discussed to answer the question: are there any scale effects between the BAM and Delta flume discernible and under which circumstances do these occur?

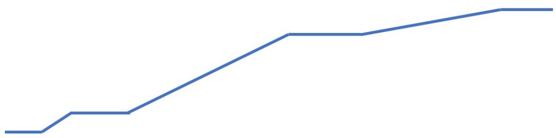

Figure 3.2: First cross-section 17a

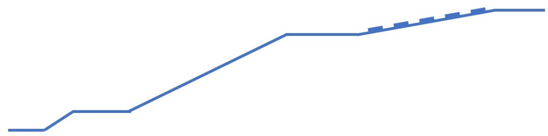

Figure 3.3: Second cross-section 17a

3.1.2. Section 8b

In Figure 3.1, the right side of the figure represents the way in which the data of dike section 8b is discussed. This data consists of overtopping tests in two different flumes; the BAM (1:32.6) and Schelde flume (1:19.8). The data of the BAM flume is generated by Levvel and the data of the Delta Flume is generated by Deltares. The tests in the BAM flume are focused on structural parameters that are influencing the roughness of the upper slope.

In the first part, γ_f of Levvel-blocs for the lower slope is discussed with data of the BAM flume. This is performed with a smooth upper slope to look only at the influence of the Levvel-blocs, the cross-section is given in Figure 3.4. This roughness coefficient for Levvel-blocs is the starting point to determine the influence of different structural parameters on the upper slope.

Secondly the reliability of Equation 2.4 to calculate γ_f of the upper slope is discussed with data of the BAM flume. This is done by looking at three different structural factors; rib height, rib pattern and amount of ribs. The tested cross-section consists of a lower slope with Levvel-blocs and a rib pattern on the upper slope, the tested cross-section is given in Figure 3.5. Three different rib heights, patterns and amount of ribs are tested in the BAM flume. Also the influence of the slope of the Levvel-blocs is discussed. The results of all these test in the BAM flume are checked with a verification test.

Thirdly the roughness coefficient of the Levvel-blocs on the lower slope is discussed with the data of the Schelde flume. Equation 2.4 is used to calculate γ_f of the upper slope for these tests. Because this equation is based on tests with the same scale as the Schelde flume.

At last the data of the BAM flume and of the Schelde Flume are discussed to answer the question: are there any scale effects between the BAM and Schelde flume discernible and under which circumstances do these occur?

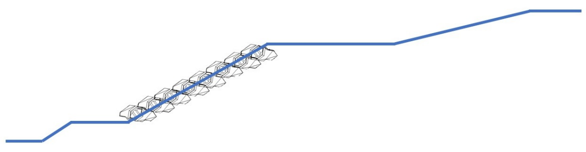

Figure 3.4: First cross-section 8b

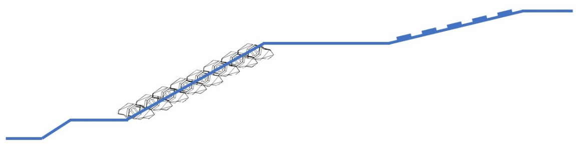

Figure 3.5: Second cross-section 8b

3.2. Method to analyze influence factors

For this research a common approach needs to be chosen for analyzing the overtopping results. As given in the previous chapter, the amount of overtopping could be calculated by Equation 2.19 because waves will break on the berm and the gentle upper slope. In Figure 3.6 both equations for non-breaking and breaking waves are given. Overtopping tests of the Afsluitdijk are used to validate the two different formulas. In Figure 3.6 can be seen that the data that is analyzed with Equation 2.19 in blue is matching better than the data that is analyzed with Equation 2.18 given in red. The black crosses are data where the berm coefficient is introduced in Equation 2.18, which will decrease the error. The calculation error on the amount of overtopping is smallest with Equation 2.19 and also the different influence factors in this research are covered better with this Equation. Equation 2.19 will be used in this document.

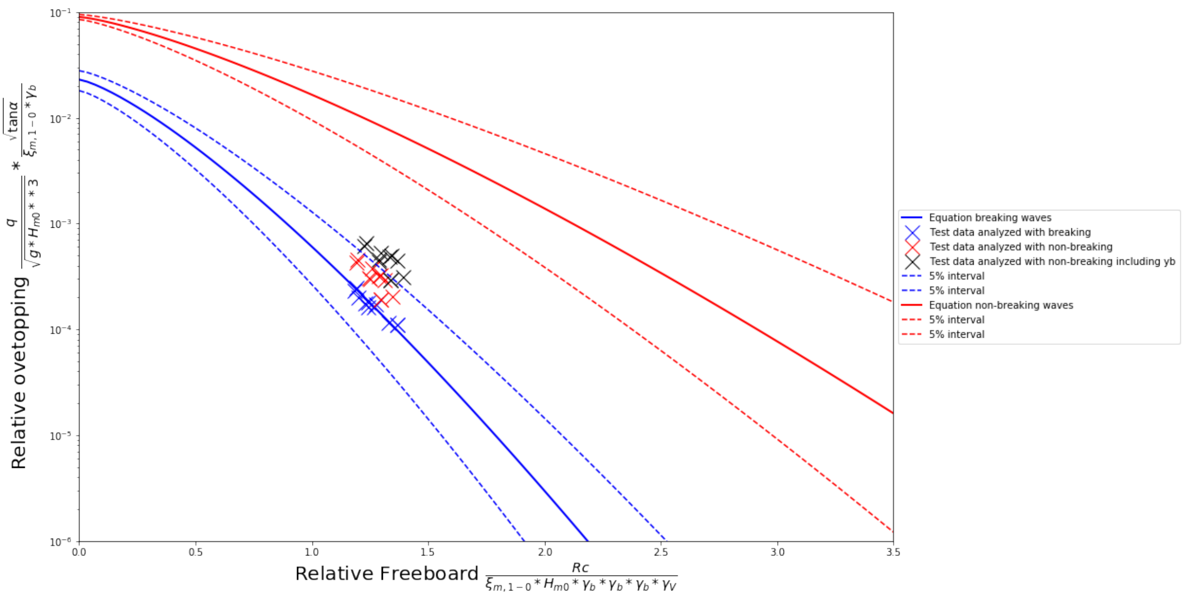

Figure 3.6: Difference between Eq 2.18 en 2.19

3.2.1. Calculated coefficient

First of all the overtopping is calculated according to the procedure which is described above. All the different influence factors are calculated as described in Chapter 2, the calculation is given in more detail in the python script in Appendix F. In Figure 3.7 the blue line representing Equation 3.1 is plotted on a logarithmic scale. The x-axis presents the relative freeboard $\frac{R_c}{H_{m0}}$ and the y-axis is presenting the relative overtopping discharge $\frac{q}{\sqrt{g \cdot H_{m0}{}^3}}$. The black dots are the measured overtopping rates of the test plotted with all the influence factors that are calculated according the theory. If these black dots are exactly on the blue line, the measured and calculated overtopping are exactly the same, the calculation method is in that case accurate. More tests with the same properties increase the reliability of the outcome. If there is a difference between the black dots and the blue line, a influence factor in the formula could be changed to match the measured overtopping with the calculated overtopping. This is described in more detail in the next section.

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}} \gamma_b \xi_{m-1,0} \, exp\left[-\left(2.7 \frac{R_c}{\xi_{m-1,0} H_{m0} \gamma_{f,calculated} \gamma_b \gamma_\beta \gamma_v}\right)^{1.3}\right] \tag{3.1}$$

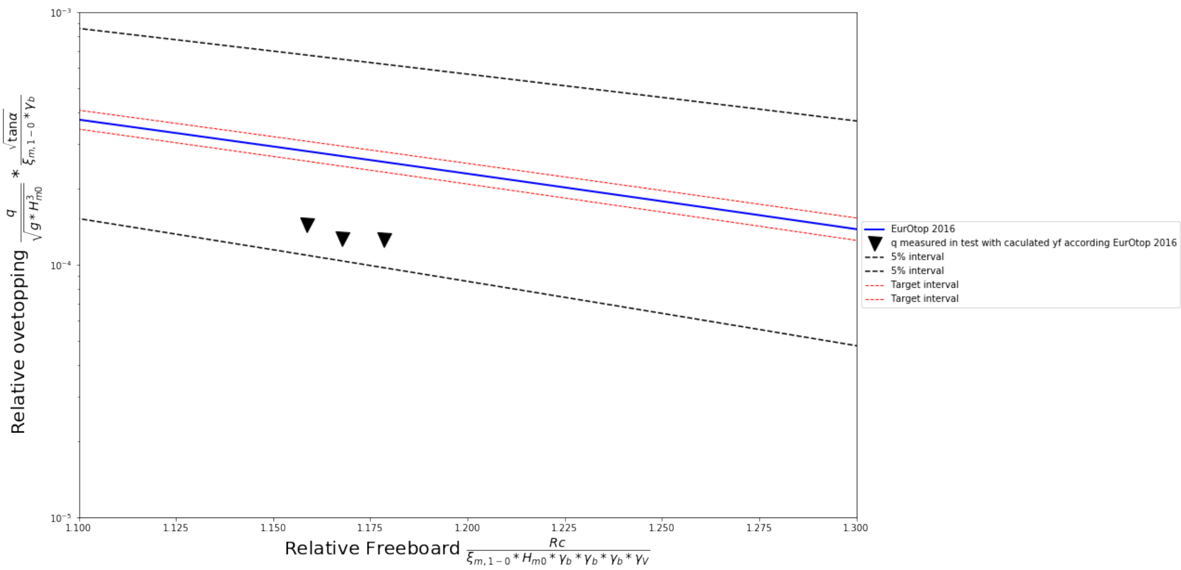

Figure 3.7: Three measured data points plotted with calculated roughness.

3.2.2. Fitted coefficient

In the previous section there is a difference between the measured overtopping and calculated overtopping. In this part one of the coefficient in Equation 3.2 is changed: γ_f of the upper slope, all the other influence factors are 1. With the change of γ_f the calculated overtopping is fitted to the measured overtopping. In Figure 3.8 the calculated and measured overtopping are matched by decreasing the roughness coefficient of the upper slope to 0.62. According to the physical model test, the roughness coefficient of the upper slope should be 0.62. This is rougher than expected by the theory. The fitted roughness coefficient is lower than the calculated roughness with the same hydraulic conditions. In this document different influence factors are fitted according this procedure, in the beginning of each fit the specific factor is mentioned.

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}}\gamma_b \xi_{m-1,0}\, exp\left[-\left(2.7\frac{R_c}{\xi_{m-1,0}\, H_{m0}\, \gamma_{f,fitted}\, \gamma_b \gamma_\beta \gamma_v}\right)^{1.3}\right] \tag{3.2}$$

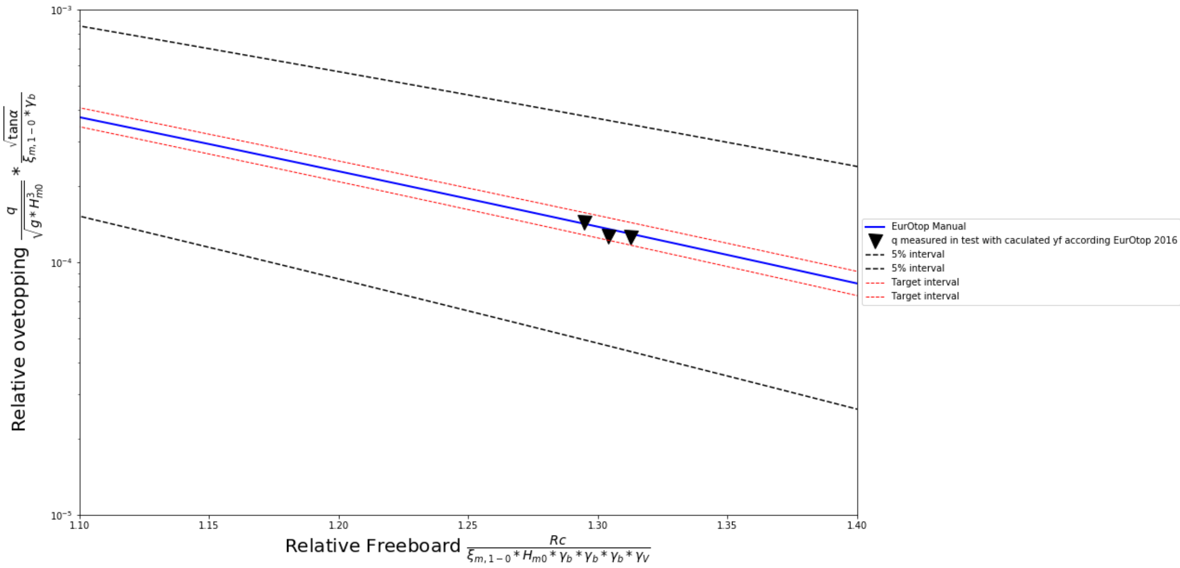

Figure 3.8: Three measured data points plotted with fitted roughness.

4

EXPERIMENTAL SET-UP

This chapter contains the laboratory set up and the testing procedure of this research. The objective of the physical tests is to obtain accurate measurements which intend to give answers to the scope of the study; the factors that are influencing amount of overtopping and the possible scale effects. Firstly the geometry and the material of the structure is mentioned. Secondly the different instrumentation and measurements are discussed. Finally the test program is defined for every test with the different conditions.

4.1. Introduction

The desirable outcome of this study is a reliable method to have an accurate estimation on the amount of wave overtopping on small scale and is this estimation applicable in prototype without any scale effects? Different tests will consider the parameters that influence the amount of overtopping on different scales.

The main step in this research is to design and construct different physical models that will express clearly the overtopping process of the Afsluitdijk. Since overtopping is an interaction of several fundamental phenomena, an issue of critical importance is the selection of parameters which are mainly influencing the amount of overtopping. The physical model has to able to simulate in the best possible way the selected parameters and at the same time provide well funded data.

4.2. Scaling process

Scale modelling must guarantee similarity in behaviour between the prototype and the model. Three types of similarity can be achieved: geometric similarity, kinematical similarity and dynamic similarity. To fulfill dynamic similarity, scaling needs to follow three laws based on Froude number, Reynolds number and Weber number. Geometric scaling between prototype and model is fulfilled by applying a certain scaling factor to the structural dimensions. However, scaling all the processes that occur in a breakwater is not possible since intrinsic properties of the fluid such as viscosity, surface tension and air content cannot be scaled for the same model. Since overtopping phenomenon can be mainly considered as free surface flow and to reproduce waves correctly, Froude scaling is applied, so inertia is assumed dominant over viscosity. Viscosity forces are governed by Reynolds' law, elasticity by Cauchy's law and surface tension forces by Weber's law, and these forces have to be neglected for most models. All effects and errors resulting from ignoring these forces are called scale effects. The problem of the quantification of all these scale effects is still unresolved, but research gave a little more insight in scale effects with wave overtopping. EurOtop (2016) lists some generic rules that should be observed for physical model studies to avoid scale effects:

- Water depths in the models should be much larger than h = 2 cm, wave periods larger than T = 0.35 s and wave heights larger than H_s = 5 cm to avoid the effects of surface tension. (Van der Meer et al., 2016)
- For rubble mound breakwaters, the Reynolds number for the stability of the armour layers should be exceed $Re = \frac{D_{n50}\sqrt{g \cdot H_{m0}}}{\nu} > 3\text{x}10^4$. (Dai and Kamel, 1969)

- For overtopping of coastal dikes $Re = \frac{h\sqrt{g \cdot H_{m0}}}{\nu} > 1x10^3$. (Andersen et al., 2011)

So in general these rules must be applied for all the different model tests. The general are calculated for the BAM flume (1:32.6):

- Water depth = 0.48 > 0.02 m
  Hs = 0.12 > 0.05 m
  T = 1.26 > 0.35 s
- Height of ribs protoype = 0.05 m, BAM flume = 0.05/32.6 = 0.0015
  $Re = \frac{0.0015\sqrt{9.81 \cdot 0.12}}{1 \cdot 10^{-6}} = 1627.48 < 3x10^4$.
  Levvel blocs protoype = 0.65 m, BAM flume = 0.65/32.6 = 0.02
  $Re = \frac{0.02\sqrt{9.81 \cdot 0.12}}{1 \cdot 10^{-6}} = 21699.8 < 3x10^4$.
- $Re = \frac{0.48\sqrt{9.81 \cdot 0.12}}{1 \cdot 10^{-6}} = 520794.47 > 1x10^3$.

With all the specification of the BAM flume the generic rules to avoid scale effects are observed. The first generic rule to avoid scale effects is exceeded. The water depth, wave height and wave period are larger than the critical lower limit so scale effects on surface tension are avoided. The d_{n50} of both the Levvel-blocs (0.65 m, smallest length) and the ribs (0.05 m, smallest height) gives a Reynolds number that is smaller than $3x10^4$, so the stability of the armour units is critical. For overtopping of coastal dikes, the Reynolds number is exceeding $1x10^3$ so their will not be any scale effects on overtopping.

4.3. Model configuration

Due to the fact that this research is part of the design of the new Afsluitdijk, the physical model tests are based on the prototype of the Afsluitdijk. The parameters of the prototype are scaled to the different physical model tests, this procedure is discussed in the previous section. In this document values of the prototype are mentioned.

The set-up of the tested cross-section has been designed by the project team of Levvel. In this document two different dike sections are discussed; 8b an 17a. The main differences between these two sections are the foreshore, hydraulic conditions, lower slope, upper slope and the length of the berm. The hydraulic conditions are discussed later on in the test program. First the foreshore of both dike section is mentioned. Secondly the cross-sections of dike section 17a and 8b are discussed.

4.3.1. Foreshore

In Table 4.1 the dimensions of the foreshores are given. These dimensions are based on the document of Rijkswaterstaat "schematizing foreshore". The foreshore represent the depth limited Waddenzee, due to this smaller depth, waves start to break and loose their energy. In Figure 4.1 the RIP-limit is 5, at this point the foreshore ends and the structure begins. In Table 4.1 the foreshore of both different dike sections is given. Building a foreshore in the flume takes a lot of time, so it is chosen to work with the same foreshore for dike section 17a and 8b. The original foreshore of section 17a is changed in such a way, that the change in bottom profile is the same as in section 8b. This will lead to the same bottom profile of 8b and 17a, which only change in vertical direction. The vertical distance between the two dike profiles is 3,35 meter. The bottom profile of 17a is used as the foreshore for all the physical model tests. For tests with dike section 8b, the level of NAP is increased with 3.35 meter, the foreshore is not changed.

Table 4.1: Foreshore Afsluitdijk.

Point	Prototype bottom location 8b	Prototype bottom location 17a
	(m NAP)	(m NAP)
1	-13.50	-10.06
2	-13.50	-10.06
3	-6.52	-3.08
4	-6.52	-3.08
5	-4.51	-1.07

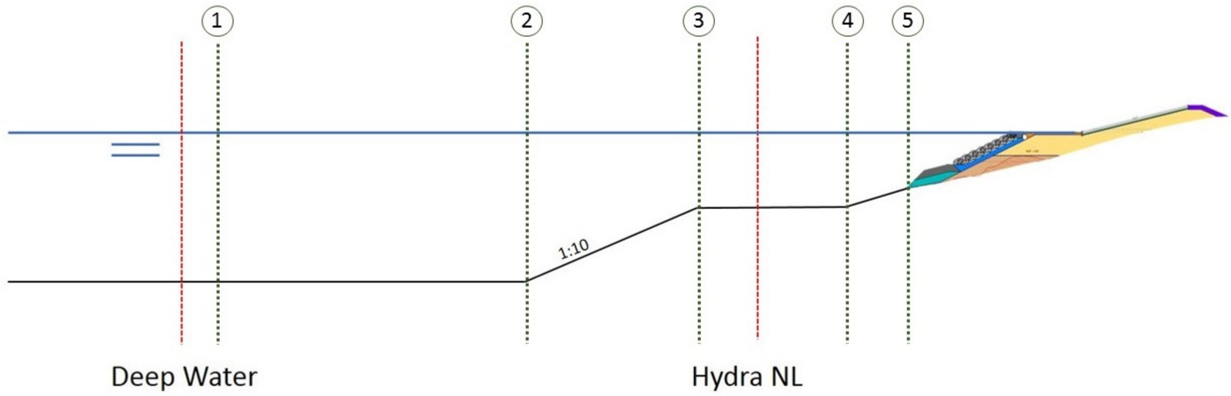

Figure 4.1: Foreshore Afsluitdijk, both 17a and 8b.

4.3.2. Cross-section section 17a

In this part the different cross-section of dike section 17a are discussed. In Figure 4.2 the five main aspects of the dike are given, these five aspects will be discussed below. In Table 4.2 all the different cross sections that are tested are mentioned. The first tests are done in the BAM flume, the other tests are done in the Delta flume.

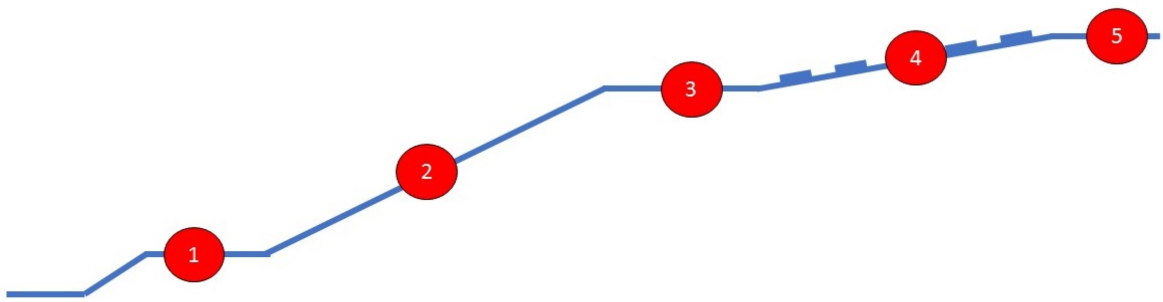

Figure 4.2: Cross section 17a.

Height toe (1)

The height of the toe is in all the test 0.5 meter above NAP.

Lower slope (2)

The lower slope of the dike is smooth, in the BAM flume this is modelled with timber. The slope of the lower part is 1:3.5 for all the tests. In the Delta flume it is modelled as prototype.

Berm (3)

The Berm is in all the test 5 meter, the berm is made of smooth asphalt. This asphalt is modelled in the BAM flume with timber. In the Delta flume it is modelled as prototype.

Upper slope (4)

The slope of the upper part is 1:4.5 in all the tests. The first cross section in the BAM flume is a smooth top slope, with this smooth top slope, the whole structure is smooth, and the influence of the berm coefficient could be discussed. The other test of the BAM flume are performed with a block revetment given in Figure 4.3. The amount of ribs is changed due to a change in crest height, which will give an longer upper slope. The height of the ribs is 0.05 meter in the prototype. The first cross section which is tested in the Delta flume consist of pattern 10c with ribs of 0.05 meter. The second cross-section consist of pattern 2 withe ribs of 0.21 meter height to increase the roughness of the upper slope and create less overtopping.

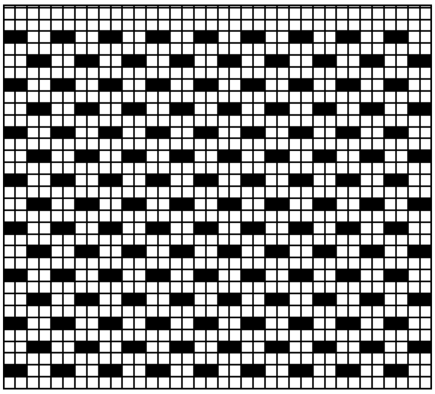

Figure 4.3: Pattern 10c

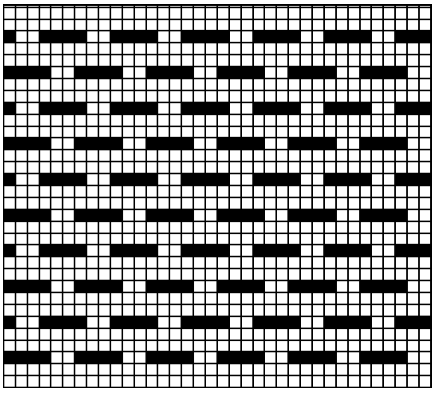

Figure 4.4: Pattern 2

The upper slope consist of Quattro-blocs, Figure 4.5, the blocs have a size of 0.55m by 0,55m , with a maximum height of 0,53 meter. The Quattro-blocs are placed over the upper slope with different heights. These different heights will introduce the ribs and create extra roughness on the slope. In Figure 4.6 there are used two different Quattro-blocs of 0,41 m and 0,2 meter. This height difference between the two blocs will introduce a rib height of 0,21 m.

Figure 4.5: Quattro-bloc.

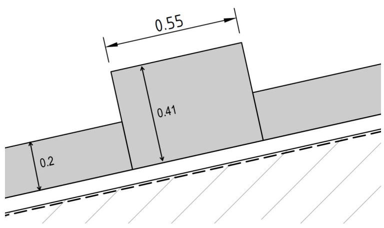

Figure 4.6: Quattro-bloc height.

Crest height (5)

The crest height is changed five times in the BAM flume. This higher crest will introduce a lower overtopping rate. With these results the effect of overtopping discharge on the roughness of the upper slope could be determined.

Table 4.2: Cross sections that are tested 17a.

Test code	Flume	Section	Crest height	Lower slope	Typ Lower slope	Berm length	Upper slope	Type Upper slope	Ribs	Height ribs	Height toe
			m NAP	α		m	α			m	m NAP
B_17a_1	BAM	17a	9	1:3.5	Smooth	5	1:4.5	Smooth			0.5
B_17a_2	BAM	17a	9	1:3.5	Smooth	5	1:4.5	10c	14	0.05	0.5
B_17a_3	BAM	17a	9.448	1:3.5	Smooth	5	1:4.5	10c	16	0.05	0.5
B_17a_4	BAM	17a	9.896	1:3.5	Smooth	5	1:4.5	10c	18	0.05	0.5
B_17a_5	BAM	17a	10.344	1:3.5	Smooth	5	1:4.5	10c	20	0.05	0.5
B_17a_6	BAM	17a	10.792	1:3.5	Smooth	5	1:4.5	10c	22	0.05	0.5
D_17a_1	Delta	17a	9	1:3.5	Smooth	5	1:4.5	10c	10	0.05	0.5
D_17a_2	Delta	17a	9	1:3.5	Smooth	5	1:4.5	2	10	0.21	0.5

4.3.3. Cross-section 8b

In this part the different cross-sections of dike section 8b are discussed. In Figure 4.7 the five main aspects of the dike are given, these five aspects will be discussed below. In Table 4.3 all the different cross sections that are tested are given. The first tests are performed in the BAM flume, the second tests are performed in the Schelde Flume.

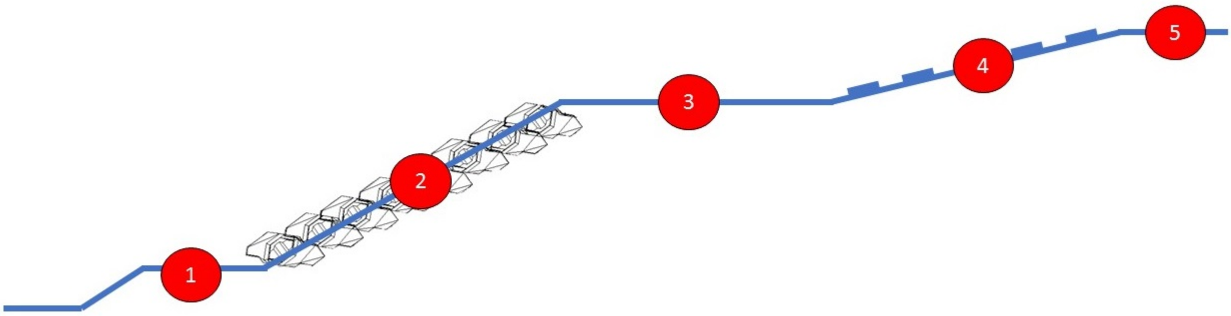

Figure 4.7: Cross section 8b.

Height toe (1)

In all the test of the BAM flume, the height of the toe is 0.5 meter below NAP. For the tests in the Schelde Flume, the toe is changing from 0.5 below and 0.16 above NAP. These changes in the toe are made to ensure the stability of the bottom Levvel-bloc, but are also important for the amount of overtopping.

Lower slope (2)

The slope of the lower part is 1:2 or 1:1.5 in all the tests. For optimization of the length of the dike, a steeper slope of 1:1.5 is introduced. In the end it came out that a slope of 1:2 fits better in the design. The influence of the slope on the roughness coefficient will be discussed later on. The lower slope of the structure consist of Levvel-blocs which are given in Figure 4.8, the original name of the Levvel-blocs is Xbloc-plus, but in the design of the Afstluitdijk they are called Levvel-blocs. These Levvel-blocs are used due to its aesthetic appearance, fast and easy placement, lowest concrete assumption. The Levvel-blocs are placed in a regular pattern on top of each other, this regular pattern is given in Figure 4.9.

Figure 4.8: Levvelbloc.

Figure 4.9: Levvelbloc pattern.

Berm (3)

The berm is different and is changed between 8 and 12 meter. The berm is a made of smooth asphalt, this asphalt is modelled in the BAM and Schelde flume with timber.

Upper slope (4)

The slope of the upper part is 1:4 in all the tests and consist of a block revetment. The first cross sections in the BAM flume are with a smooth top slope, with this smooth top slope the roughness coefficient of the Levvel-blocs of the lower slope could be discussed. The other tests of the BAM flume are performed with rib patterns given in Figure 4.10 to 4.13. For optimize of the design, the height, shape, pattern and amount of the ribs is changed during the test in the BAM and Schelde flume. These different patterns and their influence on the roughness will be discussed later on. The cross section that is tested in the Delta flume consist of pattern 2 with ribs of 0.25 meter.

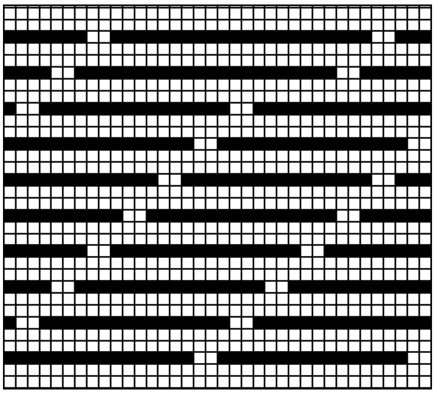

Figure 4.10: Pattern 8

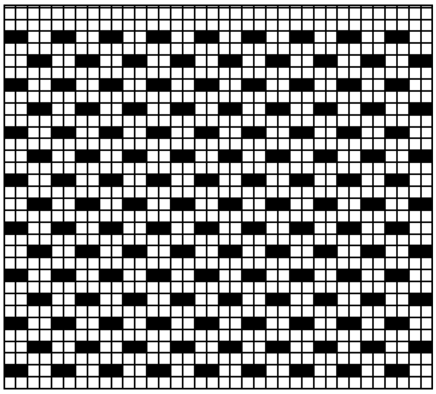

Figure 4.11: Pattern 10c

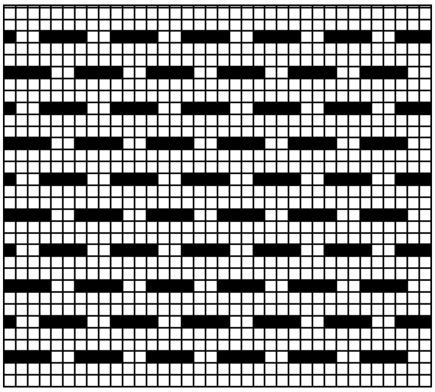

Figure 4.12: Pattern 2

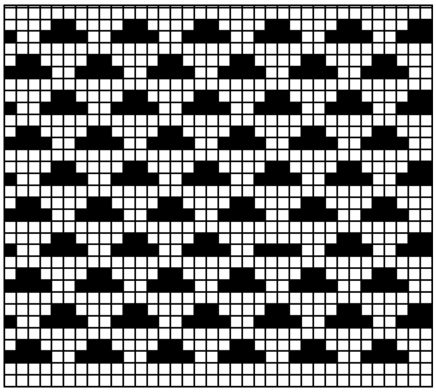

Figure 4.13: Pattern 2t

Crest height (5)

In the optimization of the roughness coefficient of the lower slope, the crest height is changed for two times in the BAM flume: 10.07 and 10.64. In the optimization of the rib patterns, the crest height is 10.05. In the Schelde en Delta flume the crest height is in all the tests 9.87 meter.

