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Applicability and Limitations of the SWASH model to predict Wave Overtopping

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The main aim of this minor thesis is to evaluate if we can accurately predict the mean wave overtopping discharge in rubble mound breakwaters with the tools we currently have. In first place, the available methods (Eurotop formulas and improvements proposed by some authors, Lykke Andersen and Burcharth formula and Neural Networks), are critically assessed focussing on how the berm influence is described. In second place, a new numerical tool called SWASH will be tested.

SWASH has been developed quite recently at TU Delft from the basis of SWAN. It is a non-hydrostatic wave-flow model that uses the non-linear shallow water equations to predict wave transformation and covers a wide range of hydraulic processes. However, it has not been verified yet for wave overtopping. Therefore, verifying if SWASH is able to model wave overtopping is the most important and innovative target of this thesis.

In order to make sure that everything works fine, intermediate steeps have been modelled and analysed. Firstly, wave propagation was tested by simulating a wave flume. Afterwards, a smooth impermeable dike was introduced to see how this structure affects the waves and if overtopping could be measured. Later, a rubble mound breakwater described as a porous structure in SWASH was studied. All simulations were performed in a flume without scaling.

On one hand, the measured overtopping at dikes is considered to be fairly good though only 13 tests were performed and analysed. Then, further and more extensive research should be conducted for different initial situations to validate the model. The SWASH output discharges are smaller than the 5% lower limit of Eurotop and Neural Network predictions. Nevertheless, they were close values and they have the same order of magnitude, which is very important when comparing overtopping values. No refection analysis was done. The smaller discharges are believed to be explained by the fact that SWASH does not describe splash, and therefore, only overtopping due to waves running up the slope, but not due to “spray” is taken into account. In addition to this, it is thought that SWASH does not model accurately enough wave breaking over abrupt changes in bottom geometry or steep slopes resulting in underestimation of overtopping.

On the other hand, SWASH is not able to predict wave overtopping at rubble mound breakwaters. A breakwater is not well modelled in SWASH, because porous structures are dealt as a numerical dissipation box and not as a physical obstacle for incoming waves. As a result, waves are damped but not diverted upwards, so there is no overtopping. The reasons why this happens and some recommendations are given in this thesis. Besides, some ideas about how to introduce a multi-layer structure are explained.

Keywords: Wave overtopping, SWASH, rubble mound breakwater, dike, Eurotop, Neural Networks, non-linear shallow water equations, wave-flow model, porous structure in SWASH, validation of SWASH.

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L'objectiu principal d'aquesta tesina és avaluar si amb les eines que tenim actualment, podem predir acuradament el cabal mitjà d'ultrapassament en dics en talús. En primer lloc, els mètodes disponibles (fórmules de l'Eurotop manual i millores proposades per diversos autors, la fórmula de Lykke Andersen i Burcharth i les xarxes neuronals) s'expliquen i es critiquen. A més, s'estudia amb més detall com descriuen la influència d'una berma. En segon lloc, una nova eina numèrica anomenada SWASH és posada a prova.

SWASH ha estat desenvolupat recentment a TU Delft a partir del programa SWAN i està basat en resoldre les equacions d'aigües somes (aproximació de les equacions de Navier-Stokes). Aquest nou model *flux-ona* no hidrostàtic és capaç de predir transformacions d'onatge i descriure un ampli ventall de processos hidràulics. Tot i això, encara no ha estat verificat per ultrapassament. Així doncs, l'objectiu més important i la component més innovadora d'aquesta tesina és validar si SWASH pot modelar ultrapassament.

Per tal d'assegurar que tot funciona correctament, s'han modelat i analitzat diversos passos intermedis. Primerament, l'onatge s'ha propagat en un *flume* numèric. A continuació, s'ha introduït un dic impermeable i llis (*dike*) per veure com aquesta estructura afecta l'onatge i si es podien mesurar els cabals d'ultrapassament. Després, s'ha descrit en SWASH un dic en talús (*breakwater*) com a estructura porosa. Totes les simulacions s'han fet en un canal sense reduir les dimensions reals.

Per una banda, el cabal mesurat en dics impermeables es pot considerar parcialment satisfactori, malgrat que només s'han fet i analitzat 13 simulacions. Així doncs, és necessari ampliar la recerca investigant més casos diferents. L'ultrapassament donat per SWASH és proper als valors predits per Eurotop i les xarxes neuronals i tenen el mateix ordre de magnitud. Això és molt important quan es parla d'aquest fenomen. Malgrat tot, les mesures sempre es troben per sota els percentils 5% predits per ambdós mètodes. Cal dir que no s'ha analitzat la magnitud de l'ona reflectida. El fet que el cabal mesurat sigui inferior s'explica pel fet que SWASH és incapaç de descriure esquitxos. Com a conseqüència, només té en compte les ones que remunten el dic fins a ultrapassar-lo, però no les "gotes" d'aigua que desprèn l'onatge en trencar i que superen el dic per inèrcia. Una altra explicació és que SWASH no modela prou acuradament trencament d'ones quan hi ha canvis bruscos en el perfil del fons.

Per altra banda, SWASH no és capaç de predir ultrapassament en dics en talús. Aquestes estructures no estan ben modelades en el programa, perquè aquest tracta les estructures poroses com un objecte que dissipa l'energia de l'onatge incident, però no com un obstacle físic que obstaculitza la propagació. Com a resultat, les ones s'esmorteixen, però no es desvien verticalment, de manera que no hi ha ultrapassament. Els motius pels quals això passa s'expliquen en aquesta tesina, junt amb algunes recomanacions sobre com es podria resoldre. A més, es comenta com es podria descriure una estructura amb diferents capes.

Paraules clau: ultrapassament, SWASH, dics en talús, dics, Eurotop, xarxes neuronals, equacions d'aigües somes, model flux-ona, estructura porosa a SWASH, validació de SWASH.

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1. Introduction

1.1. Objectives

The aim of this work is to study if the available methods are able to describe and predict accurately wave overtopping on rubble mound breakwaters. The developed empirical formulas will be analysed to see which parameters and what range do they cover and how they deal with certain aspects. A critic assessment of them will follow. Also other methods such as Neural Networks will be treated.

Furthermore, the effect of a berm will be studied with more insight. Thus, the influence on overtopping of its geometrical parameters, such as the berm width, will be analysed. Besides, the generic and specific formulas will be criticised.

New software called SWASH will be tested to see if it can be a good tool to predict wave overtopping in this kind of structures. The initial idea was to build a model with a breakwater and compare the recorded results with the predicted values by the empirical formulas and real measurements (if it was possible). However, this could be a very ambitious target for software that has just been developed and that it is likely to give some problems. For this reason, it was thought to start easy and add new features to the model step by step to be sure that it works properly. Thus, the main objective of this thesis, and then the main focus of this research, is to validate SWASH for wave overtopping.

As a last objective, but closely related to the previous one, this thesis will give some feedback about SWASH in order to help the software developers to improve it. To do that, the problems and limitations that arise when working with SWASH will be analysed, and from them, some recommendations will be given.

1.2. Wave Overtopping

Overtopping takes place when a wave breaks into the coastal structure and the water runs up on the seaward face and goes over the crest to the landward slope. It happens if the wave run-up level is higher than the crest height. However, this concept does not play any role in vertical structures, since it is only important in smooth and rough slopes. It is considered the wave run-up height that is only exceeded by the *2%* of all incoming waves. This parameter is represented by the symbol $\mathrm{R_{u2\%}}$.

Wave overtopping occurs in a very random way, as it is not constant in time and volume of water, but a mean discharge over the crest is considered. Therefore, a mean discharge per linear meter of width q is defined as wave overtopping.

This mean discharge is easy to be measured and lets us identify if we have or not an overtopping problem related to a certain structure, but it does not give us enough information about the magnitude of the problem or the caused damage. To evaluate that, we need to know the number of overtopping waves (or a ratio) and the wave overtopping volume.

The volume of water, *V*, that comes over the crest of the structure is given in volume per unit of width and per wave. Large overtopping volumes are associated to a small number of overtopping waves. The maximum overtopped volume, which allows us to evaluate a flood or storm damage, depends on the storm duration, the ratio of overtopping waves and the mean discharge *q*.

In chapter 2, 3 methods to compute wave overtopping are commented. They are the formulas recommended by the Eurotop manual (and some improvements proposed by several authors), one formula developed by Lykke Anderson and Burcharth for rubble mound berm breakwaters and the use of Neural Networks. Then, the influence of the different factors are shown and analysed. Nevertheless, it is known that overtopping reduces when there's a berm, a gentle slope or a large roughness. If the structure is permeable, wave run up and overtopping reduces too. Wave run-up also depends on the wave period and it increases in case of long waves. We will see if these methods take that into account or not and also which are their weak points.

Besides these empirical methods, we will look in chapter 3 if a new numerical model called SWASH can be a valid tool to compute overtopping.

For the analysis, it is considered the water depth at the toe of the structure, which gives us an upper bound of the wave height. Hence an increase of the sea water level due to any surge (storm, low pressure), wind set-up, greenhouse effect, astronomical tide or expansion may increase wave heights and reduce the structure height related to the Sea Water Level, resulting in larger wave run-up and overtopping.

There are three different forms of overtopping characterized by the way the water goes over the structure. In first place, the "green water" overtopping occurs when the wave run-up reaches a higher level than the crest height and a "continuous" sheet of water passes over it. The second one, the "white water" or "spray" overtopping, takes place when the waves break onto the seaward side and produce large volumes of splash. Many droplets of water are kept in the air and cross over the seawall due to their own momentum or an onshore wind. There is not a continuous overtopping. The last one, and the least important and negligible, is generated by the onshore wind that carries over the structure spray of the wave crests. The spray does not generate significant volumes of overtopping, but it is dangerous because it reduces visibility, for instance. It may be large when the overtopping discharge is low and there are very strong winds. Both "green water" and "white water" were recorded during model tests and predicted in empirical formulas.

1.3. Types of structures

Wave overtopping affects any type of structures built on the shoreline, independently of its design purpose. They could even be flood defences that protect the hinterland from the sea or rivers, revetments build against coastal erosion or harbour breakwaters that offer protected areas for ships.

Four types of structures can be distinguished: sloping dikes and embankment seawalls; armoured rubble slopes and mounds; vertical and steep structures and dunes. However, this essay was initially focussed on overtopping over rubble mound structures though dikes are treated when validating SWASH.

It is very important to be aware of which kind of structure are we studying, because they will not have close responses to wave run-up. Indeed, their behaviour can be too different. For this reason, the three kinds of structures are presented to avoid misunderstandings.

1.3.1. Coastal dikes and embankment seawalls

Sloping dikes have been used along the coasts and rivers in many countries to protect the land behind from flooding. They are simple and cheap structures, easy to build and to raise their level, with a low visual impact (or lower than others) and do not need especial materials.

They consist on an earth embankment, built with clay to make it impervious, with some protection against erosion like grass, asphalt, rock or concrete (blocks). The crest could have a vertical wall. A berm or some rough elements may be present on the seaward slope to dissipate wave energy and reduce run-up.

The most important feature of this type of structure is that it is impermeable. Therefore, water cannot go through the structure by means of porous media flow. Water can only overtop the dike.

Figure 1.1. Coastal dike covered by grass (source: Eurotop, photo from Schüttrumpf).

1.3.2. Armoured rubble slopes and mounds

This kind of structure built with layers of quarried rock is used to build breakwaters, groins, revetments and protections of the toe of other structures. They dissipate a large proportion of the incoming wave energy when the waves break without significant displacement of armour units. Significantly more energy dissipation occurs than with an equivalent smooth (and impermeable) slope because of the higher roughness and porosity of the armour layer.

They are quite easy to build, but hard and heavy rock or concrete blocks are needed, which make them more expensive than sloping dikes.

Like the dikes and embankment seawalls, the armoured rubble slopes and mounds can be complemented with a berm on the seaward slope or a vertical wall on the crest.

Figure 1.2. Rubble mound breakwater at the Malamocco inlet of the Venice Lagoon (http://www.salve.it/it/gallery/cantierimalamocco.htm).

In this kind of structures, wave overtopping has been often more important than wave run-up. For this reason, the crest height of the rock slopes and rubble mound structures has been designed in order to limit overtopping or transmission (in case of low crested structures or port breakwaters) to acceptable levels rather than to prevent them.

The under layer and the core also play a role. If the core is permeable, the run-up water can percolate into it and the run-up on the seaward slope surface decreases to a constant maximum height. However, if the core is not permeable, the water accumulates in the armour layer between the impermeable core and the slope surface. Then, the run-up water can not go down and the run-up increases. It may be significant for large Iribarren numbers.

Reshaped and non-reshaped berm breakwaters

There is a difference between a breakwater with a berm and a berm breakwater. The first one is a common rubble mound with a berm that is designed in such a way that the berm reduces the wave loads and wave overtopping. The idea comes from the dikes with a berm. In contrast, the second one is built with a rock slope that is allowed to be reshaped by the (storm) waves into a stable profile. The idea of this kind of breakwaters is not far from the beach equilibrium profile indeed. The name berm breakwater comes from the lower part of the reshaped profile, which resembles a berm although the equilibrium profile is different from the initial one and determines the breakwater stability against wave attack.

Then, berm breakwaters are totally a different design concept than a rubble mound breakwater. They require a large amount of material, but smaller sizes to be stable, because the reshaped profile “gives stability”. This allows constructors to use rock in a more effective and cheaper way, when it is available. There are 3 types of berm breakwaters depending on the allowed reshaping:

- the statically stable non-reshaping berm breakwaters: only some stones are allowed to move in a similar way to a conventional rubble mound breakwater. It is a complete multi-layered structure to optimize all the rock with the heaviest blocks at the berm.
- statically stable reshaping berm breakwater: the profile is allowed to reshape into a profile (in 1-3 storms) that is stable and where rocks do not move. It is formed by a core and a rock armour.
- Dynamically stable reshaping berm breakwater: the profile is reshaped into a stable profile for the current wave conditions, but the individual stones move up and down the slope with the changing wave conditions. It is formed by a core and a rock armour.

1.3.3. Vertical and steep structures

Vertical and steep structures are often used in cities where there is a lack of available space to build defences, and in deep water maritime works (harbour breakwaters and quays) where a huge amount of material would be necessary and now, saved.

Using stone or concrete blocks, concrete floating caissons or other materials vertical or very steep walls can be build and give a more optimal solution to the previous situations. However, waves are reflected and break directly on the wall, which produces high impact pressures and sudden and large overtopping.

Composite vertical structures where the emergent part of it is vertical, even if it has a small berm in front of it, and vertical structures which have curve parapet on the upper part of the wall are also included in this group.

In this kind of structures, wave overtopping can be reduced with a curve return parapet that deviates the upward flow, or a rough slope in front of it or dissipation chambers that dissipate energy.

Figure 1.3. Vertical breakwater in Jersey (http://commons.wikimedia.org/wiki/File:St_Catherines_Breakwater.jpg).

1.4. Methodology and essay structure

This thesis consists of two clear different parts:

In the first one, the actual knowledge about wave overtopping in rubble mound breakwaters is summarized and critically assessed. The berm behaviour and description is more deeply analysed. However, spatial distribution of wave overtopping behind the crest is not studied. In section 2.1 the Eurotop manual formulas and the improvements suggested by A. Lioutas, J.C. Krom, S. Sigurdson and J.W. van der Meer are commented. A specific formula for berm breakwaters proposed by Lykke Andersen and Burcharth is analysed in section 2.2. Finally, in section 2.3 Neural Networks are briefly introduced. This part, which comprises chapter 2, is merely a review of the state of the art and it is not new research itself. Therefore, readers who already have insight in these topics can perfectly skip it, because most of the information would be known for them. They can jump to next chapter then, which focusses on the main research point.

The second part is focused on SWASH and it is described in chapter 3. As it has been told before, this is the main research topic of this thesis and its highlight. SWASH software is introduced and their features are described in section 3.1. Later in section 3.2, SWASH has been tested to see if it could be used as a tool to compute wave overtopping. To do that, a model has been built step by step by adding new things until we are able to simulate a real breakwater. All these phases or steps are described and the arisen problems and limitations are commented together with the results. Afterwards in section 3.3, the way how SWASH deals with porosity is explained with deeper insight. Partial conclusions are discussed in detail in each section and they are summarized at the end of the chapter (section 3.4).

Finally, the conclusions and recommendations of this thesis are discussed in chapter 4. The majority of the conclusions are about SWASH validation for wave overtopping, but few of them come from the methods commented in chapter 2. All recommendations are given as a feedback to SWASH developers to improve the model.

The thesis report includes two appendices:

The appendix A is a summary of the majority of the SWASH simulations performed during this work. The input parameters and results are commented in detail. As a consequence, this appendix is very long and it has 108 pages. For this reason it has its own table of contents and lists of tables and figures. The codes of 3 SWASH scripts are also included.

The summary of SWASH scripts was written as a laboratory notebook at the same time tests were run. Therefore, the main purpose was to collect and analyse the results as soon as the simulations were performed, and give a useful tool to anyone starting working with SWASH, so he could benefit from the acquired experience. The partial conclusions written there are nothing more than ideas and thoughts and should not be considered as strong and definitive arguments or statements. All the valid explanations and conclusions are commented in chapter 3.

In appendix B the most important Matlab programmes used to analyse the results are commented. They are either scripts or functions that analyse the wave records obtained with SWASH or compute wave overtopping to speed up calculations.

2. Overtopping Formulas

After doing some research and studying the present literature, three different ways to calculate the wave overtopping discharge in rubble mound structures can be distinguished: TAW/Eurotop guidelines, Lykke Andersen & Burcharth formula and Neural Networks. All of them are based or supported on experimental tests and are presented in this chapter.

These empirical formulas depend on tests data and include parameters from the waves and the structure geometry. Sometimes the slope is included with the Irribaren number (also known as breaker parameter). Also the variables may be dimensionless, for example freeboard height referred to the wave height.

2.1. Eurotop

The Eurotop manual (2007) recommends using the formulas given by the TAW 2002, which was a Technical Report of wave run up and wave overtopping at dikes that was used in the Netherlands as a design guideline. The Eurotop manual presents these formulas with some improvements and modifications replacing the previous one.

First of all, it must be known that most research about wave overtopping has been done over smooth dikes and little over rough sloping structures. As a result, the formulas of slopping dikes are rather developed and work quite well.

Secondly, rubble mound structures are similar to dikes as they have close profiles and they are formed by different layers. However, they have steeper slopes than dikes and larger permeability and porosity because of the fact that they are built with big blocks, which reduces wave run-up and overtopping. For example, the steepness of the slopes of these structures is around 1:1.5.

As a consequence of both arguments, the basic formulas of the dikes and embankment seawalls can be applied with some modifications.

In fact, they specially suffer from a lack of research over some specific topics such as berms. This results in absence of specific formulas (and then formulas from smooth slopes are also adopted even though they may be wrong) or enough data to test or improve the accepted formulas.

In this method, the slope is included through the Irribaren number and some influence factors take into account the role played by rough elements, wave walls, berms, oblique wave attack, etc. The berm influence factor is the same as in dikes.

Different authors have proposed some improvements to these formulas and they are commented at the end in sections 2.1.7, 2.1.8 and 2.1.9.

2.1.1. Overtopping due to waves

Equation (2.1) predicts the mean $R_{u2\%}$ for rock or rough slopes, and also for slopes armoured with concrete armour units with the right roughness factor.

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \leq \gamma_b \cdot \gamma_{f,surging} \cdot \gamma_\beta \cdot \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right) \tag{2.1}$$

$\gamma_{f,surging} = \gamma_f + (\xi_{m-1,0} - 1.8) \cdot (1 - \gamma_f)/8.2 \quad \text{if } 1.8 \leq \xi_{m-1,0} \leq 10$
$\gamma_{f,surging} = 1.8 \quad \text{if } \xi_{m-1,0} > 10$
For a permeable core a maximum is reached for $R_{u2\%}/H_{m0} = 1.97$

γ_b = influence factor for a berm
γ_f = influence factor for rough elements on a slope
γ_β = influence factor for oblique wave attack
$\xi_{m-1,0} = \frac{\tan\alpha}{(s_{m-1,0})^{0.5}}$ = Iribarren number / breaker parameter

This equation can be used for probabilistic design with a normal distribution and variation coefficient $\sigma' = \sigma/\mu = 0.07$, and for prediction and comparison of measurements, with the 5% lower and upper exceedance curves.