Table 4.3: Cross sections that are tested 8b.

Test code	Flume	Section	Crest height	Lower slope	Typ Lower slope	Berm length	Upper slope	Type Upper slope	Ribs	Height ribs	Height toe
			m NAP	α		m	α			m	m NAP
B_8b_1	BAM	8b	10.07	1:2	Levelbloc	11	1:4	Smooth			-0.5
B_8b_2	BAM	8b	10.64	1:2	Levelbloc	11	1:4	Smooth			-0.5
B_8b_3	BAM	8b	10.07	1:2	Levelbloc	8.5	1:4	Smooth			-0.5
B_8b_4	BAM	8b	10.07	1:2	Levelbloc	12	1:4	Smooth			-0.5
B_8b_5	BAM	8b	10.05	1:2	Levelbloc	11	1:4	8	11	0.12	-0.5
B_8b_6	BAM	8b	10.05	1:2	Levelbloc	11	1:4	8	11	0.18	-0.5
B_8b_7	BAM	8b	10.05	1:2	Levelbloc	11	1:4	8	11	0.23	-0.5
B_8b_8	BAM	8b	10.05	1:2	Levelbloc	11	1:4	8	6	0.23	-0.5
B_8b_9	BAM	8b	10.05	1:2	Levelbloc	11	1:4	8	3	0.23	-0.5
B_8b_11	BAM	8b	10.05	1:2	Levelbloc	11	1:4	2	11	0.23	-0.5
B_8b_12	BAM	8b	10.05	1:2	Levelbloc	11	1:4	2t	11	0.23	-0.5
B_8b_13	BAM	8b	10.05	1:1.5	Levelbloc	11	1:4	2	11	0.23	-0.5
B_8b_14	BAM	8b	10.05	1:1.5	Levelbloc	11	1:4	10c	11	0.23	-0.5
B_8b_15	BAM	8b	9.87	1:2	Levelbloc	12	1:4	2	11	0.23	-0.5
B_8b_16	BAM	8b	9.87	1:1.5	Levelbloc	12	1:4	2	11	0.23	-0.5
S_8b_1	Schelde	8b	9.75	1:1.5	Levelbloc	12	1:4	10c	15	0.23	0.16
S_8b_2	Schelde	8b	9.75	1:1.5	Levelbloc	12	1:4	2	10	0.23	0.16
S_8b_3	Schelde	8b	9.75	1:2	Levelbloc	12	1:4	2	10	0.25	-0.55
S_8b_4	Schelde	8b	9.75	1:2	Levelbloc	11	1:4	2	10	0.25	-0.55
S_8b_5	Schelde	8b	9.75	1:2	Levelbloc	11	1:4	2	10	0.25	-0.55
S_8b_6	Schelde	8b	9.75	1:2	Levelbloc	9.5	1:4	2	10	0.25	-0.55

4.4. Laboratory

4.4.1. Wave flumes

The experimental tests have been performed in the 2D wave flume of Delta Marine Consultants located in Utrecht, given in Figure 4.14. The flume has a length of 25 m, a width of 0.6 m and a height of side walls of 1.0 m. It allows water depths between 40 and 75 cm and it can be filled and emptied with pump valves which are placed on both sides of the flume.

Figure 4.14: Wave flume BAM.

The Schelde Flume is a wave flume of Deltares with a total length of 110 m, width of 1 m and height of 1.2 m (Deltares, 2015), given in Figure 4.15. The flume has wave generators at both sides. There is a pumping system installed at one side of the flume. The model tests for the Afsluitdijk are performed in half of the wave flume.

The Delta flume is 300 metres long, 9.5 metres deep and 5 metres wide of Deltares, given in Figure 4.16. The depth makes it possible to generate waves up to 4.5 metres high. This requires a 10-metre-high wave board that moves to and for using hydraulic cylinders (Deltares, 2015).

Figure 4.15: Schelde flume

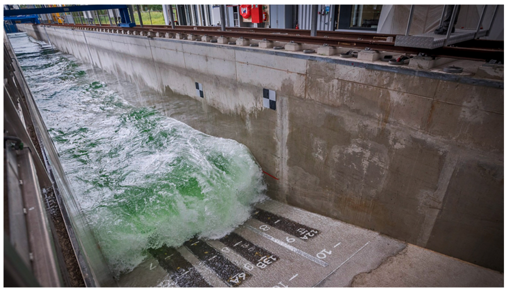

Figure 4.16: Delta flume

4.4.2. Wave generator

The wave are generated by a fully absorbing piston type spectral wave maker provided by Edinburg Designs. The wave maker is able to measure the reflected wave and correct the paddle motion to absorb it. The incoming and reflected wave fields are measured by resistance wave gauges and based on the method of Mansard and Funke (Barthel et al., 1983), which is implemented in the WaveLab software.

The wave generation of the piston is based on a file which contains all the wave information such as the significant wave height and peak period, the type of the spectrum, its characteristics and the duration, this is outlined in the test program.

The maximum wave height which can be generated in the BAM flume is a significant wave height of H_{m0} = 0.2 m, this is higher than the wave height which are tested. The maximum water depth in the flume is 0.75 m.

4.4.3. Wave gauges

In the BAM flume two sets of wave gauges are placed in the wave flume, one near the structure and one in deep water conditions, each set consist of three wave gauges. Changes in wave conditions during the propagation through the wave flume can be noticed in this way. The wave gauges at deep water are located just before the foreshore of the structure. The position of the other wave gauges is as close as possible to the structure. Water depth should be constant for all 3 wave gauges. Thus, these 3 wave gauges need to be placed on the horizontal part of the foreshore and in front of the toe. The differences in voltage between the two poles of the wave gauge are converted in the differences in water level. The water levels and corresponding voltages are established by several calibrations of the wave gauges. This is executed by measuring the voltage for several known water levels.

In the Schelde flume two sets of wave gauges are placed in the wave flume, one near the structure and one in deep water conditions, each set consist of four wave gauges. Changes in wave conditions during the propagation through the wave flume can be noticed in this way. The wave gauges at deep water are located just before the foreshore of the structure.

In the Delta flume one set of wave gauges is placed in the wave flume at deep water conditions and consist of three wave gauges. The Hydra NL conditions just in front of the structure are not noticed in this case. Therefore the decrease and increase in wave height and wave period are interpolated with the relation of the Schelde flume, this method is mentioned later on.

4.4.4. Overtopping tank and chute

In the BAM flume the overtopping water is collected behind the structure into a bucket. The volume of water is weighed using an electronic balance. The dimensions of the chute are 0.25 m or 0.6 m depending on the structure, in the last tests a cute of 0.6 m is used. The overtopping water is collected behind the structure and into a bucket, this bucket floats behind the structure. If the bucket is floating, there isn't any loss of water in the flume. A loss of around 10 liters will lead to a lowering SWL of a couple of millimeters. This lowering SWL in the model will lead to a lowering SWL of a couple of centimeters at prototype level. The bucket is placed at a balance as a result of which the overtopping amount of water can be measured. Together with the run time, which is measured with Wave lab, the mean overtopping discharge in m^3/s can be calculated.

In the Schelde en Delta flume the overtopping water is collected behind the structure into a tank. The water elevation in the tank is measured, this is multiplied with the area to calculate the volume of overtopped water. Together with the run time, the mean overtopping discharge in m^3/s can be calculated.

4.4.5. Test procedure

The performance of a test consists of three main phases, these are described next.

Before the test

Before start testing, laboratory equipment need to be calibrated and checked. The initial water level need to be checked. The input files with the corresponding wave characteristics are created and the overtopping bucket need to be weighted empty in the BAM flume. The water level in the Tank need to me measured in the Schelde en Delta flume.

Wave generation

During the performance of the test wave data is recorded by the wave gauges. In case of initiation of armour failure, the test is stopped before the filter layer is damaged.

After the test

After the wave generation is completed, the wave generator is switched off. After this the recorded wave data is analyzed in order to confirm the reliability of the test. Finally, the water depth is measured and the overtopping water is weighted or measured. The overtopping water is returned in the BAM flume in order to keep a constant water depth during the tests.

4.5. Hydraulic conditions

Different hydraulic conditions are tested with different cross sections, the target hydraulic conditions are given in Table A.1. Due to different effects, the actual measured conditions could be different. In the analysis phase, the actual measured conditions at the Hydra NL point are used. These conditions are used to calculate the amount of overtopping and compare it to the actual measured overtopping.

4.5.1. Cross-section 17a

Test 01b is the condition to determine the maximum overtopping and S02 to determine the stability for dike section 17a. These conditions are prescribed by Rijkswaterstaat and need to be calibrated to Deep water in the different flumes. H1a-H1d and S01a-S01b in Table A.1 are not prescribed by Rijkswaterstaat and are calibrated to Hydra NL. Calibrate these conditions to Hydra NL will lead to less scatter between the measured and target data at this point. H1a-H1d are conditions whereby the the wave height is increased. This increase of wave height is performed in combination with a crest height, to analyze the influence of the wave height on the roughness of the upper slope. S01a-S01b are conditions whereby the wave steepness is decreased. This decrease of wave steepness is performed to analyze the influence of the wave steepness on the roughness of the upper slope.

4.5.2. Cross-section 8b

The first three test codes are conditions that are tested on section 8b. These conditions are prescribed by Rijkswaterstaat and need to be calibrated to Deep water conditions in the different flumes. This is done by increase or decrease the input waves that are generated by the wave generator. 01a is the condition for section 8b to determine the maximum amount of overtopping. S2MHW and S2MTW are testscodes to test the stability of the structure, also the amount of overtopping is measured in these test and could be used in this research. S2MTW is the test code for stability with mean low water, and is given in Figure A.1 at the left. S2MHW is a stability with mean high water and is given in Figure A.1 in the middle. After a mean high water stability test their is always another mean low water test.

4.6. Test program

In the test program both the cross-sections and hydraulic conditions are changed. In Table B.1 of the appendix all the tests are outlined. In the rest of this document the test code is noted if these test is discussed in a specific part. Some of the test are performed a couple of times to increase the reliability of the test, this is given in the last column of the table.

5

ANALYSIS 17a

In this chapter the results of the tests for dike section 17a are analyzed. Firstly the hydraulic conditions of the model tests in the BAM flume are discussed. After this the data is analyzed on different components. First of all the berm coefficient for a smooth slope is discussed. The influence of wave overtopping, wave height and wave steepness on the roughness coefficient of the upper slope is discussed secondly. After the analysis of the tests in the BAM Flume the hydraulic conditions in the Delta flume are discussed. With these hydraulic conditions the final design that is tested in the Delta Flume is discussed.

5.1. Hydraulic conditions BAM Flume

Prior to analyzing the results, wave conditions that are performed need to be analyzed. Due to different hydraulic effects the wave height at deep water conditions differs from the waves near the structure. This is caused due to the limited depth and result in shoaling and wave breaking. The wave is partly reflected by the structure. In order to optimize the desired wave conditions, a previous calibration without the structure is performed. The best fitted gain factor of the wave generator is used to perform as exact as possible the waves. In the test series, all the conditions are fitted to the Hydra NL point, in order to get best wave conditions at the structure. The conditions are fitted to the Hydra NL to predict as exact as possible the amount of overtopping that will be measured in the test.

5.1.1. Wave height

In the test program, Table B.1, different conditions are described to expect an amount of overtopping that is calculated with the theoretical equation. It is not possible to get exact the conditions in the wave flume as prescribed, due to uncertainty of different hydraulic processes. In the analysis of the results later on, the measured wave conditions at the Hydra NL point in the wave flume are used. In Figure D.1 the target wave height at deep water is plotted against the measured wave condition. In Figure 5.1 the same procedure is done for the Hydra NL point in front of the structure. In this test series the wave conditions are fitted to the Hydra NL point. This can also be seen in the figures, the wave conditions are better fitted for the Hydra NL point. The three black points in Figure 5.1 are not taking into account, these wave heights measured incorrect due to an defective wave gauge.

During the performance of the test, the wave height was measured at deep water and near the structure. Figure D.2 presents the relationship between the spectral significant wave height, H_{m0}, at those two locations. Wave breaking caused by depth limited conditions makes the measured wave height decreases. This can be observed in Figure D.2 where it is clearly noticed that wave height near the structure is lower than in deep water. Due to wave breaking, the spectral significant wave height, H_{m0}, also varies from the statistical significant wave height $H_{2\%}$. Figure 5.2 describes the relationship between these two measured wave height near the structure, the ratio between $H_{2\%}$ and H_{m0} is 1.1.

5.1.2. Wave period

In this document, the wave period is estimated from spectral analysis performed with WaveLab. Figure 5.3 & 5.4 show the relationship between wave periods measured at deep water and near the structure which rep-

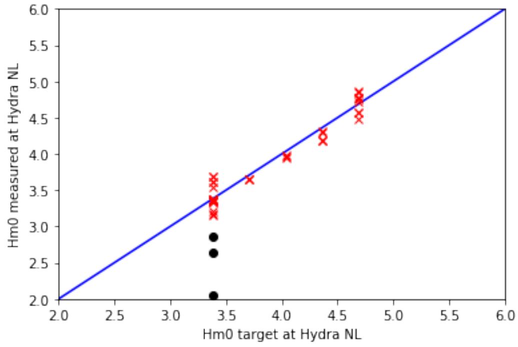

Figure 5.1: Hm0 target plotted against the measured Hm0 at Hydra NL.

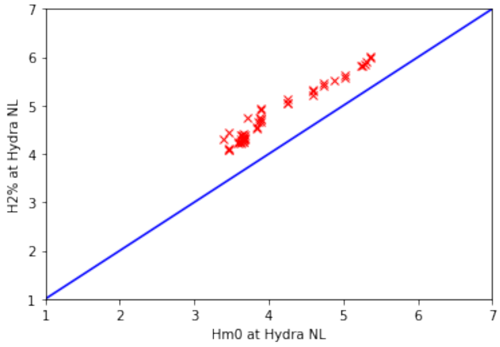

Figure 5.2: Hm0 plotted against H2% at Hydra NL.

resents intermediate waters. Less scatter is obtained for the average spectral wave period compared with the peak period. In the EurOtop Manual (Van der Meer et al., 2016) a relationship between the spectral peak

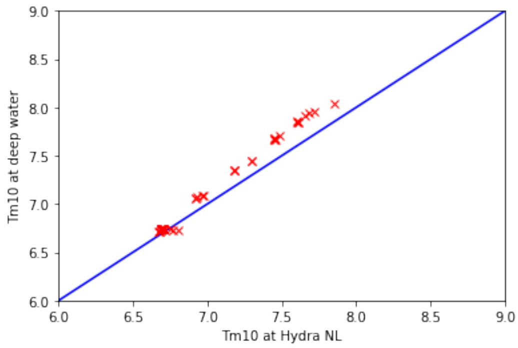

Figure 5.3: Tm-1,0 at Hydra NL plotted against Tm-1,0 at deep water.

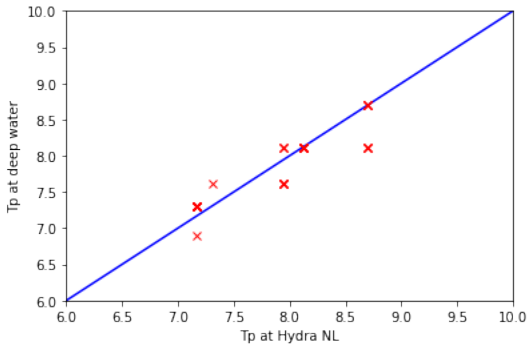

Figure 5.4: Tp at Hydra NL plotted against Tp at deep water.

wave period (T_p) and the spectral average wave period ($Tm_{-1,0}$) for a single peaked spectrum with Rayleigh distribution in deep water where T_p is assumed 1.1 times $Tm_{-1,0}$. However, in shallow water this relationship is not always valid as the spectrum in shallow water deviates from the spectrum in deep water due to wave breaking. Figure 5.5 shows the relationship between these two spectral wave periods at the Hydra NL point.

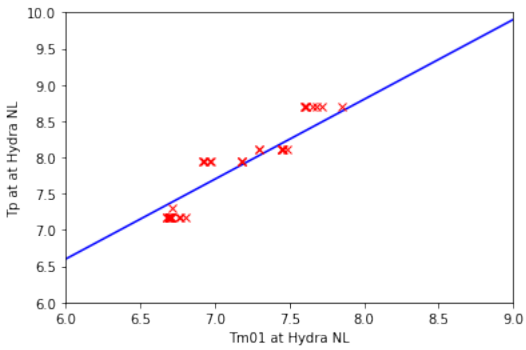

Figure 5.5: Tm-1,0 plotted against Tp at Hydra NL.

5.1.3. Wave steepness

The wave steepness is kept constant during the tests. The measured wave nominal wave steepness near the structure is presented in Figure 5.6. This is the nominal wave steepness at Hydra NL plotted against the wave steepness at deep water. The three black dots are measurement errors that are described earlier at the wave height section.

It can be observed that the target wave steepness near the structure is achieved fairly well in most of the tests. However, the difference between measured and target wave steepness should be more noticeable for the higher wave steepness. This is due to the loss of energy occurring when waves break (Van der Meer et al., 2016). In Figure 5.7 this difference is not noticeable, this is mainly because the lack of tests for the lower steepness.

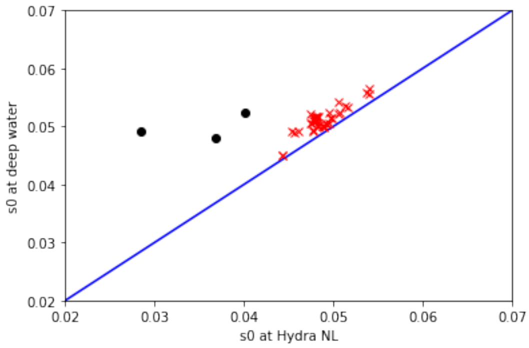

Figure 5.6: s0 at deep water plotted against s0 at Hydra NL.

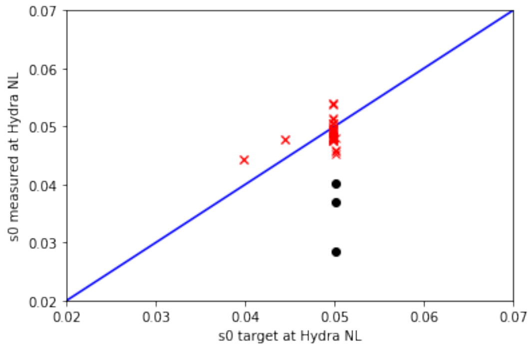

Figure 5.7: s0 target plotted against s0 measured at Hydra NL.

5.2. Analysis results BAM flume

In this part the test 1 to 13 from Table B.1 are discussed. These tests are focused on the influence of the berm coefficient with a total smooth slope, given in Figure 5.8. Secondly the influence of three different hydraulic parameters on the roughness of the upper slope is discussed, these tests are performed with a smooth lower slope and ribs on the upper slope, given in Figure 5.9.

Figure 5.8: First cross-section 17a

Figure 5.9: Second cross-section 17a

5.2.1. Berm coefficient

With the total smooth slope, given in Figure 5.8, the influence of the berm coefficient is determined. The difference in the measured overtopping and calculated overtoping are given in Figure E.2. The points in the figure outline the measured overtopping in this test series and are higher than the blue line. A wrong estimation of the γ_b could be a factor for this problem. In Table 5.1 a constant value is added to γ_b to perform reduce the error between the measured and calculated overtopping.

Table 5.1: Adjusted berm coefficients.

Change in Equation	Mean difference
+0.026	0.465
+0.027	0.459
+0.028	0.464

With this adjusted factor the value of the γ_b is calculated by the following Equation:

$$\gamma_b = max(0.6, 1.0 - r_B \cdot (1.0 - r_{db})) + 0.027 \tag{5.1}$$

This adjusted berm coefficient is used as a starting point to look at the influence factors on upper slope. All the tests in the BAM flume for 17a are performed under the same conditions with the same berm length. It is doubtful if the difference is only from the berm coefficient, or due to model errors. The difference is with the adjusted berm coefficient covered in the berm, but could also be the cause of another phenomena.

5.2.2. Mean overtopping on roughness coefficient

In this part test numbers 2,3,5,7,9 from Table B.1 are discussed. These tests are carried out with a smooth lower slope and a upper slope with ribs and a different crest height, given in Figure 5.9. Due to a higher crest height, the overtopping and the water depth over the structure decrease. With these tests the influence of the amount of overtopping and the water depth on the upper slope is discussed.

Firstly, the test result are discussed with a roughness factor for the upper slope that is calculated according Equation 2.4. Secondly a roughness coefficient is fitted to the test results, so the measured conditions are matching with the theory. In the end the difference between the fitted- and calculated roughness coefficient is discussed.

Calculated roughness coefficient

In Chapter 2 the approach to calculate the amount of overtopping is outlined, this approach is used to calculate the amount of overtopping on a specific test. In Figure E.3 the test are outlined, where the calculated amount of overtopping is plotted against the measured amount of overtopping. The measured wave conditions at the Hydra NL point are used. In this test seriess the amount of overtopping is changed, the wave height and wave steepness are kept constant, described in the test program. Small differences in the measured wave height and wave steepness are taken into account because these are influencing Equation 2.4. In the test program the crest height is increased by lengthening the upper slope with two extra rows, this will reduce the amount of overtopping. These extra rows and the lower overtopping results in a decrease of the roughness coefficient which is calculated with Equation 2.4 and is outlined in Table 5.2.

Table 5.2: Calculated roughness coefficients upper slope.

upper slope	Test number	Roughness coefficient
14 rows	1	0.8927
14 rows	2	0.8916
14 rows	3	0.8925
16 rows	4	0.8783
16 rows	5	0.8774
16 rows	6	0.8772
18 rows	7	0.8652
18 rows	8	0.8654
18 rows	9	0.8658
20 rows	10	0.852
20 rows	11	0.852
20 rows	12	0.852
22 rows	13	0.838
22 rows	14	0.839
22 rows	15	0.838
22 rows	16	0.839

The γ_f for the lower slope is 1.0 and the γ_f of the upper slope is used in the weighted roughness coefficient γ_f, which is calculated with Equation 2.9. The relative freeboard of all the tests is plotted against the relative overtopping in Figure E.3. The blue line is representing Equation 2.17, if the data point lying exact at the blue line, the calculation method is matching with the physical model test. It can be seen in Figure E.3 that the error between the measured overtopping and the calculated overtopping is larger for small overtopping, this will be discussed in the next section.

Fitted roughness coefficient

In order to get a better fit for the measured overtopping with the calculated overtopping the roughness of the upper slope is changed. This is done with a python script, given in the appendix, whereby the variance between the calculated and measured overtopping is minimized. The variance for each test series, three overtopping tests, is given in Table 5.3. The roughness coefficient which will lead to this smallest mean difference is given in Table 5.3.

Table 5.3: Fitted roughness coefficients upper slope.

# of Ribs	Test number	Roughness coefficient	Variance (σ^2)
14 rows	1&2&3	0.930	0.0426
16 rows	4&5&6	0.908	0.0237
18 rows	7&8&9	0.828	0.0021
20 rows	10&11&12	0.749	$3.91 \cdot 10^{-5}$
22 rows	13&14&15&16	0.716	0.0011

These fitted roughness coefficients will lead to a better fit in the relative overtopping rate and relative freeboard, this can be seen in Figure E.3. The measured relative overtopping rate and relative overtopping rate are following both the blue line of the EurOtop 2016 better. The variance of the measured data is smaller with the fitted roughness in contrast with the calculated roughness.

Calculated roughness compared with scaled

The equation to determine the roughness of the upper slope is depended on the amount of wave overtopping, this can be seen in Equation 2.4. In Figure 5.10 the fitted roughness is plotted against the calculated roughness, dependent on the amount of overtopping. It can be seen that with a lower overtopping discharge, the fitted roughness coefficient of the slope is lower than the calculated roughness coefficient. The difference in calculating and fitted roughness coefficient increase if the discharge decrease, this is plotted in more detail in Figure 5.11.

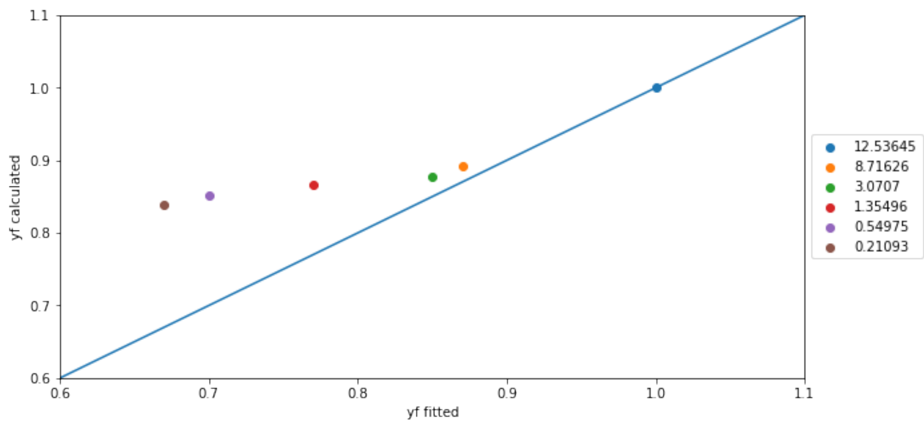

Figure 5.10: Fitted roughness plotted against calculated roughness with adjusted berm coefficient.

For low overtopping discharges, the theoretical Equation 2.4 overestimate the roughness factor. This effect could be explained by several factors. Firstly the equation is based on overtopping discharges between 0.5 and 30 l/m/s (Capel, 2015), low overtopping discharges are not covered in the equation. Another effect could be that the material behaves more rough during low overtopping discharge, because the flow depth is less on a smaller scale. Equation 2.4 is verified on a large scale compared to the BAM, so scale effect could occur

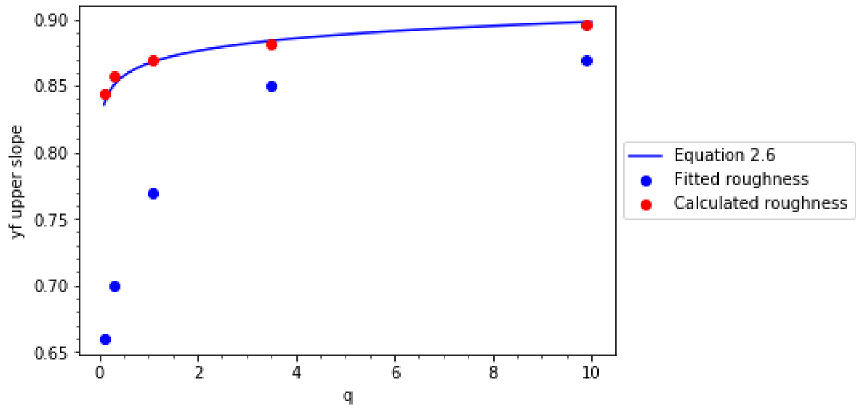

Figure 5.11: Roughness plotted against the overtopping discharge

between the equation and the test in the BAM flume. According to the EurOtop manual, scale effects do only occur in situation with overtopping rates smaller than 1 l/m/s. These scale effects can be calculated with the following relation (Van der Meer et al., 2016):

$$f_q = (-\log(q_{us}) - 2)^5/14 + 1 \tag{5.2}$$

With an upper limit of:

$$f_{qmax} = 10\cot(\alpha) \tag{5.3}$$

$$f_{qmaxred} = 5 \cdot \gamma_f \cdot (1 - f_{qmax}) + 4.5 \cdot (f_{qmax} - 1) + 1 \tag{5.4}$$

With these scaling laws in the EurOtop manual the up-scaled overtopping is calculated and plotted in Figure 5.12. It can be seen that the difference between the calculated and measured discharge could be referred partly to the scale effects described in the EurOtop manual, but this is not covering the error. The difference between the calculated overtopping discharge and the measured overtopping discharge is higher than the scale effects, so Equation 2.4 could overestimate the amount of overtopping. It is not possible to change the influence of the discharge in Equation 2.4 alone, in order to be dimensionless. Another explanation could be that the scale effects are higher than calculated according the EurOtop manual.

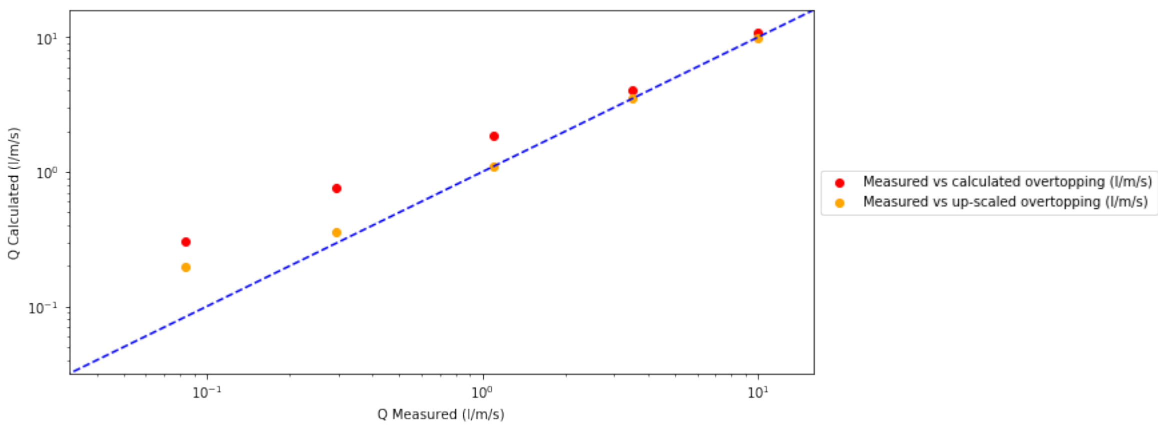

Figure 5.12: Measured overtopping plotted against the Calculated and up-scaled overtopping.

5.2.3. Wave steepness on roughness coefficient

In this part the test numbers 11,12,13 from Table B.1 are discussed. These tests consist of a smooth lower slope and a upper slope with ribs and a berm, given in Figure 5.9. The wave steepness is changed in the test and its influence on the roughness coefficient is discussed in this part.

Calculated roughness coefficient

As can been seen in Figure E.5, with a high wave steepness, the overtopping is overestimated with the roughness coefficient that is calculated with Equation 2.4. With a high wave steepness the amount of overtopping is underestimated and the measured overtopping in the flume is more than calculated. The roughness is calculated with Equation 2.4 and is given in Table 5.4.

Table 5.4: Calculated roughness coefficients upper slope.

Wave steepness	Roughness coefficient
0.051	0.904
0.048	0.902
0.044	0.901

Fitted roughness coefficient

To perform a better estimation of the amount of overtopping, the roughness coefficient is fitted to the test results. In Figure E.5 it can be seen that the test result are matching with the calculated amount of overtopping.

The fitted roughness coefficient are mentioned in Table 5.5. The fitted roughness of the test with a low steepness will result in a roughness coefficient above one. This is physical impossible, but with this roughness coefficient, the measured amount of overtopping could be calculated.

Table 5.5: Fitted roughness coefficients upper slope.

Wave steepness	Roughness coefficient	Variance
0.051	0.85	0.041761
0.048	0.97	0.010203
0.044	1.06	0.026201

In Figure 5.13 the calculated roughness coefficient is plotted against the fitted roughness. It can be seen that with a higher steepness (0.051) and lower overtopping the calculated roughness coefficient is higher than the fitted roughness coefficient. With a low wave steepness (0.044) and a high measured overtopping, the fitted roughness coefficient is higher than the calculated roughness coefficient, given in Figure 5.13. This could be explained by the fact that with high overtopping volumes, the water depth over the structure is larger so the impact of the ribs is smaller. For the lowest wave overtopping (0.044) the fitted roughness coefficient is more than 1. This couldn't be explained by the effect of the water depth on the ribs, because a roughness coefficient of 1 refers to a smooth slope. So the difference is caused by other phenomena, for example a wrong estimated berm coefficient. This test seriess outlines that tests with a wave steepness of around 0.048-0.049 can be predicted good with the theory. Tests with a higher wave steepness will overestimate the amount of overtopping and tests with a lower wave steepness underestimate the overtopping. It is difficult to say this is caused mainly by the wave steepness because the amount of overtopping is also change in these test.

Table 5.6: Wave steepness with corresponding wave overtopping.

Wave steepness	Wave overtopping (l/m/s)
0.051	9.739
0.048	20.665
0.044	36.621

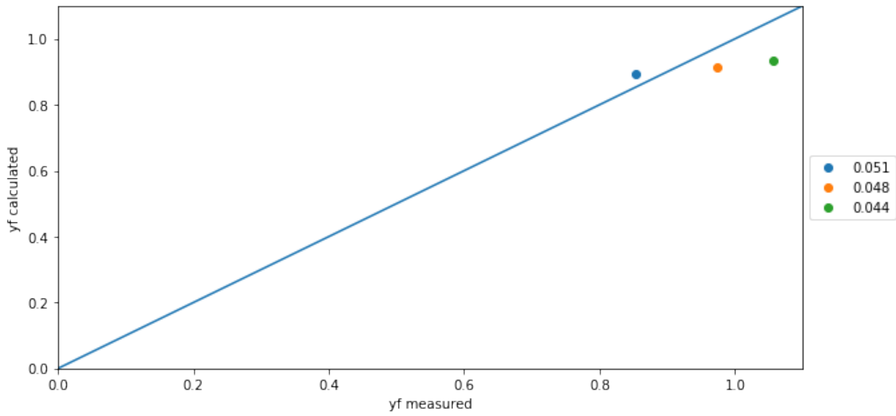

Figure 5.13: Fitted roughness plotted against calculated roughness.

5.2.4. Wave height on roughness coefficient

In this part the test numbers 2,4,6,8,10 from Table B.1 are discussed. These tests consist of a smooth lower slope, a rough upper slope with ribs and a berm, given in Figure 5.9. The crest height is increased in each test by lengthening the upper slope. The wave height is changed in the test and its influence on the roughness coefficient is discussed. These wave heights are fitted to the tests to create the same amount of overtopping if the upper slope is lengthened. Because the calculation show scatter, this gives also scatter in these results. In the flume the wave height is fitted to get approximately the same amount of overtopping. In the end there are some differences in the amount of overtopping, these are included in the approach.

Calculated roughness coefficient

As can been seen in Figure E.7, that the overtopping is underestimated with the roughness coefficient that is calculated with Equation 2.4 in most of the tests. Four of the five test are underestimate die amount of overtopping. The test with 14 ribs is overestimating the amount of overtopping. The roughness is calculated with Equation 2.4 and is given in Table 5.7.

Table 5.7: Calculated roughness coefficients upper slope.

Wave Height	Roughness coefficient
3.353	0.895
3.652	0.893
3.969	0.891
4.265	0.889

Fitted roughness coefficient

To perform a better estimation of the amount of overtopping, the roughness coefficient is fitted to the test results. In Figure E.5 it can be seen that the test result are matching with the calculated amount of overtopping. The fitted roughness coefficients are mentioned in Table 5.8.

Table 5.8: Fitted roughness coefficients upper slope.

Wave height	Roughness coefficient	Mean difference
3.353	0.87	0.166
3.652	0.94	0.219
3.969	0.97	0.134
4.265	0.94	0.165

In Figure 5.14 the calculated roughness coefficient is plotted against the fitted roughness. All the points are lying on a line, because the calculated roughness is almost the same. The wave height is not a sensitive parameter in Equation 2.4. The physical model test show a different dependency on the wave height, if the wave height increase the roughness coefficient increase as well, given in Figure 5.14. The blue dot is the standard wave height, all the other dots are increased wave heights which are targeted with the same overtopping discharge and wave steepness. It can bee seen that with an higher wave height, the roughness in the physical model test is higher than calculated with Equation 2.4. The green dot is a test with a significant higher overtopping, due to this effect, the roughness coefficient is even higher. It is difficult to fit Equation 2.4 to these test results. An explanation could be that the area of influence of the roughness of the lower slope is different. If the wave height increase, $Ru_{2\%}glad$ increase whereby the lower slope is of more influence on the weighted roughness. In Figure E.9 the results are given if the weighted roughness of Equation 2.9 is calculated with 40 % influence of the lower slope. The calculated roughness coefficient is matching better with the physical model tests. An important conclusion is that the 2.4 calculated the roughness in the right way with the wave height of condition O2a given in Table A.1. If the wave height becomes larger than the 02a conditions, the upper slope tend to be smoother or the smooth lower slope need to be increased in the influence on the weighted roughness.

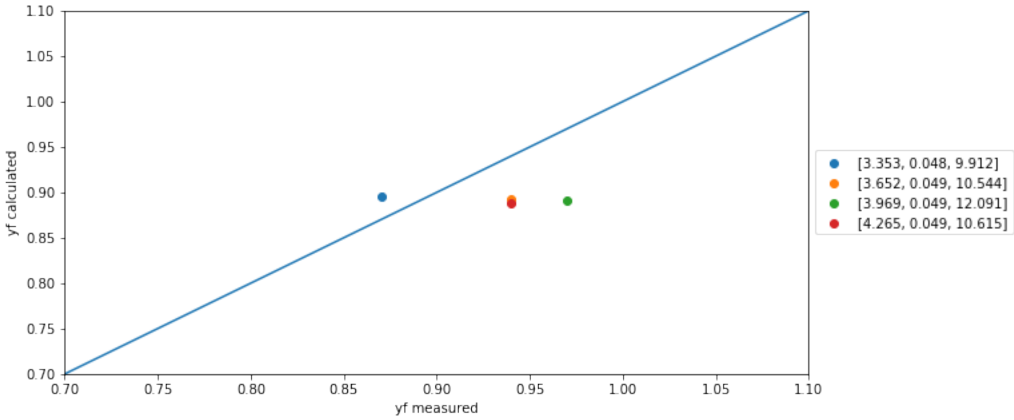

Figure 5.14: Fitted roughness plotted against calculated roughness.

5.3. Hydraulic conditions Delta Flume

In the test program two different hydraulic conditions are described for the Delta flume: 01b and S02. The first test is focused mainly on overtopping, the second test is focused on the initial stability of the blocs. Both conditions create overtopping and will be used in the analysis. It is not possible to get exact the conditions in the wave flume as prescribed, due to uncertainty of different hydraulic processes. In the analysis of the results later on, the measured wave conditions at the Hydra NL point in the wave flume are used. In the Delta Flume the waves are measured only at deep water. The average relation between deep water and Hydra NL in the Schelde and BAM flume is used to calculate the Hydra NL conditions in the Delta flume.

5.3.1. Wave height

The relation between Hm0 at deep water and Hydra NL is 1.08 in the Schelde flume. The wave height at deep water is multiplied with 1.08 to calculate the wave height at Hydra NL. In Figure 5.15 the target wave height at deep water is plotted against the measured wave condition. In Figure 5.16 the same procedure is done for the Hydra NL point in front of the structure. In this test series the wave conditions are fitted to deep water. This can also be seen in the figures, the wave conditions are better fitted at deep water.

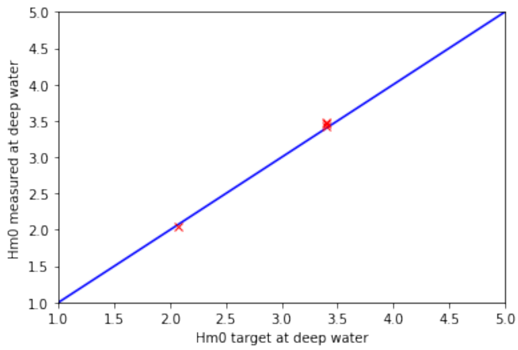

Figure 5.15: Hm0 target plotted against the measured Hm0 at deep water.

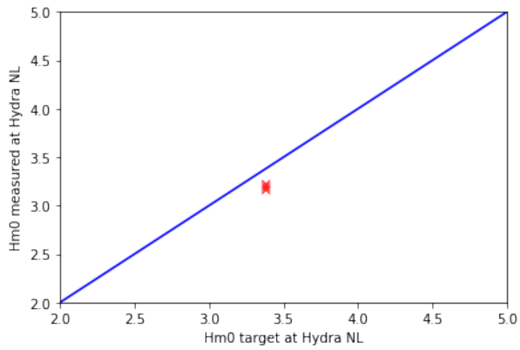

Figure 5.16: Hm0 target plotted against the measured Hm0 at Hydra NL.

5.3.2. Wave period

The relation between ($Tm_{-1,0}$) and T_p at deep water and Hydra NL is 0.98 in the Schelde flume. The wave period at deep water is multiplied with 0.98 to calculate the wave period at Hydra NL. Figure 5.17 & 5.18 describes the relationship between wave periods measured at deep water and near the structure.

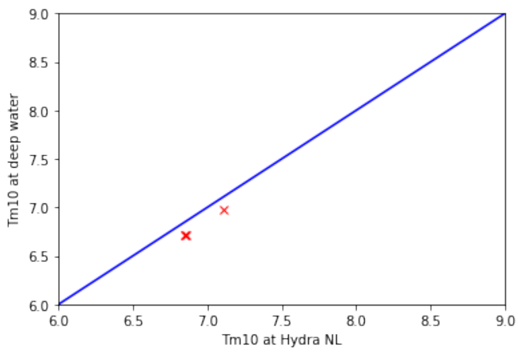

Figure 5.17: Tm-1,0 at Hydra NL plotted against Tm-1,0 at deep water.

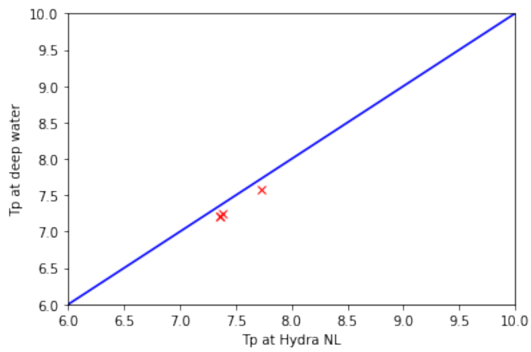

Figure 5.18: Tp at Hydra NL plotted against Tp at deep water.

5.3.3. Wave steepness

Wave steepness maintained constant trough out the duration of each test. The measured wave nominal wave steepness near the structure is presented in Figure 5.19. This is the nominal wave steepness at Hydra NL plotted against the wave steepness at deep water. In Figure 5.20, it can be observed that the measured wave steepness is smaller than target wave steepness. This measured wave steepness is used for the calculation of the amount of overtopping.

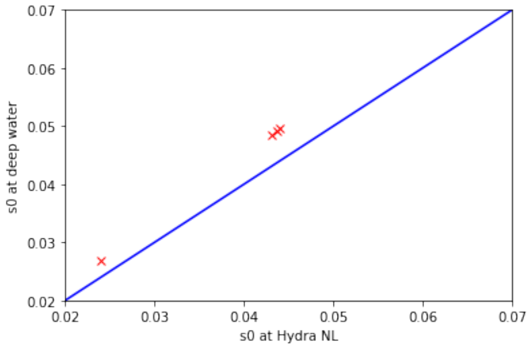

Figure 5.19: s0 at deep water plotted against s0 at Hydra NL.

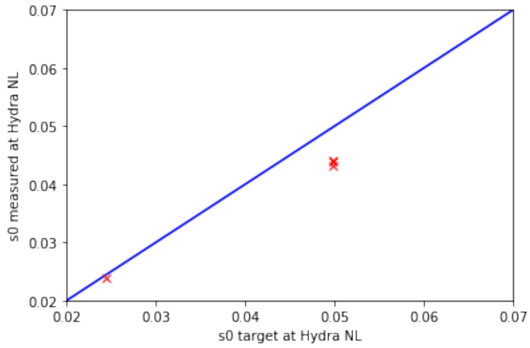

Figure 5.20: s0 target plotted against s0 measured at Hydra NL.

5.4. Analysis results Delta flume

In this part the results of the Delta flume of dike section 17a are discussed. Four test are analyzed with different hydraulic conditions and structures, they are mentioned in Table B.1 with test code 45 to 48. In Figure 5.21 the tested structure is given, a smooth lower slope a berm and a rib pattern on the upper slope. In the first tests, the height of the ribs is 0.05 meter, in the last test (48 in Table B.1) the height of the ribs is 0.21 meter.

Figure 5.21: Tested cross section 17a Delta flume.

5.4.1. Calculated roughness coefficient

As can been seen in Figure 5.22, three test show significant difference with the the EuropOtop manual line. The scatter is in two of the three cases more than the 5 % interval of Equation 2.16. The last physical test with ribs of 0.21 m show hardly any difference, this is discussed later on.

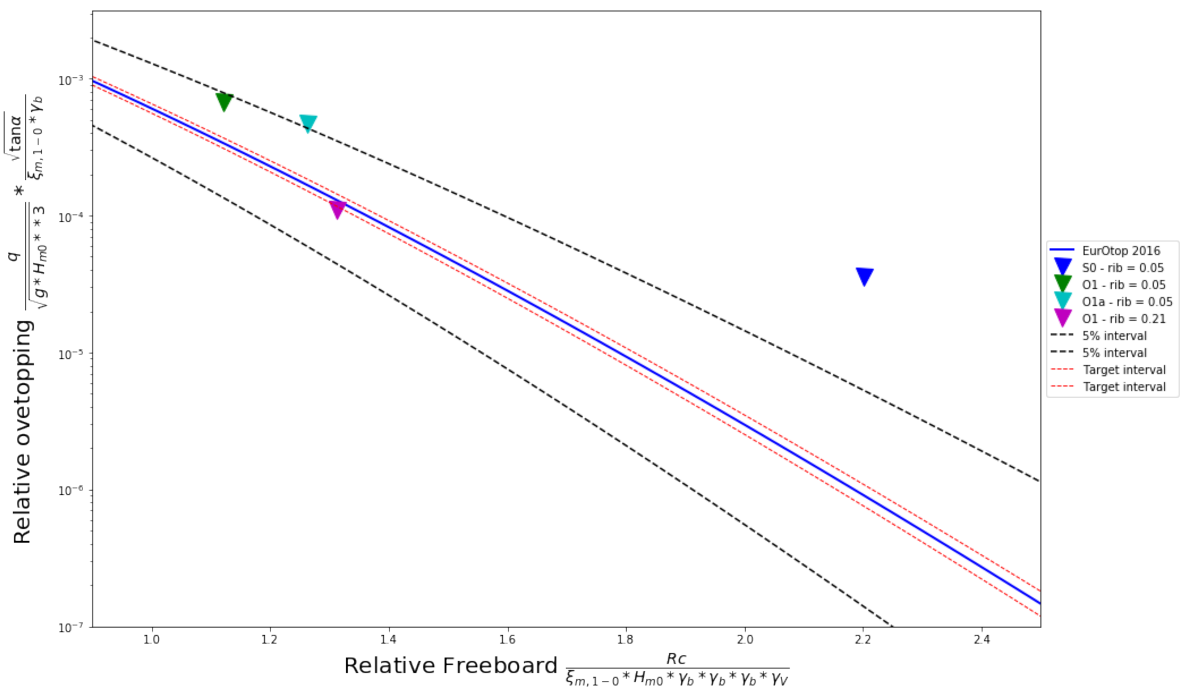

Figure 5.22: Measured data plotted with different ribs and conditions with a calculated roughness

The roughness is calculated with Equation 2.4. The first test consist of a significant lower roughness compared to the other tests, because the amount of overtopping is significant lower. The last test consist of a lower roughness compared to the other two tests, due to the higher ribs on the upper slope.

5.4.2. Fitted roughness coefficient

To perform a better estimation of the amount of overtopping, the roughness coefficient is fitted to the test results. In Figure 5.23 it can be seen that the measured overtopping is matching with the calculated overtopping.

Table 5.9: Calculated roughness coefficients upper slope.

Measured overtopping	Roughness coefficient
0.7	0.51
25.0	0.88
17.0	0.88
4.0	0.73

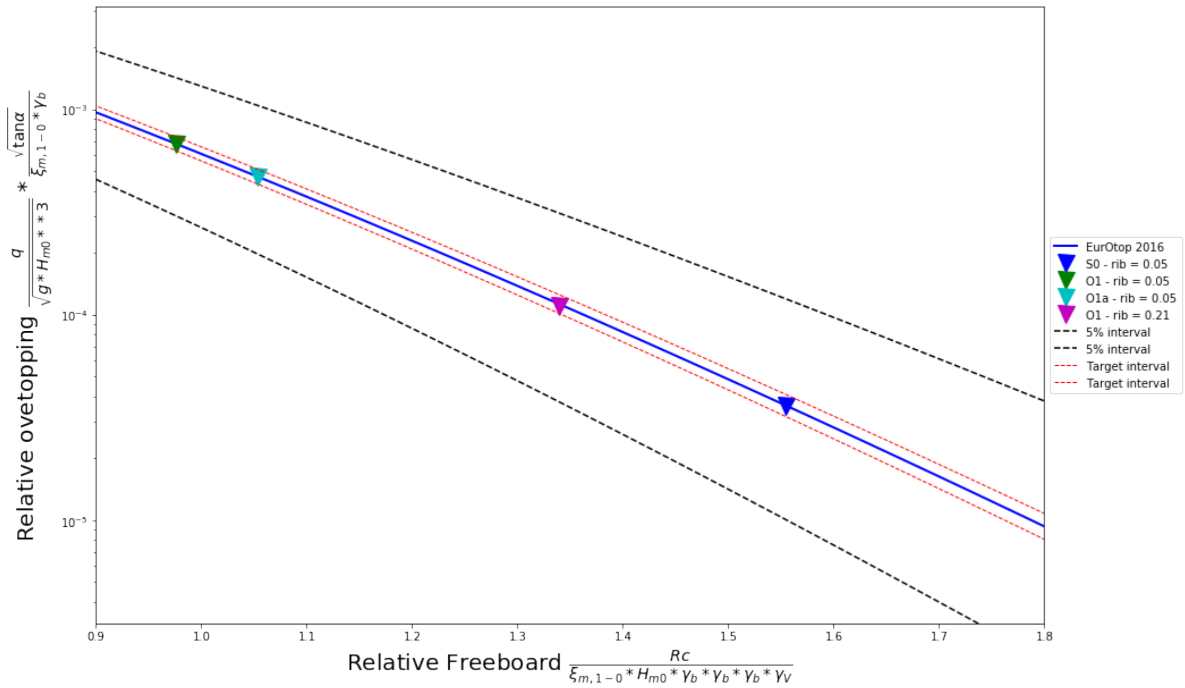

Figure 5.23: Measured data plotted with different ribs and conditions with a fitted roughness

The fitted roughness coefficient are mentioned in Table 5.10. In the test with ribs of 0.05 m, the fitted roughness coefficient is higher than one. This is physical impossible, but with this roughness coefficient, the amount of overtopping could be calculated.

Table 5.10: Fitted roughness coefficients upper slope.

Measured overtopping	Roughness coefficient	Difference
0.7	1.24	0.003312
25.0	1.12	0.052369
17.0	1.22	0.061932
4.0	0.72	0.07432

The difference in amount of overtopping in the test could be referred to any scale effects. In the Delta flume the calculated overtopping is underestimated if the ribs have a height of 0.05 meter. The test with a height of ribs of 0.21 meter show a good fit. The procedure in which the data of the Delta flume is analyzed is verified in the BAM flume. The ribs have less influence on the amount of overtopping on bigger scale.

As mentioned in the literature only scale effects could occur if the overtopping discharge is lower than 1 l/m/s. The first test consist of a overtopping discharge of 0.7 l/m/s so this could refer to any scale effects.

Test 2 and test 3 are test with a overtopping discharge of around 10 l/m/s. It's expected that no scale effects occur with this overtopping rate. In test 2 and test 3 the amount of overtopping is much higher than calculated, the fitted roughness need to be higher than 1. The wave steepness of these tests is around 0.044. In the analysis of the BAM flume also the influence of the wave steepness on the amount of overtopping is discussed. There was concluded that tests with a higher wave steepness will overestimate the amount of overtopping and with a lower wave steepness underestimate the overtopping. In the BAM flume the same conditions were tested with a wave steepness of 0.044, the fitted roughness factor was in this case 1.06, this van be seen in 5.6. This could explain partly the difference between the measured overtopping and the calculated. The difference could not only be referred to the wave-steepness, so scale affects need to be present.

The origin of these scale effects could be that wave breaking on a slope and the front wedge flow past the Levvel-blocs enclose air in the water. This water is transported the dissolved air over the upper slope. Due to this effect relatively more air will be enclosed in the Delta flume. Due to differences in relative air contents, the average mass density of the up-rushing water is smaller in the Delta flume compared with the BAM flume. Water can flow easily over the upper slope with entrapped air and this will lead to more overtopping. If the up-rushing water does not have enough time to flow downwards, there will be water between the ribs when the next wave is facing the structure. The influence of the ribs for the next wave is smaller, because water is flowing between the ribs and gives a smoother slope.

In test 4, with ribs of 0.21 meter, the wave overtopping discharge is 4 l/m/s. The error in case the theory is calculating the amount of overtopping in the Delta flume is small. The height of the ribs will give extra roughness to the upper slope which is calculated correctly according Equation 2.4. In contrast with ribs of 0.05 which creates an underestimation of the amount of overtopping. In Table 5.11 the different overtopping values are given with there scale factor. The scale factor in test 2 & 3 is between 2-3 (high overtopping), the scale factor of test 1 is around 50 (low overtopping).

Table 5.11: Overtopping results.

Method	Test 1	Test 2	Test 3	Test 4
	q $(l/m/s)$	q $(l/m/s)$	q $(l/m/s)$	q $(l/m/s)$
Calculated (verified in BAM flume)	0.013	12.21	11.13	4.33
Delta Flume	0.7	25.0	17.0	4.0
Difference	0.69	12.79	11.13	0.33
Scale factor	51.818	2.05	2.90	0.92

6
ANALYSIS 8b

In this chapter the results of the different test for dike section 8b are performed. The hydraulic conditions of the different physical test in the BAM flume are discussed. After this, the data is analyzed on different structural components. Firstly the influence of Levvel-blocs on the lower slope is discussed. Secondly this the influence of the pattern of the upper slope is discussed, what is the influence of this pattern on the roughness coefficient. At third the influence of the slope of the lower slope is discussed, what is the difference between a 1:2 slope and 1:1.5 slope. After this the The hydraulic conditions in the Schelde flume of the different physical test are discussed. After this the data is analyzed on different components. Firstly the influence of Levvel-blocs on the lower slope is discussed. Secondly the influence of the slope of the lower slope is discussed, what is the difference between a 1:2 slope and 1:1.5 slope.

6.1. Hydraulic conditions BAM FLume

Prior to analyzing the results, wave conditions that are performed need to be analyzed. Due to different hydraulic effects the wave height at deep water conditions differs from the waves near the structure. This is caused due to the depth limit and result in shoaling and wave breaking. The wave is partly reflected due to the structure. In order to optimize the desired wave conditions, a previous calibration without the structure is performed. The best fitted gain factor of the wave generator is used to perform as exact as possible the waves. In this test serie, the conditions are fitted to deep water this is prescribed by Rijkswaterstaat.

In total three test series are performed, one in the tender phase in 2017 and two in the design phase in June and Augustus 2018. The first test serie was mainly performed to look at the different overtopping rates and stability effects of Levvel-blocs on the lower slope. The second test serie to optimize the effect of the top slope on the overtopping and the last serie a verification test of the final design. Prior for analyzing the results, the wave conditions that are performed need to be analyzed. In the tests, the conditions are fitted to deep water, the hydra NL conditions are used to analyze the results.

6.1.1. Wave height

In the test program different conditions are described to expect an amount of overtopping that is calculated with the theoretical formula. In Figure D.3 the target wave height at deep water is plotted against the measured wave condition. In Figure 6.1 the same procedure is done for the Hydra NL point in front of the structure. In this test serie the wave conditions are fitted to the deep water point. This can also be seen in the figures, the wave conditions are better fitted for the Hydra NL point. The black points in Figure 6.1 are measured with an steeper slope (1:1:5).

During the performance of the test, the wave height was measured at deep water and near the structure. Figure D.4 presents the relationship between the spectral significant wave height (H_{m0}), at those two locations. Wave breaking caused by depth limited conditions makes the measured wave height decreases. This can be observed in Figure D.4 where it is clearly noticed that wave height near the structure is lower than in deep water. Due to wave breaking, the spectral significant wave height, H_{m0}, also varies from the statistical significant wave height $H_{2\%}$. Figure 6.2 describes the relationship between these two measured wave height near the structure, the ratio between $H_{2\%}$ and H_{m0} is 1.25.

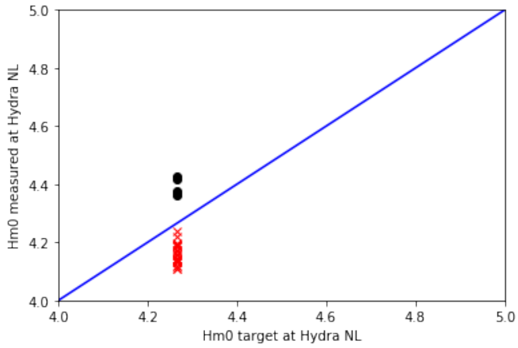

Figure 6.1: Hm0 target plotted against the measurd Hm0 at Hydra NL.

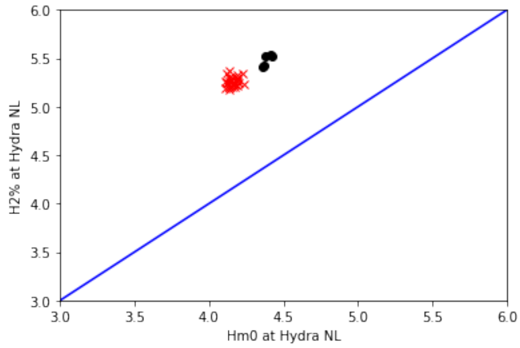

Figure 6.2: Hm0 plotted against H2% at Hydra NL.

6.1.2. Wave period

In this research, the wave period is estimated from spectral analysis performed with WaveLab. Figure 6.3 & 6.4 describes the relationship between wave periods measured at deep water and near the structure which represents intermediate waters.

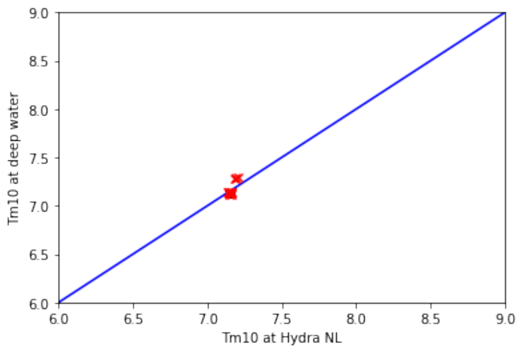

Figure 6.3: Tm-1,0 at Hydra NL plotted against Tm-1,0 at deep water.

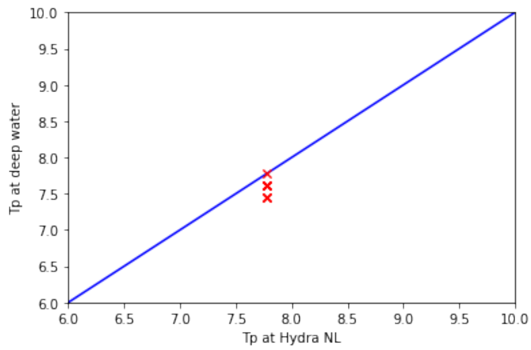

Figure 6.4: Tp at Hydra NL plotted against Tp at deep water.

In the EurOtop Manual (Van der Meer et al., 2016) a relationship between the spectral peak wave period (T_p) and the spectral average wave period ($Tm_{-1,0}$) of a single peaked spectrum with Rayleigh distribution in deep water where T_p is assumed 1.1 times $T_{m_{-1,0}}$. However, in shallow water this relationship is not always valid as the spectrum in shallow water deviates from the spectrum in deep water due to wave breaking. Figure 6.5 shows the relationship between these two spectral wave periods at the Hydra NL point.

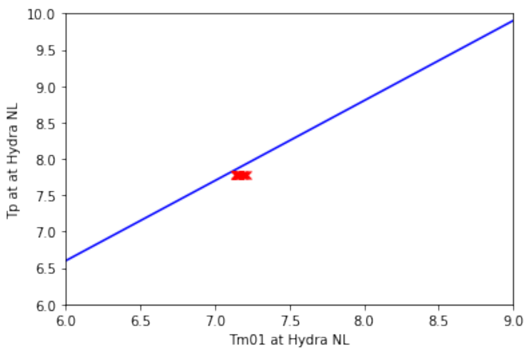

Figure 6.5: Tm-1,0 plotted against Tp at Hydra NL.

6.1.3. Wave steepness

Wave steepness maintained constant trough out the duration of each test. The measured wave nominal wave steepness near the structure is presented in Figure 6.6. This is the nominal wave steepness at Hydra NL plotted against the wave steepness at deep water. The black dots are measurement with the lower slope of 1:1.5.

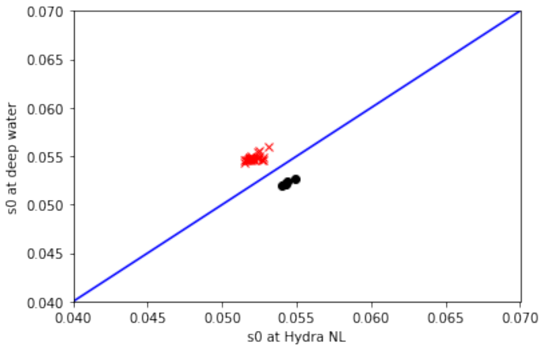

Figure 6.6: s0 at deep water plotted against s0 at Hydra NL.