For deterministic design or safety assessment of the crest is advised to use the equation (2.2), which includes the uncertainty of the prediction.

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1,0} \leq \gamma_b \cdot \gamma_{f,surging} \cdot \gamma_\beta \cdot \left(4.3 - \frac{1.6}{\sqrt{\xi_{m-1,0}}}\right) \tag{2.2}$$

$\gamma_{f,surging} = \gamma_f + (\xi_{m-1,0} - 1.8) \cdot (1 - \gamma_f)/8.2 \quad \text{if } 1.8 \leq \xi_{m-1,0} \leq 10$
$\gamma_{f,surging} = 1.8 \quad \text{if } \xi_{m-1,0} > 10$
For a permeable core a maximum is reached for $R_{u2\%}/H_{m0} = 2.11$

$$P_{ov} = \frac{N_{ov}}{N_w} = \exp\left[-\sqrt{-\ln 0.02} \cdot \left(\frac{R_C}{R_{u2\%}}\right)^2\right] \tag{2.3}$$

Using the equation (2.3), both the probability of overtopping for a given structure and the required freeboard height for an allowable ratio of overtopping waves can be computed.

It is important to be aware where the wave run-up and the wave overtopping are measured. The run-up is measured on the straight slope, while the overtopping is measured behind a crown wall on the crest or simply some at some distance from the seaward slope. Hence, these equations always give an overestimation of the number of overtopping waves, because they show us the number of waves that reach the crest, but some of them might not go over the crest and then be considered overtopping.

In chapter 1 a mean wave overtopping discharge has been defined as an average value per unit of width. It lets us model this random phenomenon, but it has a clear limitation: it can only be computed for almost constant wave and water level conditions. This should not surprise us, since when we characterise a sea-state we are used to work with short time records, so that quasi-stationary conditions could be easily assumed.

The mean overtopping discharge formula has the following pattern:

$$\mathrm{Q}_* = \mathrm{Q}_0 \cdot \exp(-\mathrm{b} \cdot \mathrm{R}_*) \tag{2.4}$$

$Q_* = q/(gH_{m0}^3)^{0.5} = dimensionless\ overtopping\ discharge$
$R_* = R_C/H_{m0} = dimensionless\ freeboard\ height$
$Q_0 = wave\ overtopping\ discharge\ for\ zero\ freeboard\ structure$
$b = influence\ of\ the\ structure$

First, it is necessary to distinguish whether waves are in breaking or non-breaking conditions, because the overtopping response may be different in both cases. To do that, we can compute the Iribarren number and check if the waves are in the "plunging" $0.2 < \xi_{m-1,0} < 2-3$ regime (they usually break) or in the "surging" regime $\xi_{m-1,0} > 2-3$ (they do not usually break). The TAW 2002 Report sets the limit in $\gamma_b \cdot \xi_{m-1,0} \cong 2$. Hence, in the interface would be better to compute both cases.

With no breaking conditions, the waves are not critically influenced by the structure toe or the preceding slope. As a consequence, overtopping waves run up over the wall resulting in "green water" overtopping and apply a smooth-varying load.

With breaking conditions, the waves are mainly influenced by the water depth and, as a result, some waves break violently against de structure.

To calculate the overtopping mean discharge, the same equations for smooth slopes can be used. These equations are:

$$\begin{aligned} \frac{q}{\sqrt{g \cdot H_{m0}^3}} &= \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(\frac{-4.75 \cdot R_C}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \\ \frac{q}{\sqrt{g \cdot H_{m0}^3}} &\le 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \end{aligned} \tag{2.5}$$

$$\begin{aligned} \frac{q}{\sqrt{g \cdot H_{m0}^3}} &= \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(\frac{-4.3 \cdot R_C}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \\ \frac{q}{\sqrt{g \cdot H_{m0}^3}} &\le 0.2 \cdot \exp\left(\frac{-2.3 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \end{aligned} \tag{2.6}$$

Equation (2.5) should be used for probabilistic design and prediction or comparison of measurements. Wave overtopping discharge can be calculated for breaking and non-breaking waves. It is assumed that the coefficients 4.75 and 2.6 have a normal distribution with standard deviations $\sigma = 0.5$ and $\sigma = 0.35$ respectively. For deterministic design or safety assessment it is advised to use equation (2.6), which increases the average discharge by one standard deviation.

The upper bound appears for values near to $\xi_{m-1,0} \cong 1.8$, which is close to the borderline defined in the TAW 2002 $\gamma_b \cdot \xi_{m-1,0} \cong 2$. Therefore, it can be said that the wave breaking behaviour plays a role. As in rubble mound structures the slope steepness is around 1:1.5, large breaking parameters result and the previous equations reduce to:

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \tag{2.7}$$

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = 0.2 \cdot \exp\left(\frac{-2.3 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \tag{2.8}$$

The Eurotop manual only presents the last 2 equations for rubble mound structures ((2.7) and (2.8)). Even though it may work well for the common slopes and as an upper bound they are conservative, the first part should not be forgotten, since in certain situations, large overestimations may be considered.

Type of armour layer	Number of layers	Roughness factor (γ_f)
Smooth impermeable slope	-	1.00
Rock (two layers; permeable core)	2	0.40 (0.37-0.43)
Rock (two layers; impermeable core)	2	0.55
Rock (one layer; permeable core)	1	0.45
Rock (one layer; impermeable core)	1	0.60
Cube	2	0.47 (0.44-0.50)
Cube	1	0.49 (0.46-0.52)
Antifer	2	0.50 (0.46-0.55)
HARO	2	0.47 (0.44-0.50)
Tetrapod	2	0.38 (0.35-0.42)
Accropode	1	0.46 (0.43-0.48)
Core-Loc ®	1	0.44 (0.41-0.47)
Xbloc ®	1	0.44 (0.41-0.49)
Dolosse	2	0.43
Berm breakwater	2	0.40
Icelandic breakwater (reshaped)	2	0.35

Table 2.1. Roughness factors for 1:1.5 slope. These values (T. Bruce, J.W. van der Meer, L. Franco, J.M. Pearson, 2009) are an update of the values given at Eurotop.

Notice that with these formulas there is no influence of the slope and the wave period. These may seem surprising, but for simple breakwaters it may be a reasonable assumption according to tests.

The different roughness factors are shown in table 2.1. They have been obtained from experimental research. Eurotop collects the results of the European Project CLASH, which provided an extensive database from European scale models and structures. Later, some updates were made. In the consulted tables it is not clear what is considered as a “layer”. The point of confusion is that armour rock breakwaters have one or 2 armour layers with two-rock-diameter thickness each (so 2 layers of rock in each armour layer), and the concrete mounds have one or two layers of concrete units placed over a filter or directly over the core.

The experimental tests show that the roughness of the structure slightly depends on the slope angle and the breaking parameter. Besides it, some experimental research found these values low. Therefore, this factor should be checked with experimental tests for each structure.

2.1.2. Oblique waves

The angle of wave attack β is defined at the toe of the structure after the wave propagation to this point (refraction, shoaling and diffraction).

Wave run-up and wave overtopping can be assumed to be equally distributed along the axis of the dike. If it is curved, run-up and overtopping will change. They will increase on concave curves due to fact that the flux of wave energy will concentrate on a shorter length of seaward slope, and decrease in case of convex curves because of the opposite reason.

In CLASH some specific tests were performed to evaluate the effect of the oblique waves. Test were made on a rubble mound breakwater with a slope of 1:2 and a rock or cubes armour (for further information, Eurotop refers to Andersen and Burcharth, 2004). A linear relationship between the angle of wave attack (β) and the influence factor was found. However, overtopping reduces much faster with an increasing β.

$$\gamma_\beta = 1 - 0.0063|\beta|; \quad \text{if } 0° \leq |\beta| \leq 80° \tag{2.9}$$

For $|\beta| > 80°$ the result $|\beta| = 80°$ can be applied and no overtopping is assumed when $|\beta| > 110°$.

Valérie Vanlishout studied in her Master Thesis (2008) oblique wave transmission through rough impermeable rubble mound submerged breakwaters. It is a completely different topic than the one treated in this thesis, but during her research she studied the uncoupled effect of the roughness and the core permeability, which is an interesting point. After some laboratory tests and analysing the obtained data, Vanlishout concluded that the permeability of the core has no influence on the breakwater response to oblique waves. Thus, it is the roughness of the structure what matters most rather than the permeability of the core when talking of oblique waves transmission.

This conclusion comes from studying oblique wave transmission in submerged breakwaters, however, this conclusion might be general. Then, it would be expected that the roughness factor that appears in the Eurotop formulas is more influenced by the roughness than the permeability of the core.

2.1.3. Influence of a berm

A berm is defined when a part of the structure profile has a slope that varies between horizontal and 1:15, while if it is steeper than 1:8, it is called a slope.

When the Eurotop Manual was written little research about rubble mound berm breakwaters had been done. Despite the lack of information, the definition of the berm influence parameter used for smooth slope structures was adopted (2.10). Hence, it was assumed that a rubble permeable berm would behave as an impermeable (dike) berm.

Berms reduce wave run-up and wave overtopping. The influence factor of a berm depends on the width (*B*) and the vertical difference between the middle of the berm and the Sea Water Level (d_B), which describe the berm.

Equation (2.10) lets us compute the reduction factor.

$$\gamma_b = 1 - r_B(1 - r_{db}); \qquad 0.6 \leq \gamma_b \leq 1.0 \tag{2.10}$$

$$r_b = \frac{B}{L_{Berm}}$$

$$r_{db} = 0.5 - 0.5\cos\left(\pi \frac{d_b}{R_{u2\%}}\right); \qquad \text{for a berm above still water line}$$

$$r_{db} = 0.5 - 0.5\cos\left(\pi \frac{d_b}{2 \cdot H_{m0}}\right); \qquad \text{for a berm below still water line}$$

$$r_{db} = 1.0; \qquad \text{for a berm lying outside the area of influence}$$

L_{Berm} is the horizontal length measured between the points of the down and upper slope that differ $1 \cdot H_{m0}$ from the berm level. The geometry of the berm is drawn in figure 2.1.

The reduction of wave run-up or wave overtopping is maximum for a berm on the still water level ($r_{db} = 0$) and decreases with raising d_B. If the berm lies below $2 \cdot H_{m0}$ or above $R_{u2\%}$ (from SWL[1]), it has no influence on the process.

The assumption made that if the berm depth is too deep ($d_B \leq 2 \cdot H_{m0}$), it does not affect run-up or overtopping could be right, because it is assumed that waves break when $H_{break} = \gamma_{break} \cdot h_{break} \leq h_{break}$, so it does not influence wave breaking and the waves break on the main slope. Nevertheless, I do not think that the other statement is completely true. My argument is that if a berm lies above $R_{u2\%}$ (or some times H_{m0}) it works as a breakwater with a large crest width. It is true that wave run-up will not change, because the maximum run-up level will be below the berm height (or little water will reach the berm height and there will stop or percolate). But overtopping will have a significant reduction, since splash water must go over the berm and the crest width. To give an idea of the reduction although the physical

[1] SWL: Sea Water Level

phenomenon is not the same, it is advised to compare the reduction factors of the table 2.2. Just compare the initial crest width against an equivalent crest width that includes the berm. As a result, I think that a different reduction factor should be used for run-up and overtopping, or at least a modification of the formula of overtopping discharge when the berm lies above the SWL. In the last alternative, a correction of the crest width that takes into account the slope berm can be useful.

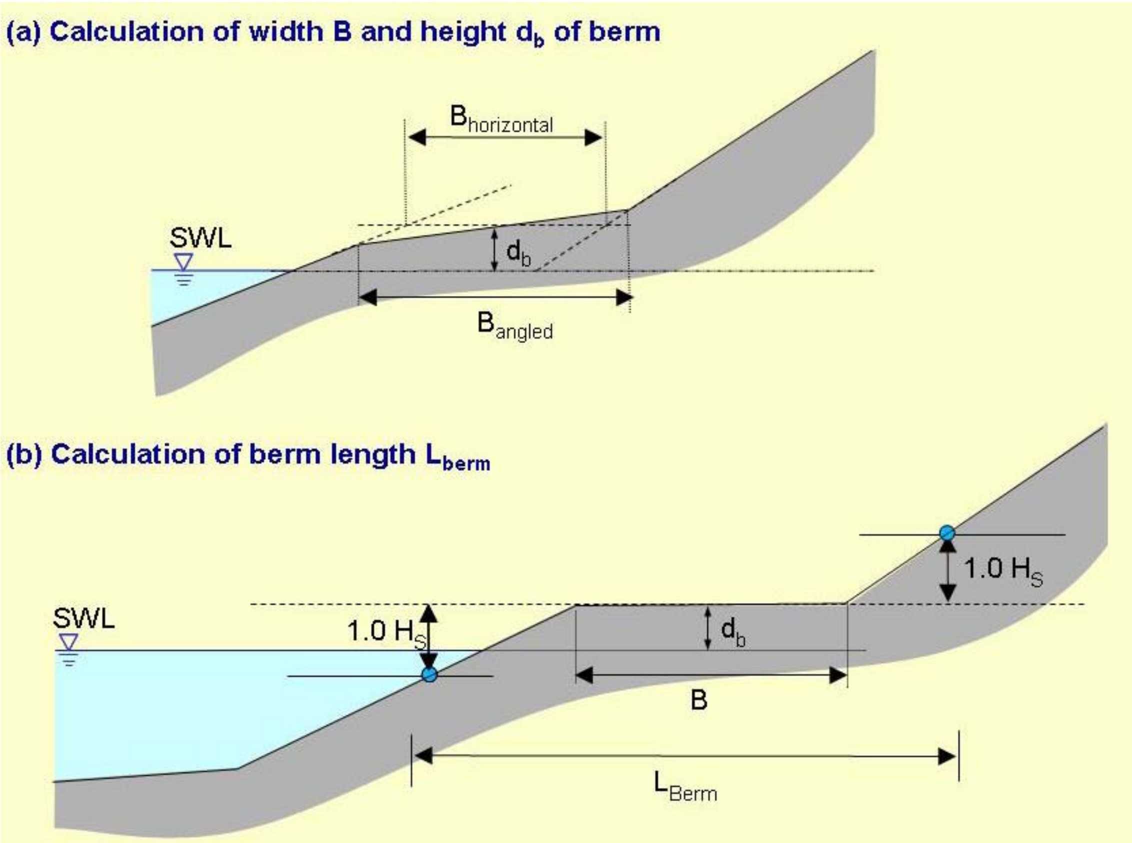

Figure 2.1. Geometric description of the berm parameters (Eurotop).

As the dikes and embankment seawalls have smooth gentle slopes, the breaker parameter is low and has a large influence in the formulas ((2.5) and (2.6)). In contrast to this situation, rubble slopes and mound have steep slopes and the breaker parameter is quite high. As a consequence, there is a situation of non-breaking waves and the maximum value for the previous equations is used (that corresponds to equations (2.7) and (2.8)), and no berm influence factor is found. Then, we conclude that the berm only plays a role in rubble slopes whose slope is so gentle that the upper bound of the commented equations is not reached (and then, can be used).

This conclusion comes from assuming the formulas from dikes because of little previous research. However, that may be wrong, since the berm must have an influence on overtopping, even a little one. Thus, it is clear that there is a big spot in the Eurotop manual for rubble mound structures and berms have to be deeply studied.

J.C. Krom (2012) studied the influence of a berm on overtopping in rubble mounds in his MSc Thesis. He confirmed what I had previously thought about how EurOtop formulae deal with berms, and after performing some laboratory tests he proposed some improvements, which are given in section 2.1.8.

Sigurdson and van der Meer (2012) have proposed an influence factor for berm breakwaters that would replace γ_f and γ_b . It is commented in section 2.1.9.

2.1.4. Effect of the armoured crest

The width of the slope crest can be used to reduce overtopping. The wider the crest, the more water can percolate through, more energy can be dissipated and thus, the more overtopping decreases. This reduction takes place when the crest width is at least 3 nominal diameters ($\mathrm{D_n}$) long.

Eurotop refers to Besley (1999) to describe how to calculate the influence of a wide armoured crest. First the wave overtopping for a simple slope with a crest width of $3 \cdot \mathrm{D_n}$ must be computed[2]. Secondly, the reduction coefficient (2.11) should be applied to the overtopping discharge. The table 2.2 shows some results.

$$\mathrm{C_r} = 3.06 \cdot \exp(-1.5 \cdot \mathrm{G_c}/\mathrm{H_{m0}}) \qquad \text{with a maxiumum } \mathrm{C_r} = 1.0 \tag{2.11}$$

$G_c = crest\ width$
$C_r = reduction\ coefficient$

Crest width (G_C)	Reduction coefficient (C_r)
$0.75H_{m0} \cong 3 \cdot D_n$	0.9934
$1.0 \cdot H_{m0}$	0.6828
$1.5 \cdot H_{m0}$	0.3225
$2.0 \cdot H_{m0}$	0.1523
$3.0 \cdot H_{m0}$	0.0340
$4.0 \cdot H_{m0}$	0.0076

Table 2.2. Relation between the crest width and the overtopping reduction coefficient (Eurotop).

A larger reduction was obtained for an accropode slope, since equation (2.11) was found for a rock slope and may be conservative.

2.1.5. Definition of the crest height and effect of wave walls

Equations (2.7) and (2.8) consider a crest with a width of $3 \cdot \mathrm{D_n}$ and a wall behind the slope that has the same height as the armour layer crest ($\mathrm{A_C} = \mathrm{R_C}$). Since part of the overtopping waves will percolate through the armour and will not go over the crest (and the wall), a less higher wave wall ($\mathrm{A_C} > \mathrm{R_C}$) will not reduce significantly overtopping. Nevertheless, the elimination of the wall would lead to an overtopping increase. It is known from the available

[2] The width of the crest of a rubble mound structure like breakwaters or revetments is usually $3 \cdot \mathrm{D_n}$.

data that the crest width is more important than the wave wall height when reducing overtopping.

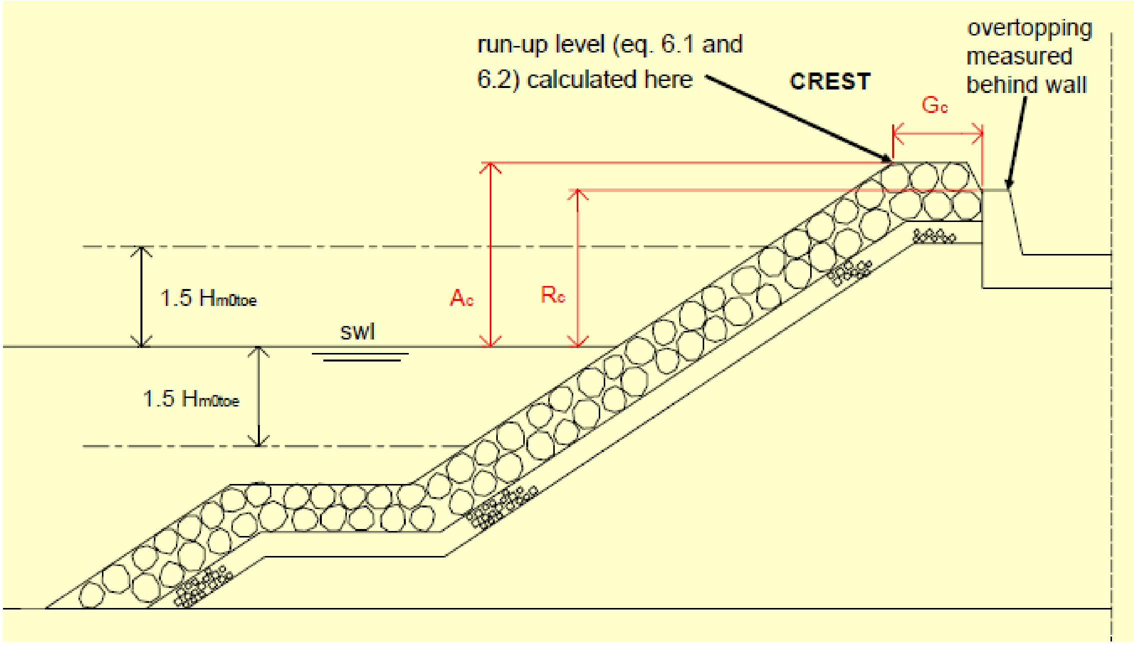

Figure 2.2. Section of a berm breakwater section (Eurotop).