6.2. Analysis results BAM flume

In this part the test 14 to 28 from Table B.1 are discussed. The first test serie consist of a lower slope of Levvelblocs and smooth upper slope, given in Figure 6.7. In the second part the influence of different aspects on the upper slope is discussed, given in Figure 6.8. The lower slope which is determined in the first part is used in this analysis as a starting point. In the end the difference between a 1:1.5 and 1:2 of the lower slope on the roughness coefficient is discussed, these results are checked with a verification test.

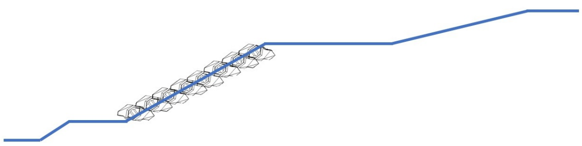

Figure 6.7: First cross-section 8b

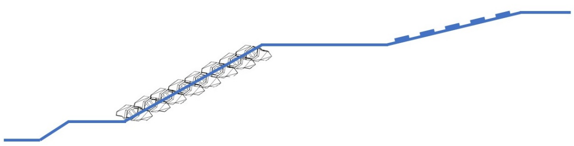

Figure 6.8: Sevond cross-section 8b

6.2.1. Roughness coefficient of lower slope

In this first test serie there is focused on the the lower slope. In test program it can be seen that the different designs are quite similar, there is a small change in crest height and the length of the berm. The upper slope is considered to be smooth with a roughness factor of 1. Test 14-17 in Table B.1 are used in this analyses.

Calculated roughness coefficient

The roughness coefficient of the lower slope is considered to be constant if the water level is around the berm (Van der Meer et al., 2016). The roughness of the lower slope is 0.49 according (Jiménez Moreno, 2017). This roughness coefficient gives different results in the physical test, this can be seen in Figure E.10. All the data points are lying above the blue line of Equation 2.16, so the measured overtopping values are higher than the calculated overtopping values. The roughness of the structure is lower in the model which creates more overtopping.

Fitted roughness coefficient

To perform a better estimation of the total overtopping, one coefficient in Equation 2.16 is changed. In this part the roughness coefficient of the lower slope is changed. The best fitted roughness coefficient of the lower slope is 0.96. In Table 6.1 three different roughness coefficient are given. A roughness coefficient of 0.96 introduce the smallest error between all the four tests. This roughness coefficient is used in the next analysis of the upper slope.

Table 6.1: Fitted roughness coefficients lower slope.

Roughness coefficient	Total difference
0.95	2.43
0.96	2.41
0.97	2.47

6.2.2. Influence Rib height on roughness coefficient of upper slope

This test serie there is focused on the influence of rib height on the roughness of the upper slope. Test 18-20 in Table B.1 are used in this analyses.

Calculated roughness coefficient

The roughness of the lower slope is 0.96 according the previous section. The roughness of the upper slope is calculated according Equation 2.4. In Figure E.12 the measured overtopping is smaller than the calculated overtopping according Equation 2.16. The material is behave rougher in the BAM flume than Equation 2.4 is calculated.

Fitted roughness coefficient

For fitting the physical measurements to the calculation, the roughness coefficient of the upper slope need to be decreased. The roughness coefficient of the upper slope is calculated quite well for ribs of 0.12 meter. The difference between calculated roughness and fitted roughness is higher if the height of the ribs increase, this can also be seen in Figure 6.9. Equation 2.4 is based on test on a scale of 1:19.8. These test are performed on a 1:36.5 scale. The difference between the the formula and the fitted could be caused by any scale effects. If the height of the Ribs is higher, their influence on the roughness is higher on small scale. The ribs are dominant over the water depth that is flowing over the highest part of the upper slope.

Table 6.2: Fitted roughness coefficients top slope.

	Fitted roughness	Calculated roughness
Ribs of 0.12 m	0.81	0.836
Ribs of 0.18 m	0.73	0.794
Ribs of 0.23 m	0.56	0.766

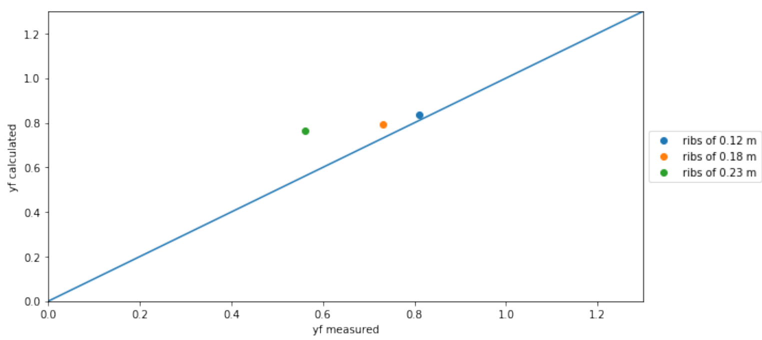

Figure 6.9: Fitted roughness plotted against calculated roughness.

6.2.3. Influence Rib pattern on roughness coefficient of upper slope

This test serie there is focused on the influence of rib pattern on the roughness of the upper slope. Test 20,24 & 25 in Table B.1 are used in this analyses.

Calculated roughness coefficient

In this section, three different rib patterns are tested which are given in Figure 4.10, 4.12 & 4.13. The height of the ribs is in all the test 0.23 meter. The calculated roughness according to Equation 2.4 gives a higher overtopping discharge than the measured overtopping in the model, this can be seen in Figure E.14.

Fitted roughness coefficient

To fit the physical measurements to the calculation, the roughness coefficient of the upper slope need to be increased. The difference between calculated roughness and fitted roughness is the highest if rib pattern 8 is used, this can be seen in Figure 6.9. All the overtopping results from physical model test are lower than is calculated. An explanation of this effect is outlined in the previous section, the same rib height is used in these tests.

Rib pattern 2 is given the smallest fitted roughness for the upper slope, given in Table 6.3. The fitted roughness coefficient of rib pattern 2t is the largest. This can be explained by the effect that the flow of water on the upper slope is smoother. Water can flow easily around the ribs which will lower the roughness of the upper slope. With rib pattern 8 the front width face is the highest. Due to this wider front face water can flow difficult between the ribs compared to the other two rib patterns. Water can also flow more difficult downwards of the slope. If there is water between the ribs and a new wave is flowing over the structure, the roughness of the upper slope is lower. In order to create the minimum overtopping, pattern 2 should be used because the roughness coefficient is the lowest.

Table 6.3: Fitted roughness coefficients top slope.

	Fitted roughness	Calculated roughness
Rib pattern 8	0.56	0.766
Rib pattern 2	0.55	0.792
Rib pattern 2t	0.61	0.791

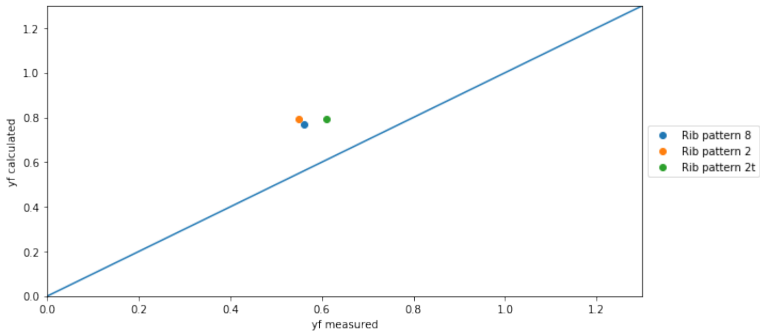

Figure 6.10: Fitted roughness plotted against calculated roughness.

6.2.4. Influence of amount of Ribs on roughness coefficient of upper slope

This test serie there is focused on the influence of the amount of ribs of the upper slope. Test 20-23 in Table B.1 are used in this analyses.

Calculated roughness coefficient

In this section three different amount of ribs are tested. The height of the ribs is in all the test 0.23 meter. The calculated roughness according to Equation 2.4 gives a higher overtopping discharge than the measured overtopping in the model, this can be seen in Figure E.16. All the data point are lying under the the blue line, so the overtopping is overestimated according Equation 2.16.

Fitted roughness coefficient

To fit the physical measurements to the calculation, the roughness coefficient of the upper slope need to be increased. The difference between calculated roughness and fitted roughness is the lowers for 3 ribs, this can be seen in Figure 6.9. Equation 2.4 is mostly based on tests where ribs are placed direct after another rib or with space of around 1 rib between two consecutive ribs. In test 20, with 11 ribs, there is 1 rib between two consecutive ribs. The space between two consecutive ribs is much larger with 3 and 6 ribs on the upper slope. If three ribs are used, the roughness is so low that Equation 2.4 is calculating the roughness correctly. It is doubtful if this match is referred to a good estimation of the roughness, because the tests that are used for Equation 2.4 are more based on a slope of 11 ribs.

Table 6.4: Fitted roughness coefficients top slope.

	Fitted roughness	Calculated roughness
11 Ribs	0.60	0.766
6 Ribs	0.70	0.835
3 Ribs	0.88	0.884

6.2.5. Influence of slope on roughness coefficient of lower slope

This test serie there is focused on the influence of the amount of ribs of the upper slope. Test 20,23,25 & 26 in Table B.1 are used in this analyses.

Calculated roughness coefficient

The roughness coefficient of the lower slope is considered to be constant if the water level is around the berm. The roughness of the lower slope is 0.96 according previous section for a slope of 1:2. In Figure E.18 tests with a slope of 1:2 and 1:1.5 are given. All the test are performed with ribs of 0.23 meter, the difference in the section about the influence of rib height on the roughness of 0.23 meter is corrected in this example by multiply the rib height with a factor of 2.6, as described earlier. The influence of rib height is than calculated correctly and only the influence of the lower slope can be determined. With a roughness factor of 0.96 for a slope of 1:1.5 the amount of overtopping is overestimated.

Fitted roughness coefficient

In order to calculate the amount of overtopping correctly for a structure with a lower slope of 1:1.5 the roughness coefficient should decrease. In Table E.19 it can be seen that with a roughness factor of 0.51 for the lower slope of 1:1.5 the data points are more at the blue line. To conclude: with a lower slope of 1:2 the roughness coefficient is 0.96 and a with a lower slope of 1:1.5 the roughness coefficient is 0.51.

Table 6.5: Fitted roughness coefficients top slope.

Slope	Fitted roughness	Calculated roughness
1:2.0	0.96	0.96
1:2.0	0.96	0.96
1:1.5	0.51	0.96
1:1.5	0.51	0.96

6.2.6. Verification test

This test serie there is a verification test of the final design. Test 27 & 28 in Table B.1 are used in this analyses.

Calculated roughness coefficient

The roughness coefficient of the lower slope is 0.96 according previous section for a slope of 1:2 and 0.51 for a slope of 1:1.5. All the test are done with ribs of a height of 0.23 meter, as described earlier these height need to be corrected by multiplying with a factor of 2.6 in order to give a reliable roughness coefficient of the upper slope. With this corrected roughness factor for the upper slope and the roughness coefficients for the lower slope the calculated and measured overtopping are matching good, see Figure E.20.

Fitted roughness coefficient

In order to reduce the difference between the calculated overtopping and the measured overtopping in the BAM flume the roughness coefficient of the lower slope is changed. In Table 6.6 the fitted roughness coefficient for the lower slope is given. The roughness coefficient of the lower slope for 1:2 is changed with 0.03 and for a slope of 1:1.5 with 0.06. These changes are small, because the influence of the lower slope is small in Equation 2.9.

Table 6.6: Fitted roughness coefficients top slope.

Slope	Fitted roughness	Calculated roughness
1:2.0	0.98	0.96
1:1.5	0.45	0.51

6.3. Hydraulic conditions Schelde Flume

In the test program three different hydraulic conditions are described for the Schelde flume: 01a, S2WHW & S2WLW. The first test is focused mainly on overtopping, the other two tests are focused on the stability of the blocs. Both conditions create overtopping so will be used in the analysis. It is not possible to get exact the conditions in the wave flume as prescribed, due to uncertainty of different hydraulic processes. In the analysis of the results later on, the measured wave conditions at the Hydra NL point in the wave flume are used.

6.3.1. Wave height

In Figure 6.11 the target wave height at deep water is plotted against the measured wave condition. In Figure 6.12 the same procedure is done for Hydra NL in front of the structure. In this test serie the wave conditions are fitted deep water. This can also be seen in the figures, measured wave conditions matching better to deep water.

During the performance of the test, the wave height was measured at deep water and near the structure. Figure D.5 presents the relationship between the spectral significant wave height, Hm0, at those two locations. Wave breaking caused by depth limited conditions makes the measured wave height decreases. This can be observed in Figure D.5 where it is clearly noticed that wave height near the structure is lower than in deep water.

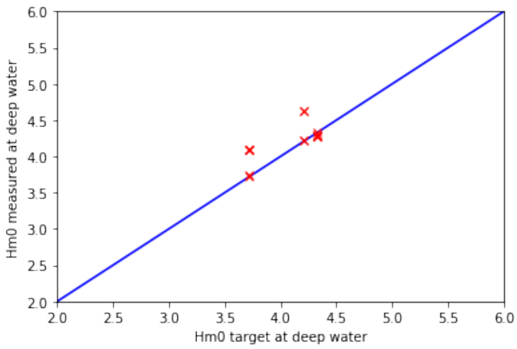

Figure 6.11: Hm0 target plotted against the measurd Hm0 at deep water.

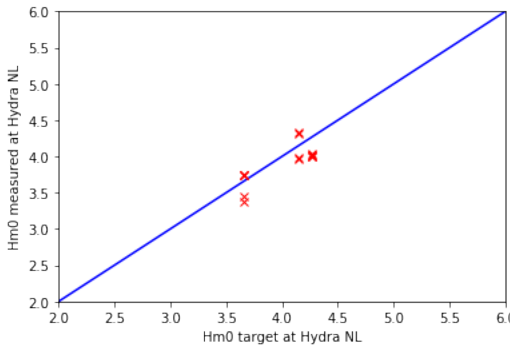

Figure 6.12: Hm0 target plotted against the measurd Hm0 at Hydra NL.

6.3.2. Wave period

In this research, the wave period is estimated from spectral analysis performed with WaveLab. Figure 6.13 & 6.14 describes the relationship between wave periods measured at deep water and near the structure.

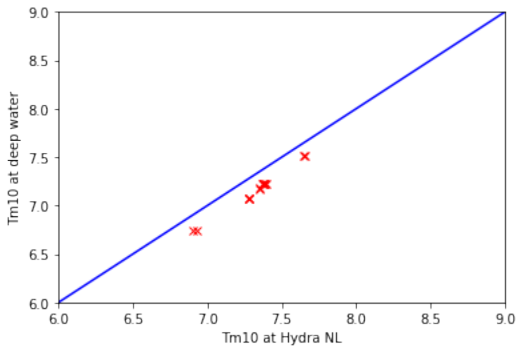

Figure 6.13: Tm-1,0 at Hydra NL plotted against Tm-1,0 at deep water.

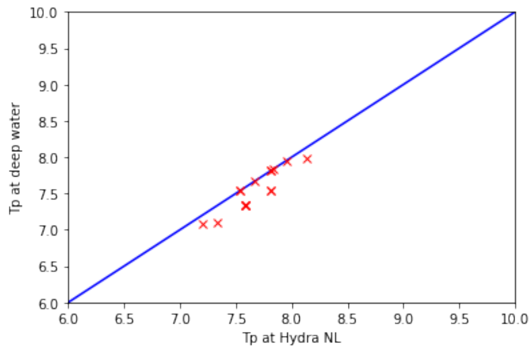

Figure 6.14: Tp at Hydra NL plotted against Tp at deep water.

In the EurOtop Manual (Van der Meer et al., 2016) a relationship between the spectral peak wave period (Tp) and the spectral average wave period ($Tm_{-1,0}$) of a single peaked spectrum with Rayleigh distribution in deep water where Tp is assumed 1.1 times $T_{m_{-1,0}}$. However, in shallow water this relationship is not always valid as the spectrum in shallow water deviates from the spectrum in deep water due to wave breaking. Figure 6.15 shows the relationship between these two spectral wave periods at the Hydra NL point.

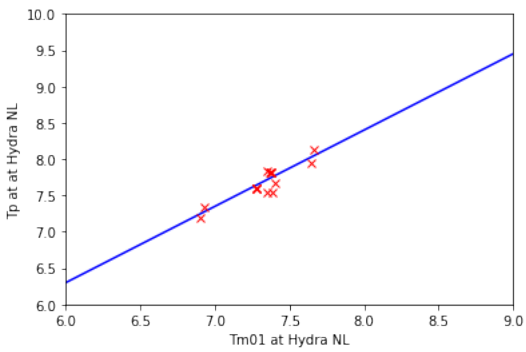

Figure 6.15: Tm-1,0 plotted against Tp at Hydra NL.

6.3.3. Wave steepness

Wave steepness maintained constant trough out the duration of each test. The measured wave nominal wave steepness near the structure is presented in Figure 6.16. This is the nominal wave steepness at Hydra NL plotted against the wave steepness at deep water.

6.4. Analysis results Schelde flume

In this part test 29 to 44 from Table B.1 are discussed. First the tests with a lower slope of 1:2 are discussed. Secondly tests with a lower slope of 1:1.5 are discussed. After this an analysis of low overtopping discharges is performed.

6.4.1. Analysis results lower slope 1:2

In this section test 34 to 44 with hydraulic conditions 02 and S2MHW from Table B.1 are discussed. With these conditions the overtopping discharge is around 4 to 12 l/m/s. In the test serie of the BAM flume there is first focused on the roughness of the lower slope as a constant to determine only the influence of the upper slope.

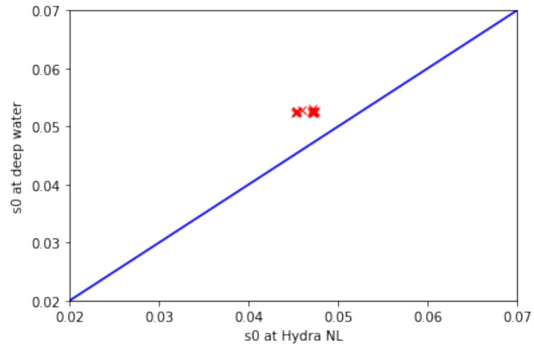

Figure 6.16: s0 at deep water plotted against s0 at Hydra NL.

In this section the roughness coefficient of the upper slope is calculated with Equation 2.4. The tests that are used for this equation are based on a scale of 1:22 and are validated on a scale of 1:11 according to Capel (Capel, 2015). The tests in the Schelde flume are performed on a scale of 1:19.8 between these two scales. Equation 2.4 is valid and can be used as a given so only the influence of the lower slope could be considered in this section.

Calculated roughness coefficient

The roughness coefficient of the lower slope is considered to be constant if the water level is around the berm. The roughness of the lower slope is 0.49 according to Jiminez (Jiménez Moreno, 2017). In the section of the BAM flume the roughness of the lower slope is changed to 0.96, any measurements and possible model and scale effects where included in this coefficient to look only at the influence of the upper slope. This roughness coefficient of 0.96 is overestimate the overtopping for the test in the Schelde flume, this can be seen in Figure 6.17. All the data points are lying under the blue line of Equation 2.16, so the measured overtopping values are lower than the calculated overtopping values. The roughness coefficient of the lower slope should be lower than 0.96.

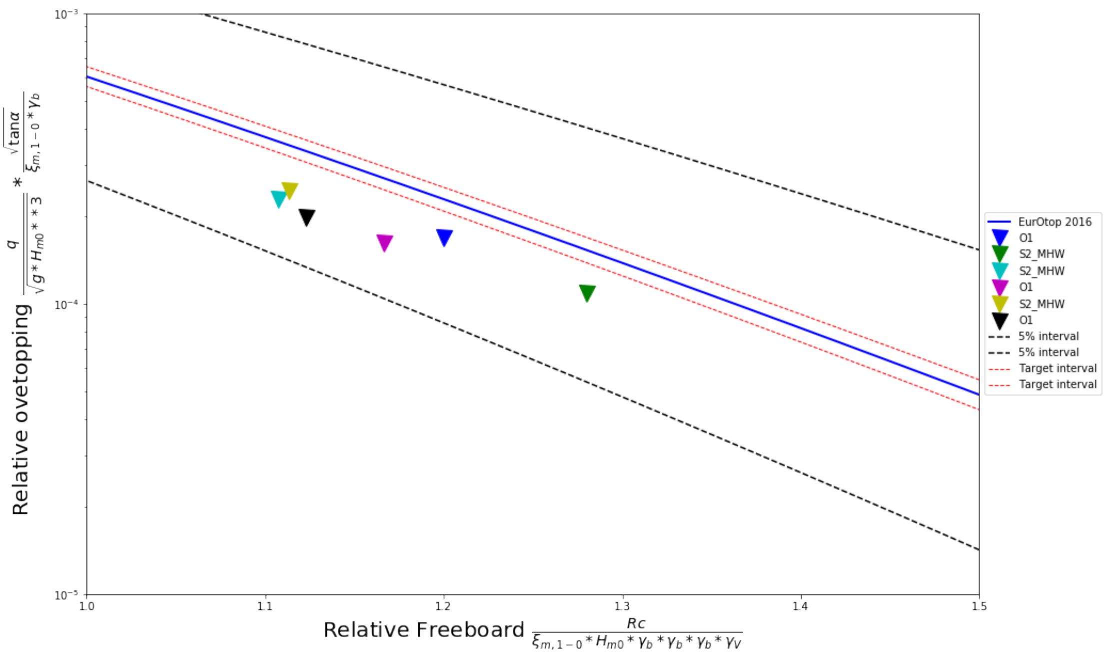

Figure 6.17: Measured data plotted with lower slope of 1:2 and calculated roughness

Fitted roughness coefficient

To perform a better estimation of the total overtopping, the roughness coefficient of the lower slope in Equation 2.16 is changed. The best fitted roughness coefficient for the particular tests are given in Table 6.7. A roughness coefficient of the lower slope between 0.48 and 0.65 introduce an average error of 0.0596 l/m/s between the calculated overtopping and measured overtopping in the flume, this can be seen in Figure 6.18. The average roughness coefficient of the different tests is 0.568, the average error is in that case 0.75 l/m/s.

Table 6.7: Fitted roughness coefficients lower slope.

Roughness coefficient lower slope	Difference
	l/m/s
0.65	0.039
0.65	0.098
0.53	0.077
0.48	0.039
0.62	0.081
0.48	0.024

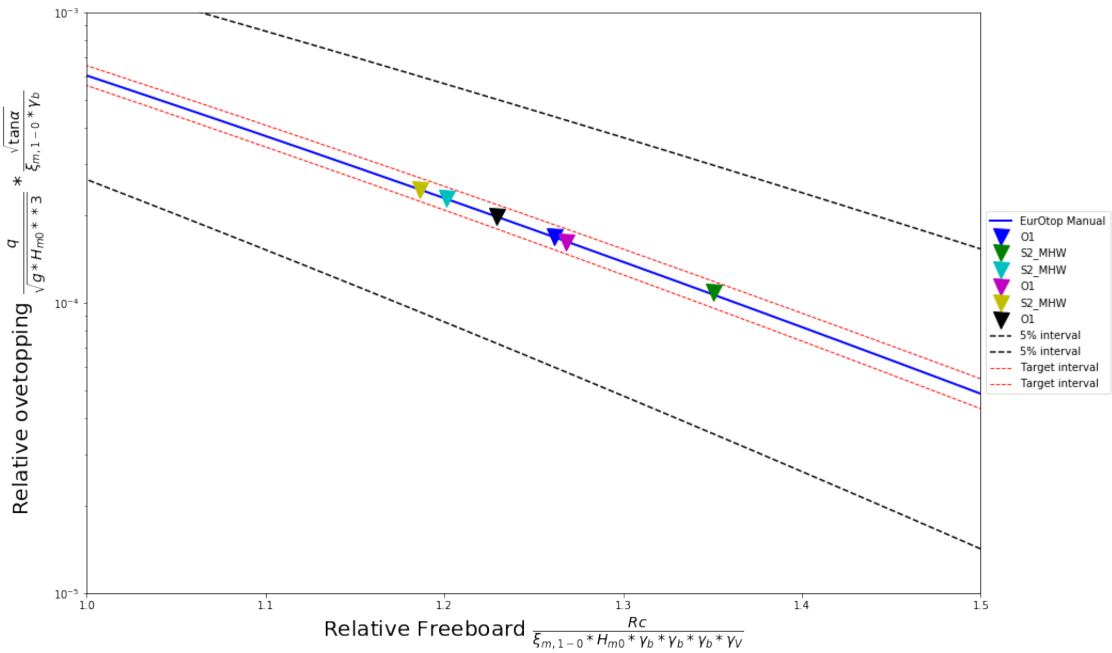

Figure 6.18: Measured data plotted with lower slope of 1:1.5 and fitted roughness

6.4.2. Analysis results lower slope 1:1:5

In this section the test 29 to 33 with hydraulic conditions 02 and S2MHW from Table B.1 are discussed. With these conditions the overtopping discharge is around 5 to 8 l/m/s. In the section above, the roughness factor of the lower slope with a slope of 1:2 is discussed. In this section a cross section with a lower slope of 1:1.5 is tested. The roughness coefficient of the upper slope is calculated with Equation 2.4.

Calculated roughness coefficient

The roughness coefficient of the lower slope is considered to be constant if the water level is around the berm (Van der Meer et al., 2016). The roughness of the lower slope is 0.49 according to Jiminez (Jiménez Moreno, 2017). In the section of the BAM flume the roughness coefficient of the lower slope is considered to be 0.49. This roughness coefficient is used to calculate to the amount of overtopping, this is overestimate the amount of overtopping and can be seen in Figure 6.19. The blue data points are lying under the blue line of Equation 2.16.

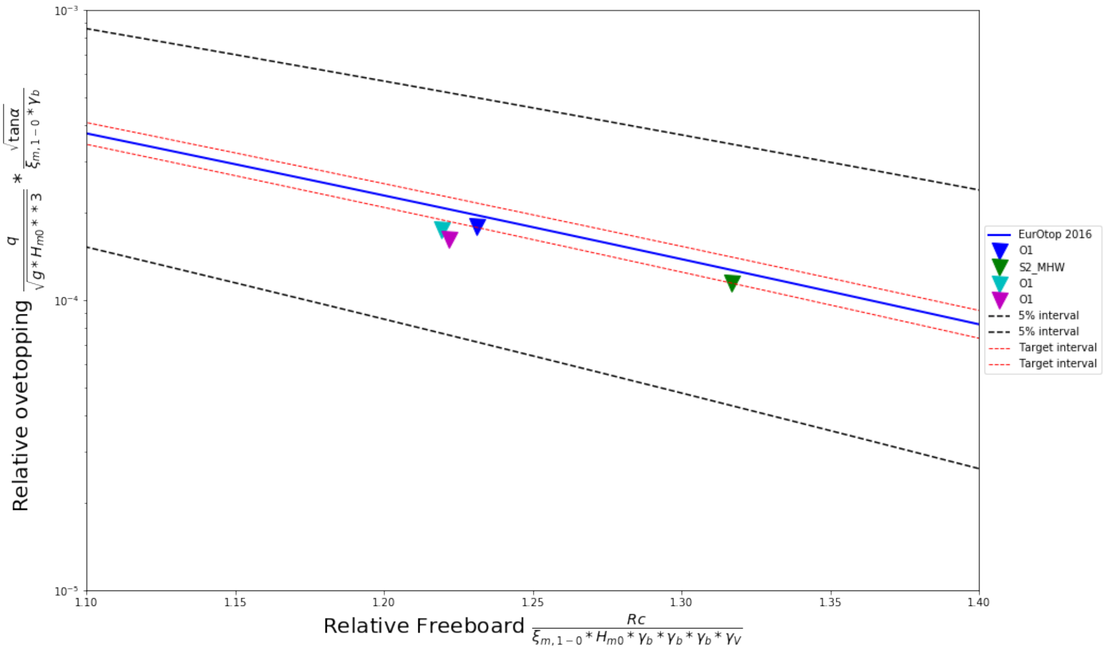

Figure 6.19: Measured data plotted with lower slope of 1:1.5 and calculated roughness

Fitted roughness coefficient

To perform a better estimation of the total overtopping, the roughness coefficient of the lower slope in Equation 2.16 is changed. The best fitted roughness coefficient for the particular tests are given in Table 6.8. A roughness coefficient for the lower slope between 0.28 and 0.39 introduce an average error of 0.193 l/m/s between the calculated overtopping and measured overtopping in the flume, this can be seen in Figure 6.20. The average roughness coefficient of the different tests is 0.35, the average error is in that case 0.435 l/m/s.

Table 6.8: Fitted roughness coefficients lower slope.

Roughness coefficient lower slope	Difference
	l/m/s
0.38	0.218
0.39	0.144
0.35	0.198
0.28	0.212

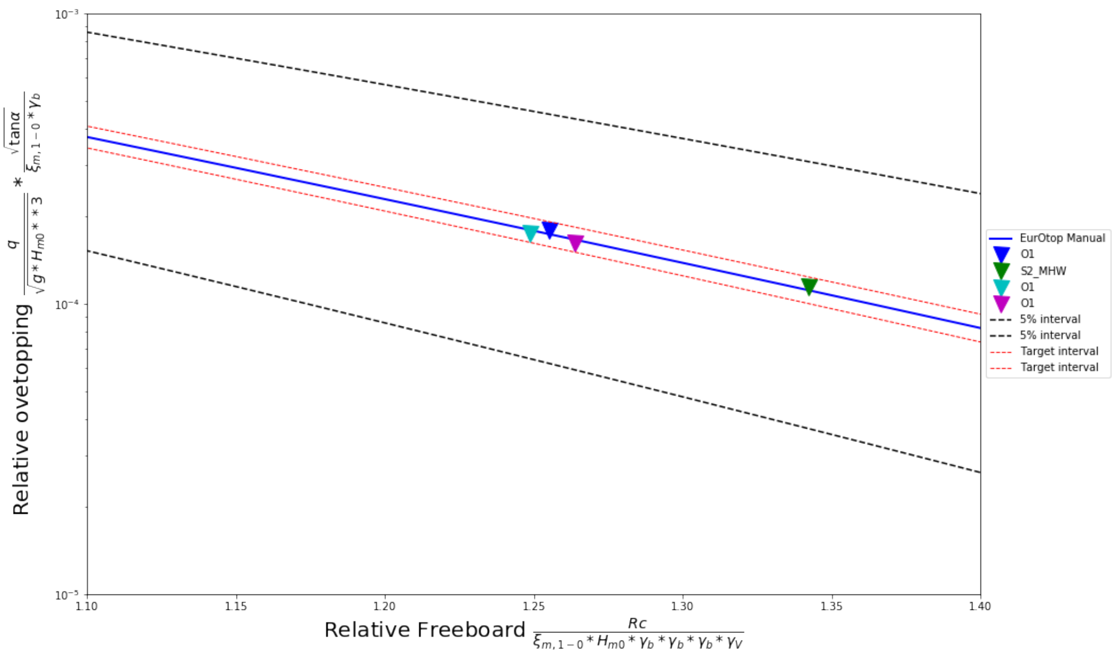

Figure 6.20: Measured data plotted with lower slope of 1:1.5 and fitted roughness

6.4.3. Analysis of low overtopping rates

In the test serie of the Schelde flume also low overtopping discharge of 0.03-0.07 are measured. With these low overtopping rates, scale effects start to occur (De Rouck et al., 2005). Test that are used for Equation 2.4 are done with overtopping rates larger than 0.5 l/m/s. To look at the possible scale effects Equation 2.4 is used to calculate the roughness coefficient of the upper slope. The roughness of the lower slope is 0.35 (1:1.5) and 0.568 (1:2) according the previous section. In Figure 6.21 it can be seen that most of the data points are lying above the blue line of Equation 2.16, so the calculation underestimate the amount of overtopping. In Table 6.9 the calculated and measured overtopping rates are given in the first two columns. The difference between these two is the measured scale factor that is applied between the measured overtopping and calculated overtopping. It can be seen that there are scale effects and that the amount of overtopping is higher than calculated, the scale factor varies from 1 to 10. The measurement method in the Schelde flume is not accurate for low overtopping discharges, it is design for overtopping discharges of 10 l/m/s. In the end, all the test consist of a higher overtopping than calculated, this is discussed later on. The scale factor is in all the cases positive, but it is not possible to quantify a scale factor related to the amount of overtopping discharge smaller than 1 l/m/s. The calculated scale factor according the Eurotop manual is in all the test higher than the measured scale factor. The theory of scale effects for low overtopping rates is valid in this case.

Table 6.9: Scale effect low overtopping.

Calculated overtopping	Measured overtopping	Measured scale factor	Calculated scale factor
l/m/s	l/m/s		
0.0069	0.03	4.3521	11.7332
0.0218	0.07	3.2075	6.3323
0.0223	0.07	3.1372	6.235
0.0236	0.03	1.272	5.9986
0.0236	0.04	1.696	5.9986
0.0064	0.07	10.8654	19.3866

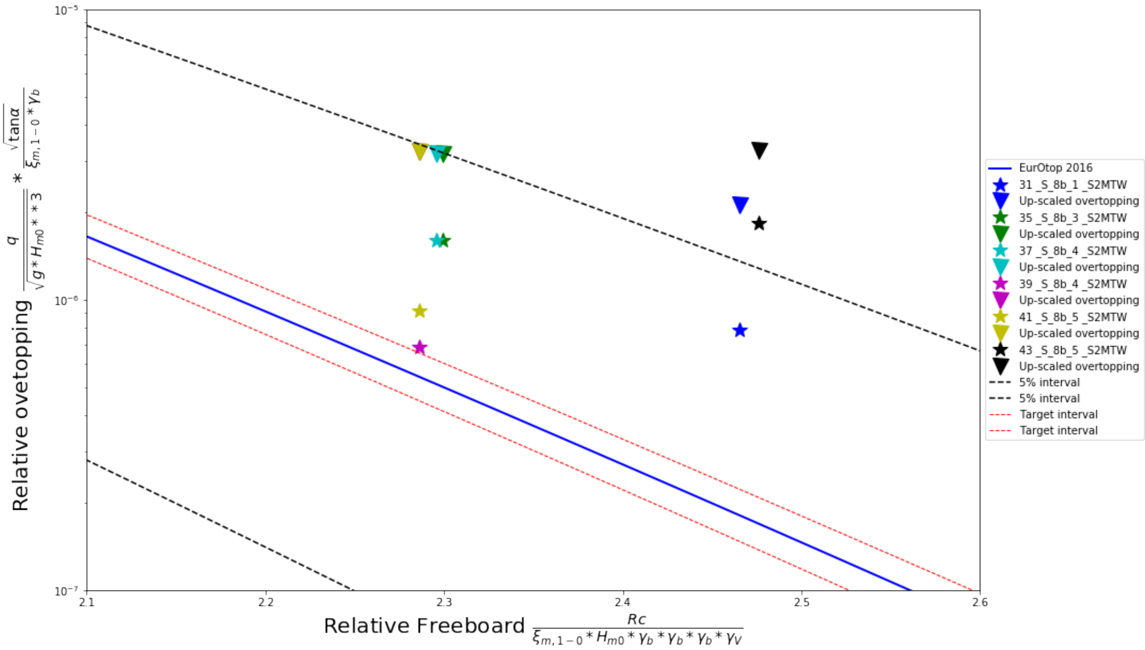

Figure 6.21: Measured data for low overtopping plotted with an up-scaled calculated overtopping

7

Discussion

During the experiments it became clear that the current experimental setups may deviate from each other and the prototype on some important points. At first the experimental set-up is discussed. Secondly the influence of different parameters is discussed. Lastly, the scale effects between the flumes and prototype of the Afsluitdijk are discussed.

7.1. Experiment set-up

7.1.1. Wave conditions in the different flumes

All the structures are situated at intermediate water depth so waves are affected by the foreshore. The decrease in wave height is measured in the BAM & Schelde flume by measuring at the deep water point and the Hydra NL point. In the Delta flume wave conditions are only measured at deep water (there were no wave gauges at the Hydra NL point). The influence of the foreshore on the wave conditions is considered to be equal in all the tests, due to the same foreshore. The decrease in wave height in the Delta flume is calculated according to the relation between deep water and Hydra NL in the Schelde flume. This assumption is valid. However if there are any changes in wave conditions, these are not taken into account in the Delta flume.

7.1.2. Wave conditions at the toe

The empirical formula for the amount of overtopping is based on wave conditions at the toe of the structure. In this document, the wave conditions at Hydra NL are used in the empirical formula. Wave breaking, shoaling, set-up and reflection could influence the wave conditions between the Hydra NL point and the toe of the structure, these processes are not taken into account. The reliability of the wave conditions at the toe of the structure are therefore smaller. Hydra NL is chosen as one particular point on the foreshore, but there could be a slightly difference in conditions around the Hydra NL point and at the toe of the structure. In order to determine the wave height in the Delta flume in a different way, simulations in Mike2 are done during this thesis. These simulations were not used for the wave conditions in the Delta flume, but they show differences between the wave conditions at the Hydra NL point and toe of the structure.

7.1.3. Measurement errors on overtopping in different flumes

In the different flumes, the overtopping is measured in different ways. In the BAM flume a floating tank is used, at the end of the tests the mass increase of the tank is weighted. In the Schelde and Delta flume the overtopping is measured in a tank with a certain water level before and after the tests. The increased water level, multiplied with the area of the tank is the amount of overtopping. The area of the tank is different in the Schelde and Delta flume. The larger the area of the tank, the larger the error in measured overtopping due to wrong measured water levels. These different measuring systems could influence the amount of overtopping and decrease the reliability of the results.

7.1.4. Amount of tests

In the BAM flume, most of the tests are performed three times, this can be seen in Table **??**. All the tests in the Schelde and Delta flume are performed once. Due to the high variation in overtopping values, more tests with the same conditions and structure increase the reliability. The reliability of an outcome is higher if the tests is performed a couple of times. The results of the BAM flume are therefore more reliable compared to the Schelde en Delta flume results.

7.1.5. Measuring low overtopping

In tests with low overtopping, the reliability of the tests is lower compared with the tests with overtopping of around 10 l/m/s. The set-up of the tests is made to measure overtopping of around 10 l/m/s. For example in a tests with low overtopping, small water drops on the chute could be important for the amount of overtopping. The reliability of the conclusions about low overtopping are therefore lower compared with overtopping of around 10 l/m/s.

7.2. Influence factors on the amount of overtopping

7.2.1. Roughness Levvel-blocs

In the BAM flume, the roughness of the lower slope is determined to be 0.96. This is based on tests with only Levvel-blocs on the lower slope and a smooth upper slope. The influence of the smooth upper slope in the weighted roughness is high, whereby the reliability of the roughness coefficient for the Levvel-blocs is lower, discussed below. In the analysis of the Schelde flume, Equation 2.4 is used as a given and the roughness coefficient of the lower slope is changed. The roughness coefficient of the lower slope is in that case similar as Jiminez; 0.49 according to Jiminez (Jiménez Moreno, 2017), 0.51 in case of the Schelde flume. In the Schelde flume, a combination of the research of Jimenez and Capel gives reliable results with a lower slope of 1:2.

If Equation 2.4 is less reliable in the Schelde flume (mentioned in next section), the roughness coefficient of the Levvel-blocs is less reliable in the Schelde flume. Because the influence of a smooth top slope in the weighted roughness is high, the reliability of roughness of the Levvel-blocs in the BAM flume is also low. It is difficult to make an exact estimation of the roughness coefficient of Levvel-blocs in the different flumes, but the reliability of the one in the BAM flume is higher.

7.2.2. Validation of Capel equation

Equation 2.4 is used to calculate the roughness of the upper slope, this equation is based on tests that are performed on a scale of 1:22 (Capel, 2015). These tests are verified on a scale of 1:11 to look at any possible scale effects. The data points of the fitted roughness on a scale of 1:11 lie in the scatter of the data on a scale of 1:22. From that it is concluded that scale effects do not occur because the scatter between the two scales show hardly any difference. Different overtopping discharges measured in the BAM flume could refer to possible scale effects because the scale of the BAM flume is 1:36.2

Equation 2.4 of Capel is based on overtopping tests between 0.5 - 30 l/m/s. Wave overtopping higher than 30 l/m/s is not tests in this thesis, wave overtopping lower than 0.5 l/m/s is tests a couple of times. With these low overtopping results, the influence of ribs on low overtopping and possible scale effects in case of low overtopping are discussed. It is difficult to distinguish one of these two processes to the difference between the calculated overtopping with Equation 2.4 and the measured overtopping in the physical model tests.

7.2.3. Amount of overtopping

For overtopping values that are larger than 3 l/m/s the roughness coefficient is calculated correctly with Equation 2.4 for ribs of 0.05 meter. If the amount of overtopping is less than 3 l/m/s the ribs start to behave rougher than the Equation 2.4 in the BAM flume. The height of the ribs becomes dominant over the water depth. Because Equation 2.4 is based on larger scale tests than the BAM flume, this could be referred to scale effects in case of low overtopping. The difference is actual larger than the theory of scale effects. Either the theory of the scale effects is wrong, or Equation 2.4 underestimates the amount of overtopping.

7.2.4. Wave steepness

The influence of the wave steepness is large in a composite slope with ribs on the upper slope. If the wave steepness is around 0.047 the calculation method with Equation 2.4 is reliable. If the wave steepness becomes larger the amount of overtopping is overestimated with the calculation method. If the wave steepness becomes smaller the amount of overtopping is underestimated. In the BAM flume more overtopping is measured with a wave steepness of 0.044 than a completely smooth slope. With a wave steepness of 0.044 the roughness coefficient of the upper slope needs to be larger than 1. This is physically impossible so another coefficient in Equation 2.19 is determined incorrectly in this case, it is difficult to say which parameter this is.

7.2.5. Wave height

There is an influence of the wave height in a composite slope, but this is not significant. If the wave height is 3.38 meter, the prediction method with Equation 2.4 is accurate. If the wave height increases, the amount of overtopping is underestimated with Equation 2.4. There are two possible explanations for this effect. Firstly, the increased wave height creates relatively a lower roughness and a more smooth slope. Secondly, the increased amount of overtopping creates a smoother slope. It is difficult to say if this is only the effect of the wave height, an increased overtopping discharge or both. The prediction method of the amount of overtopping for the physical model tests was not reliable to get exactly the same amount of overtopping with different wave heights. A conclusion about the influence of the wave height is difficult, but the significant wave height in storm conditions at the Afsluitdijk is calculated correctly.

7.2.6. Rib height

The larger the rib height, the larger the error between the measured overtopping and the calculated overtopping in the BAM flume. The tests in the BAM flume for dike section 17a with ribs of 0.05 meter and Equation 2.4 is calculating the roughness correctly. For larger rib heights (0.12-0.23 meter) that are tests for dike section 8b, the error of Equation 2.4 increases. It could be a scale effect that lower rib heights are calculate correctly according Equation 2.4 and the large rib heights show a larger error. The water depth on the structure is smaller in the BAM flume compared with the experiments that are used for Equation 2.4, the ribs are more dominant over the water layer thickness in the BAM flume.

Equation 2.4 is tested with ribs of 0.22 meter. In Figure 7.1 the turbulent mixing layer is given for a rib pattern of 0.23 meter, with 1.1 meter space between two consecutive ribs. Equation 2.4 is based on ribs of 0.22 meter and a space of 1.1 meter between two consecutive ribs, the turbulent mixing is approximately the same with ribs of 0.23 meter. The length of the turbulent mixing layer is calculated according (Schiereck, 2003).

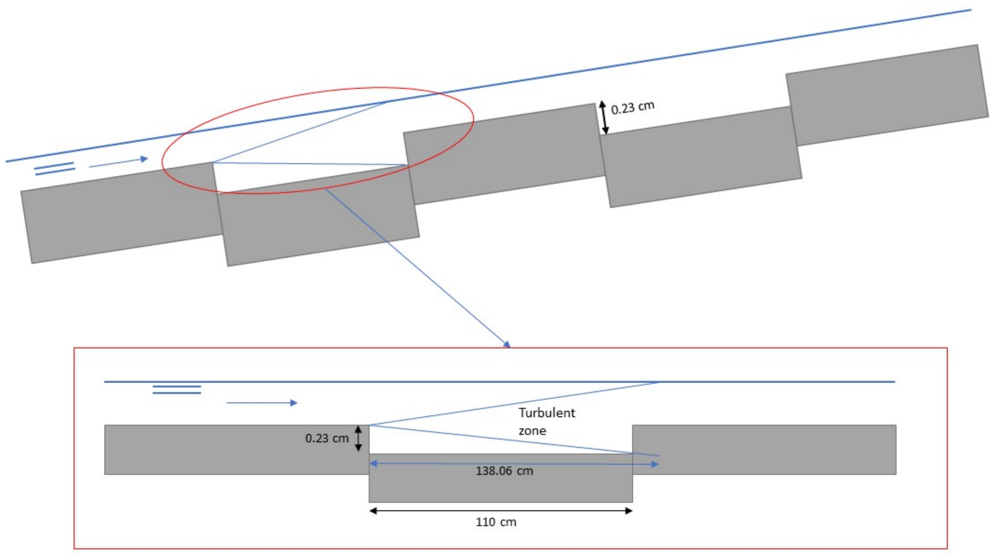

Figure 7.1: Turbulent mixing layer with a rib height of 0.23 meter.

With Ribs of 0.05 meter, the turbulent mixing layer decreases to 0.28 meter, Figure 7.2. The turbulent mixing layer is attached to the slope before the next rib, in contrast to the length of the mixing layer with ribs of 0.23 meter where the mixing layer is attached to the slope at the next layer of ribs.

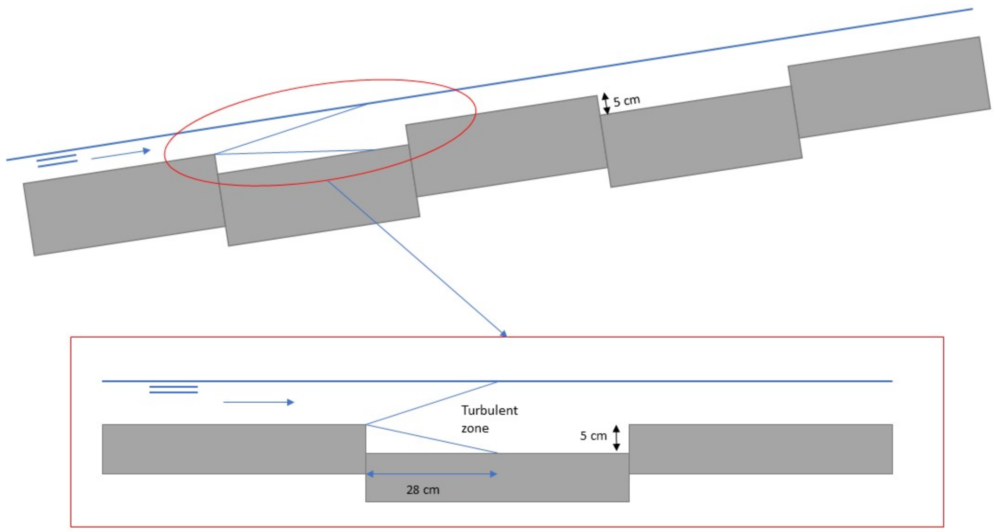

Figure 7.2: Turbulent mixing layer with a rib height of 0.05 meter.

7.2.7. Rib pattern

The different patterns that are tested in the BAM flume are all in the same range of error between the measured overtopping and the calculated overtopping. This error is caused by the height of the ribs of 0.23 meter which is discussed above. The largest roughness is created by ribs of pattern 2, see Figure 4.12. Rib pattern 8 and rib pattern 2T reduce the roughness and create more overtopping. The lowest roughness is created by pattern 2T. This can be explained by the effect that the flow of water on the upper slope is smoother. Water can flow easily around the ribs which will lower the roughness of the upper slope. With rib pattern 8 the front width face is the largest. Due to this wider front face, the flow of water between ribs is more restricted. Water also flows more difficult downwards the slope. If there is water between the ribs and a new wave is flowing over the structure, the roughness of the upper slope is lower.

7.2.8. Amount of ribs

Both 11 and 6 ribs of 0.23 meter are creating the same error if Equation 2.4 is used. Equation 2.4 is based on the pattern where the space between the ribs is equal to the width of two consecutive ribs. The space between two consecutive ribs is approximately the same with 11 and 6 ribs. If the amount of ribs decrease, the space between the ribs increase and this reduce the error between the calculated and measured overtopping. The error which is influenced by the height of the ribs is more dominant than the error by the amount of ribs.

7.2.9. Combination of lower steepness and lower ribs

With the tests of dike section 17a in the Delta flume, there can be seen that if ribs of 0.05 meter are used, there is significantly more overtopping than calculated. The wave steepness in this test is also 0.044 and extra overtopping is measured in the BAM flume. An explanation for this effect could be that the duration of the low steepness waves is much longer. The first part of the wave is flowing between the ribs and is filling the space between the ribs with water. The last part of the wave is flowing over a combination of ribs and water and is not influenced by the ribs only. This effect does not occur in a situation with ribs of 0.21 and a wave steepness of 0.044, the height of the ribs is large enough to prevent filling the space between the ribs with water. It can be concluded that a combination of a lower steepness waves and low ribs of 0.05 meter will create a high difference between the measured overtopping and the calculated overtopping in the Delta & BAM Flume.

7.2.10. Roughness density parameter

The exposed area of the front face can be characterized by a roughness density parameter, $\rho_{\gamma f}$. This roughness density parameter is defined as: the total exposed area of all protruding elements, divided by the width and length of the slope from the waterline to the crest level (Capel, 2015). This roughness density parameter is reliable, if approximately the same configuration is tested as in the Equation of Capel. If another configuration is tested, the reliability will be lower.

7.2.11. Height of the toe

The influence of the height of the toe is large in the calculation of the amount of overtopping. In Equation 2.3 it is mentioned that the lower part to calculate the length of the structure, is limited by the height of the toe. If the wave height (multiplied with 1.5) is larger than the difference between SWL and the height of the toe, the waves feel the structure earlier at the front of the toe. Waves will break differently on the slope which is covered in $\xi_{m-1,0}$. With the limitation in Equation 2.3 the influence of the wave height on $\xi_{m-1,0}$ is smaller.

7.2.12. Weighted roughness

The weighted roughness is calculated according to Equation 2.9, where roughness elements have little or no effect 0.25 $R_{u2\%,smooth}$ below the still water line or 0.50 $R_{u2\%,smooth}$ above the still water line. According to EurOtop manual roughness elements applied only underwater with a smooth upper slope have no effect and, in such a case, should be considered as a smooth slope (Van der Meer et al., 2016). The roughness elements of the Levvel-blocs in the BAM flume are determined with a smooth upper slope. The theory outlines that the influence of Levvel-blocs in this case should be minimal. With the tests in the BAM flume the fitted roughness is smaller than 1 for the lower slope, so the theory does not seem valid in this case.

7.2.13. Combination of different factors; roughness and berm

The combination of different factors that are influencing the amount of overtopping makes it difficult to make one clear definition for the amount of overtopping. In the tests of 17a in the BAM flume, the first tests was performed to discuss the influence of the berm coefficient. The berm coefficient was adjusted by a factor of 0.027 to match with the measured overtopping, given in Equation 5.1. This is the only case where the berm coefficient could be adjusted to be more specific about the other influence factors in the design. In all the other tests, their is one roughness element at the lower or upper slope, so it is difficult to say if the error is originating from the berm coefficient, from a wrongly chosen roughness coefficient or another effect.

In the tests of 8b in the BAM flume, the first tests were performed with different berm lengths. The roughness coefficient of the lower slope is a constant parameter in this situation. The berm length is changed in this situation and calculated according Equation 2.6. With a constant roughness of the lower slope of 0.96 in combination with the berm coefficient according Equation 2.6 the error of the calculated amount of overtopping is small. The estimation of the amount of overtopping for a combination of a berm and roughness element is reliable.

7.3. Possible scale effects

7.3.1. Validation of Capel equation

Equation 2.4 is used to calculate the roughness of the upper slope, this equation is based on tests that are performed on a scale of 1:22 (Capel, 2015). These tests are verified on a scale of 1:11 to look at any possible scale effects. These data points lie in the scatter which was obtained by the data on a scale of 1:22. There is concluded that scale effects do not occur because the scatter of the scale effects is equal.

7.3.2. Different scales

In Figure 7.3 different tests of dike section 17a and 8b, in the BAM, Schelde & Delta flume are outlined. The roughness coefficient for the Levvel-blocs on the lower slope in dike section 8b is chosen to be 0.95. The smooth lower slope of dike section 17a has a roughness factor of 1. The roughness of the upper slope for both dike sections is calculated according to Equation 2.4. The ribs of dike section 8b are all 0.23 meter and have slightly different patterns. The ribs of dike section 17a are all 0.05 except the last Delta flume tests with ribs of 0.21 meter.

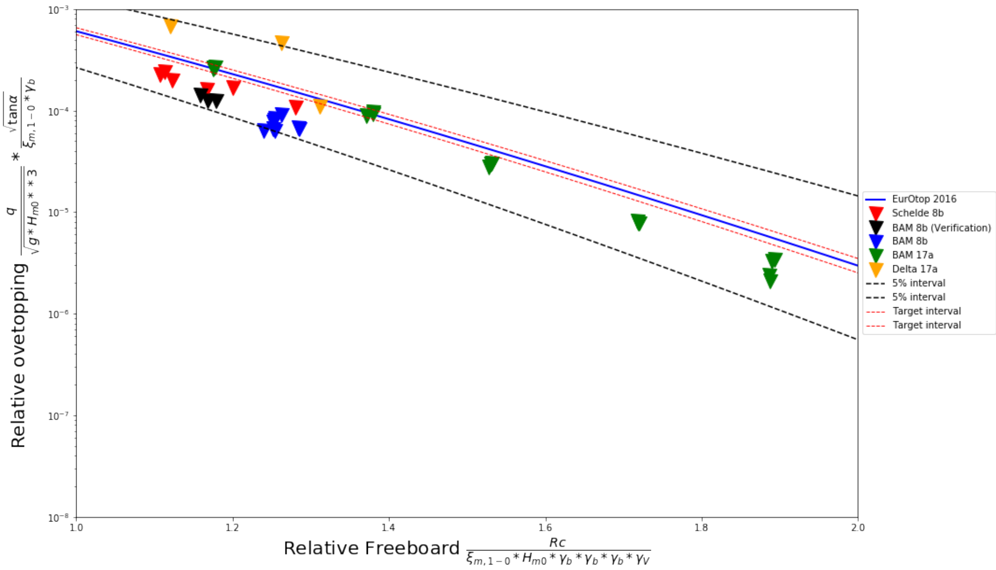

Figure 7.3: tests results of 8b and 17a with calculated roughness upper slope.

In Figure 7.4 the calculated overtopping is plotted against the measured overtopping. It can been seen that the results of the BAM flume for dike section 17a are calculated correctly. The tests of 8b in the BAM and Schelde flume show a large scatter.

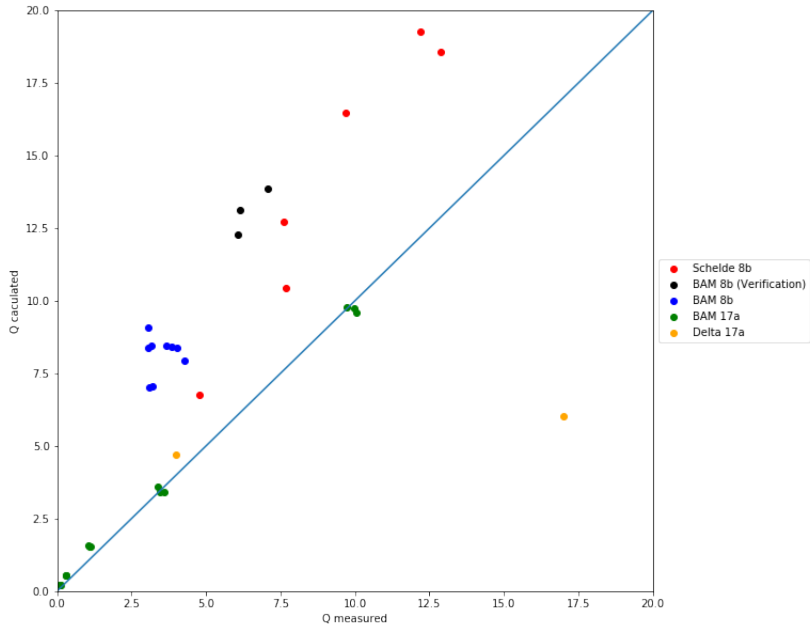

Figure 7.4: Q measured plotted against the Q calculated overtopping according the literature.

In Figure 7.5 the roughness coefficient that is calculated according to Equation 2.4 is plotted against the roughness coefficient that is fitted for the upper slope with the physical model tests. The roughness coefficient is calculated correctly for the tests in the BAM flume for dike section 17a. In Figure 7.5 it can be seen that the tests in the BAM flume give a larger error than the tests of the Schelde flume for dike section 8b. The core of the Schelde flume is permeable in contrast with the impermeable core in the BAM flume, this effect will only enlarge the difference if the core in both flumes were modelled in the same way. The difference between the BAM and Schelde flume could refer to any scale effects. In the Schelde flume there is measured more overtopping compared with the BAM flume. This effect was also seen with dike section 17a, whereby a smaller scale (BAM) underestimated the amount of overtopping for Ribs of 0.05 meter compared with a large scale (Delta). In the Delta flume the amount of overtopping with ribs of 0.21 meter is calculated correctly, in the BAM flume (with section 8b) there was seen that such high ribs are rougher on smaller scale. This implies that there are also scale effects between the BAM and Delta Flume if two identical slopes with ribs of 0.21 meter were tested.

The difference in roughness coefficient of the upper slope implies that there are scale effects between the different tests. The roughness coefficients of the upper slope is not particularly the source of the scale effects. These scale effects could also occur because of the Levvel-blocs on the lower slope or the composite slope in the different designs.

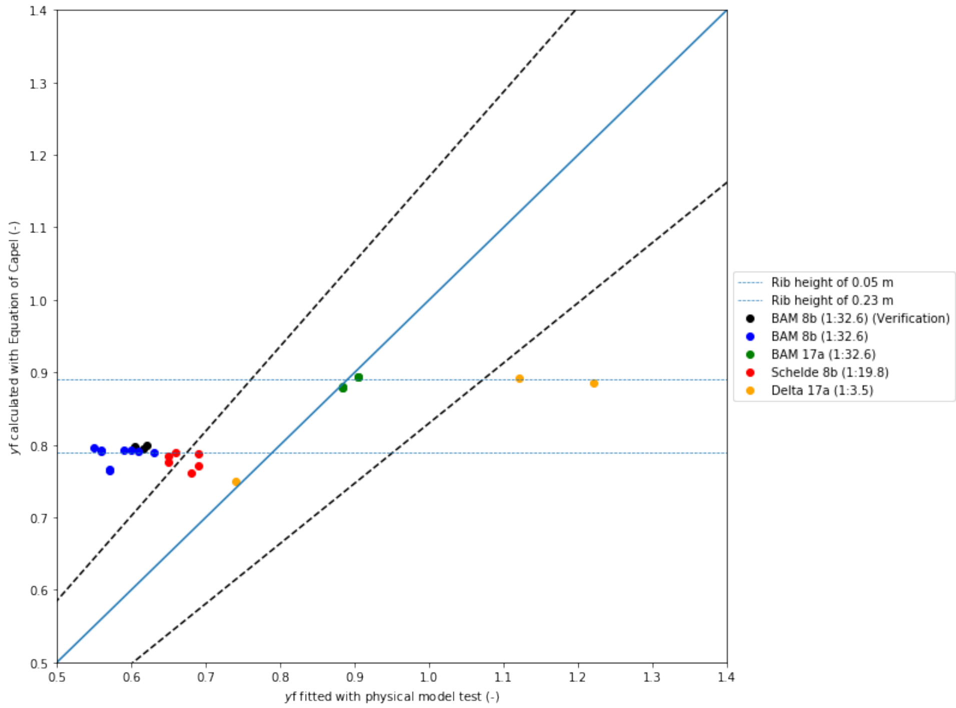

Figure 7.5: $\gamma_{f,calculated}$ according Capel plotted against $\gamma_{f,fitted}$ for different physical model tests.

7.3.3. Low overtopping

For dike section 8b in the Schelde flume, low overtopping rates are discussed. The overtopping is higher than calculated according to the literature. In all the six different tests, the overtopping is higher than calculated. As discussed earlier, the measurement system in the Schelde flume is not designed to measure low overtopping rates. A small change in water level will influence the amount of measured overtopping. In the end the different tests all have an underestimated overtopping. If you compare it with Equation 6.13-6.15 in the EurOtop manual, the underestimated overtopping is smaller than the up-scaled overtopping. It is expected that the measured overtopping for low overtopping rates will be even higher in prototype. Any quantification of a scale factor is difficult, the range is too high due to the low measurement accuracy.

7.3.4. Prototype

Mentioned in the section above, scale effects occur in the different flumes. The overtopping in the Delta flume (1:2.95) need to be smaller than 10 l/m/s. There are no physical model tests performed on prototype scale (1:1). Between all the other scales, there is measured more overtopping on a larger scale. It is expected that more overtopping will be measured in prototype. The amount of overtopping is not predictable, but will be underestimated and larger than 10 l/m/s.

8

Conclusions and recommendations

8.1. Conclusion

After analysis of the test results and discussion, conclusions can be drawn. They are presented in answers to the research questions that are mentioned in the first chapter. What is the combined effect of roughness and composite slopes on wave overtopping and are there any scale effects on overtopping for the new Afsluitdijk?

Different influence factors on the roughness of the lower slope for the Levvel-blocs

The influence on the roughness coefficient of the lower slope is determined for a dike section with a composite slope. This is a composite slope consisting of a berm around mean water level, Levvel-blocs on the lower slope and a smooth upper slope. On a scale of 1:36.2 the roughness coefficient of Levvel-blocs is higher for a 1:2 slope (γ_f = 0.96) compared to a slope of 1:1.5 (γ_f = 0.51). For a composite slope on a scale of 1:32.6, the roughness coefficient of Levvel-blocs is higher than the roughness coefficient (γ_f = 0.49) determined by Jiminez. The difference in roughness coefficient for different slopes is also noticeable in different scales. Using a slope of 1:1.5 reduces the amount of overtopping significantly on both a 1:32.6 and 1:19.8 scale.

Different influence factors on the roughness of the upper slope with a rib pattern

With the tests that are performed during this research, different hydraulic influence factors on the roughness of the upper slope are researched; amount of wave overtopping, wave steepness and wave height. These tests are performed on a scale of 1:32.6 with ribs of 0.05 meter.