A way to consider this effect is using the given equations with the larger height. If the wall height is higher than the armour crest height ($A_C < R_C$), use the wall height (R_C). Otherwise, use the armour crest height (A_C).

If we consider a wall that crosses the whole crest into an impermeable core, water can not go through the armour. With this kind of impermeable structure, when overtopping occurs, all the water must go over the crest.

A. Lioutas (2010) and J.C. Krom (2012) suggested in their MSc Thesis to take into account the permeability of the crest by using $R_C = A_C - 0.5 \cdot D_{n50}$ and $R_C = A_C - D_{n50}$, respectively. The determination of the crest height was found to be of critical importance.

2.1.6. Overtopping volumes

A Weibull distribution with two parameters is used to describe the overtopping volume per wave. These are a shape factor 0.75 and a scale factor a, which depends on the mean overtopping rate q, the mean wave period T_m and the probability of overtopping waves P_{ov}. As a result of this probability distribution, a few waves generate the larger overtopping volumes, as it can be observed in the nature.

Expression (2.12) shows the cumulative probability function of overtopping volumes per wave and (2.13), the overtopping volume per wave for a given probability of exceedance P_V. If the number of overtopping waves is known, equation (2.14) gives the maximum overtopping volume in a storm.

$$P_V = P[\nu \le V] = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right] \tag{2.12}$$

$$where\ a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 0.84 \cdot T_m \cdot \frac{q \cdot N_w}{N_{ov}} = 0.84 \cdot \frac{q \cdot t}{N_{ov}}$$

$V = volume\ per\ wave\ (value)$

$\nu = volume\ per\ wave\ (random\ variable)$

$t = T_m \cdot N_w = storm\ duration$

$$\mathrm{V} = \mathrm{a} \cdot [-\ln(1 - \mathrm{P_V})]^{4/3} \tag{2.13}$$

$$\mathrm{V_{max}} = \mathrm{a} \cdot [\ln(\mathrm{N_{ov}})]^{4/3} \tag{2.14}$$

If a Rayleigh distribution of the wave run-up height is assumed, then the probability of overtopping per wave can be computed through (2.15).

$$P_{ov} = \exp\left[-\sqrt{-\ln 0.02} \cdot \left(\frac{R_C}{R_{u2\%}}\right)^2\right] \tag{2.15}$$

$\mathrm{P_{ov}} = \frac{\mathrm{N_{ow}}}{\mathrm{N_w}} =$ probability of overtopping per wave

$\mathrm{N_w} =$ number of incoming waves

$\mathrm{N_{ow}} =$ number of overtopping waves

2.1.7. Improved formula by Lioutas

A. Lioutas proposed in his Master Thesis (2010) a prediction formula which is an improvement of the TAW and Eurotop Manual equations. It is called “the Adjusted TAW formula”. This formula (2.16) defines the entire process of overtopping and joins the two previous formulas in one. As a result, it can be used in all wave conditions. It also incorporates a decay factor, which allows us to estimate the overtopping discharge at a certain distance behind the crest.

That formula was developed after some experimental research in which he concluded that the formula used for non-breaking waves (the upper bound) had a considerable scatter, due to the fact that the Iribarren number was not taken into account. He also found that the other stretch of the formula, which corresponds to breaking conditions, fitted very well (probably because of considering $\xi_{m-1,0}$), but it underestimated and overestimated overtopping for large and small relations $R_*/\xi_{m-1,0}$ respectively. Therefore he included the breaking parameter with a power smaller or equal to 1, which resulted in a good fit.

$$\frac{q}{\sqrt{g \cdot H_{m0}^{\ 3}}} = (0.2 - 0.133 \cdot k)\left(\frac{\gamma_b \cdot \xi_{m-1,0}}{\sqrt{\tan\alpha}}\right)^k \cdot \exp\left(\frac{-(2.6 + 2.15 \cdot k) \cdot R_C}{\xi_{m-1,0}^{\ k} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{2.16}$$

The value k included in the improved formula (2.16) varies between 0 and 1. If it takes the value 1, the (long) formula for breaking waves given in the Eurotop Manual is obtained. In contrast, the upper bound which corresponds to non-breaking conditions is obtained when $\mathrm{k} = 0$. A. Lioutas proposes using the value $\mathrm{k} = 0.5$, since it fits best his experimental tests. After using this formula all the data follow the same trend line and no groups are formed, resulting in a low spread.

The decay factor is not included in the above written formula, since it is not the study of the overtopping distribution behind the crest the aim of this thesis. However, the reduction factor is $\gamma_c = -0.164 \cdot \frac{x}{G_C} + 0.677$, where $x = 0$ is the landward end of the crest (x increases landwards). γ_c is a crest width reduction factor like C_r explained in section 2.1.4., but it is added in the denominator of the exponential function.

As I commented in section 2.1.5, Lioutas defined the crest freeboard as $R_C = A_C - 0.5 \cdot D_{n50}$ to take into account the crest permeability.

Afterwards in Lioutas et al. (2012), this formula was re-adjusted (2.17) and a better fit was obtained with $k = 0.6$ and $R_C = A_C - 0.9 \cdot D_{n50}$. For overtopping just behind the crest $\gamma_c = 0.577$ is recommended (see 2.18).

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = (0.2 - 0.133 \cdot k)\left(\frac{\gamma_b \cdot \xi_{m-1,0}}{\sqrt{\tan\alpha}}\right)^{k} \cdot \exp\left(\frac{-(2.6 + 2.15 \cdot k) R_C}{(\xi_{m-1,0} \cdot \gamma_b \cdot \gamma_v)^{k} \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_c \cdot H_{m0}}\right) \tag{2.17}$$

$$\gamma_c = -0.142 \cdot \frac{x}{G_C} + 0.577 \tag{2.18}$$

2.1.8. Improved formula by Krom

J.C. Krom continues in his Master Thesis the research that A. Lioutas had begun. He performed more experiments with a permeable non-reshaping berm and analysed and compared all the new and previous results.

His first conclusion was that the existing formulations are not able to predict accurately the wave overtopping discharges of the physical small-scale models. The latest formula given by Liotas (2.17) had the better fit though it was not enough.

Based on his experimental data, a good fit was achieved when the crest freeboard was defined as $R_C = A_C - D_{n50}$ to take into account the crest permeability.

According to his reasoning, with an impermeable berm part of the wave energy is concentrated over the berm because of the smaller water depth and the rest is dissipated. Therefore, it is the most efficient mechanism to dissipate wave energy and this is the way it works for dikes, which explains how the influence factors are defined in EurOtop. This is called the “fully impermeable” berm, for which the Eurotop berm parameters apply.

In contrast, the waves can partially propagate through a permeable berm, so less energy is dissipated and the waves can reach the main slope before breaking. However, the irregularities of the berm and its roughness dissipate some energy through turbulence. If the berm lies above water level this turbulent dissipation increases. Despite the more turbulence, the propagation through the berm makes that impermeable berms dissipate more energy before reaching the main slope.

For permeable berms below SWL, the berm affects the waves and make them steeper. As a result, there is more energy dissipated in wave breaking and run-up and overtopping discharge reduces, but less than for impermeable berms.

For permeable berms above SWL, it does not affect waves until they break on the berm slope, so the berm acts as an extended crest rather than a berm, because energy is not dissipated before the main slope. This berm is more permeable, irregular and rougher and dissipates more energy, and thus, run-up and overtopping are reduced. Wind waves do not “generate” a flow of water over the berm, because it percolates. There is a kind of decay. Only some swell waves propagate through the berm and reach the main slope, where they run-up and may overtop. Hence, the reduction is stronger for sea waves than for swell waves. This situation dissipates more energy than equivalent impermeable berms and that permeable berms below SWL due to the larger importance of roughness.

The first change was to incorporate the breaking parameter in the formula through the roughness factor as it was already done for run-up. Since for long waves, the roughness of the slope does not reduce overtopping anymore, the upper limit is changed from $\xi_{m-1,0} = 10$ to $\xi_{m-1,0} = 6$. The resulting expression is:

$$\gamma_{f,surging} = \begin{cases} \gamma_f & \text{for } \xi_{m-1,0} < 1.8 \\ \gamma_f + (\xi_{m-1,0} - 1.8) \cdot (1 - \gamma_f)/4.2 & \text{for } 1.8 < \xi_{m-1,0} < 6 \\ 1.0 & \text{for } 6 < \xi_{m-1,0} \end{cases} \quad (2.19)$$

$$\frac{q}{\sqrt{g \cdot H_{m0}{}^3}} = 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_b \cdot \gamma_\beta \cdot \gamma_{f,surging}} \right) \quad (2.20)$$

The equation 2.20 incorporates the berm influence factor and the Iribarren number through the new roughness factor. The spreading caused by the different wave steepness is not found anymore. Although it was developed from tests with berm breakwaters, this new equation gives a better fit for simple rough slopes.

From the comparison of the results of the new tests and the previous formulas with the expected physical behaviour, the berm influence factor was redefined in equation (2.21).

$$\gamma_b = \begin{cases} 1 - f_{dB\geq 0} \cdot r_B \cdot (1 - r_{db}) & \text{for } d_B \geq 0 \\ \exp\left(\frac{-f_{dB<0} \cdot B}{H_{m0} \cdot \xi_{m-1,0}} \right) & \text{for } \frac{d_B}{A_C} \approx -0.5 \end{cases} \quad (2.21)$$

$f_{dB\geq 0}$ and $f_{dB<0}$ are parameters that define the material permeability (and maybe more properties). They are defined to be 1 for a fully impermeable berm and 0 for a theoretical fully permeable berm made of air. Since only one type of material was used, Krom fitted these paramaters with the values $f_{dB\geq 0} = 0.4$ and $f_{dB<0} = 0.43$. It is important to recall that only one berm above SWL was tested, which explains that the vertical position is not included. Notice that for berms above water level a decay function is used whereas the same type of function is used below SWL. The other factors are computed according to the EurOtop (see equation 2.10 in section 2.1.3).

When this new berm factor is used in the Adjusted TAW formula from Lioutas a significantly better fit is found.

2.1.9. Improved influence factor for berm breakwaters

S. Sigurdarson and J.W. van der Meer proposed in an ICCE conference on Coastal Engineering in 2012 an improved influence factor for berm breakwaters (2.23). This factor γ_{BB} should be used instead of γ_{f} on the formulas presented in the Eurotop Manual (2.22). Using this new factor and re-analysing some experimental data on berm breakwaters, they found that the improved formula is as reliable as the formulas given for steep smooth (dikes) and rough (rubble mound) slopes. However, this formula has been recently developed and has been roughly validated.

$$\frac{q}{\sqrt{g \cdot H_{m0}{}^{3}}} = 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \tag{2.22}$$

This new factor takes into account the wave steepness and the berm width. As a consequence, the wave period through the wave steepness is now in the formula (2.23).

$$\frac{q}{\sqrt{g \cdot H_{m0}{}^{3}}} = 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta} \right) \tag{2.23}$$

$$\gamma_{BB} = 0.68 - 4.5 \cdot s_{op} - 0.5 \cdot \frac{B}{H_s} \qquad for\ hardly\ and\ partially\ reshaping\ breakwaters$$

$$\gamma_{BB} = 0.70 - 9.0 \cdot s_{op} \qquad for\ fully\ reshaping\ berm\ breakwaters$$

The authors commented that in contrast to steep slope profiles, it was known that wave overtopping in berm breakwaters depend on the wave period. This is true for permeable berms, which dissipate quite effectively the energy of short waves. This wave period influence was found by the authors although it depends on the type of berm breakwater. As in the berm influence factor of Eurotop (found in dikes and applied in rubble mounds), a wider berm reduces wave overtopping. The mean discharge also reduces with shorter and steep waves.

2.1.10. Conclusions

Eurotop can give reasonably good predictions for rubble mound simple slopes, but not for breakwaters with a berm, since the behaviour of a permeable armour layer is very much different than a smooth an impermeable dike. Besides modelling the berm as it was a dike, reshaped berm breakwaters can only be introduced as a composite slope and the mean slope is used for the calculations, which is not realistic.

The influence of the permeable crest is not totally known and further research is necessary. As a consequence, it is not clear which freeboard should be used in the formulas.

Lioutas; Krom; and Sigurdson and van der Meer have proposed quite recently different improvements to the Eurotop formulations. They were developed at the same time from two different approaches and have improved the predictions for the analysed data set. However, they have not been compared and we do not know which of them performs better.

2.2. Lykke Andersen and Burcharth formula

Lykke Andersen deeply studied rubble mound breakwaters and realised his PhD Thesis over scale effects and berm breakwaters (Hydraulic Response of Rubble Mound Breakwaters. Scale Effects – Berm Breakwaters, 2006). He and his fellow Burcharth from Aalborg University presented in 2004 a formula to compute overtopping of berm breakwaters. This formula (improved in 2005) is presented in this section.

Data of 634 tests (565 from Lykke Andersen and Burcharth and the rest from other five authors) was used to fit the formula through multi-parameter fitting, based on a routine that minimise the square errors of the overtopping discharge logarithms.

The developed formula (2.24) is valid for berm breakwaters with no superstructure. Overtopping is given at the back of the crest and $\mathrm{R_C} = \mathrm{A_C}$.

$$Q_* = 1.79 \cdot 10^{-5} \cdot (f_{H0}^{1.34} + 9.22) \cdot s_0^{-2.52} \cdot$$

$$\exp\left(-5.63 \cdot \mathrm{R}_*^{0.92} - 0.61 \cdot \mathrm{G}_*^{1.39} - 0.55 \cdot \mathrm{h}_{b*}^{1.48} \cdot \mathrm{B}_*^{1.39}\right) \tag{2.24}$$

$$Q_* = \frac{Q}{\sqrt{g \cdot {H_{m0}}^3}} \ ; \ R_* = \frac{R_C}{H_{m0}} \ ; \ G_* = \frac{G_C}{H_{m0}} \ ; \ B_* = \frac{B}{H_{m0}}$$

$$h_{b*} = \frac{3 \cdot H_{m0} - h_b}{3 \cdot H_{m0} + R_C} \geq 0$$

$$f_{H0} = \begin{cases} 19.8 \cdot exp\left(-\dfrac{7.08}{H_0}\right) \cdot {s_{m0}}^{-0.5} & for\ T_0 \geq {T_0}^* \\ 0.07 \cdot H_0 \cdot T_0 + 11 & for\ T_0 < {T_0}^* \end{cases} = berm\ recession\ parameter$$

$$H_0 = \frac{H_{m0}}{\Delta \cdot D_{n50}} \ ; \ \Delta = \frac{\rho_{rock} - \rho_w}{\rho_w}$$

$$T_0 = \sqrt{\frac{g}{D_{n50}}} \cdot T_m \ ; \ {T_0}^* = \frac{19.8 \cdot exp\left(-\dfrac{7.08}{H_0}\right) \cdot {s_{m0}}^{-0.5} - 11}{0.07 \cdot H_0}$$

The factor $1.79 \cdot 10^{-5}$ has a variation coefficient 2.22. All parameters involved are referred to the non-reshaped profile (initial slope) to make it easier. The stability parameter $\mathrm{f_{H0}}$ describes the reshaping of the berm breakwater. In case of non-reshaping berm breakwaters, $\mathrm{f_{H0}} = 0$. From observing the variables, it is noticed that the rock parameters (size and density) only affect $\mathrm{f_{H0}}$. However, the large number of parameters involved makes it not handy to use the formula.

The data of other researchers used to fit the formula also includes tests with multi-layer berm breakwaters. For this reason, the formula can be used for this type of breakwaters when the largest size is applied in the formula due to the limited allowed reshaping in this kind of structures.

To conclude with, it is demonstrated that the formula can predict overtopping of homogeneous and multi-layer berm breakwaters with high accuracy. A little overestimation of the discharge has been found for reshaped profiles in contrast to non-reshaped.

2.2.1. Laboratory tests

Before their study, there was very limited and no systematic research over berm breakwaters. They presented a dimensionless overtopping formula for berm breakwaters based a large parametric model test study with berm breakwaters. 565 tests were performed and the data from other authors was used, resulting in a 695-test database. The formula was derived for statically and dynamically stable berm breakwaters as well as non-reshaping statically stable berm breakwaters, all with homogeneous berms.

Their tests were conducted in a wave flume at Aalborg University. The model structure had no coronation structure and overtopping was measured behind the crest. Table 2.3 shows the range of parameters tested. Most of the tests (594) were performed with reshaping berm breakwaters and a few (59) with fixed front geometry.

Parameter	Range
Peak wave steepness (s_0)	0.010 - 0.054
Wave height at the toe of the structure ($\mathrm{H_{m0}}$)	0.064 - 0.164 m
Water depth at the toe of the structure (h)	0.24, 0.34, 0.44 m
Crest and armour crest freeboard (R_C/A_C)	0.08, 0.11, 0.14, 0.17 m
Crest width (G_C)	0.17, 0.24, 0.31, 0.38 m
Berm width (B)	0.00, 0.20, 0.30, 0.40, 0.50. 0.65 m
Berm elevation (h_b)	0.04 (above SWL), 0.00, 0.04, 0.12 m
Front slope below berm	1:1.25
Front slope above berm	1:1.25
Rear slope	1:1.25
Stability number ($\mathrm{H_0}$)	0.96 – 4.86
Stability index including wave period ($H_0 \cdot T_0$)	16.8 – 163
Reynolds number for berm stones (Re_D)	Armour 1: $3.32 \cdot 10^4$ to $4.92 \cdot 10^4$
	Armour 2: $3.13 \cdot 10^4$ to $3.91 \cdot 10^4$
	Armour 3: $1.60 \cdot 10^4$ to $2.44 \cdot 10^4$

Table 2.3. Range of parameters tested by Lykke Andersen in his PhD Thesis (Lykke Andersen, 2006).

The formula was based on tests with initial slopes of 1:1.25, which was the natural repose angle for the tested armour materials. However, guidelines on how to modify this formula for other slopes are also given (see section 2.2.2).

As data for multi-layer berm breakwaters (Icelandic type) are included, the conclusions on the usage of the derived formula will cover also multi-layer berm breakwaters.

To study the influence of the breakwater reshaping, 59 tests were done with fixed front geometry. The structure was the same used in other tests, but a net was put to avoid rock movements. Then, the influence of rock displacements in overtopping could be studied. However, I think that these models did not actually represent non-reshaping breakwaters, in which the layers of rock should be disposed so that they do not move on stable slopes. Therefore it is not clear to me if this formula was thought for stable berm breakwaters, whose (design) slope is stable for most of the wave conditions, although Lykke Andersen wrote in his conclusions that non-reshaping breakwaters were completely covered by his formula.

Although the formula was developed for reshaping breakwaters and it has a factor that takes into account the effect of profile reshape, better overtopping estimations are found for non-reshaping profiles, since some overestimation occur in reshaped profiles. Besides this accuracy, higher overtopping discharges values take place in reshaped berm breakwaters because of their smoother slope.

2.2.2. Guidelines for initial slopes other than 1:1.25

Since the present tests were all performed with initial slopes 1:1.25, some guidelines were given by the authors to use the formula for other initial slopes.