- For overtopping values that are larger than 3 l/m/s the roughness coefficient is calculated correctly with Equation 2.4. If the amount of overtopping is lower than 3 l/m/s the ribs start to behave rougher than Equation 2.4, because the ribs are dominant over the water layer thickness on the upper slope. The upper slope is rougher than Equation 2.4. If Rijkswaterstaat wants to predict any overtopping (highway should be closed), it is important to notice that Equation 2.4 behaves rougher and there is less overtopping expected.
- The influence of the wave steepness on the roughness coefficient is large in a composite slope. If the wave steepness is around 0.047 the prediction method with Equation 2.4 is reliable. If the wave steepness becomes larger the amount of overtopping is overestimated with the calculation method. If the wave steepness becomes smaller the amount of overtopping is underestimated.
- There is an influence of wave height on the roughness coefficient in a composite slope, but this is not significant. If the wave height is according to the storm conditions of the Afsluitdijk, the prediction method with Equation 2.4 is reliable.

In tests with ribs of 0.05 meter and a low wave steepness, there can be seen that for both a 1:2.95 and 1:36.2 scale there is significantly more overtopping than calculated. There can be concluded that a combination of low steepness waves and low ribs of 0.05 meter will create an underestimation of the amount of overtopping on different scales.

With the tests that are performed during this research, different structural influence factors on the roughness of the upper slope are discussed; rib height, rib pattern and amount of ribs:

- The larger the rib height, the larger the error between the measured overtopping and the calculated overtopping on small scale. This research shows that on a scale of 1:32.6 with ribs of 0.05 meter, Equation 2.4 calculates the roughness correctly. For larger ribs (0.12-0.23 meter), the error of Equation 2.4 increases on a scale of 1:36.2.
- The highest roughness is created by rib patterns where the next consecutive rib slightly overlaps with the previous one. If there is no overlap or too much overlap, the pattern behaves smoother.
- If the amount of ribs in longitudinal direction decreases, the space between two consecutive ribs increases. In case of a larger space between two consecutive ribs, the error of Equation 2.4 increases.

The estimation of the amount of overtopping for a combination of different coefficients; roughness and berm

The combination of different factors that influence the amount of overtopping makes it difficult to make a reliable estimation of the amount of overtopping. The way in which the coefficients are determined step by step in this research, gives a more reliable solution. If a lot of coefficients are determined simultaneously, it is difficult to say if the error is originating from a particular coefficient or combination of two coefficients. The way in which the EurOtop manual describes the influence of different coefficients on the amount of overtopping gives a reliable estimation.

Scale effects on wave overtopping

The overtopping values in the BAM flume with a rib height of 0.05 show hardly any difference compared to Equation 2.4. The BAM tests are performed on a 1:36.2 scale and Equation 2.4 is calibrated on a 1:22 scale. If the height of the ribs increase on a scale of 1:36.2, the difference between the calculated roughness according Equation 2.4 and the fitted roughness increases. The higher ribs on a scale of 1:32 are, the more dominant their effect on the water that is flowing over the structure is, compared to a scale of 1:22. If the ribs are more dominant over the water layer thickness, the roughness of the upper slope is increased and a higher influence of the ribs is measured on a smaller scale.

According to the literature, scale effects could only occur for overtopping discharges that are lower than 1 l/m/s. With the tests of this research, it can be noted that if ribs of 0.05 meter are used, the amount of overtopping is underestimated on a scale of 1:2.95 and calculated correctly on a scale of 1:36.2. The calculation error on a scale of 1:2.95 reduces if ribs of 0.21 meter are tested. On a scale of 1:36.2, ribs of 0.21 meter behave rougher than calculated. In both situations scale effects occur for overtopping discharges larger than 1 l/m/s. In Figure 7.5 can be seen that there is more overtopping on a scale of 1:19.8 compared with a scale of 1:36.2, scale effects occur also in this situation for overtopping discharges larger than 1 l/m/s. The origin of these scale effects could be that wave breaking on a rubble slope and the front wedge flow past the structure enclose air in the water. This water includes the dissolved air that is flowing over the upper slope. Relatively more air will be enclosed on large scale. Due to differences in relative air contents, the mass density of the up-rushing water is smaller on a larger scale. Water can flow easily over the upper slope with entrapped air which will lead to more overtopping. If there is more water between the ribs, the up-rushing water does not have enough time to flow downwards. The influence of the ribs for the next wave is smaller, because water between the ribs creates a smoother slope. In the end, all these effects will create more overtopping on a larger scale.

During this research, overtopping discharges lower than 1 l/m/s are measured on small scale. The roughness of the upper slope in these situations is larger than calculated with Equation 2.4 and less overtopping is measured. Because Equation 2.4 is based on a larger scale compared to the experiments in this research, scale effects occur. Less overtopping is measured on small scale than calculated with the theory. The amount of overestimated overtopping is larger than the up-scaled overtopping according to the EurOtop manual.

In this research, overtopping discharges of 0.03-0.07 l/m/s are measured on a scale of 1:19.8 and 1:2.95. The amount of overtopping is underestimated with Equation 2.4. In all the low overtopping tests the measured overtopping is higher than the calculated overtopping. This difference could be explained by scale effects. The EurOtop manual outlines a method for scale effects. All the test results are within the range of the EurOtop manual and are therefore predictable with the theory.

8.2. Recommendations

- This research is based on the wave overtopping over a period of 1000 waves. To get a better impression of the influence of the ribs compared to the water layer thickness, overtopping could be researched wave by wave. A relation between water layer thickness, turbulence generated by the ribs and the flow speed between and over the ribs. The process in which the water is influenced by the ribs could be visualized with slow motion cameras on the side of the structure or with laser scanning above the structure.
- To increase the reliability of scale effects on overtopping, it is preferable to continue making comparisons of new large and small scale overtopping tests in the future. In this research cross sections are not exactly the same in across different flumes. More tests with three identical slopes will introduce more knowledge about possible scale effects. Uniform hydraulic conditions are recommended, this research shows that the influence of the wave height, wave steepness and wave overtopping is important. A primary consideration should be at the influence of different factors in the design with the same hydraulic conditions. After this the influence of hydraulic conditions could be researched.
- Research the influence of the combination of Levvel-blocs on the lower slope and ribs on the upper slope better. Start with only a smooth slope to determine the influence of the berm should be investigated. Secondly a smooth lower slope in combination with ribs on the upper slope. Thirdly a smooth upper slope with Levvel-blocs on the lower slope. Lastly, a structure which consist of a lower slope with Levvel-blocs and a upper slope with ribs. The effect of the two different roughness factors is measured separately. After this, the effect of both the lower and upper slope on the weighted average could be discussed in more detail.
- This research outlines that the roughness coefficient of Levvel-blocs varies by the slope of the blocs. Analyses about the influence of the different parameters that might affect the roughness coefficient should be conducted to give a better representation of the roughness coefficient in different situations. For the upper slope is discussed what the influence of the wave height, wave steepness and amount of overtopping is. This could also be performed for the Levvel-blocs. The influence of the Levvel-blocs, due to the average roughness coefficient, is very small. It is therefore difficult in this situation to discuss the influence of the wave height, wave steepness and amount of overtopping on the roughness coefficient.
- All the structures are situated at intermediate water depths so waves are affected by the foreshore. The wave height is decreased by the foreshore due to energy dissipation. In some of the cases it is difficult to predict what the wave conditions are at the toe of the structure. Particularly for the Afsluitdijk where the foreshore is necessary in the design. To investigate the influence of the structure on overtopping and possible scale effects, it is better to do the tests without a foreshore. Hydraulic processes such as wave breaking and shoaling on the foreshore could be removed which will increase the reliability on the estimation of overtopping.
- The influence of the height of the toe is dominant in the calculation method of the amount of overtopping. In Equation 2.3 it is mentioned that the lower part to calculate the length of the structure, is limited by the height of the toe. Future research about the influence of the height of toe in front of the structure on the amount of overtopping is valuable.

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A
Hydraulic-Conditions

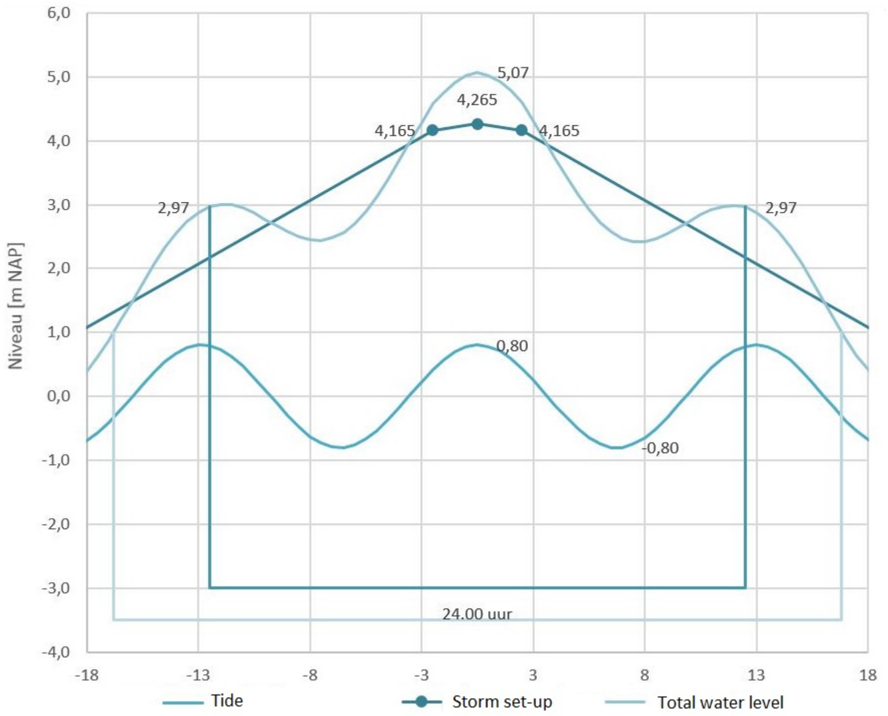


Figure A.1: Wave conditions

Table A.1: Target hydraulic conditions.

		Deep water			Hydra NL		
Test code	Waterlevel	Hm0	Tp	Tm-0,1	Hm0	Tp	Tm-0,1
O1a	5,21	4,33	7,63	6,92	4,27	7,69	6,97
S2MHW	5,07	4,21	7,56	6,9	4,15	7,62	6,95
S2MTW	2,97	3,72	7,09	6,37	3,66	7,13	6,42
O1b	5,32	3,4	7,21	6,55	3,38	7,23	6,59
S02	5,12	2,07	7,59	6,90	1,91	7,72	7,08
H1a	5.32	-	-	-	3.38	7.23	6.59
H1b	5.32	-	-	-	3,71	7,60	6.91
H1c	5.32	-	-	-	4.04	7.93	7.20
H1d	5.32	-	-	-	4,36	8,24	7.49
H1e	5.32	-	-	-	4,69	8,54	7.76
S01a	5.32	-	-	-	3.38	7.23	6.59
S01b	5.32	-	-	-	3,38	8,11	7.42
S01c	5.32	-	-	-	3.38	9.37	8.45

B

Test-Program

Table B.1: Test program.

	Cross section	Hydraulic conditions	Test code	Amount of test
	Table 4.2 & 4.3	Table A.1		
1	B_17a_1	O2a	1 _B_17a_1 _O2a	4
2	B_17a_2	O2a	2 _B_17a_2 _O2a	3
3	B_17a_3	O2a	3 _B_17a_3 _O2a	3
4	B_17a_3	H1b	4 _B_17a_3 _H1b	3
5	B_17a_4	O2a	5 _B_17a_4 _O2a	3
6	B_17a_4	H1c	6 _B_17a_4 _H1c	3
7	B_17a_5	O2a	7 _B_17a_5 _O2a	3
8	B_17a_5	H1d	8 _B_17a_5 _H1d	3
9	B_17a_6	O2a	9 _B_17a_6 _O2a	4
10	B_17a_6	H1e	10 _B_17a_6 _H1e	3
11	B_17a_1	S01a	11 _B_17a_1 _S01a	2
12	B_17a_1	S01b	12 _B_17a_1 _S01b	2
13	B_17a_1	S01c	13 _B_17a_1 _S01c	2
14	B_8b_1	O1a	14 _B_8b_1 _O1a	3
15	B_8b_2	O1a	15 _B_8b_2 _O1a	3
16	B_8b_3	O1a	16 _B_8b_3 _O1a	3
17	B_8b_4	O1a	17 _B_8b_4 _O1a	3
18	B_8b_5	O1a	18 _B_8b_5 _O1a	3
19	B_8b_6	O1a	19 _B_8b_6 _O1a	3
20	B_8b_7	O1a	20 _B_8b_7 _O1a	2
21	B_8b_8	O1a	21 _B_8b_8 _O1a	3
22	B_8b_9	O1a	22 _B_8b_9 _O1a	3
23	B_8b_11	O1a	23 _B_8b_11 _O1a	4
24	B_8b_12	O1a	24 _B_8b_12 _O1a	3
25	B_8b_13	O1a	25 _B_8b_13 _O1a	3
26	B_8b_14	O1a	26 _B_8b_14 _O1a	3
27	B_8b_15	O1a	27 _B_8b_15 _O1a	3
28	B_8b_16	O1a	28 _B_8b_16 _O1a	1
29	S_8b_1	O1a	29 _S_8b_1 _O1a	1
30	S_8b_1	S2MHW	30 _S_8b_1 _S2MHW	1
31	S_8b_1	S2MTW	31 _S_8b_1 _S2MTW	1
32	S_8b_1	O1a	32 _S_8b_1 _O1a	1
33	S_8b_2	O1a	33 _S_8b_2 _O1a	1
34	S_8b_3	O1a	34 _S_8b_3 _O1a	1
35	S_8b_3	S2MTW	35 _S_8b_3 _S2MTW	1
36	S_8b_3	S2MHW	36 _S_8b_3 _S2MHW	1
37	S_8b_4	S2MTW	37 _S_8b_4 _S2MTW	1
38	S_8b_4	S2MHW	38 _S_8b_4 _S2MHW	1
39	S_8b_4	S2MTW	39 _S_8b_4 _S2MTW	1
40	S_8b_4	O1a	40 _S_8b_4 _O1a	1
41	S_8b_5	S2MTW	41 _S_8b_5 _S2MTW	1
42	S_8b_5	S2MHW	42 _S_8b_5 _S2MHW	1
43	S_8b_5	S2MTW	43 _S_8b_5 _S2MTW	1
44	S_8b_5	O1a	44 _S_8b_5 _O1a	1
45	D_17a_1	S02	45 _D_17a_1 _S02	1
46	D_17a_1	O1a	46 _D_17a_1 _O1a	2
48	D_17a_2	O1a	48 _D_17a_2 _O1a	1

C

Test results

Table C.1: Test results.

		Deep water			Hydra NL			
Test code	Waterlevel	Hm0	Tp	Tm-0,1	Hm0	Tp	Tm-0,1	q
1 _B_17a_1 _O2a	5.32	3.46	7.31	6.72	3.18	7.17	6.68	12.21
	5.32	3.46	7.31	6.72	3.21	7.17	6.68	12.13
	5.32	3.46	7.31	6.71	3.15	7.17	6.67	12.43
	5.32	3.66	6.90	6.73	3.35	7.17	6.69	14.86
2 _B_17a_2 _O2a	5.32	3.66	7.31	6.74	3.36	7.17	6.70	9.72
	5.32	3.68	7.61	6.73	3.34	7.31	6.71	10.03
	5.32	3.65	7.31	6.74	3.36	7.17	6.70	9.99
3 _B_17a_3 _O2a	5.32	3.62	7.31	6.74	3.35	7.17	6.71	3.38
	5.32	3.59	7.31	6.74	3.33	7.17	6.70	3.46
	5.32	3.59	7.31	6.74	3.33	7.17	6.70	3.59
4 _B_17a_3 _H1b	5.32	3.90	7.61	7.07	3.65	7.94	6.93	10.42
	5.32	3.90	7.61	7.06	3.65	7.94	6.92	10.42
	5.32	3.90	7.61	7.06	3.65	7.94	6.92	10.79
5 _B_17a_4 _O2a	5.32	3.62	7.31	6.75	3.36	7.17	6.70	1.12
	5.32	3.63	7.31	6.74	3.36	7.17	6.70	1.10
	5.32	3.63	7.31	6.74	3.37	7.17	6.70	1.05
6 _B_17a_4 _H1c	5.32	4.24	8.12	7.35	3.95	7.94	7.18	12.01
	5.32	4.25	8.12	7.35	3.97	7.94	7.18	11.93
	5.32	4.26	8.12	7.35	3.98	7.94	7.18	12.33
7 _B_17a_5 _O2a	5.32	3.64	7.31	6.75	3.36	7.17	6.70	0.30
	5.32	3.65	7.31	6.75	3.35	7.17	6.70	0.29
	5.32	3.65	7.31	6.74	3.35	7.17	6.70	0.29
8 _B_17a_5 _H1d	5.32	4.60	8.12	7.66	4.19	8.12	7.45	9.31
	5.32	4.59	8.12	7.66	4.19	8.12	7.45	9.37
	5.32	4.59	8.12	7.66	4.19	8.12	7.46	9.56
	5.32	4.74	8.12	7.67	4.31	8.12	7.45	11.23
	5.32	4.74	8.12	7.67	4.30	8.12	7.45	11.05
9 _B_17a_6 _O2a	5.32	3.65	7.31	6.74	3.36	7.17	6.69	0.13
	5.32	3.66	7.31	6.74	3.37	7.17	6.69	0.12
	5.32	3.67	7.31	6.74	3.37	7.17	6.70	0.08
	5.32	3.67	7.31	6.74	3.38	7.17	6.69	0.09
10 _B_17a_6 _H1e	5.32	5.25	8.12	7.71	4.72	8.12	7.49	7.67
	5.32	5.30	8.70	8.04	4.77	8.70	7.85	12.96
	5.32	5.29	8.70	7.96	4.77	8.70	7.72	10.83
	5.32	5.25	8.70	7.94	4.75	8.70	7.69	10.59
	5.32	4.87	8.70	7.92	4.48	8.70	7.66	10.24
	5.32	5.01	8.12	7.85	4.58	8.70	7.61	9.37
	5.32	5.02	8.12	7.85	4.58	8.70	7.61	9.59
	5.32	5.36	8.12	7.86	4.87	8.70	7.61	9.99
	5.32	5.36	8.12	7.85	4.85	8.70	7.61	10.01
11 _B_17a_1 _S01a	5.32	3.84	7.31	6.74	3.54	7.17	6.69	9.74
	5.32	3.83	7.31	6.73	3.54	7.17	6.69	9.53
12 _B_17a_1 _S01b	5.32	3.90	8.12	7.45	3.68	8.12	7.30	36.62
	5.32	3.90	8.12	7.45	3.68	8.12	7.30	36.35
13 _B_17a_1 _S01c	5.32	3.86	7.61	7.09	3.61	7.94	6.97	20.66
	5.32	3.87	7.61	7.09	3.62	7.94	6.97	21.02
14 _B_8b_1 _O1a	5.21	-	-	-	4.21	7.65	7.21	17.12
	5.21	-	-	-	4.28	7.65	7.21	17.12
	5.21	-	-	-	4.29	7.65	7.21	17.68
15 _B_8b_2 _O1a	5.21	-	-	-	4.18	7.65	7.21	9.87
	5.21	-	-	-	4.28	7.65	7.21	10.98
	5.21	-	-	-	4.28	7.65	7.22	10.42
16 _B_8b_3 _O1a	5.21	-	-	-	4.21	7.65	7.14	26.62
	5.21	-	-	-	4.16	7.65	7.21	25.69
	5.21	-	-	-	4.16	7.65	7.21	26.24

Table C.2: testresults.

	Deep water				Hydra NL			
Test code	Waterlevel	Hm0	Tp	Tm-0,1	Hm0	Tp	Tm-0,1	q
17 _B_8b_4 _O1a	5.21	-	-	-	4.19	7.65	7.19	13.40
	5.21	-	-	-	4.15	7.65	7.20	13.22
	5.21	-	-	-	4.17	7.65	7.18	13.77
18 _B_8b_5 _O1a	5.21	4.35	7.61	7.14	4.14	7.78	7.15	9.37
	5.21	4.36	7.61	7.13	4.14	7.78	7.15	8.72
	5.21	4.37	7.61	7.14	4.16	7.78	7.16	8.42
19 _B_8b_6 _O1a	5.21	4.34	7.61	7.13	4.11	7.78	7.15	6.80
	5.21	4.34	7.61	7.13	4.12	7.78	7.15	6.39
	5.21	4.36	7.61	7.13	4.14	7.78	7.15	6.58
20 _B_8b_7 _O1a	5.21	4.34	7.61	7.13	4.14	7.78	7.15	3.10
	5.21	4.34	7.61	7.13	4.14	7.78	7.15	3.19
21 _B_8b_8 _O1a	5.21	4.34	7.61	7.12	4.16	7.78	7.15	5.16
	5.21	4.35	7.61	7.14	4.19	7.78	7.14	4.52
	5.21	4.35	7.61	7.14	4.19	7.78	7.14	4.80
22 _B_8b_9 _O1a	5.21	4.35	7.61	7.14	4.17	7.78	7.17	9.02
	5.21	4.33	7.61	7.12	4.14	7.78	7.16	8.97
	5.21	4.34	7.61	7.13	4.17	7.78	7.16	9.13
23 _B_8b_11 _O1a	5.21	4.37	7.61	7.14	4.19	7.78	7.15	3.84
	5.21	4.40	7.61	7.13	4.19	7.78	7.15	3.07
	5.21	4.41	7.61	7.13	4.20	7.78	7.15	3.16
	5.21	4.45	7.61	7.14	4.24	7.78	7.15	3.06
24 _B_8b_12 _O1a	5.21	4.34	7.61	7.13	4.16	7.78	7.15	4.29
	5.21	4.36	7.61	7.13	4.19	7.78	7.15	4.04
	5.21	4.36	7.61	7.13	4.20	7.78	7.15	3.66
25 _B_8b_13 _O1a	5.21	4.32	7.78	7.29	4.36	7.78	7.19	7.43
	5.21	4.31	7.46	7.28	4.37	7.78	7.18	7.95
	5.21	4.32	7.46	7.28	4.37	7.78	7.19	7.57
26 _B_8b_14 _O1a	5.21	4.35	7.61	7.30	4.42	7.78	7.21	8.38
	5.21	4.35	7.46	7.28	4.43	7.78	7.19	8.37
	5.21	4.38	7.78	7.29	4.43	7.78	7.21	8.23
27 _B_8b_15 _O1a	5.21	4.55	7.61	7.19	4.18	7.78	7.17	7.07
	5.21	4.47	7.46	7.16	4.16	7.78	7.14	6.16
	5.21	4.47	7.46	7.16	4.12	7.78	7.15	6.06
28 _B_8b_16 _O1a	5.21	4.53	7.61	7.16	4.14	7.78	7.15	5.90
29 _S_8b_1 _O1a	5.21	4.27	7.81	7.22	4.01	7.81	7.38	7.70
30 _S_8b_1 _S2MHW	2.97	3.75	7.10	6.74	3.45	7.34	6.93	0.03
31 _S_8b_1 _S2MTW	5.07	4.22	7.54	7.19	3.97	7.54	7.35	4.80
32 _S_8b_1 _O1a	2.97	4.09	7.34	7.07	3.74	7.59	7.28	0.07
33 _S_8b_2 _O1a	5.07	4.63	7.98	7.51	4.34	8.13	7.66	12.20
34 _S_8b_3 _O1a	2.97	4.10	7.34	7.07	3.75	7.59	7.28	0.07
35 _S_8b_3 _S2MTW	5.21	4.28	7.54	7.23	4.02	7.54	7.39	7.60
36 _S_8b_3 _S2MHW	3	4.10	7.34	7.07	3.75	7.59	7.28	0.03
37 _S_8b_4 _S2MTW	5.07	4.62	7.95	7.51	4.32	7.95	7.65	12.90
38 _S_8b_4 _S2MHW	3	4.10	7.34	7.07	3.75	7.59	7.28	0.04
39 _S_8b_4 _S2MTW	5.21	4.29	7.54	7.22	4.02	7.81	7.38	9.70
40 _S_8b_4 _O1a	5.21	4.29	7.54	7.22	4.00	7.81	7.38	8.14
41 _S_8b_5 _S2MTW	5.07	4.22	7.84	7.17	3.99	7.84	7.35	5.10
42 _S_8b_5 _S2MHW	2.97	3.72	7.09	6.74	3.37	7.20	6.90	0.07
43 _S_8b_5 _S2MTW	5.21	4.33	7.67	7.23	4.03	7.67	7.40	8.09
44 _S_8b_5 _O1a	5.21	4.28	7.81	7.22	4.00	7.81	7.37	7.36
45 _D_17a_1 _S02	5.13	2.04	7.57	6.97	1.89	7.72	7.11	0.70
46 _D_17a_1 _O1a	5.47	3.46	7.24	6.71	3.20	7.39	6.85	25.00
47 _D_17a_1 _O1a	5.20	3.48	7.21	6.71	3.22	7.36	6.85	17.00
48 _D_17a_2 _O1a	5.32	3.42	7.21	6.72	3.17	7.36	6.86	4.00

D

Measured-conditions

D.1. BAM flume 17a

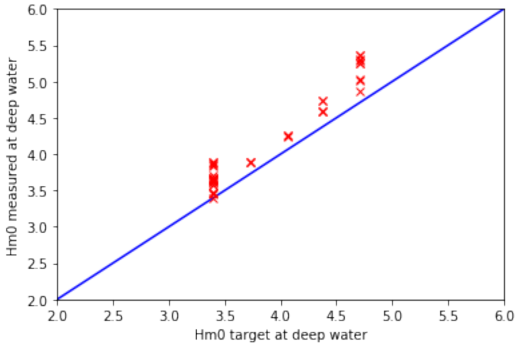


Figure D.1: Hm0 target plotted against the measured Hm0 at deep water.

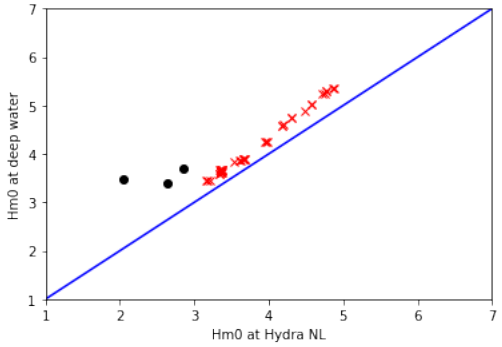


Figure D.2: Hm0 Hydra NL plotted against Hm0 at deep water.

D.2. BAM flume 8b

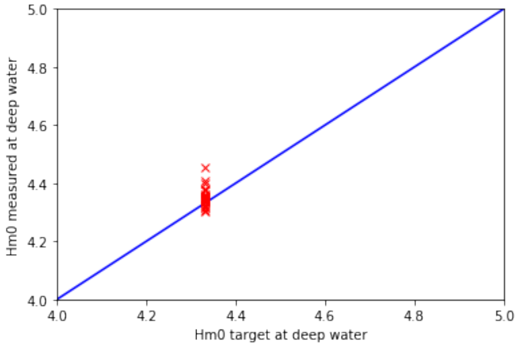


Figure D.3: Hm0 target plotted against the measurd Hm0 at deep water.

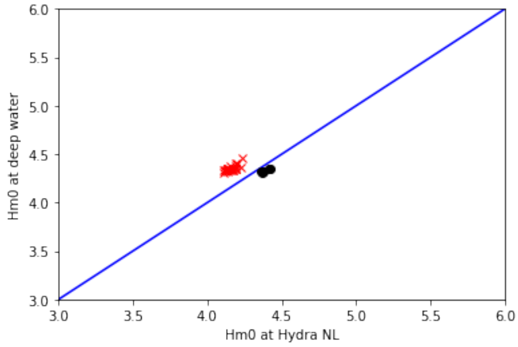


Figure D.4: Hm0 Hydra NL plotted against Hm0 at deep water.

D.3. Schelde flume 8b

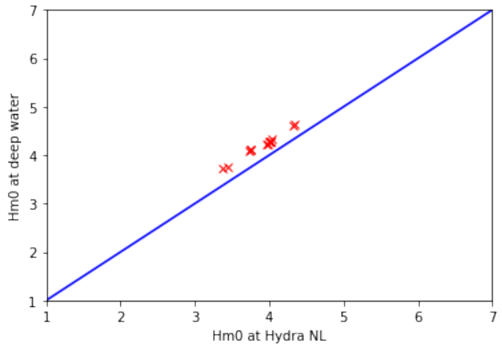


Figure D.5: Hm0 Hydra NL plotted against Hm0 at deep water.

E
Plotted test results

In this appendix, the graphs that are used in the document are given. In the document the results of the graphs are discussed.

E.1. 17a BAM flume

E.1.1. Influence berm

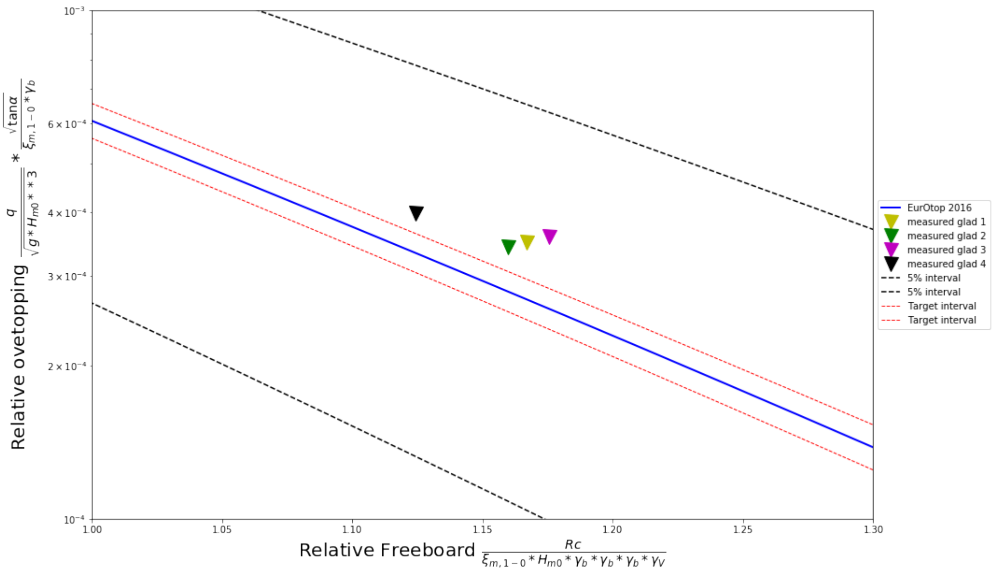


Figure E.1: Measured data plotted with normal berm coefficient.

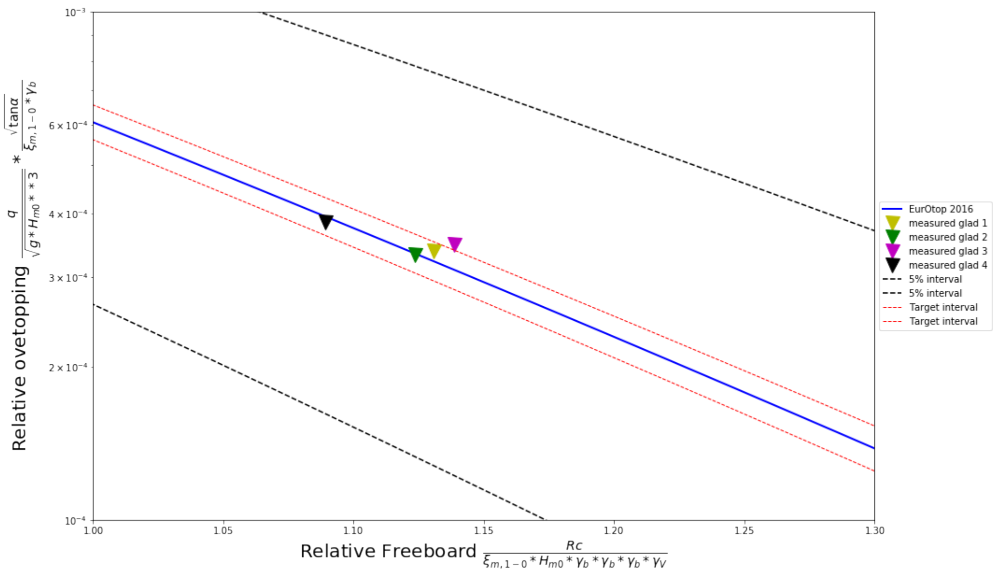


Figure E.2: Measured data plotted with adjusted berm coefficient.

E.1.2. Influence mean overtopping

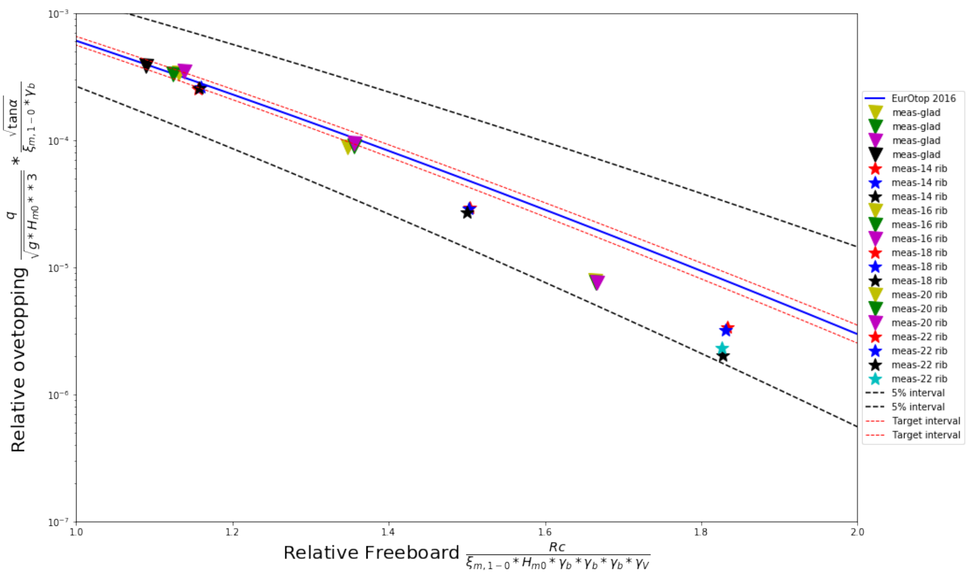


Figure E.3: Measured data plotted with different overtopping discharges and calculated roughness.

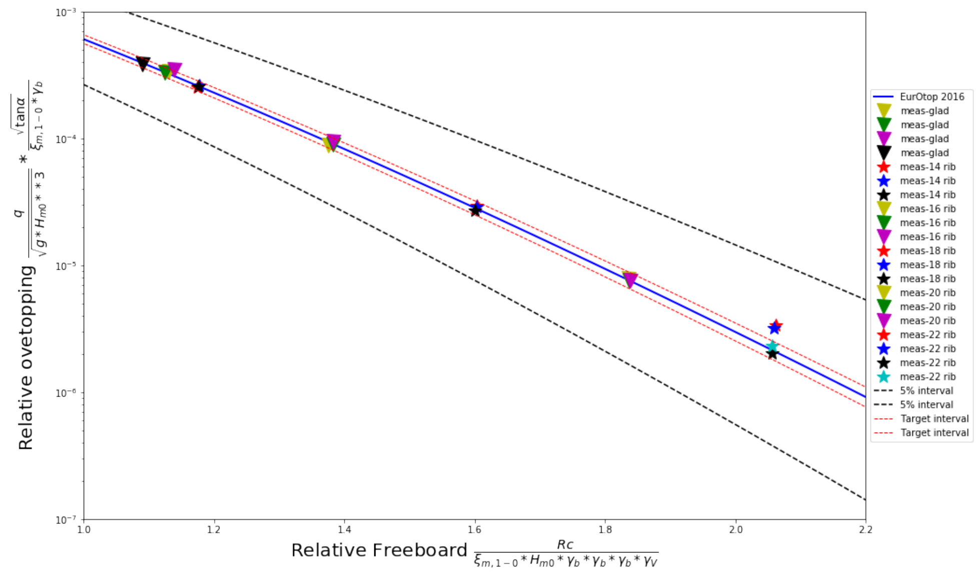


Figure E.4: Measured data plotted with different overtopping discharges and fitted roughness.

E.1.3. Influence wave steepness

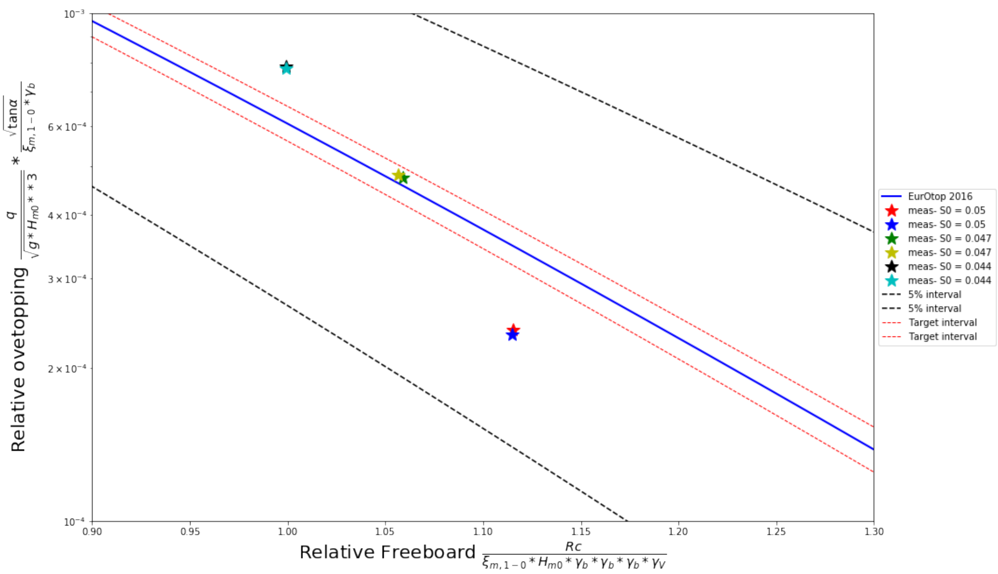


Figure E.5: Measured data plotted with different wave steepness and calculated roughness

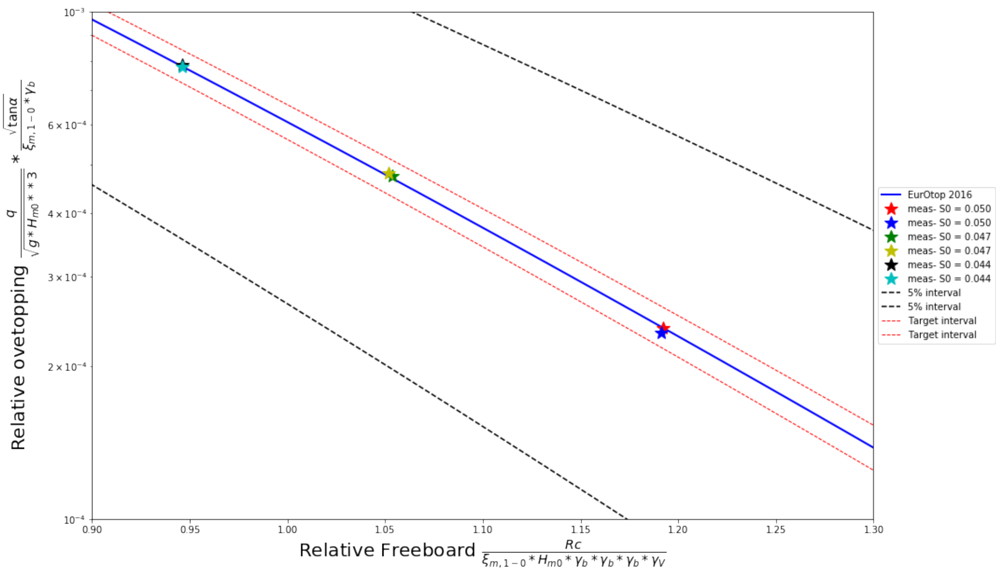


Figure E.6: Measured data plotted with different wave steepness and fitted roughness

E.1.4. Influence wave height

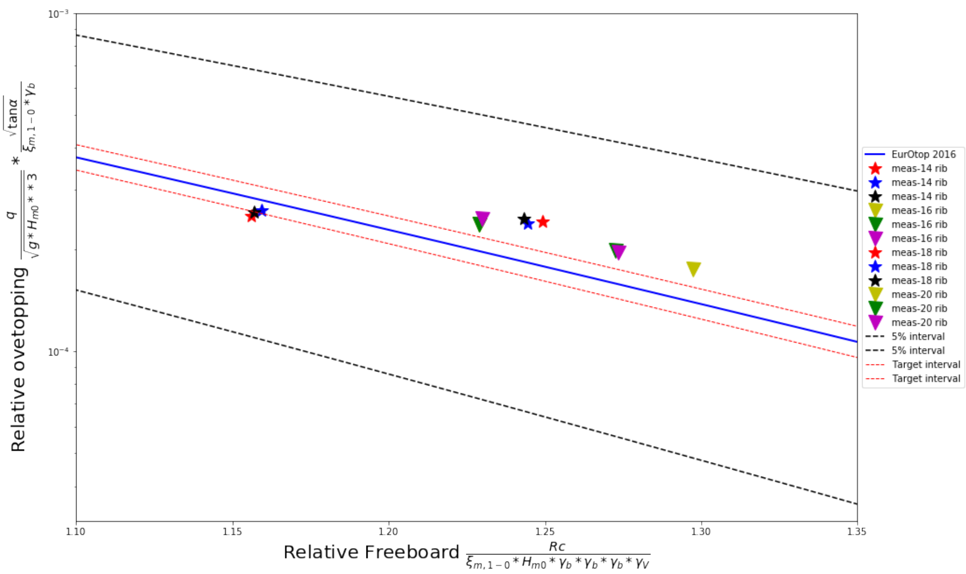


Figure E.7: Measured data plotted with different wave height and calculated roughness

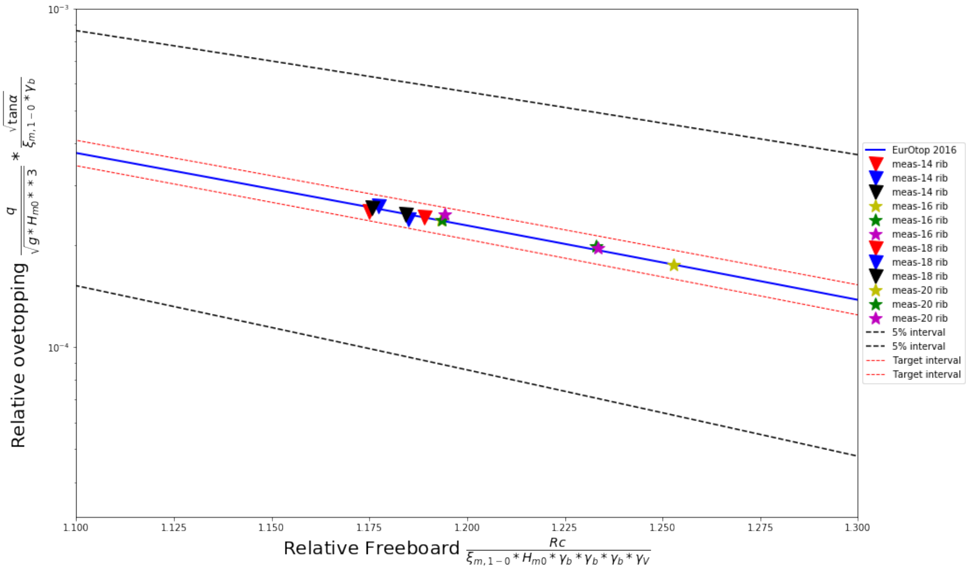


Figure E.8: Measured data plotted with different wave height and fitted roughness

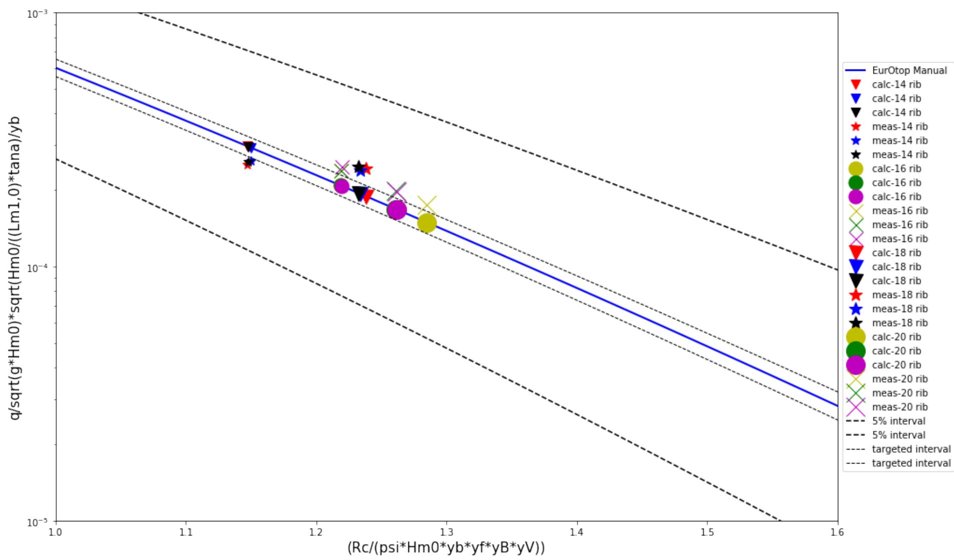


Figure E.9: Measured data plotted with different wave height and calculated roughness, with a different area of influence

E.2. 8b BAM flume

E.2.1. Influence lower slope

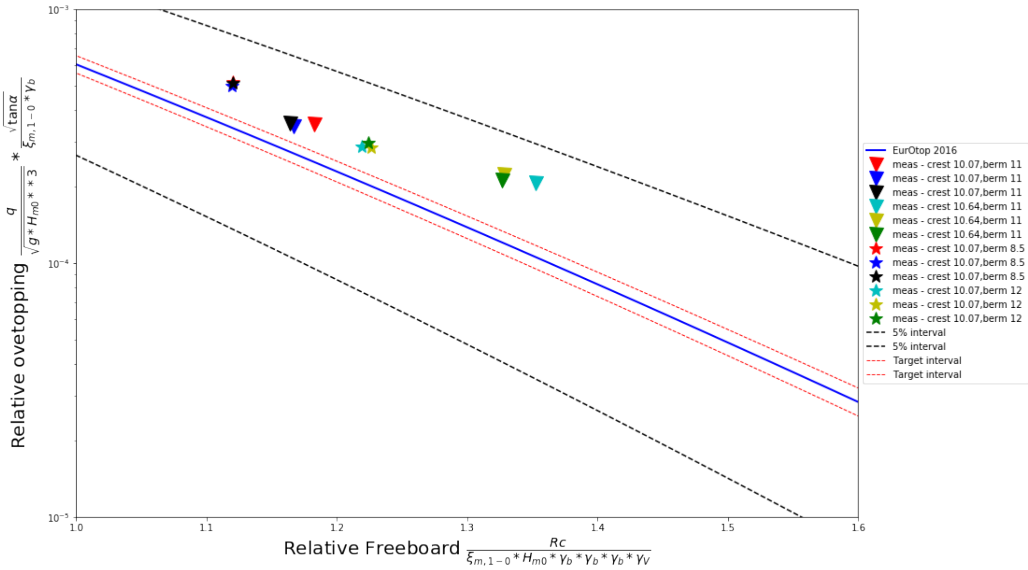


Figure E.10: Measured data plotted with a lower slope of Levvelblocs and calculated roughness

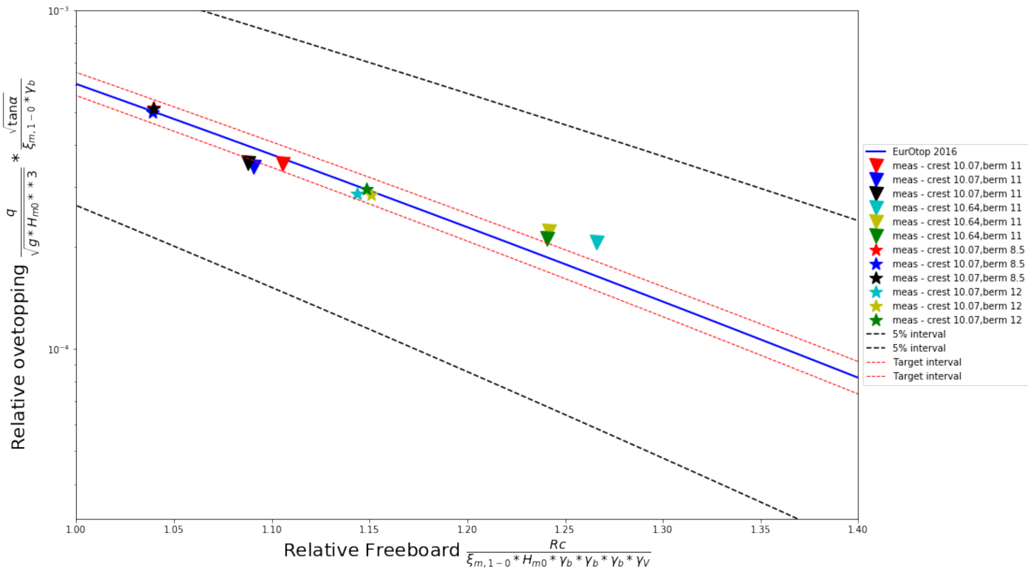


Figure E.11: Measured data plotted with a lower slope of Levvelblocs and fitted roughness

E.2.2. Influence rib height

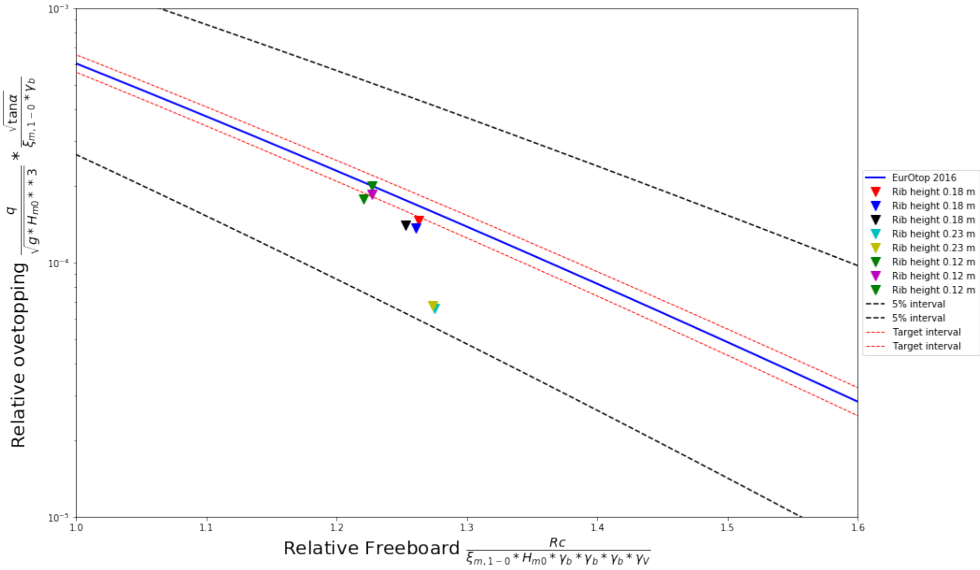


Figure E.12: Measured data plotted with different rib heights and calculated roughness

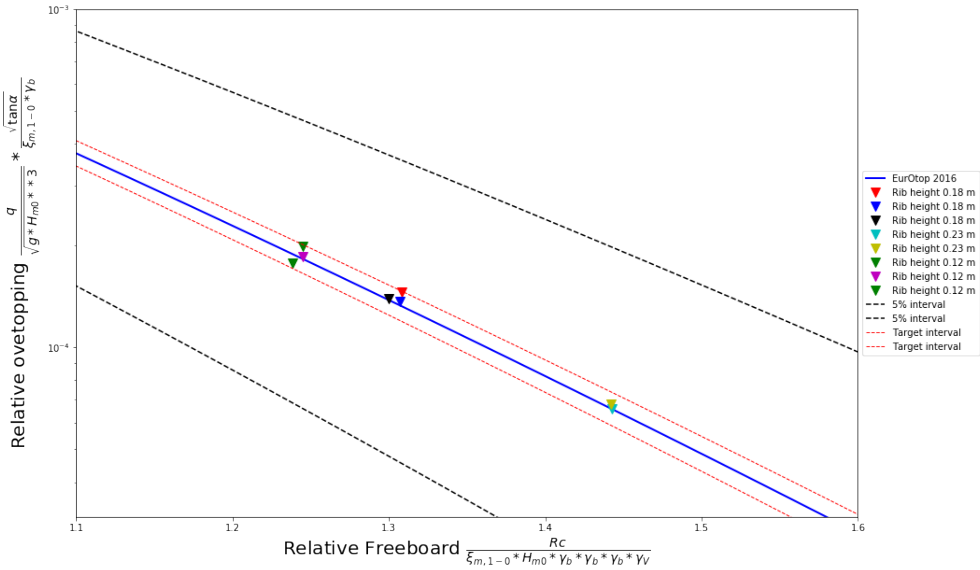


Figure E.13: Measured data plotted with different rib heights and fitted roughness

E.2.3. Influence rib pattern

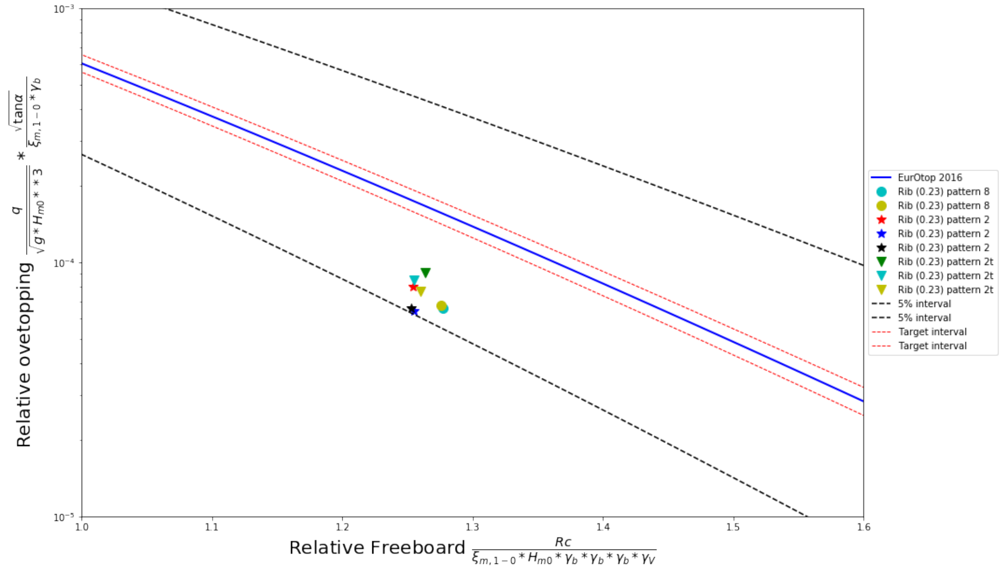


Figure E.14: Measured data plotted with different rib patterns and calculated roughness

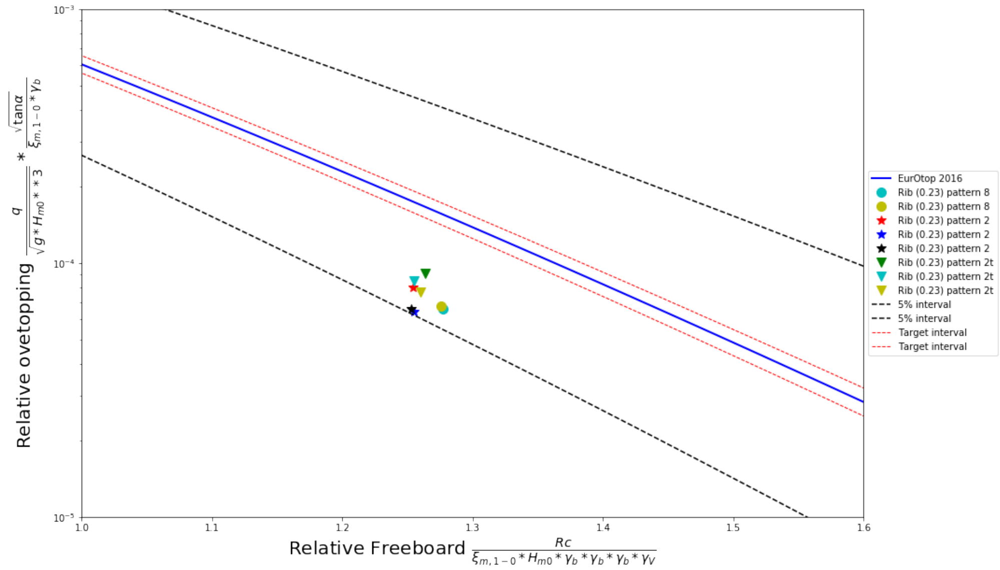


Figure E.15: Measured data plotted with different rib patterns and fitted roughness

E.2.4. Influence amount of ribs

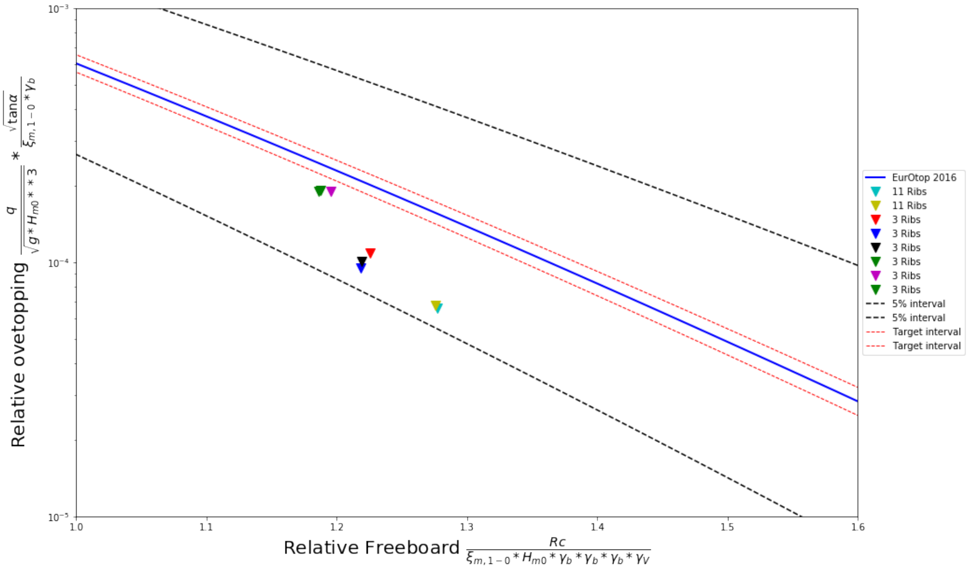


Figure E.16: Measured data plotted with different amount of ribs and calculated roughness

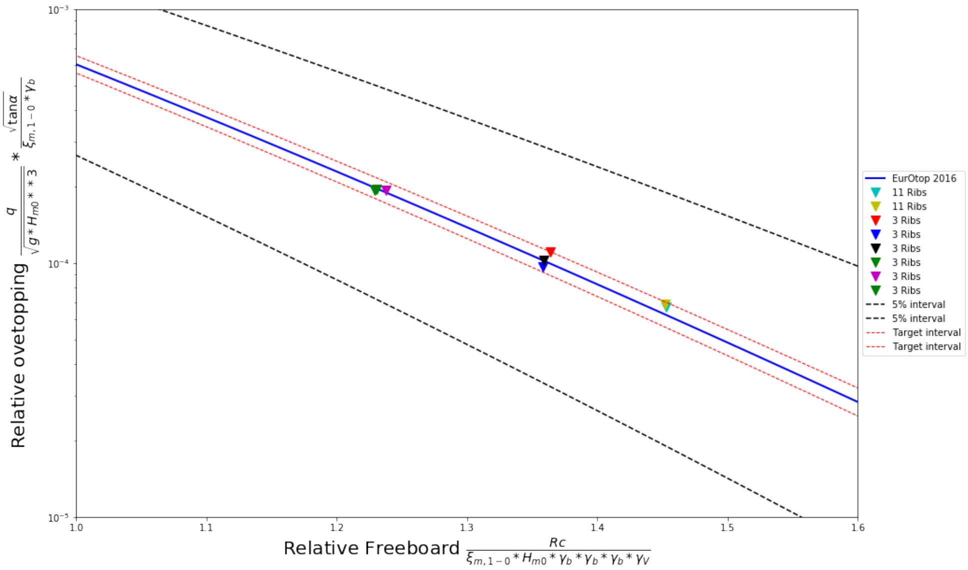


Figure E.17: Measured data plotted with different amount of ribs and fitted roughness

E.2.5. Influence of slope

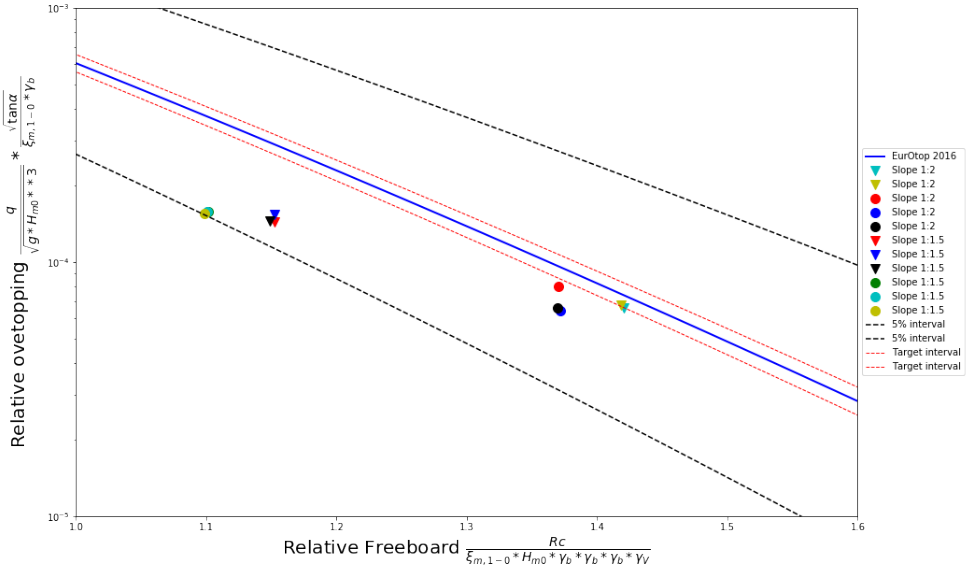


Figure E.18: Measured data plotted with different lower slopes and calculated roughness