The argument is based on the fact that the reshaped profiles are almost identical for a given volume of stones and do not depend on the initial lower slope. At least, this seems to be true for dynamically stable profiles and drives to the conclusion that B has to be enlarged by $0.5 \cdot (\mathrm{h} - \mathrm{h_b}) \cdot (\cot(\alpha_\mathrm{d}) - 1.25)$. For a very stable structure with very limited damage it is believed that the down slope has very little influence on the overtopping discharge. Therefore the correction should not be done in such cases as it could lead to unsafe results for slopes flatter than 1:1.25.

In case of different initial front slopes above the berm, it is advised to modify B and GC so the distance to the back of the crest is the same as for a slope 1:1.25. That means increasing both B and $\mathrm{G_C}$ with the length $0.5 \cdot (\mathrm{R_C} + \mathrm{h_b}) \cdot (\cot(\alpha_\mathrm{d}) - 1.25)$.

Note that the berm length may be modified twice if the upper and lower slopes differ from 1:1.25.

2.2.3. Other restrictions

Some dependency on the overtopping discharge with the water depth was found for $\mathrm{h}/\mathrm{H_{m0}} < 2.5$ due to wave breaking (higher threshold for flatter foreshores). Despite the influence of the foreshore slope and the water depth in the wave height distribution, these two parameters were not included in the formula, since only one foreshore was tested and mostly non-breaking waves were performed in the flume. Therefore, it is advised to verify the formula for cases with heavy wave breaking on the breakwater slope and in deep water. Conservative results are expected in case of heavy wave breaking.

The influence of the grading of the armour rocks on the overtopping discharge was not studied in their study even though it may exist, since grading plays a role in the structure porosity and reshaping.

Remember that the formula was developed for breakwaters with no superstructure. In that situation, a reduction will surely occur, but is not modelled.

2.2.4. Conclusions

Lykke Andersen and Burcharth developed a very specific formula for a very specific case looking at many variables. As a result, they obtained a very complex formula that gives a good fit for all kind of berm breakwaters. The reshaping behaviour is introduced through a factor, so that the input parameters come from the non-reshaped profile description, which makes it easier. However, because the formula was obtained from a multivariable fitting method, the formula includes lots of parameters and it is very complex. Therefore, the formula is not very handy to use and there is no physical approach behind it, only statistics.

Few breakwaters profiles were tested although it is justified that the reshaped profiles will be the same whatever the building profile. However, I do not think that the non-reshaping berm breakwaters are well studied, since only one slope was tested. There are guidelines to adapt the input values for different slopes, but flatter slopes than 1:1.25 are said to give unsafe predictions. Besides it, no superstructure was studied. Furthermore, it should be verified for heavy wave breaking though the authors comment that they expect conservative results.

Despite the fact that there is no direct influence of the porosity or multiple layers in the formula, accurately predictions both for homogeneous and multi-layered berm breakwaters are obtained, which apparently looks contradictory.

For these reasons, further research with different sections and wave conditions should be performed although this specific formula seems to work well for berm breakwaters.

2.2.5. Comparison with Eurotop

The formula recommended in the Eurotop manual has the following generic form:

$$Q_* = \min\left\{\begin{array}{c} \dfrac{A\cdot\sqrt{\tan\alpha}}{\sqrt{s_0}}\gamma_b\cdot\exp\left(\dfrac{-C\cdot R_*\cdot s_0}{\tan\alpha\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_v}\right) \\ A\cdot\exp\left(\dfrac{-C\cdot R_*}{\gamma_f\cdot\gamma_\beta}\right) \end{array}\right\} \tag{2.25}$$

In case of breaking waves, the formula includes the following variables: H_{m0}, R_C, s_0, $\tan\alpha$, γ_f, β, B, h_b. When there are non-breaking waves, the parameters that represent the berm disappear (commented in 2.1.3).

In reshaped berm breakwaters, the final berm has a steeper slope than the range considered in the berms definition. As a result, the resulting berm must be introduced with a composite slope profile and the average slope is used. It must be said that this formula does not cover well this type of structure, since there is too much scatter.

The formula proposed by Lykke Andersen and Burcharth is:

$$Q_* = A \cdot \left(f_{H0}^{1.34} + C\right) \cdot s_0^{-2.52} \cdot \exp\left(-D \cdot R_*^{0.92} - E \cdot G_*^{1.39} - F \cdot h_{b*}^{1.48} \cdot B_*^{1.39}\right) \tag{2.26}$$

We can observe that more variables are present in this formula: H_{m0}, R_C, G_C, s_0, $T_{m-1,0}$, D_{n50}, ρ_{sea}, ρ_{rock}, B, h_b, but the slope angle is not present since only one was tested. The wave obliquity may be considered by multiplying the discharge by an orientation factor. D_{n50}, ρ_{sea}, ρ_{rock} may be taken into account within γ_f, and G_C in the reduction factor for crest wider than $3 \cdot D_{n50}$, but it is not inside the formula as it is in Lykke's. If we insert this coefficient in the Eurotop formula, we get:

$$\begin{aligned} Q_* &= \min\left\{\begin{array}{c} \dfrac{0.067 \cdot \sqrt{\tan\alpha}}{\sqrt{s_0}} \gamma_b \cdot \exp\left(\dfrac{-4.75 \cdot R_* \cdot s_0}{\tan\alpha \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \\ 0.2 \cdot \exp\left(\dfrac{-2.6 \cdot R_*}{\gamma_f \cdot \gamma_\beta}\right) \end{array}\right\} \cdot 3.06 \cdot \exp\left(-1.5 \cdot G_*\right) = \\ &= \min\left\{\begin{array}{c} \dfrac{0.20502 \cdot \sqrt{\tan\alpha}}{\sqrt{s_0}} \gamma_b \cdot \exp\left(-\dfrac{4.75 \cdot R_* \cdot s_0}{\tan\alpha \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} - 1.5 \cdot G_*\right) \\ 0.612 \cdot \exp\left(\dfrac{-2.6 \cdot R_*}{\gamma_f \cdot \gamma_\beta} - 1.5 \cdot G_*\right) \end{array}\right\} \end{aligned} \tag{2.27}$$

If we consider front waves:

$$\begin{aligned} Q_* = 1.79 \cdot 10^{-5} \cdot \left(f_{H0}^{1.34} + 9.22\right) \cdot s_0^{-2.52} \\ \cdot \exp\left(-5.63 \cdot R_*^{0.92} - 0.61 \cdot G_*^{1.39} - 0.55 \cdot h_{b*}^{1.48} \cdot B_*^{1.39}\right) \end{aligned} \tag{2.28}$$

$$Q_* = \min\left\{\begin{array}{c} \dfrac{0.20502 \cdot \sqrt{\tan\alpha}}{\sqrt{s_0}} \gamma_b \cdot \exp\left(-\dfrac{4.75 \cdot R_* \cdot s_0}{\tan\alpha \cdot \gamma_b \cdot \gamma_f} - 1.5 \cdot G_*\right) \\ 0.612 \cdot \exp\left(\dfrac{-2.6 \cdot R_*}{\gamma_f} - 1.5 \cdot G_*\right) \end{array}\right\} \tag{2.29}$$

Applying logarithms and considering 1:1.25 slope and $\gamma_f = 0.55$, we respectively get:

$$\begin{aligned} \ln Q_* = -4.4178 + \ln\left(f_{H0}^{1.34} + 9.22\right) - 2.52 \cdot \ln s_0 - 5.63 \cdot R_*^{0.92} - 0.61 \cdot G_*^{1.39} - 0.55 \cdot \\ h_{b*}^{1.48} \cdot B_*^{1.39} \end{aligned} \tag{2.30}$$

$$\ln Q_* = -1.6962 - 0.5 \cdot \ln s_0 + \ln \gamma_b - 10.7954 \cdot R_* \cdot s_0 \cdot \gamma_b^{-1} - 1.5 \cdot G_* \tag{2.31}$$

$$\ln Q_* = -0.4910 - 4.7272 \cdot R_* - 1.5 \cdot G_* \tag{2.32}$$

Few terms can be directly compared, because of the different relations between variables obtained from experimental research. Only Lykke's and Eurotop's upper bound can be compared.

2.3. Neural Networks

The wave overtopping phenomenon involves too many variables from the sea hydrodynamics and the structure geometry and properties that can not be properly analysed in current experimental tests. They can describe quite well the response of a certain "common and simple" structure, but not all the relations between all variables and some influence factors are often added to model uncertainties or unknowns.

These are some of the general disadvantages of the empirical formulas derived from experimental tests. Nevertheless, there is a way to improve the predictions made on the basis of the available data: the use of Neural Networks.

2.3.1. Description

Neural Networks are data driven models that work as a multiple regression fit for cause-effect relations. They do not give a relation between parameters as a result, but they are very useful for solving difficult modelling, where there is large amount of experimental data and an unclear interrelationship of parameters.

Provided that a huge amount of (reliable) data is available, a very precise solution (output) will be given for the considered variables (input). However, the input parameters should be according to the ones used in the initial database from which the Neural Network was developed, i.e. the problem to be solved must be similar to the experimental data.

A method that uses Neural Networks can estimate wave overtopping discharges (or run-up or whatever) for a wide range of coastal structures, so wide that they do not have anything in common such as vertical walls, rubble mounds and dikes. Given an accurate description of the structure and the waves as an input (that should be treated in a certain way), the same method can deal with any kind of structure as long as enough data from other structures are in the database. This point overcomes the restrictions from experimental tests, which are done with very strict models, and, therefore, whose results are only valid for these intervals of parameters. Nevertheless, the initial database must be very extensive, since it should include variation intervals of all input parameters.

In one hand, the main advantages of the Neural Networks are that only one method is used for all types of structures, instead of different equations for each simplified case (Vertical, Mounds and Dikes as in most guidelines) without restrictions of the number of parameters; and that good predictions are obtained in spite of our ignorance of the relationship between the different parameters.

In the other hand, an endless large amount of data is needed to build the database and the NN[3]. This data has to be reliable and must be accurately treated to build an homogeneous database, which is a titanic work before preparing the NN.

2.3.2. Database

The European project CLASH collected around 10000 overtopping tests from several laboratories before 2004. This extensive database has been used for setting up the few Neural Networks proposed by some authors.

Each test is described by 3 hydraulic parameters ($\mathrm{H_{m0}}$, $\mathrm{T_{m-1,0}}$[4] and β); 12 structural parameters (h, $\mathrm{h_t}$, $\mathrm{B_t}$, γ_f, $\cot\alpha_\mathrm{d}$, $\cot\alpha_\mathrm{u}$, B, $\mathrm{h_b}$, $\tan\alpha_\mathrm{b}$, $\mathrm{R_c}$, $\mathrm{A_c}$ and $\mathrm{G_c}$); a Reliability Factor (RF) that estimates the reliability and uncertainties of the test procedure and results; and a Complexity Factor (CF) which evaluates if the structure has been correctly described with the structural parameters. A Weight Factor was built combining the Reliability and Complexity Factors, so that the most reliable tests had a larger weight in the NN configuration and training (see next points). The most unreliable or most complex data was taken out to avoid unreliable results. Figure 2.3 shows all input parameters.

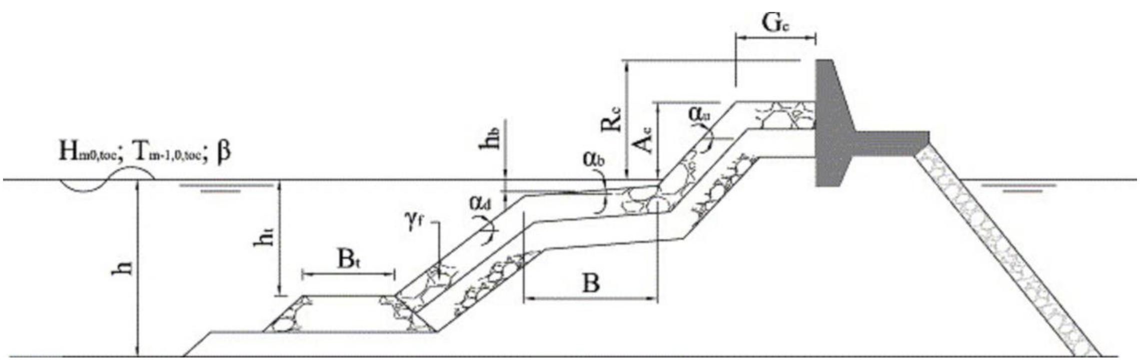

Figure 2.3. Scheme of the structure profile with all the parameters. (Van Gent et al., 2007).

All the input and output variables were scaled to $\mathrm{H_{m0}} = 1\ \mathrm{m}$ using Froude's similarity law. Roughness and slopes are not affected by this scaling.

2.3.3. Modelling

The Neural Network is organized in three layers that contain several neurons and the connections between them and the neurons of other layers. The neurons of the same layer are not connected and work as a processing units. The key idea of this structure is embracing all the possible relationships between the input parameters. Each neuron gets the information from the preceding layer as a weighted sum of outputs, applies a non-linear activation function and generates an output.

The first layer is formed by all the input parameters and the third one by the output parameter (only one in this NN). The middle layer (hidden layer) and their connections are set during a

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[3] NN: Neural Network
[4] The mean spectral period at the toe of the structure.

training process. The number of neurons within a hidden layer work as adjustable parameters. Therefore, an increase of them reduces the differences between the predicted output (by the NN) and the observed one due to a larger number of degrees of freedom. However, it cannot be increased without limitations, because the NN would model fluctuations in the data set and the accuracy will not decrease.

The training process with the database assigns some weights to the connectivities. Then, the stronger the influence of some input parameters, the stronger their links will be. It sets the number of neurons of the hidden layers too.

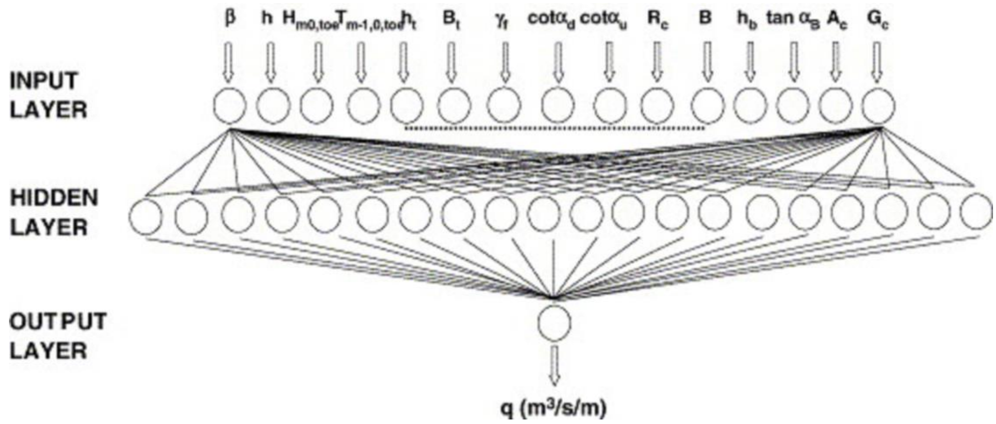

Figure 2.4. Schematization of the Neural Network. (Van Gent et al., 2007).

2.3.4. Training and Evaluation

First of all, the data set must be divided in two sets (it is a partition). A larger one is used to "train" the NN, and a shorter one "tests" the NN. Each set is used only once, i.e. the test set does not train the NN and vice versa.

In second place comes the training or learning phase, in which the NN is adjusted and its weights are calibrated until the predicted results meet the registered ones, which are considered as targets. During this process, the differences between the estimated and measured overtopping (quantified in an error function) are minimised through an iterative process.

Finally, in the test phase a testing data set evaluates the calibrated NN. As a conclusion, the predictions of the NN may be influenced by the chosen training set, so the partition must be made carefully. Nevertheless, there is a way to overcome this limitation by using Resampling Techniques. This strategy consists in training many NN with different training and testing sets. As a result, a set of NN (which may not have the same output for the same input) is obtained. This set lets us evaluate the uncertainty of the model (not the database, which once treated is considered right).

2.3.5. Applications

Besides predicting the overtopping discharge for a given structure and wave conditions, the Neural Network is capable to predict the influence of a certain input parameter, such as wave angle, crest freeboard or slope. Furthermore, a confidence interval can be estimated from using Resample Techniques.

The NN can not predict $q = 0$ as an output, even when these situations are included in the training set. This is another weak point of the NN (a part from all of them related to the database). Zero output is simply not possible and large (and wrong) overestimations of the overtopping discharge occur in the area of low overtopping. However, it is possible to “cheat” the NN through a combined classifier-quantifier model. This consists in using two neural networks. The first one just computes an estimation of the output discharge and decides whether it can be neglected or not. If it can be neglected it is considered null. In the other case, the second NN predicts a more accurate discharge for the non-zero cases. This model was developed within the CLASH project as a part of the PhD Thesis of Verhaeghe (2005).

2.3.6. Conclusions

Neural Networks are useful tools that let us predict the mean overtopping discharge in a wide range of coastal structures using one single model. This model includes all the relations between the wave and structure parameters and offers a rather accurate estimation. It is a stronger and more robust model than empirical formulae, but it needs a painstaking work.

Neural Networks can not predict a null output, but this problem can be “easily” solved by using a classifier NN that works as a filter.

The prediction reliability is totally based on the reliability of the used database and, up to a point on the training set. For these reasons the used database must be carefully treated and chosen. However, the uncertainties of the prediction can be estimated.

Besides to the data reliability, the model would only predict accurately the overtopping discharge over those structures whose description is included in the database’s input intervals. As a consequence, its extensive range of applicability reduces to the data used to train the Neural Network.

Lykke Andersen deeply studied rubble mound berm breakwaters, proposed a formula and compared it with the CLASH Neural Network. He concluded that his experimental formula performed better than the NN due to the few berm breakwaters included in the database (82 over approximately 8000). Although this NN worked very well for berm breakwaters included in the training set, the predictions for other berm breakwaters not included were bad. It was expected that, if his laboratory tests data were included in the database, the NN would perform better.

Due to Lykke Andersen conclusion, few data of the rubble mound berm breakwaters and the large time needed to prepare NN, they will not be used in this thesis for this purpose.

3. SWASH

SWASH (Simulating Waves till Shore) is a non-hydrostatic wave-flow model recently developed by TU Delft from the basis of the SWAN software. The main aim of this new software is to predict the wave transformation from offshore conditions to the shore in order to study wave phenomenon in beaches, harbours and coastal waters.

This software uses the non-linear shallow water equations (Navier-Stokes depth averaged) and can describe complex phenomenon related to rapidly varied flows, such as wave breaking and run-up at the shoreline, between other coastal processes. A large range of conditions can be simulated and thus, many processes modelled of different hydraulic interests.

In the next section, the properties of the software will be briefly described. For more detailed and extensive explanation, the reader is referred to the SWASH website and the paper written by Marcel Zijlema et al. (2011).

3.1. Description and properties of the software

Since SWASH is ruled by the non-linear shallow water equations, the model runs in a depth-averaged mode or a multi-layered mode, in which the vertical domain is divided in many layers. To obtain a better prediction the model would use more layers rather than higher order derivatives. In fact, it does not use more than second order spatial derivatives and gets second order accurate approximations in time and space. The use of different layers allows us to overcome the limitations of using these “simplified” Navier-Stokes equations. However, using more layers means more operations and thus, larger computation time and more numerical errors.

This numerical model works with second order finite differences and different grids. Each parameter (water depth, velocity, etc.) can be defined independently of the others in their own special grid. This is valid for the input and output values. Afterwards, all these grids are superposed. If their nodes coincide with the ones of the computational mesh, the model uses those values. Otherwise, it makes a linear interpolation of the values of the “property grids”. On the one hand no fine meshes must be defined if the values of some parameters are constant or have just a linear variation, which is a main advantage. On the other hand, if the parameters can not be described with a simple pattern, interpolations between meshes with different sizes can lead to major unexpected and unreliable results. Be aware that 2 interpolations may be done (from the input grid to the computational one and from the last one to the output grid) and thus, errors might be amplified.

Mass and momentum are strictly conserved at a discrete level (i.e. each element). This allows the model to track the actual location of incipient wave breaking. Broken waves can propagate and change their form due to the momentum conservation. This approach is valid for hydraulic jumps, dam-break problems and flooding situations. Then a similarity between wave breaking and hydraulic jumps is set, and, therefore, energy dissipation is already considered. Furthermore, non-linear wave properties of breaking waves are preserved.

As commented before, SWASH can be used to study a large list of physical phenomena and predicts a large amount of outputs, including elevations, depth-averaged velocities and discharges. Therefore, it can be used as a numerical model flume to test hydraulic structures, such as breakwaters and estimate wave overtopping.

However, this new software has some limitations.

The first one is that the developers of this software put more emphasis on the hydraulic computation and processes rather than in the physical objects. For this reason, some limitations appear when describing structures such as breakwaters, since some parameters can only be introduced as constant values. For example, a multilayer breakwater can not be described with layers with different porosities. Then, a constant value must be used. In fact, a single value is introduced for each node of the grid. Also there are some limitations in describing objects with very low porosity. New updates are expected to solve these limitations.

The second one is related to the main advantage. The model was designed for a wide range of possible applications, because it can compute everything, but the model has not been verified yet. So, we do not know yet if the results are good or not. As there are lots of different situations, this process is thought to take long. In a first phase the model has to be calibrated with some real data, so that from a certain input values we get a measured real output. During this process, some unknown coefficients that appear in the equations will be given a certain value. Once it is done, in a second phase, the model has to be tested and compared with other results from other methods such as empirical equations or Neural Networks or measured data. Then we will know if the software works and give acceptable results or must be improved.

3.2. Modelling and testing

3.2.1. Procedure

In this thesis SWASH will be used to predict wave overtopping of berm breakwaters, and the results will be compared with the already commented empirical formulas. This software has not been verified for overtopping yet. Thus, this thesis will also check if it works properly in this specific situation.

Before testing a specific structure (berm breakwater), a simpler model needs to be described. Then, all the initial and boundary conditions and all the commands that the software needs to start the computation and give a result will be fixed. Since it is a kind of non-scaled flume test (with the real dimensions) and the description of the breakwater is not complex, this is the most difficult task. In this part, simple waves and the breakwater geometry will be introduced. Afterwards, the “real” structure to be tested and the design waves will be introduced, but the model will not change much.

This simple model will be built following different phases. In each step, some new inputs or boundary parameters will be introduced and checked to make sure that they have been correctly processed. This procedure allows us to identify problems earlier and solve them faster and in an easier way.

The first step consists in checking if the spectrum wave has been correctly introduced and if it propagates right. To do that, a deep water flume (h=100 m) is built, so that the waves are not affected by the bottom. In this flume, the waves are generated at one side and the significant wave height is measured at the other side. From these measured values, a wave spectrum will be built using Matlab, which should coincide with the input.

The second step introduces the limitation of water depth, since it is reduced from 100 to 10 meters. This is the only difference with the previous model. Wave heights are also measured at the end of the flume and then a wave spectrum is built. If the wave height is small compared to the water depth, no big differences should be appreciated.

In the third step a smooth and impervious structure is introduced at one side. Wave overtopping can be measured in this simple dike. The main objective is checking if it can be correctly computed.

Afterwards in the fourth step a rubble mound breakwater will be introduced in order to look what happens when a permeable structure is described. It will be a simple geometry, but a porosity and porous size field will be defined.

Finally, the real geometry will be described in the fifth step. As all the other parameters have been previously verified, no problems are expected to come to light in this last step.

3.2.2. Simulation tests to check the model

3.2.2.1. Wave propagation in deep and shallow water

First I tried the model with a water depth of $\mathrm{h} = 100\ \mathrm{m}$ and a wave with a significant height of 3.5 m and a period of 10 s. We are in deep water conditions and no problems should be expected. However, at the beginning the computations did not work properly. For this reason (and after some tests in shallow water), I reduced to a water depth of $\mathrm{h} = 80\ \mathrm{m}$ and a wave defined with $\mathrm{H_s} = 2\ \mathrm{m}$ and $\mathrm{T_m} = 7\ \mathrm{s}$. Waves were introduced as a regular waves and spectrum waves.

To analyse the output waves, which were recorded on a time-water level table, two Matlab scripts were created. One analyses the register wave per wave using the zero down-crossing criterion to split the waves. The mean and significant wave height and the wave period are given. Easily the range of wave periods and wave heights can be obtained. It also gives as an output the mean water level and $\mathrm{H_{rms}}$. Besides it, $\mathrm{H_s}$ and $\mathrm{H_{rms}}$ are computed through the water elevation variance. A second script does the spectral analysis. First the spectrum and the graphs with the Power Spectral Density (or Variance Density Spectrum) are built through averaging some periodograms and, afterwards, many wave parameters are computed from the previous spectrum and the spectral moments. They are: $\mathrm{H_{m0}}$, $\mathrm{T_{m01}}$, $\mathrm{T_{m-1,0}}$, $\mathrm{T_p}$, $\mathrm{T_0}$ and $\mathrm{H_{rms}}$. However, the latter two are hardly used.

For further information about these Matlab scripts, look at the appendix B (Summary of the Matlab scripts), where is explained how they work.

These tests worked and I could get some interesting conclusions, which are given later with some comparisons with shallow water tests.

One of the objectives of this thesis is to check if SWASH works properly when talking of overtopping as a part of the verification tests of this software, which was recently developed. Because it is quite new and only a few people have worked with it, some definition errors and other problems were likely to occur. For this reason, it was decided to include some of the problems faced during this thesis where it is explained how the model was built. It is thought to be an interesting point for people who want to know more about SWASH.

I worked with SWASH version 1.10A. The current software version is 1.20A.

The first tests with shallow water were performed with a water depth of 10 m and a wave defined by a significant wave height of 3.5 m and a mean period of 10 s. These are intermediate water conditions indeed though they are close to the shallow water boundary, but they will be referred as shallow water tests. The majority of the tests run without any problem though some commands used in deep water did not work. The following conclusions were drown from these tests:

- The wave spectrum was not well defined. The built spectrum is not close to the introduced data, since smaller and much shorter waves were measured.
- When spectrum waves were introduced as an input in deep water, strong dissipation occurred. This phenomenon did not take place in shallow water. The reasons were that the generated spectrum waves were so steep that broke. The dissipative behaviour of wave propagation in deep water conditions enhanced the wave height reduction.
- Be careful with how the wave spectrum is computed with Matlab. It is important to check it first with regular waves. However, do not compare the significant wave heights, but $\mathrm{H_{rms}}$, since the first one is a concept that makes no sense with regular waves, especially when it is obtained from the variance!
- Regular waves were well introduced both in deep and shallow water.
- A strong reduction of mean water level took place in shallow water tests. It was larger when regular waves were introduced instead of spectrum waves. An increase of water depth reduced this effect.
- SWASH computes $\mathrm{H_s}$ and $\mathrm{H_{rms}}$ as output parameters from the variance of the water level elevation. For this reason, the significant wave height value given for regular waves makes no sense. SWASH does not compute the wave spectrum and does not give wave periods as an output either.

The SWASH scripts and their results are not commented in this section. For further information about the used commands in each script, comments, comparisons and conclusions, please look at the appendix A (Summary of the SWASH scripts).

Finally these problems could be solved. The large water level changes were a sort of resonance effect in the wave flume that could be solved with an appropriate sponge layer at the end. Realize how important is to fix the right length for this layer, that should be at least three times the wave length (even though in the User Manual version 1.10 the necessary length was not specified). Although the wave spectrum can be defined through the mean and peak wave periods as input parameters, the spectrum signal is built from the peak period. Hence, the equivalent peak period was computed from the mean one in the first step. It resulted that this computation was wrong introduced and the bug needed to be fixed. With the newer SWASH version 1.20 or using the peak wave period and the appropriate sponge layer length these problems could be overcome. Then (and using the peak period), a structure can be introduced in the next step.

There is some dissipation along the flume. The reason could be the influence of the bottom rather than numerical dissipation. The tests were performed in intermediate water depth close to shallow water conditions, so there is some dissipative behaviour. The spectrum graph at the end of the flume is wider, the peak periods do not change and the mean periods show little variations. For these reasons, the interaction with the bottom would explain the little wave dissipation that is observed.

In the next section, some results of propagated waves are shown and compared with the measured waves under the influence of a structure.

3.2.2.2. Impermeable smooth dike

After reducing the time step, which was not stable although I had managed to simulate wave propagation, I started working with an impervious smooth dike and wave overtopping was measured over it. SWASH adjusts the time step according to the given computation grid and the range of Courant numbers, so in fact the time step was reduced by changing the range of Courant numbers, which had led to larger and unstable time steps.

The computational grid has a size of 1 m and the input range of Courant numbers is between 0.1 and 0.25. An initial time step of 0.001 s was introduced, which fits this range of Courant numbers and also fits with the output tables, which give results every 0.05 s.

The commands DISCRET UPW UMOM H BDF and DISCRET UPW WMOM H BDF have been used to discretize the horizontal advective terms of the momentum equations. A second order backwards upwind scheme has been applied to solve the equations in a stable way.

The smooth and impervious dike is introduced as a bottom variation. In fact, the dike is a part of the bottom that remains above the initial water level. This is the only way to introduce an impervious structure, because small porosity values can not be introduced in SWASH, since some terms of the momentum equations have (a power of) the porosity in the denominator (see section 3.3). For this reason, it is not advised to use porosity values smaller than 0.1 to avoid problems. As no bottom friction was assumed, it was already a smooth bottom. However, it would be good to compare a model with a real case to see, for instance, if some friction should be added.

Wave overtopping is measured as a time-discharge record at the given output points. At least, measurements should be done at the back of the crest. It can not be directly measured as an average discharge value, because is a “dry” point, and then the mean value of the time series must be computed. The maximum peak discharge can be obtained as an instantaneous value from the time-discharge record.

The recorded mean overtopping discharge must be compared with the result given by the empirical formulas of Eurotop for probabilistic design and prediction or comparison of measurements, as wave overtopping measures (equations 3.1 and 3.2) are being compared.

Equation (3.1) should be used for values $\xi_{m-1,0} < 5$. The coefficients 4.75 and 2.6 are stochastic parameters that follow normal distributions described by $\mu = 4.75$ and $\sigma = 0.5$, and $\mu = 2.6$ and $\sigma = 0.35$ respectively. With this values the confidence interval curves can be easily computed as well as the deterministic value.

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(\frac{-4.75 \cdot R_C}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right)$$
$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} \leq 0.2 \cdot \exp\left(\frac{-2.6 \cdot R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right) \tag{3.1}$$

For values $\xi_{m-1,0} > 7$ equation (3.2) should be used. The constant $c = -0.92$ has a standard deviation of $\sigma = 0.24$. For intermediate breaking parameters ($5 < \xi_{m-1,0} < 7$) the results of both formulas should be interpolated.

$$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} \leq 10^{-0.92} \cdot \exp\left(\frac{-R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta \cdot (0.33 + 0.022 \cdot \xi_{m-1,0})} \right) \tag{3.2}$$

The empirical formulas of the Eurotop Manual for dikes are the same as the formulations of Rubble Mounds changing some coefficients (γ_β and γ_f). These influence factors can be checked on the Eurotop Manual. Notice that there is an extra stretch for breaking parameters $\xi_{m-10} > 7$ and interpolation is needed between 5 and 7. This is not done with rubble mounds.

The input values are the spectral parameters measured at the toe of the dike. It is very important to notice that they include the reflected wave, which can not be dissipated in the flume, because it is laterally confined. However, wave overtopping should be computed with the parameters of the incoming wave without any interaction with the structure, so reflection must be subtracted. For this reason, the recorded values from some previous wave propagation tests without any structure at the toe of the dike to compute the input parameters are used.

To start with, 4 simulations with SWASH with 3 different slopes and 2 freeboards were performed. Different dike profiles were tested varying these two parameters, because they are thought to have the larger influence. This is a kind of rough and limited sensitivity analysis, but some clear conclusions can be drawn. These tests are not commented here in detail, only the general observations and conclusions. The results are shown below in table 3.5. For further reading and comparison of the results of each test, have a look on Appendix A (section A.5. Impermeable smooth dikes).

These 4 simulations were performed to see if SWASH worked. For this reason the simulated test duration was quite short. 50 minutes were simulated and the last 45 minutes were recorded. The input wave spectrum in all simulations has a duration of 2 h and it is defined by $\mathrm{H_{m0}} = 3.5$ m, $\mathrm{T_p} = 11$ s.

Measured data at x=0 m (Propagation01.sws)			
Wave per wave analysis		Spectral Analysis	
$H_s = 3.59\,m$	$T_m = 8.72\,s$	$H_{m0} = 3.71\,m$	$T_p = 10.91\,s$
$4 \cdot \sigma = 3.71\,m$	$3.8 \cdot \sigma = 3.52\,m$	$T_{m01} = 9.30\,s$	$T_{m-1,0} = 11.02\,s$

Table 3.1. Measured wave data at the wave generator in a flume without any hydraulic structure. SWASH only simulates here wave propagation.

Measured data at x=780 m (Propagation01.sws)			
Wave per wave analysis		Spectral Analysis	
$H_s = 3.00\,m$	$T_m = 9.18\,s$	$H_{m0} = 3.23\,m$	$T_p = 14.11\,s$
$4 \cdot \sigma = 3.23\,m$	$3.8 \cdot \sigma = 3.06\,m$	$T_{m01} = 9.36\,s$	$T_{m-1,0} = 15.42\,s$

Table 3.2. Measured data in a flume without any hydraulic structure at the location of the toe of the structure.

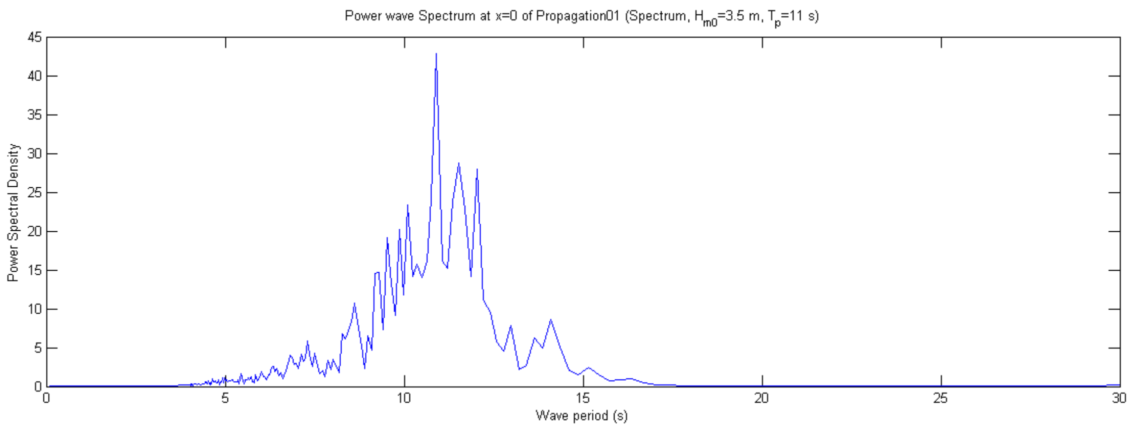

Figure 3.1. Wave spectrum at the wave generator in simulation Propagation01.sws.

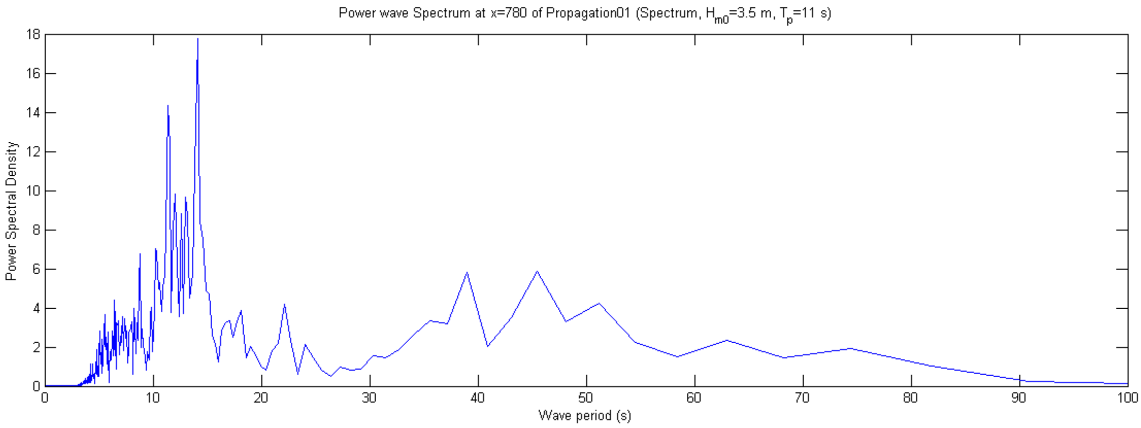

Figure 3.2. Wave spectrum at the location of the toe of the structure in a wave propagation test (Propagation01.sws) without the structure itself. A second peak around 11 s and long waves can be observed. This would be the result from some wave dissipation during the wave propagation.

It is important to realize how important is to use the values without reflection. For this reason, the measured wave parameters with the dike in the flume are also enclosed.

Measured data at x=0 m (Dike01.sws)			
Wave per wave analysis		Spectral Analysis	
$H_s = 3.88\ m$	$T_m = 8.87\ s$	$H_{m0} = 4.16\ m$	$T_p = 10.91\ s$
$4 \cdot \sigma = 4.16\ m$	$3.8 \cdot \sigma = 3.95\ m$	$T_{m01} = 9.40\ s$	$T_{m-1,0} = 12.85\ s$

Table 3.3. Measured wave data at the wave generator in a flume with an impermeable dike. Notice that there is some reflection due to the smooth and impermeable dike although many waves overtopped in the simulation.

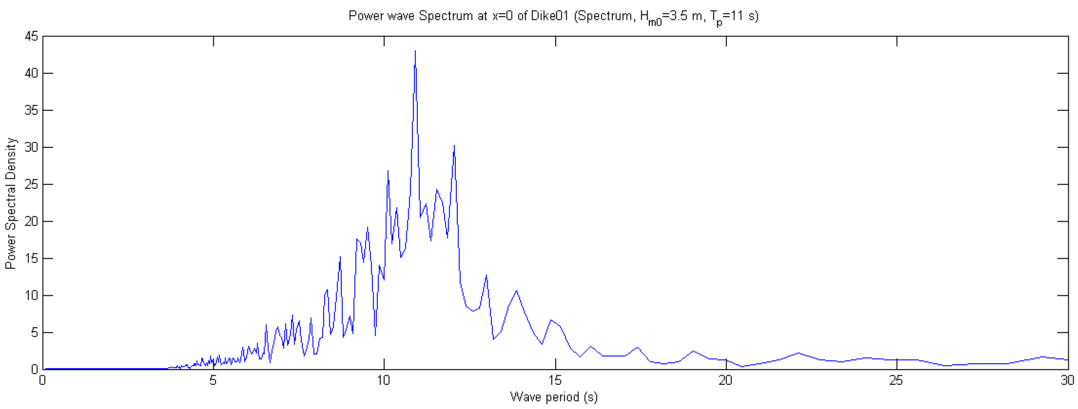

Figure 3.3. Wave spectrum at the wave generator with a dike (Dike01.sws). The shape is similar to the recorded without dike.

Measured data at x=780 m (Dike01.sws)			
Wave per wave analysis		Spectral Analysis	
$H_s = 3.24\ m$	$T_m = 8.22\ s$	$H_{m0} = 3.75\ m$	$T_p = 38.98\ s$
$4 \cdot \sigma = 3.75\ m$	$3.8 \cdot \sigma = 3.56\ m$	$T_{m01} = 10.37\ s$	$T_{m-1,0} = 23.53\ s$

Table 3.4. Measured wave data at the toe of the dike. Long waves are observed.