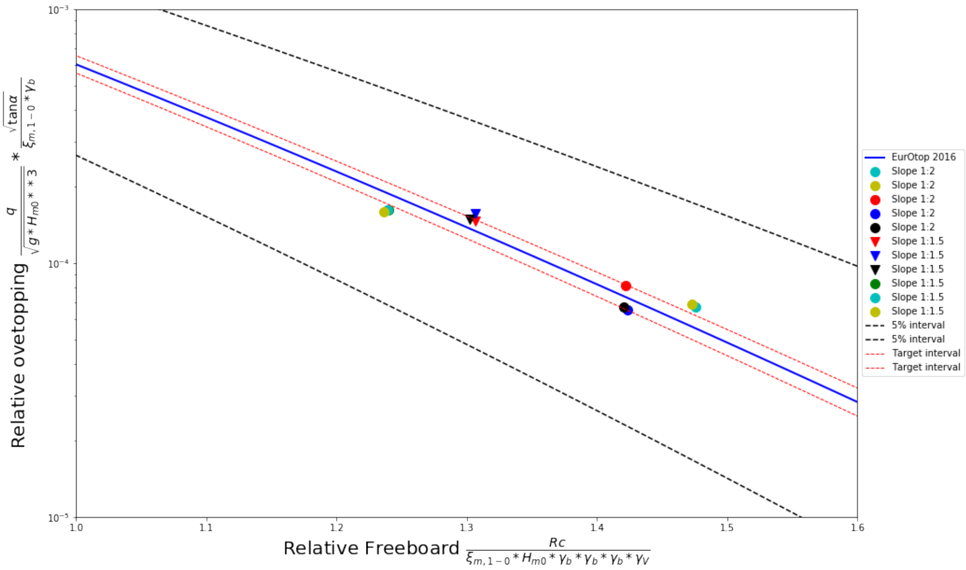


Figure E.19: Measured data plotted with different lower slopes and fitted roughness

E.2.6. Verification

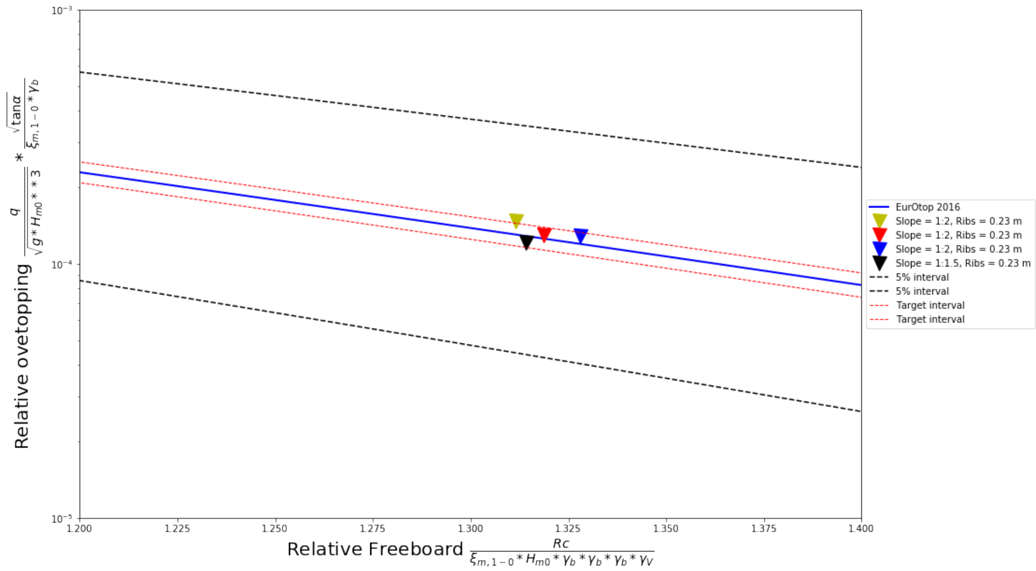


Figure E.20: Measured data plotted in verification test with fitted roughness

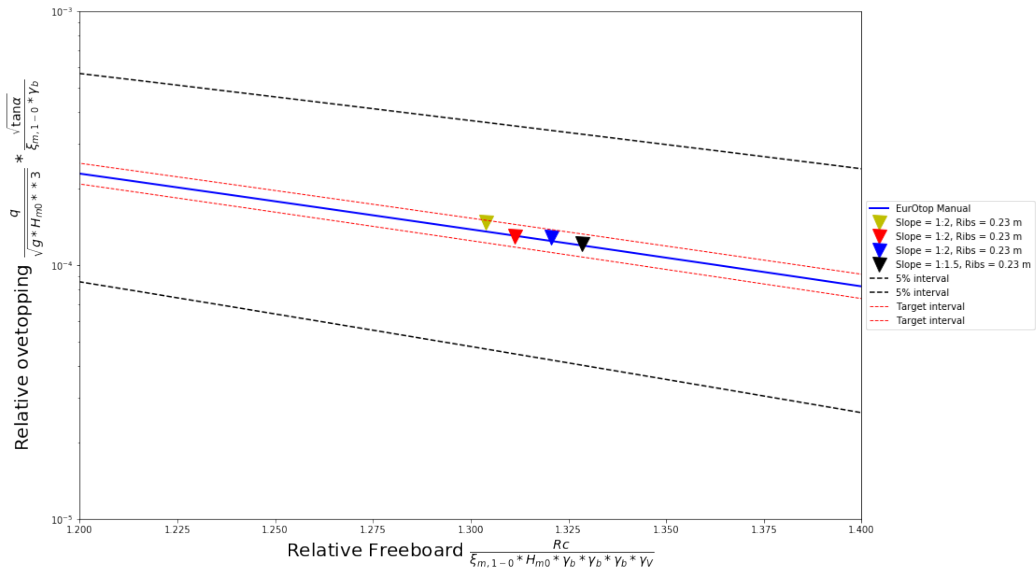


Figure E.21: Measured data plotted in verification test with calculated roughness

E.3. All the tests

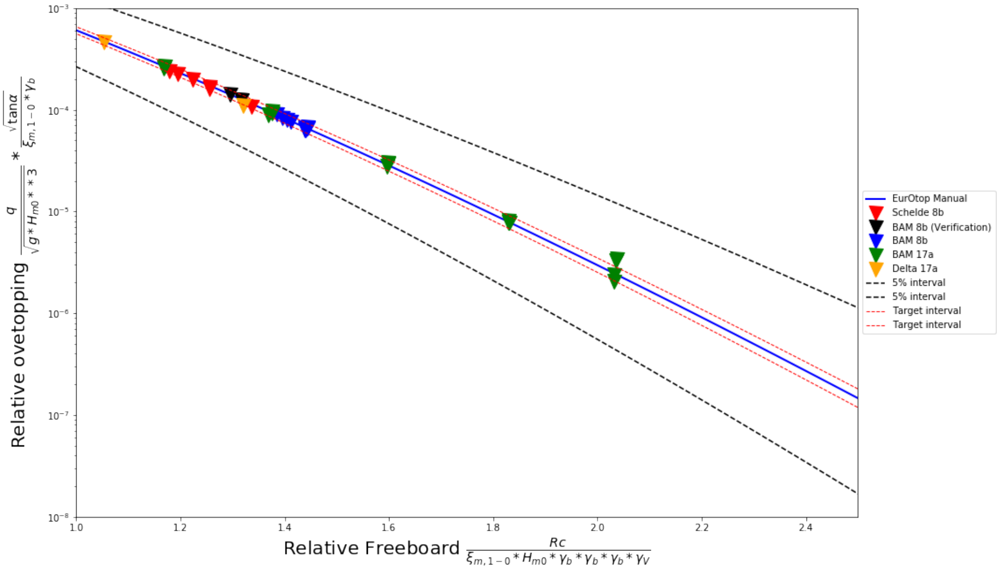


Figure E.22: Test results of 8b and 17a with fitted roughness upper slope.

F

Pythonscript

In [2]:
```
##This is eurotop equation
g = 9.81
yB = 1.0
yV = 1.0
yfBerm1 = 1.0
yfBerm2 = 1.0
N = 10
bteen = 0.5

def EurOtop(SWL,Hm0,Tm10,Kn,Bh,yfLEv,yfQuat1,B,cotAlow,cotAtop,py,qmeas):
    Rc = Kn - SWL
    #print ('Rc =', Rc)

    #print ('Hm0 =', Hm0)

    #Tm10 = Tp / 1.06
    #print ('Tm10 =', Tm10)

    L0 = g * Tm10**2 / (2*np.pi)
    #print (L0)

    s0 = Hm0 / L0
    #print (s0)

    if SWL<Bh:
        Lberm = Hm0 * cotAlow +  (Bh - SWL ) * cotAlow + B + (min((SWL + Hm0), Kn) - Bh) * cotAtop
    if SWL==Bh:
        Lberm = Hm0 * cotAlow + B + (min((SWL + Hm0), Kn) - Bh) * cotAtop
    else :
        Lberm = (Hm0 - (SWL - Bh)) * cotAlow +  B + (min((SWL + Hm0), Kn) - Bh) * cotAtop


    db = Bh-SWL
    rB = B / Lberm

    R2prec = np.zeros(N+1)
    R2prec [0] = 1.5 * Hm0
    #print (len(R2prec))
    #print  ('R2prec =', R2prec)

    Lslope = np.zeros(N)
    tana = np.zeros(N)
    cota = np.zeros(N)

    psi = np.zeros(N)
    rdb = np.zeros(N)

    yb = np.zeros(N)
    yf = np.zeros(N)
    Lbot = np.zeros(N)
    Ltop = np.zeros(N)
    yfQuat = np.zeros(N+1)
    #Hoi = np.zeros(N)
    yfQuat[0] = yfQuat1
    q = np.zeros(N)
    #print(yf)

    R2precg = np.zeros(N)

    for i in range(N):
        Lslope[i] = (min((1.5 * Hm0),(SWL-bteen)) - (SWL - Bh)) * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
      # if SWL<Bh:
       #    Lslope[i]= 1.5 * Hm0 * cotAlow +  (Bh - SWL ) * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
       #if SWL==Bh:
        #   Lslope[i] = 1.5 * Hm0 * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
       #else:
           #Lslope[i] = (1.5 * Hm0 - (SWL - Bh)) * cotAlow + B + min((Kn-SWL), R2prec[i]) * cotAtop
        #   Lslope[i] = (min((1.5 * Hm0),(SWL-bteen)) - (SWL - Bh)) * cotAlow + B + min((Kn-SWL), R2prec[i]) * cotAtop


        tana[i] = ( min((1.5 * Hm0),(SWL-bteen)) + min(R2prec[i], Kn-SWL) ) / (Lslope[i] - B )
        cota[i] = 1 / tana[i]
        psi[i] = tana[i] / np.sqrt(s0)

        if SWL<Bh:
            rdb[i] = 0.5 - 0.5 * np.cos(np.pi * (Bh - SWL) / R2prec[i])
        if SWL == Bh:
            rdb[i] = 0
        else :
            rdb[i] = 0.5 - 0.5 * np.cos(np.pi * (Bh - SWL) / R2prec[i])

        yb[i] = max(0.6, 1.0 - rB * (1.0 - rdb[i]))

        R2precg[i]= min((1.65 * yb[i] * 1.0 * yB * yV * psi[i] * Hm0), 1.0 * 1.0 * yB * (4 - (1.5/np.sqrt(yb[i] * psi[i])))* Hm0)

        Lbot[i] = np.sqrt ((Bh-(SWL-0.25*R2precg[i]))**2 + (cotAlow*(Bh-(SWL-0.25*R2precg[i])))**2)
        Ltop[i] = np.sqrt ((Bh-(SWL+0.5*R2precg[i]))**2 + (cotAtop*(Bh-(SWL+0.5*R2precg[i])))**2)

        #Lbot[i] = np.sqrt ((0.25*R2precg[i])**2 + (cotAlow*(0.25*R2precg[i]))**2)
        #Ltop[i] = np.sqrt ((0.5*R2precg[i])**2 + (cotAtop*(0.5*R2precg[i]))**2)

        yf[i] = (Lbot[i] * yfLEv + (B-2.5) * yfBerm1 + 2.5 * yfBerm2 + Ltop[i] * yfQuat[i]) / (Lbot[i] + B+ Ltop[i])

        R2prec[i+1]= min((1.65 * yb[i] * yf[i] * yB * yV * psi[i] * Hm0), 1 * yf[i] * yB * (4 - (1.5/np.sqrt(yb[i] * psi[i])))* Hm0)

        q[i] = 0.023/np.sqrt(tana[i])*yb[i]*psi[i]*np.exp(-(2.7*(Rc/(psi[i]*Hm0*yb[i]*yf[i]*yB*yV)))**1.3)*np.sqrt(g*Hm0**3)*1000

        yfQuat[i+1]= 1 - (0.585*np.sqrt(0.075-s0)*np.sqrt(py)*(-np.log(((q[i]/1000)/(np.sqrt(g*Hm0**3))))))
        #yfQuat[i+1]= 1 - (0.585*np.sqrt(0.075-s0)*np.sqrt(py)*(-np.log(((q[i]*(0.8))**(2.6)/1000)/(np.sqrt(g*Hm0**3)))))**(0.78)

        #Lbot[i] = np.sqrt ((SWL-0.25*R2precg[i])**2 + (cotAlow*(SWL-0.25*R2precg[i]))**2)
        #Ltop[i] = np.sqrt ((SWL+0.5*R2precg[i])**2 + (cotAtop*(SWL+0.5*R2precg[i]))**2)

        #yf[i] = (Lbot[i] * yfLEv + (B-2.5) * yfBerm1 + 2.5 * yfBerm2 + Ltop[i] * yfQuat) / (Lbot[i] + B+ Ltop[i])

    #print ('L0= ', L0)
    #print ('s0= ', s0)
    #print ('Lberm= ', Lberm)
    #print ('Lslope= ', Lslope)
```

```
    qmean = (qmeas/1000)/(np.sqrt(g*Hm0**3))
    #print (q)
    y = q[N-1]/1000/np.sqrt(g*Hm0**3)*np.sqrt(Hm0/(L0*tana[N-1]))/yb[N-1]
    y1= qmeas/1000/np.sqrt(g*Hm0**3)*np.sqrt(Hm0/(L0*tana[N-1]))/yb[N-1]
    x = Rc/(psi[N-1]*Hm0*yb[N-1]*yf[N-1]*yB*yV)

    #print  ('qmeas =',qmeas)
    #print  ('q =',q)
    verschil = np.abs(qmeas-q[N-1])
    #print  ('vershil=', np.abs(qmeas-q))
    return q[N-1],y,y1,x,verschil,qmean,Rc,cotAtop,yfQuat[N-1],s0,Hm0,psi[N-1],qmeas
```

In [3]:

```
#ASD wave overtopping - SMO III.xlsx

book1 = xlrd.open_workbook("ScheldegootresultatenDV8b.xlsx")
data1 = book1.sheet_by_index(0)

#for j in range(23):
 #  for i in range(9,27):
      #print (data.cell_value(i,j))

#print (data[23,0])
#df1 = df.replace(np.nan, '', regex=True)
data1.movies_skip_rows = pd.read_excel("ScheldegootresultatenDV8b.xlsx", sheet_name='Scheldegoot-resultaten',skiprows=(1),usecols=(2,4,17,18,19,21,))
data1.proeven = data1.movies_skip_rows.replace(np.nan, '', regex=True)


data1.proeven
#print (df[0,0])

#df_subset = df[[SWLHm0,]]
```

Out[3]:

	test	SWL	Hm0	Tp	Tm10	q
0						
1	T2		2.74	6.41	6.38	0
2	O1	5.21	4	7.81	7.38	8.14
3	S2	5.07	3.99	7.84	7.35	5.1
4	S2	2.97	3.37	7.2	6.9	0.07
5	O1	5.21	4.03	7.67	7.4	8.09
6	O1	5.21	4	7.81	7.37	7.36

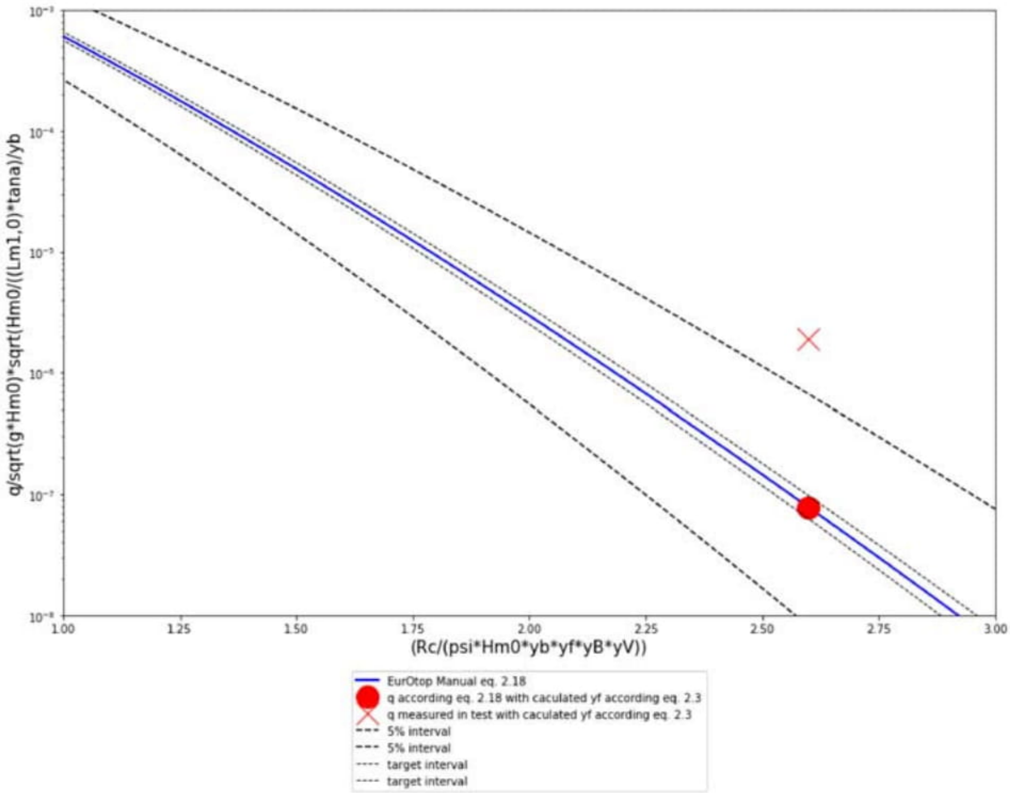
In [4]:

```
kn  = 9.75
berm = 12.0
yflevelbloc15 = 0.39

s = (1,13)
q_10c_1 = np.zeros(s)
yfonder = yflevelbloc15
yfboven = 0.9
for i in range (1):
    q_10c_1[i] = EurOtop(data1.proeven.SWL[i+4],data1.proeven.Hm0[i+4],data1.proeven.Tm10[i+4],kn,5.2,yfonder,yfboven,berm,1.5,4.0,
                     0.5*15*0.23*np.sin(1/4.0)/(kn-data1.proeven.SWL[i+4]),data1.proeven.q[i+4])
print (q_10c_1)
print ('..............................')
```

```
[[2.90507250e-03 7.81301288e-08 1.88260672e-06 2.59943898e+00
  6.70949275e-02 3.61259008e-06 6.78000000e+00 4.00000000e+00
  6.02809398e-01 4.53359354e-02 3.37000000e+00 1.31107451e+00
  7.00000000e-02]]
..............................
```

In [ ]:

In [5]:
```
x = np.zeros(100)
y = np.zeros(100)
p = np.zeros(100)
q = np.zeros(100)
b = np.zeros(100)
d = np.zeros(100)
#print (len(y))

x = np.linspace(0,3,100)
#print (len(x))
#print (x)
for i in range(len(x)):
    y[i] = 0.023*np.exp(-((2.7*x[i])**(1.3)))
    p[i] = (0.023+1.64*0.003)*np.exp(-(((2.7-1.64*0.2)*x[i])**(1.3)))
    b[i] = (0.023+1.64*0.003*0.1)*np.exp(-(((2.7-1.64*0.2*0.1)*x[i])**(1.3)))
    q[i] = (0.023-1.64*0.003)*np.exp(-(((2.7+1.64*0.2)*x[i])**(1.3)))
    d[i] = (0.023-1.64*0.003*0.1)*np.exp(-(((2.7+1.64*0.2*0.1)*x[i])**(1.3)))

#print (y)
#print (p)

plt.figure(figsize=(15,10))
plt.semilogy(x,y,'b',linewidth = 2, label= 'EurOtop Manual eq. 2.18')

plt.semilogy(q_10c_1[0][3],q_10c_1[0][1],'ro', markersize=20, label = 'q according eq. 2.18 with caculated yf according eq. 2.3')
plt.semilogy(q_10c_1[0][3],q_10c_1[0][2],'rx', markersize=20, label = 'q measured in test with caculated yf according eq. 2.3')

plt.semilogy(x,p,'k--', linewidth=1.5 ,label= '5% interval')
plt.semilogy(x,q,'k--', linewidth=1.5 ,label= '5% interval')
plt.semilogy(x,b,'k--', linewidth=1,label= 'target interval')
plt.semilogy(x,d,'k--', linewidth=1,label= 'target interval')

#plt.title('EurOtop formula')
plt.ylabel('q/sqrt(g*Hm0)*sqrt(Hm0/((Lm1,0)*tana)/yb', size=15)
plt.ylim(10**(-8),10**(-3))
plt.xlim(1,3)
plt.xlabel('(Rc/(psi*Hm0*yb*yf*yB*yV))', size=15)
plt.legend(loc='lower center', bbox_to_anchor=(0.5, -0.3));
```



In [6]:

```python
##This is eurotop equation to fit
g = 9.81
yB = 1.0
yV = 1.0
yfBerm1 = 1.0
yfBerm2 = 1.0
N = 10
bteen = 0.5

def EurOtopFit(SWL,Hm0,Tm10,Kn,Bh,yfLEv,yfQuat,B,cotAlow,cotAtop,py,qmeas):
    Rc = Kn - SWL
    #print ('Rc =', Rc)

    #print ('Hm0 =', Hm0)

    #Tm10 = Tp / 1.06
    #print ('Tm10 =', Tm10)

    L0 = g * Tm10**2 / (2*np.pi)
    #print (L0)

    s0 = Hm0 / L0
    #print (s0)

    if SWL<Bh:
        Lberm = Hm0 * cotAlow +  (Bh - SWL ) * cotAlow + B + (min((SWL + Hm0), Kn) - Bh) * cotAtop
    if SWL==Bh:
        Lberm = Hm0 * cotAlow + B + (min((SWL + Hm0), Kn) - Bh) * cotAtop
    else :
        Lberm = (Hm0 - (SWL - Bh)) * cotAlow +  B + (min((SWL + Hm0), Kn) - Bh) * cotAtop


    db = Bh-SWL
    rB = B / Lberm

    R2prec = np.zeros(N+1)
    R2prec [0] = 1.5 * Hm0
    #print (Len(R2prec))
    #print  ('R2prec =', R2prec)

    Lslope = np.zeros(N)
    tana = np.zeros(N)
    cota = np.zeros(N)

    psi = np.zeros(N)
    rdb = np.zeros(N)

    yb = np.zeros(N)
    yf = np.zeros(N)
    Lbot = np.zeros(N)
    Ltop = np.zeros(N)
    #yfQuat = np.zeros(N+1)
    #Hoi = np.zeros(N)
    #yfQuat[0] = yfQuat1
    q = np.zeros(N)
    #print(yf)

    R2precg = np.zeros(N)

    for i in range(N):
        Lslope[i] = (min((1.5 * Hm0),(SWL-bteen)) - (SWL - Bh)) * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
       # if SWL<Bh:
        #    Lslope[i]= 1.5 * Hm0 * cotAlow +  (Bh - SWL ) * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
        #if SWL==Bh:
         #   Lslope[i] = 1.5 * Hm0 * cotAlow + B + min((Kn-Bh), R2prec[i]) * cotAtop
        #else:
            #Lslope[i] = (1.5 * Hm0 - (SWL - Bh)) * cotAlow + B + min((Kn-SWL), R2prec[i]) * cotAtop
         #   Lslope[i] = (min((1.5 * Hm0),(SWL-bteen)) - (SWL - Bh)) * cotAlow + B + min((Kn-SWL), R2prec[i]) * cotAtop


        tana[i] = ( min((1.5 * Hm0),(SWL-bteen)) + min(R2prec[i], Kn-SWL) ) / (Lslope[i] - B )
        cota[i] = 1 / tana[i]
        psi[i] = tana[i] / np.sqrt(s0)

        if SWL<Bh:
            rdb[i] = 0.5 - 0.5 * np.cos(np.pi * (Bh - SWL) / R2prec[i])
        if SWL == Bh:
            rdb[i] = 0
        else :
            rdb[i] = 0.5 - 0.5 * np.cos(np.pi * (Bh - SWL) / R2prec[i])

        yb[i] = max(0.6, 1.0 - rB * (1.0 - rdb[i]))

        #yb[i] = 0.868

        R2precg[i]= min((1.65 * yb[i] * 1.0 * yB * yV * psi[i] * Hm0), 1.0 * 1.0 * yB * (4 - (1.5/np.sqrt(yb[i] * psi[i])))* Hm0)

        Lbot[i] = np.sqrt ((Bh-(SWL-0.25*R2precg[i]))**2 + (cotAlow*(Bh-(SWL-0.25*R2precg[i])))**2)
        Ltop[i] = np.sqrt ((Bh-(SWL+0.5*R2precg[i]))**2 + (cotAtop*(Bh-(SWL+0.5*R2precg[i])))**2)

        #Lbot[i] = np.sqrt ((0.25*R2precg[i])**2 + (cotAlow*(0.25*R2precg[i]))**2)
        #Ltop[i] = np.sqrt ((0.5*R2precg[i])**2 + (cotAtop*(0.5*R2precg[i]))**2)

        yf[i] = (Lbot[i] * yfLEv + (B-2.5) * yfBerm1 + 2.5 * yfBerm2 + Ltop[i] * yfQuat) / (Lbot[i] + B+ Ltop[i])

        R2prec[i+1]= min((1.65 * yb[i] * yf[i] * yB * yV * psi[i] * Hm0), 1 * yf[i] * yB * (4 - (1.5/np.sqrt(yb[i] * psi[i])))* Hm0)

        q[i] = 0.023/np.sqrt(tana[i])*yb[i]*psi[i]*np.exp(-(2.7*(Rc/(psi[i]*Hm0*yb[i]*yf[i]*yB*yV)))**1.3)*np.sqrt(g*Hm0**3)*1000

        #yfQuat[i+1] = 1.0 + (0.585*np.sqrt(0.075-s0)*np.sqrt(py)*(np.log((q[i]/1000)/(np.sqrt(g*Hm0**3)))))

        #Lbot[i] = np.sqrt ((SWL-0.25*R2precg[i])**2 + (cotAlow*(SWL-0.25*R2precg[i]))**2)
        #Ltop[i] = np.sqrt ((SWL+0.5*R2precg[i])**2 + (cotAtop*(SWL+0.5*R2precg[i]))**2)

        #yf[i] = (Lbot[i] * yfLEv + (B-2.5) * yfBerm1 + 2.5 * yfBerm2 + Ltop[i] * yfQuat) / (Lbot[i] + B+ Ltop[i])

    #print ('L0= ', L0)
    #print ('s0= ', s0)
    #print ('Lberm= ', Lberm)
```

```
    #qmean = 0.023/np.sqrt(tana[N-1])*yb[N-1]*psi[N-1]*np.exp(-(2.7*(Rc/(psi[N-1]*Hm0*yb[N-1]*yf[N-1]*yB*yV)))**1.3)*1000
    qmean = (qmeas/1000)/(np.sqrt(g*Hm0**3))
    #print (q)
    y = q[N-1]/1000/np.sqrt(g*Hm0**3)*np.sqrt(Hm0/(L0*tana[N-1]))/yb[N-1]
    y1= qmeas/1000/np.sqrt(g*Hm0**3)*np.sqrt(Hm0/(L0*tana[N-1]))/yb[N-1]
    x = Rc/(psi[N-1]*Hm0*yb[N-1]*yf[N-1]*yB*yV)

    #print  ('qmeas =',qmeas)
    #print  ('q =',q)
    verschil = np.abs(qmeas-q[N-1])
    #print  ('vershil=', np.abs(qmeas-q))
    return q[N-1],y,y1,x,verschil,qmean,Rc,cotAtop,yfQuat,s0,Hm0,psi[N-1],qmeas
```

In [7]:

```
kn   = 9.75
berm = 12.0
yflevelbloc = 0.6

s = (1,13)
q_10c_1_meas = np.zeros(s)
yfonder = yflevelbloc
yfboven = 1.69
for i in range (1):
    q_10c_1_meas[i] = EurOtopFit(data1.proeven.SWL[i+4],data1.proeven.Hm0[i+4],data1.proeven.Tm10[i+4],kn,5.2,yfonder,yfboven,berm,1.5,4.0,
                     0.5*15*0.23*np.sin(1/4.0)/(kn-data1.proeven.SWL[i+4]),data1.proeven.q[i+4])
print ('totaal verschil',np.mean(q_10c_1_meas[:,4]))
```

```
totaal verschil 4.437323545110339e-05
```

In [ ]:

In [8]:

```
x = np.zeros(100)
y = np.zeros(100)
p = np.zeros(100)
q = np.zeros(100)
b = np.zeros(100)
d = np.zeros(100)
#print (len(y))

x = np.linspace(0,3,100)
#print (len(x))
#print (x)
for i in range(len(x)):
    y[i] = 0.023*np.exp(-((2.7*x[i])**(1.3)))
    p[i] = (0.023+1.64*0.003)*np.exp(-(((2.7-1.64*0.2)*x[i])**(1.3)))
    b[i] = (0.023+1.64*0.003*0.1)*np.exp(-(((2.7-1.64*0.2*0.1)*x[i])**(1.3)))
    q[i] = (0.023-1.64*0.003)*np.exp(-(((2.7+1.64*0.2)*x[i])**(1.3)))
    d[i] = (0.023-1.64*0.003*0.1)*np.exp(-(((2.7+1.64*0.2*0.1)*x[i])**(1.3)))

#print (y)
#print (p)

plt.figure(figsize=(15,10))
plt.semilogy(x,y,'b',linewidth = 2, label= 'EurOtop Manual')

plt.semilogy(q_10c_1_meas[0][3],q_10c_1_meas[0][1],'ro', markersize=20, label = 'q according eq. 2.18 with fitted yf according eq. 2.3')
plt.semilogy(q_10c_1_meas[0][3],q_10c_1_meas[0][2],'rx', markersize=20, label = 'q measured in flume with fitted yf according eq. 2.3')

plt.semilogy(x,p,'k--', linewidth=1.5 ,label= '5% interval')
plt.semilogy(x,q,'k--', linewidth=1.5 ,label= '5% interval')
plt.semilogy(x,b,'k--', linewidth=1,label= 'target interval')
plt.semilogy(x,d,'k--', linewidth=1,label= 'target interval')

#plt.title('EurOtop formula')
plt.ylabel('q/sqrt(g*Hm0)*sqrt(Hm0/((Lm1,0)*tana)/yb', size=15)
plt.ylim(10**(-8),10**(-3))
plt.xlim(1,3)
plt.xlabel('(Rc/(psi*Hm0*yb*yf*yB*yV))', size=15)
plt.legend(loc='lower center', bbox_to_anchor=(0.5, -0.3));
```