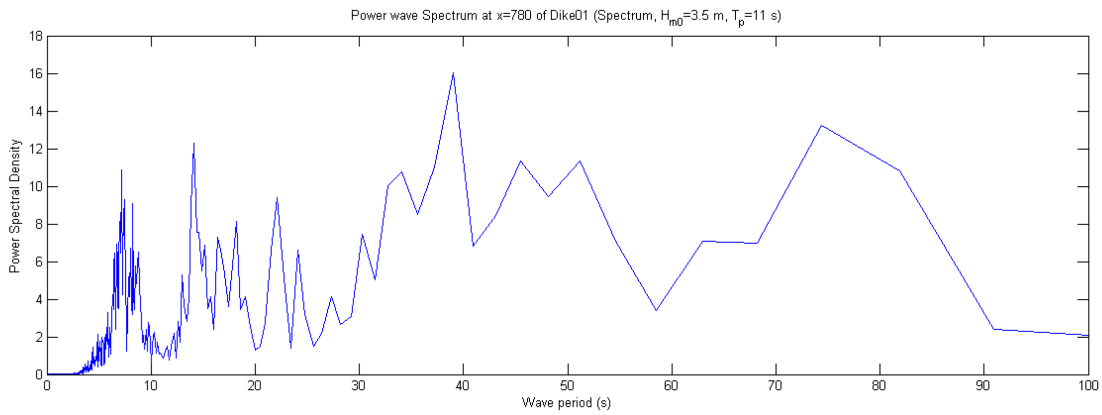

Figure 3.4. Wave spectrum recorded at the toe of the dike (Dike01.sws). Many peaks and long waves are observed compared with the situation without a dike. The reflection produced by the smooth dike and the waves running up and down the slope would explain them.

Model	R_c (m)	Slope	$\xi_{m-1,0}$	$q_{Eurotop}$ (l/sm)	q_{SWASH} (l/sm)	q_{NN} (l/sm)	$q_{NN,5\%} - q_{NN,95\%}$ (l/sm)
Dike01.sws	2.5	1:2	5.36	478.1	267.0	487.6	207.8 - 1106
Dike02.sws	5	1:2	5.36	69.3	22.8	190.7	84.81 - 421.6
Dike03.sws	5	1:3	3.57	65.0	21.1	262.0	98.14 - 678.3
Dike04.sws	2.5	1:4	2.68	486.1	269.3	577.3	258.9 - 1318

Table 3.5. Estimations of mean wave overtopping over impermeable smooth dikes given by Eurotop empirical formulas, SWASH and Neural Networks. Only the incoming wave without reflection has been considered.

As shown in table 3.5, SWASH computes an overtopping discharge that is always smaller than the estimated values from the formulas of the Wave Overtopping Manual (Eurotop). They have the same order of magnitude, which is very important, although the relative differences look quite large (56%, 33%, 32% and 55% respectively). Overtopping increases with a lower freeboard. Despite the fact that SWASH gives smaller discharge values, this is reasonable according to the Wave Overtopping Manual, because the empirical formulas tend to overestimate the wave overtopping discharge, as wave overtopping is measured behind the crest and the number of waves that reach the crest on the front slope is computed. Hence, SWASH gives results which should be “on the safer side” of the empirical formulations. For these reasons, it is concluded at this stage that SWASH computes quite accurate the mean wave overtopping discharge when compared to the Eurotop guidelines.

Furthermore, the on-line free Neural Network available at Deltares[5] website was used to check the results. This NN[6] is incorporated in the software BREAKWAT, developed by Deltares, although the latter software requires purchasing a licence, and it was developed with the database created through the European Project CLASH.

[5] Deltares, formally WL | Delft Hydraulics, is a research institute that collaborates with TU Delft. It is an independent non-profit company.
[6] NN: Neural Network

It can be seen that for low freeboards (Dike01.sws and Dike04.sws), the predictions given by the NN are rather close to the Eurotop estimations. Besides it, the discharge computed by SWASH is within the 90% confidence interval of the NN. Therefore, the results from SWASH look to be quite accurate to the existing tools. In contrast, for the high freeboards and low overtopping, these predictions are much higher than the empirical results and neither the SWASH and Eurotop outputs are within the 90% and 95% confidence intervals. This looks quite strange. There may not be many data with a close range of input parameters in the NN database, but this would be unlikely for simple dikes.

In addition to this, Suzuki et al. (2011 and 2012) compared SWASH model with a physical model and found that there was a very good agreement between both of them. Their conclusion for a concrete case study with a shallow and very mild foreshore and a wall does not show the differences that been observed here. Probably the explanation is related to the fact that SWASH is directly compared to a physical model and not with the empirical formulas although scale effects are known to be very small and are already included in the formulas, and to the geometry itself. The last one may have a significant effect, because it represents a real situation closer to the basis of real sea dikes (and what Eurotop describes) than the SWASH model, which works as a smooth impermable breakwater built in intermediate water indeed. However, the wave conditions at the toe (should) take into account the foreshore, so it is not uncoupled. Furthermore, waves shoal on the gentle foreshore and there is a progressive bottom variation, while in the simulations there is a sharp edge between the horizontal bottom and the steep dike slope. This is makes a big difference on waves (and in SWASH)!

After comparing the results from Dike01.sws and Dike02.sws, it can be seen that the influence of the crest level on wave overtopping is very important. This difference is even clearer after having a look to the plots of recorded discharge along time in the following figures 3.5 and 3.6. In these two situations $5 < \xi_{m-1,0} < 7$, so interpolation between the equations (3.1) and (3.2) must be done to compute wave overtopping. This first formula only depends on the relative crest freeboard, while the second one also depends on $\xi_{m-1,0}$ although the influence of this parameter is smaller.

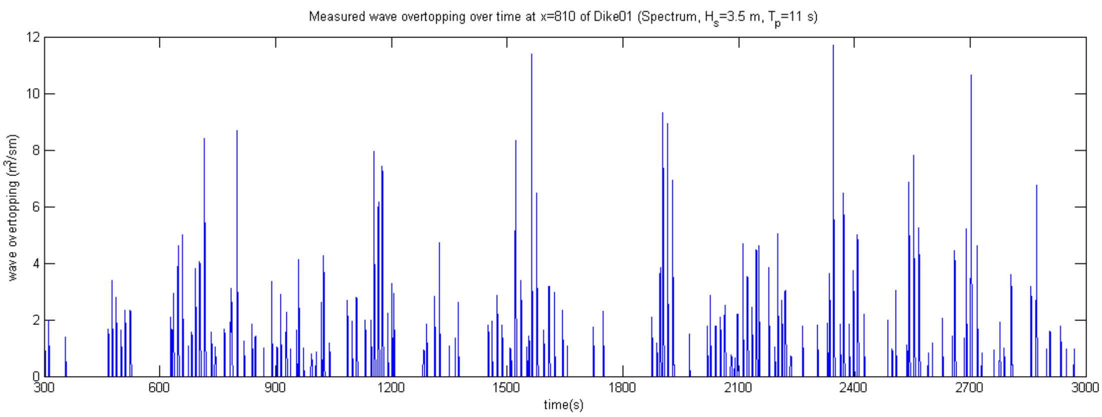

Figure 3.5. Measured discharge over time at the end of the crest of Dike01.sws (2.5 m freeboard). Many waves overtop the dike.

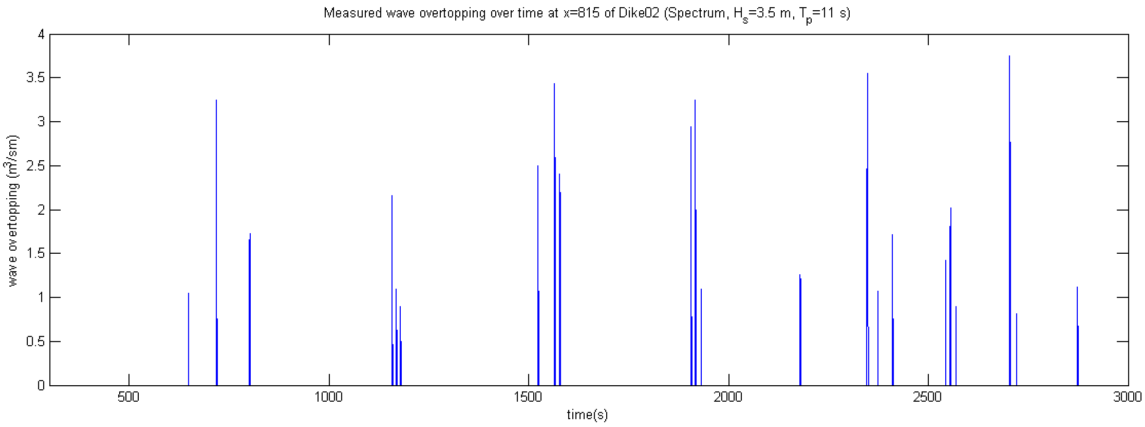

Figure 3.6. Measured discharge over time at the end of the crest of Dike02.sws (5 m freeboard). Only a few waves go over the crest and the overtopping volumes are small.

The other 2 dikes have a relatively large $\xi_{m-1,0} < 5$ and wave overtopping should be computed through the second branch of the equation (3.1), which is an upper bound that does not depend on $\xi_{m-1,0}$ and only on the relative freeboard and the influence factors of slope roughness and wave obliqueness. As a result, no comparisons strictly related to the slope can be made between these two tests. The slope does not affect much the waves at the toe (there is not a significant difference, only 0.1 m). In addition to this and as it was expected, much larger overtopping is measured with a lower crest.

If we want to compare the results from the Dike01.sws and Dike02.sws with Dike03.sws and Dike04.sws, we must be aware that we are comparing different formulas, or different stretches.

A small and ridiculous reduction of the overtopping discharge is found with milder slopes when there is a high crest, but the "same result" is found with low freeboards. The numbers are very close and have the same order of magnitude. It seems that there is no influence of the slope though and only on the freeboard. These observations agree with the second stretch of equation (3.1), which does not depend on the slope, and with (3.2), where the coefficient that multiplies $\xi_{m-1,0}$ or $(\tan\alpha)$ is 15 times smaller than the other, reducing the influence of the slope. Then, in this brief sensitivity analysis, the results measured in SWASH and the differences between the different cases are satisfactory and in line with the predictions given by the Eurotop formulas.

It would be interesting to test SWASH for even smaller Iribarren numbers. It thought that the most interesting option would be decreasing the wave period instead of increasing the wave height or decreasing the slope, since quite high and long waves and a very gentle slope of 1:4 have been already tested.

Remember that the values of the spectral analysis at the toe of the dike have been used to compute the expected wave overtopping through the empirical formulations of Eurotop, and that the wave height at the that place includes the reflected wave (which can not be dissipated in the flume). The data from only the incoming wave is obtained from Propagation01.sws, a

SWASH simulation that propagates the same wave spectrum and measures the waves at the same point without the structure.

Since the results of these 4 simulations were thought to be satisfactory and no problems related to SWASH were found, more tests with a longer simulated duration time and different wave conditions were performed. The results are shown in table 3.6.

The dimensionless discharges have been also plotted in figure 3.7. In the graph, the measured discharge in SWASH can be compared to the predictions of Eurotop and Neural Networks. The exponential function that fits the measured discharges has a correlation factor $\mathrm{R}^2 = 0.8746$. The mean and the 90% confidence interval of both methods are drawn. As it can be seen, the fitted function of the measured overtopping discharge is always below the lower 5% limit predicted by both methods. The measured values are very close to the limit for high overtopping, but the difference increases for lower discharges. The latter could be explained due to the low reliability of small values of wave overtopping. In addition to this, the tests Dike11.sws, Dike12.sws and Dike13.sws were not included on the graph, because no overtopping was recorded. Previously, it has been commented that the Eurotop formulas tend to overestimate the discharges, which may explain why SWASH underestimation, although these overestimations are found to be small in other cases. Furthermore, absolutely no calibration of friction and reflection was done. Friction should not be important, because a smooth dike was considered and with friction discharges would reduce. In contrast, the interaction with the reflected wave may be quite important, since it may slow down the incoming overtopping wave and reduce it. This is just a hypothesis. After analysing the magnitude of the reflected wave, it will be known if this has an influence on the results.

Simulation	R_c (m)	Slope	Incoming wave at the toe without reflection		$\xi_{m-1,0}$	$q_{Eurotop}$ (l/sm)	q_{SWASH} (l/sm)	q_{NN} (l/sm)	$q_{NN,5\%} - q_{NN,95\%}$ (l/sm)
			H_{m0} (m)	$T_{m-1,0}$ (s)					
Dike01.sws	2.5	1:2	3.23	15.42	5.36	478.1	267.0	487.6	207.8 - 1106
Dike02.sws	5	1:2	3.23	15.42	5.36	69.3	22.8	190.7	84.81 - 421.6
Dike03.sws	5	1:3	3.23	15.42	3.57	65.0	21.1	262.0	98.14 - 678.3
Dike04.sws	2.5	1:4	3.23	15.42	2.68	486.1	269.3	577.3	258.9 - 1318
Dike05.sws	2.5	1:2	3.19	14.35	5.02	464.8	261.5	456.5	211.0 – 1003
Dike06.sws	5	1:2	3.19	14.35	5.02	60.9	20.9	174.1	81.3 – 326.9
Dike07.sws	5	1:3	3.19	14.35	3.35	60.6	18.3	252.5	93.6 – 617.4
Dike08.sws	2.5	1:4	3.19	14.35	2.51	465.2	260.4	547.8	272.3 – 1170
Dike09.sws	2.5	1:2	1.82	8.00	3.70	43.2	3.8	64.1	25.3 – 165.8
Dike10.sws	2.5	1:2	1.40	7.46	3.94	10.0	0.43	20.3	6.5 – 64.9
Dike11.sws	5	1:2	1.82	8.00	3.70	1.22	0	2.3	0.8 – 5.7
Dike12.sws	5	1:3	1.82	8.00	2.47	1.22	0	1.6	0.7 – 3.5
Dike13.sws	2.5	1:4	1.82	8.00	1.85	43.2	0	30.3	16.8 – 55.3

Table 3.6. Estimations of mean wave overtopping over impermeable smooth dikes given by Eurotop empirical formulas, SWASH and Neural Networks. Only the incoming wave without reflection measured at the location of the structure toe has been considered.

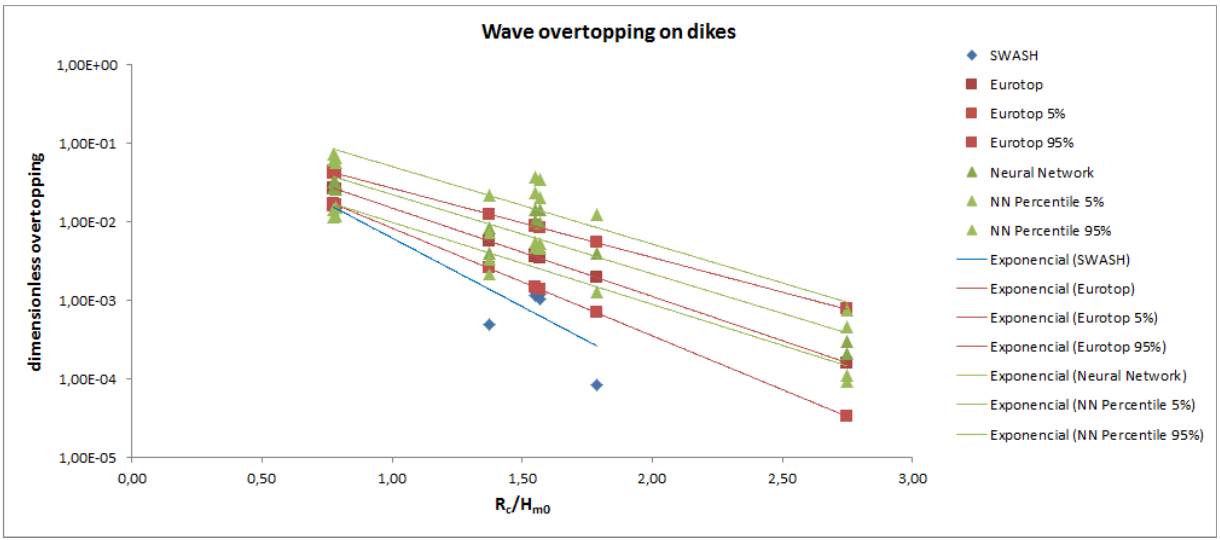

Figure 3.7. Dispersion graph comparing the measured discharge in SWASH with the predicted overtopping by the Eurotop formulas and the Neural Network. The lower and upper limits of Eurotop and NN are also plotted. The three results of SWAH that gave nil overtopping are not drawn.

The majority of the measured discharges in SWASH have the same order of magnitude of the Eurotop predictions. This means that the results are quite good though some relatively important differences are found for low crests and small waves. As the Eurotop formulas do in the considered $\xi_{m-1,0}$ range, practically no differences are found between different slopes and same freeboard. In both cases, the differences are negligible. In contrast, a completely different behaviour was found in NN. It was surprising that with steeper slopes, less overtopping was predicted with large waves. However, the opposite was observed with smaller waves. The NN was mainly developed from small-scale models and a prototype was here considered. Then, it is likely that some scaling and modelling effects would influence the predictions. In addition to this, for impermeable crests it was asked to consider a width of zero meters (CLASH workpackage 8, Coveld et al., 2005). Then, the measured discharge was in fact the recorded at the beginning of the crest, which is slightly larger than at the back, because there are waves that reach the top, but not the end of the crest. These two factors would explain some overestimation with respect to Eurotop.

Finally, and after thinking about the different types of wave overtopping (see section 1.2) and how SWASH models water motion, it became clear that SWASH does not describe all of them. Since SWASH uses the Navier-Stokes shallow water equations, it can only describe the motion of a continuous fluid body. Therefore, “green water” overtopping (continuous water sheet that runs up the slope and overtops) is taken into account, but not “white water” or “spray” overtopping (due to splash). The latter one needs a model such as MAC (Marker And Cell), VOF (Volume Of Fluid) or SPH (Smoothed-Particle Hydrodynamics) to describe the behaviour of the water particles that are suspended after breaking and travel over the structure. These methods are not described in this thesis. SWASH models wave breaking in a simpler way taking into account the integral aspects of the process such as bulk energy dissipation and wave height reduction, but not splash. Thus, SWASH is only able to give reasonable good predictions in situations where “green water” overtopping is predominant, e.g. beaches and mild dikes.

For steeper structures (also relative to the wave length) splash becomes more important due to more "violent wave impacts". This explanation is in line with which has been measured in this thesis, because the closer values were recorded for lower relative freeboards, where there is more overtopping. For higher relative freeboards, "spray" represents a larger share of the total overtopping although it is still small, and hence, the differences increase. Furthermore, it would in part explain why Suzuki et al. (2011, 2012) conclude that SWASH gave a good prediction in their study case.

3.2.2.3. Rubble mound breakwater with an impermeable core

Later, I began working with a rubble mound breakwater. I started with an intermediate structure to see if the porosity and the structure were introduced right, but this unreal structure was difficult to contrast with empirical formulations. For this reason is better to jump to a real breakwater instead of testing an intermediate situation first.

First a rubble mound breakwater with an impervious core was tested. If this did not work in SWASH, a breakwater with a permeable core would not work either.

The introduced breakwater has a rock size of 1 m and one single rock layer over an impermeable core, which can be characterized by the roughness factor $\gamma_f = 0.6$ and the porosity $n = 0.35$. According to the empirical formulae, the expected wave overtopping should be much smaller for rubble mounds than for dikes.

In the first simulation a very large value of mean discharge was recorded, very much larger than the expected one (15 times). The explanation is simple and it is related to one of the software limitations. The rubble mound structures are porous structures that allow water to flow through them, while energy is dissipated. Since SWASH measures the discharge that goes through a certain cross-section, all the water that flows through the breakwater is taken into account. This problem did not appear with dikes, because they were impermeable and introduced as a bottom variation, so there was only water travelling over it, and this was what was measured. Now there is water that overtops and water that goes through the breakwater, which is a kind of box with an average porosity for SWASH rather than 2 layers with different permeability. In fact, it was water elevation and waves inside the breakwater that were measured. Waves had reduced due to the lower porosity compared with water. The structure simply works as a sponge layer.

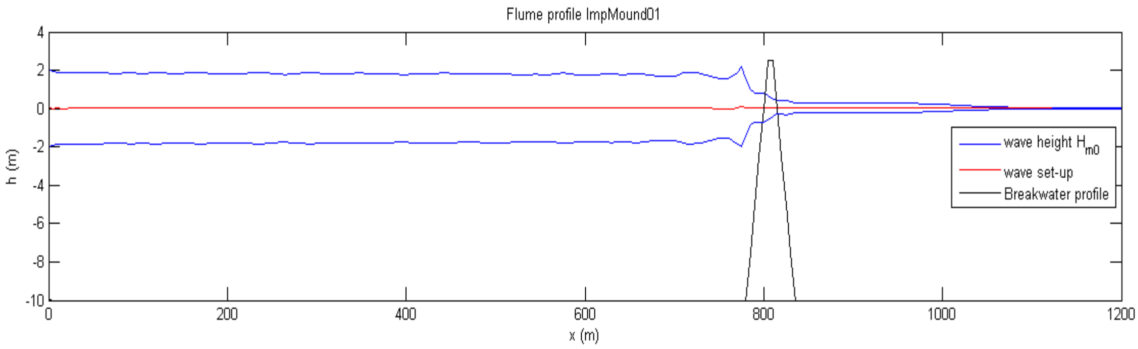

Figure 3.8. Profile of the flume with the breakwater profile (Imp_Mound01.sws), the mean water level (set-up) and the crest and trough limits of the spectral wave height.

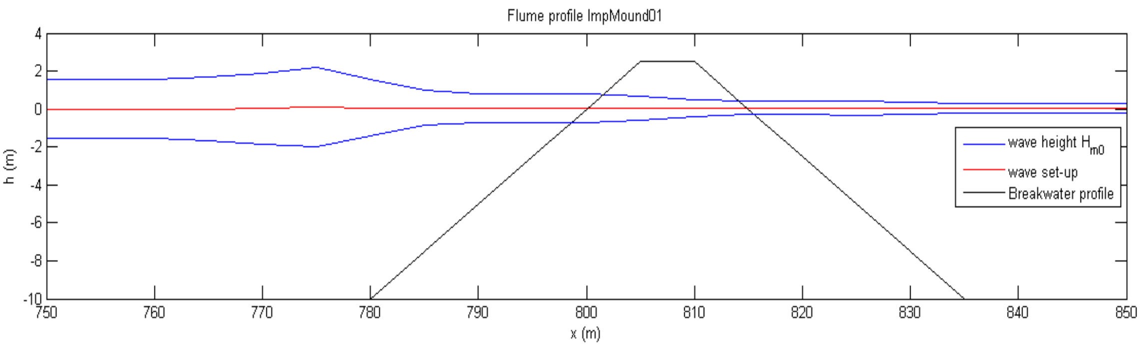

Figure 3.9. Profile of the breakwater with the waves going through (Imp_Mound01.sws). The waves break approximately 5 m in front of the toe, while they should do so on the first third of the slope.

Besides, due to the waves, here the net discharge and not the absolute discharge must be considered. It is important to remember about the oscillatory motion of waves, so water moves forward and backwards. If the net discharge is computed using an output vector, a value close to the predicted overtopping (about the 46%) is obtained. This includes the flow through the breakwater. In dikes it was not necessary to worry about the discharge vector, because over the crest there was water moving only in the same direction. This was checked and the same results were reached.

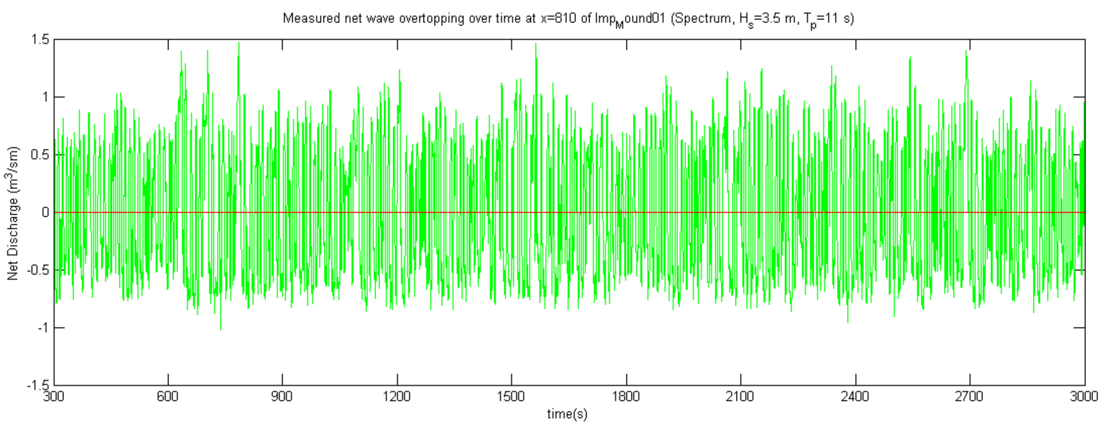

Figure 3.10. Recorded net discharge over time at the end of the crest (Imp_Mound01.sws). The positive values mean discharge in the propagating wave direction.

The software can not split the flow in these two components (over and through the breakwater), because SWASH does not consider the breakwater as a physical obstacle with some porosity that allows water to go through and stops part of it making it go over through splash or a water tongue. The software models the rubble mound as a kind of sponge layer in which the wave goes through and some energy is dissipated. As a result, there is no water travelling over the structure. From the measured water depth and absolute and net velocities the same absolute and net discharge are computed. In fact, the introduced structure height is only used (in combination with the structure and water porosities and the water depth) to compute an average porosity value to be used in the equations. As a result, SWASH does not consider the structure geometry (freeboard, slope, etc.) as a physical boundary as it is done with the bottom layer or as I did with dikes. One example of this is the location of the breaking waves (see figure 3.9). This will be commented in section 3.3, where it is explained with more detail how SWASH deals with porosity.

Nevertheless, the introduced breakwater has an impervious core and one layer of one-meter size rock placed on it, which means that water can only flow through the armour layer and over it. Assuming this stone size and this single rock layer, the armour width will be around 2 meters ($2 \cdot \mathrm{D_{n50}}$), so little water can go through. Thus, I think that it can be assumed that a considerable part of the water flow through the cross-section would be overtopping. However, this is a risky assumption since little is really known about permeable crests and its influence on overtopping.

Later, different breakwaters with a lower expected overtopping were simulated and much larger net discharges were measured. As a consequence, it is clear that the measured net discharge in SWASH cannot be trusted to be an indicator of the wave overtopping. The table 3.7 includes the results. The Eurotop prediction takes into account the width of the permeable crest (section 2.1.4).

Simulation	$\boldsymbol{R_c}$ $(\boldsymbol{m})$	**Slope**	$\boldsymbol{H_{m0}}$ $(\boldsymbol{m})$	$\boldsymbol{T_{m-1,0}}$ $(\boldsymbol{s})$	$\boldsymbol{\xi_{m-1,0}}$	$\boldsymbol{q_{Eurotop}}$ $(\boldsymbol{l/sm})$	$\boldsymbol{q_{SWASH}}$ $(\boldsymbol{l/sm})$	$\boldsymbol{q_{NN}}$ $(\boldsymbol{l/sm})$	$\boldsymbol{q_{NN,5\%} - q_{NN,95\%}}$ $(\boldsymbol{l/sm})$
Imp_Mound01.sws	2.5	1:2	3.23	15.42	5.36	38.1	17.45	195.4	21.39 - 2245
Imp_Mound02.sws	5	1:2	3.23	15.42	5.36	1.3	15.5	8.8	1.317 – 65.57
Imp_Mound03.sws	5	1:1	3.23	15.42	10.72	1.3	18.9	38.5	3.86 – 339.9
Imp_Mound04.sws	2.5	1:2	1.87	8.20	14.99	0.27	6.2	3.5	0.554 – 21.56

Table 3.7. Estimations of mean wave overtopping over rubble mound breakwaters with an impermeable core given by Eurotop empirical formulas, SWASH and Neural Networks. Only the incoming wave without reflection measured at the location of the structure toe has been considered.

The discharge measured by SWASH in the first 3 simulations hardly changes. Furthermore, the four dimensionless discharges (not included in the table) are within a very small range between $\mathrm{Q}_* = 7.74 \cdot 10^{-4}$ and $\mathrm{Q}_* = 1.04 \cdot 10^{-3}$, so it can be assumed that there is a kind of constant discharge value that depends on the wave properties and the introduced porosity in SWASH.

This list of new problems was thought to be overcome by modelling the impermeable core as it had been done with dikes. A more realistic model would result from this approach and the discharge over the core crest, which would be the discharge going over and through the permeable crest, could be measured. The impermeable core was described as a bottom variation and a porous structure over it representing the armour layer was added. Two different tests were performed: the first one with only the seaward slope covered by this permeable layer and the second one with the complete armour layer over the core. However, none of them worked. SWASH interpreted the emerged part of the core as an impermeable wall and nothing was measured beyond it. It is not know why it did not work, but it is thought to be related to how the software deals with the porous structures. Probably, SWASH did not understand that a porous structure was placed over a part of the bottom that is above the initial water level and remains dry during part of the time.

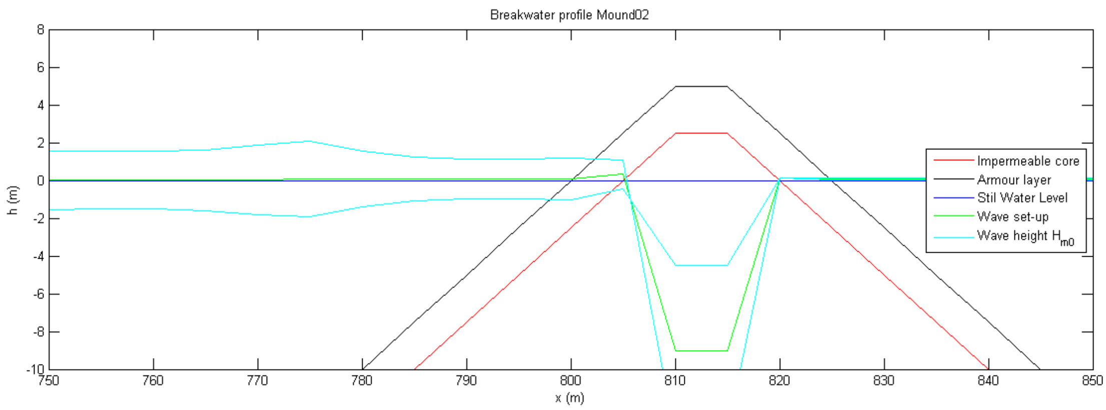

Figure 3.11. Breakwater profile with a thin porous structure (armour) placed over a bottom variation (impermeable core). The impermeable core is the same profile as one of the previously tested dikes. No agitation is recorded behind the structure. The strange values plotted inside the core correspond to the wave output values given by SWASH, which are non-sense in the dry points. The script file is Mound02.sws.

To sum up, SWASH models the breakwater as a kind of sponge box that dissipates energy of the waves going through rather than a set of big stones. The water body is continuous and there is no splash going over the breakwater or water tongue running up and down the slope. As a consequence, the only water particles that will go over the crest will be the wave itself if it is enough big. The net water discharge through the cross-section was measured, but it includes all the water, so it is not possible to distinguish wave overtopping discharge from the flow inside the breakwater. From the considerations made about our rubble mound breakwater cross-section, I think that the inner flow will be small, because of the thickness of the single rock porous layer over an impervious thick core. As a result, big part of the water flow should be overtopping, which is enhanced by the impermeable core, because water can not go through and cumulates (as commented in Eurotop). However, I did not manage to introduce a multi-layered structure with an impermeable core in SWASH, which would let me directly measure the discharge over it; research results do not agree how much water goes over and through the crest, so any hypothesis about this division is doubtful; and it is clear that SWASH does not model waves breaking on rubble mound breakwaters as we physically understand. Therefore, I conclude that this is not a good model to study wave overtopping on rubble mound breakwaters (yet).

3.3. Porosity in SWASH

SWASH models porous flow through the Forchheimer law (3.3), which is an improvement of the Darcy law by adding a turbulent term to the laminar one:

$$I = a \cdot u_f + b \cdot u_f \cdot \left| u_f \right| = -\frac{1}{\rho_w \cdot g} \cdot \frac{\partial p}{\partial x} \tag{3.3}$$

$u_f = n \cdot u =$ filter or averaged velocity

$$n = \frac{pore\ volume}{total\ volume} = porosity$$

The Forchheimer formula can be derived from the Navier-Stokes equations and it results in the following expression (Andersen and Burcharth, 1995):

$$I = \frac{\alpha \cdot (1-n)^2}{n^3} \cdot \frac{\nu}{g \cdot D_{n50}{}^2} \cdot u_f + \beta \cdot \frac{(1-n)}{n^3} \cdot \frac{1}{g \cdot D_{n50}} \cdot u_f{}^2 \tag{3.4}$$

The Forchheimer equation is only valid for stationary conditions and the coefficients α and β must be determined experimentally. Polubarinova-Kocina (1952) extended the Forchheimer formula by adding a local acceleration term and a new coefficient. However, the initial 2 coefficients are different in non-stationary conditions according to Van Gent (1993).

This brief introduction to porous flow summarizes what B. Mellink (2012) explained with more detail in his Master Thesis, where he referred to all the original publications.

A porous structure is introduced in SWASH through 3 input grids, in which the structure height above the bottom level, the grain size and the porosity are introduced. As it has been commented previously, porosity values smaller than 0.1 can not be introduced (because a power of 3 is at the denominator) and this value is considered to represent an impermeable wall. Water has got porosity 1. From the introduced structure porosity and height and the water depth from the surface to the structure height, an average porosity value is computed. This value will be used in the computations with the total water depth going from the surface to the bottom. So the structure height is only taken into account as a weight factor to compute the average porosity to be applied in the total water depth and the physical structure is ignored afterwards. Therefore, if there is a slope, SWASH will consider a varying average porosity along it (see figures 3.12 and 3.13), but not the slope as a physic boundary (as it does with dikes), because the bottom is below. As a result, a porous structure acts as a sponge layer that slows down the water flowing through it rather than an obstacle that diverts the flow (and thus, there is no overtopping and only a continuous wave).

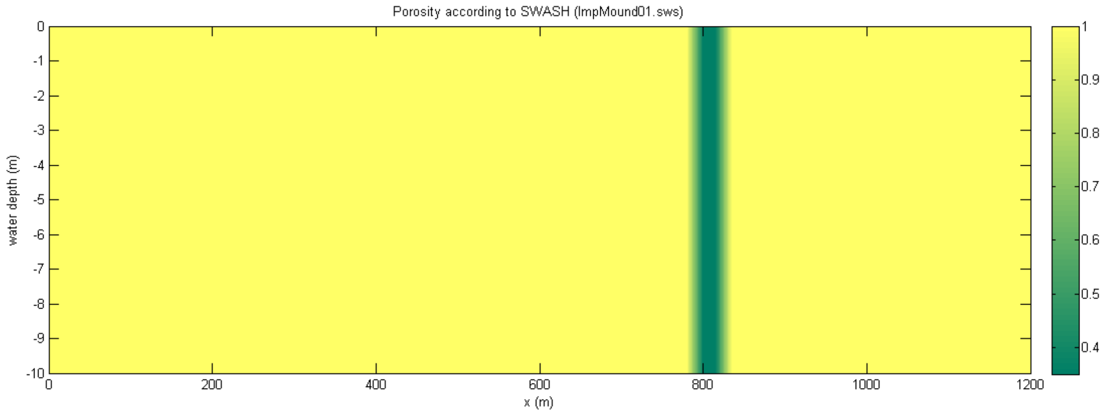

Figure 3.12. Porosity considered in the water body by SWASH along the flume in script Imp_Mound01.sws.

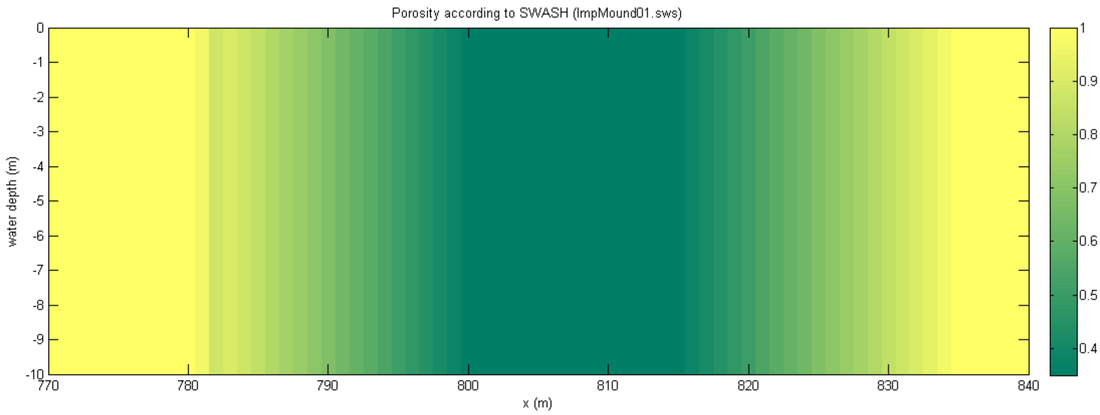

Figure 3.13. Porosity considered by SWASH at the breakwater location. It is constant in the entire water depth. A porosity grading can be seen along the slopes and constant values under the crest and where there is no structure. These values were used in Imp_Mound01.sws.

In addition to this, the porosity is only taken into account in the horizontal component of the momentum conservation equations. Hence, only the horizontal velocity component is damped by the porous structure, while the porosity does not affect the vertical component. This is quite an unreal situation, because it is assumed that the water particles can move up and down inside a porous structure as easy as outside without any kind of friction and energy losses. The consequence is that there is a continuous water surface even inside the porous structure, because it simply propagates slower in the horizontal plane.

In reality, there is a discontinuity in the water elevation inside a rubble mound breakwater (or any kind of permeable structure), because the water level inside can not follow the water surface variations from outside due to grain friction. Then, both flows are “uncoupled”, since the vertical propagation accelerations are different. Only close to the interphase zone, inertia of the incoming waves is enough strong to overcome internal friction and keep the inner surface variations.

As it can be seen in figures 3.14, 3.15 and 3.16, there is not a (fast) decrease of the water level or the wave height inside the breakwater.

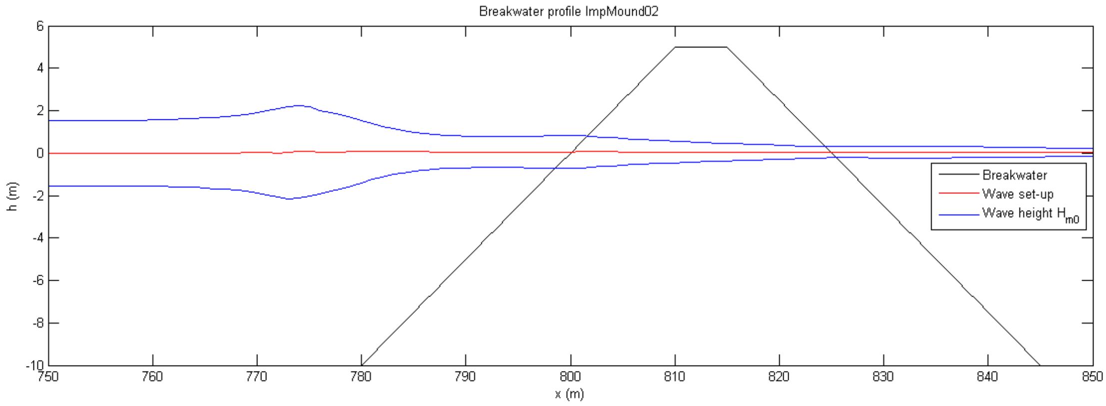

Figure 3.14. Waves propagating through the breakwater in simulation Imp_Mound02.sws. The blue lines represent the crest and trough of the recorded significant wave height. Only a progressive damping is observed.

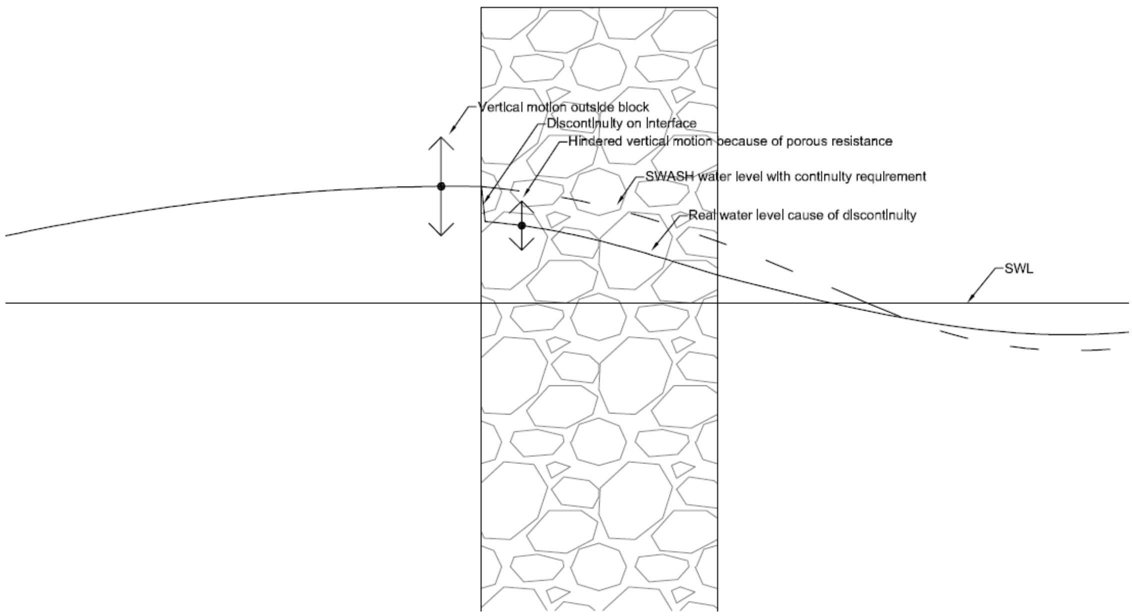

Figure 3.15. Difference between the real water surface inside the porous structure and the water level measured in SWASH. (Mellink, 2012).

What can be observed in all simulations with a porous structure is that the waves break between 5 and 10 m in front of the breakwater toe, which is not true. They should break on the slope at the point where the current wave height is larger than the maximum allowed by the water depth $H_{s,break} = \gamma_{break} \cdot h_{break} \sim [0.4 - 0.5] \cdot h_{break}$.

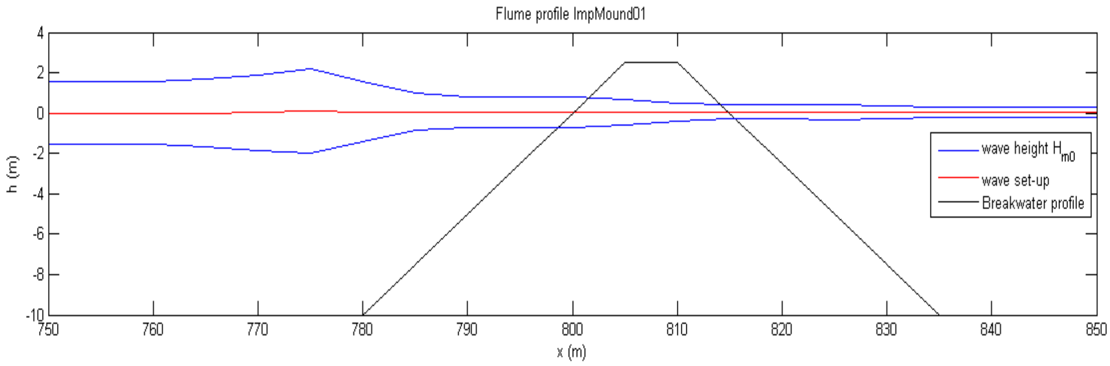

Figure 3.16. Wave breaking 5 meters in front of the breakwater toe (Imp_Mound01.sws). No run-up or overtopping takes place.

This contrasts to what happens with dikes, where the bottom variation reduces the water depth inducing wave breaking. The waves break where they are expected to do so. Just compare figures 3.16 and 3.17, where the same profiles and waves were simulated. This is a proof that SWASH does not consider the structure profile as the most important feature that interacts with waves outside the breakwater. Instead, it is its porosity.

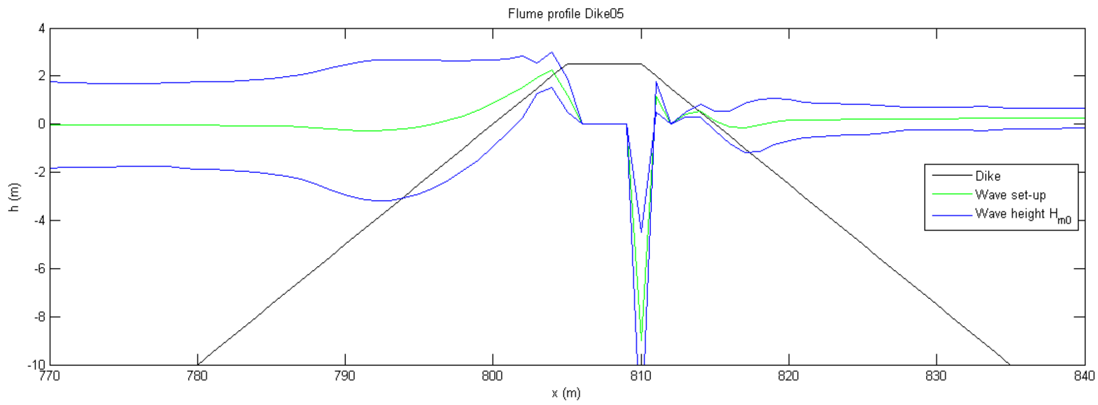

Figure 3.17. Profile of an impermeable smooth dike (Dike05.sws). Waves break on the slope at 4-meter water depth and run-up the slope. The recorded set-up includes the run-up water tongue. Some agitation is also recorded behind the dike due to large wave overtopping. SWASH is not able to give wave outputs where there is not always water, which explains the strange values shown in the plot.

For SWASH, the porous structure begins at the toe, where the porosity of the propagation medium begins to reduce. The reduction of porosity along the whole structure width increases the flow resistance and slows down the propagating wave. Part of it goes through and it is damped (transmitted wave), but the other part can not and water “accumulates” in front of the structure as it would do in a flow contraction rising the water level (reflected wave). This is how I would explain that the wave “notices” the structure “much earlier”.

Nevertheless, the (precise) numerical reason that explains it is not that clear. Probably it would be a mix of the lack of vertical damping, calibration of the transmitted and reflected wave, the way the structure itself has been modelled and the formulation of overtopping.

I could even say that part of the water that accumulates in front of the structure, and part of the wave volume that propagates through it without vertical damping, would be the overtopping volume that is missing.

B. Mellink in his Master Thesis (2012) studied the wave interaction with a porous breakwater. He did some small scale tests and compared the experimental results with a numerical model built with SWASH. Although he focused on wave reflection and transmission, he faced some problems with SWASH modelling that are thought to have the same origin. Therefore, his experience and conclusions are very important for this thesis.

Reflection and transmission was not calibrated in the tests performed in this thesis although if it is done, the computed wave overtopping on dikes may be more reliable and comparable to experimental data. The default parameters were not changed.

For a vertical porous structure, Mellink found that the experimentally determined Forchheimer constants gave a severe underestimation of the reflected wave and an overestimation of the transmitted wave in SWASH compared to the analytical and experimental data. The calibrated values were unrealistic, but gave a good fit. However, it is not sure if this is only needed for vertical structures or the same behaviour always takes place.

Mellink also comments a suggestion of Van Gent about the need of higher β values in SWASH. His hypothesis is that the interphase between the water and the porous medium is not well modelled, because vertical accelerations near the water surface behave differently in a porous medium. This would explain why there is not a discontinuity in the water surface. The transition between both phases is modelled in SWASH by a continuity formulation. This leads to a larger incoming wave, and thus, a smaller reflected wave and a larger transmitted one. This transition effect was found to be lower in wider porous blocks.

3.4. Conclusions

In this section the conclusions reached with SWASH are summarized.

Wave propagation is well modelled with SWASH. Some small dissipation takes place, but it can be explained by the physical behaviour of waves in intermediate water conditions. The wave height outputs given by SWASH ($\mathrm{H_{m0}}$ and $\mathrm{H_{rms}}$) match with the computed parameters through spectral analysis ($\mathrm{H_{m0}} = 4 \cdot \sqrt{\mathrm{m_0}}$ and $\mathrm{H_{rms}} = \sqrt{8 \cdot \mathrm{m_0}}$) and the water elevation variance ($\mathrm{H_{m0}} = 4 \cdot \sigma$ and $\mathrm{H_{rms}} = \sqrt{8} \cdot \sigma$). The measured significant wave height through wave per wave analysis is slightly smaller than the spectral value $\mathrm{H_{m0}}$, around $\mathrm{H_s} \approx 3.8 - 3.9 \cdot \sigma$, which is something already known. In order to avoid reflection, it is very important to introduce an enough big sponge layer at the end. A length of about 3 times the wave length will suffice. Otherwise a standing wave will take place.

The best way to model a dike in SWASH is with a bottom variation, because it is an impermeable bottom boundary and water can flow over it without any problem. Some roughness could be added as a bottom friction. Despite absolutely no calibration for friction and reflexion was done, the performed tests showed promising good predictions of wave overtopping. The measured discharges over the dike crest were always smaller than the predicted values using the Eurotop formulas for dikes (and also using Neural Networks), but they had the same “order of magnitude”, which is quite important when talking about overtopping. However, all measured results appeared to be below the 5% lower limit of Eurotop and Neural Network predictions.

One reason why the measured overtopping is always smaller is due to the fact that SWASH is not able to describe splash. Therefore, the “white water” or “spray” overtopping is not in the model, and so, the measured overtopping in SWASH does not include this part of the total overtopping, which is estimated to be less than 10%. Only the continuous water tongue that runs up the slope and overtops, which is called “green water”, is modelled by the shallow water equations. For this reason, the differences increase with low overtopping values in dikes with high relative freeboards, because a larger share of the total overtopping corresponds to “white water”. Besides the missing splash, differences increase because of the lower reliability of the formulas for very small overtopping discharges. As a recommendation, it would be nice to test more different situations, analyse first the reflected wave to see if the magnitude is right and if it affects the overtopping discharge, and add and calibrate friction on the slope due to grass or any kind of smooth revetment.

Furthermore, other hypothesis related to the software modelling itself could explain this systematic underestimation. The first one could be related to some possible limitations of the modelling of wave breaking on steep slopes compared to mild ones, because Suzuki et al. (2011 and 2012) showed that good estimations of wave overtopping were predicted in a very shallow foreshore with a wall at the end. However, they also comment in Suzuki et al. (2011) that some discrepancies were seen with small overtopping volumes in combination with a rapidly varying topographic feature, such as a sea-wall. They considered that the reason may be the inability of depth integrated models to represent the water tongue of the overtopping waves. The latter could be the second reason, since their conclusions are in line with the fact that there is an abrupt variation between the horizontal bottom and the dike slope (more than what they tested). Maybe, SWASH does not model it accurately. Besides, this sharp edge coincides with a computation grid node, which may have an influence on the computations.

SWASH can not predict overtopping in rubble mound breakwaters. In fact, it can not model overtopping in any kind of permeable structure. Because of the way how porous structures are dealt in SWASH, overtopping simply does not occur. The wave goes through the structure and it is damped and part of its energy dissipated, but the flow is not diverted due to the interference of the structure. The vertical velocities of the wave are not affected by the porosity, so there is a continuous smooth water surface. The structure itself is not modelled as a kind of boundary that interacts with waves, since they do not break on the slope, but before reaching the toe of the structure. As a consequence, SWASH must be improved in this field by adding the porosity to the vertical component of the flow equations and rethinking how the structure geometry is introduced in the computations as a limit of the water depth. Somehow, both inner and outer flow should be split, but at the same time linked by a small interphase zone. Furthermore, at the moment the discharge and flow velocities can only be measured as an average output value of the full (or layer) cross-section of the flume or the structure, but not over a certain height.

4. Conclusions and recommendations

In this chapter the conclusions reached in this thesis are exposed. Besides, some recommendations about further research or future improvements of SWASH are given.

4.1. Conclusions and discussion

From the study of the available methods and the critic assessment done in chapter 2, it is clear that Eurotop formulas give quite accurate good predictions of wave overtopping on simple rubble mound slopes. However, the definition of the freeboard that should be used when the crest is permeable is not clear yet. Neural Networks can be a powerful tool to compute overtopping discharges in very complex structures, but the reliability of the results depends very much on the availability of close situations in the database.

Little research had been done for breakwaters with a berm before the publication of the Eurotop manual. As a result, in this manual a berm is modelled in the same way as in dikes. However, the behaviour of a permeable berm combined with an armour layer is very much different than an impermeable one with a smooth and impervious slope. Furthermore, for reshaped berm breakwaters the mean slope was used to compute wave overtopping, which is not realistic. Neural Networks can not be used either, because a few structures with a berm were included in the database. One solution is using the formula developed by Lykke Andersen and Burcharth for berm breakwaters, but only one slope was tested for non-reshaping breakwaters and the equation was obtained through a multivariable fit, which resulted in a very complex expression that is not handy to use.

Krom (from the work of Lioutas) and Sigurdson and van der Meer have proposed quite recently different improvements to the Eurotop formulations starting from different approaches. However, they have not been compared and we do not know which of them performs better.

At the moment, SWASH can not be used as a tool to predict wave overtopping in rubble mound breakwaters, because it is not able to model this phenomenon in porous structures. In contrast reflection and transmission could be studied with this software after calibration according to Mellink. This was not done in this thesis though. As it has been commented in chapter 3 with more detail, porous structures are not modelled in SWASH as a physical obstacle for water flow, but as a dissipative box that damps the wave going through. In fact, incoming waves are stopped and they break before reaching the structure toe. As a result, the flow is not diverted upwards as overtopping and the wave is only slowed down and the wave height reduces. The breakwater geometry does not interact with the waves, because it is not a boundary for them that induce wave breaking. It is only an input to determine the average porosity value to be used in the Forchheimer equation that describes water flow through a porous media considering the entire water column. Furthermore, the porosity only affects the horizontal component of the velocity. Thus, the vertical water motion is not hindered by the porous structure, which is not realistic. For these reasons, it is clear that SWASH needs further development for porous structures.

4. Conclusions and recommendations

Despite not working with permeable structures, SWASH is able to model wave overtopping on impermeable dikes giving “promising” results although improvements must be made to get good results. The measured discharges (13) are close to the predicted ones by Eurotop and Neural Networks, they have the same order of magnitude and an exponential function can be fitted with $\mathrm{R}^2 = 0.8746$. Nevertheless, the measured trend function is smaller than the 5% lower limit of both methods. The reflected wave has not been analysed and compared to any analytical or experimental method, therefore, its influence in the results is unknown. The recorded “interaction” between the dike and the waves fits what really happens, because in low crested dikes, a large sheet of water running up the slope has been recorded through the wave set-up (“green water” overtopping), waves break on the slope when the critical depth is reached and some wave agitation due to overtopping is measured behind the dike. There are several reasons that could explain why SWASH underestimates wave overtopping.

In first place, SWASH is based on the Navier-Stokes shallow water equations, and thus, only a continuous water body can be modelled. The consequence is that splash cannot be described, so “white water” overtopping is not taken into account. This is a reason why the measured wave overtopping in SWASH is always smaller than the predicted one, because a share is (always) missing. This also could explain the larger differences found for low overtopping values, because in these cases spray overtopping has more weight. However, overtopping due to splash is always visually overestimated, it represents approximately less than 10% of the total discharge and it is only important in very violent breaking.

The second reason is the low reliability of the empirical formulas and Neural Networks for low overtopping discharges. This would explain the larger differences for this range of values, but not the systematic underestimation.

In third place, the mathematical computations done by SWASH on the slope may not be enough accurate, which has led to lower overtopping discharges. The causes are not clear though. There may be some limitations or drawbacks in wave breaking on steep slopes. Furthermore, quite an abrupt bottom variation is found between the horizontal foreshore and the dike slope. This sharp edge may not be well modelled. Perhaps a finer mesh is needed to describe better the process around the edge and the slope. It may not be a good idea to compare the results of the simulations performed in this thesis with the very gentle foreshores tested by Suzuki et al., because both situations are globally very different.

SWASH is a software tool that was developed for a very wide range of hydraulic situations. As a consequence, it must use the same code and procedure to deal with absolutely different situations. In addition to this, more emphasis was put on describing hydraulic processes rather than boundary conditions and relations with certain structures. As a result, quite a robust model was built with many applications such as wave propagation, which works quite well in 1D and 2D models, but it lacks of detail on certain issues such as flow through porous structures, breakwaters description and interaction with waves and splash. Probably, for multi-layered structures with many details such as a toe, a berm or a crown wall, specialized software would give much better results at this moment. Besides, only in situations with mild slopes (in relation to the incoming wave length) such as beaches or very gentle sloping dikes, where there are no abrupt geometry variations, and where the largest share of wave overtopping corresponds to waves running up the slope and not to water particles that travel over the crest due to splash, SWASH will give accurately good predictions to wave overtopping. In other situations where splash is more important or the geometry presents "sharp edges", it is advised to use another type of model, e.g. VOF or SPH.

4.2. Recommendations

Many recommendations can be done about aspects of SWASH that need to be improved in the future and about how to continue verifying if it is a good model.

SWASH has been roughly verified for impermeable dikes in this thesis, but only a few cases were tested and the simulations were not calibrated for friction. Therefore, it is recommended to test more dike profiles and/or wave conditions to get more results. These new simulations should include situations with a low relative freeboard (and large expected relative overtopping) to enlarge the range of tested parameters. Besides performing more tests, the amount of reflection should be validated with existing analytical, empirical or physical models, and if it were needed, the model should be calibrated.

The results show a nice trend of measured discharge, but there is still room for improvement to correct the current underestimation of the wave overtopping. Otherwise, the range of situations in which it could be used as a valid tool will be quite limited. Two main recommendations are given:

- The numerical model should describe splash either in the BREAK mode or in a new one to be able to give precise predictions of wave overtopping in any impermeable slope.

- Improve the "behaviour" of SWASH in abrupt geometry variations when modelling wave overtopping. At the moment, it looks like the software cannot properly deal with them. However, it is a finer mesh that perhaps is necessary to describe these phenomena. It is advised to look first at the transition points and sharp edges instead of at the steep elements, because the problem is related to variations from horizontal or very gentle foreshores to steep slopes or vertical walls in a short distance. When the length of the first element is very much longer than the second one, the differences decrease.

The way how SWASH deals with porous structures should be clearly improved. Mellink already proposed some measures about that (which I reaffirm) and new ones have been added:

- Porosity should be included in the vertical component of the equations that model the water motion, so that the vertical movements are damped inside the porous structure. This recommendation was also given by Mellink and it is supported with the results of this thesis.

- The interphase between water and the porous structure should be improved in such a way that there is not a (very) smooth transition between the water surface inside and outside the breakwater. This zone would have to link an inner and an outer flow that seem to be "uncoupled" rather than "continuous". Furthermore, part of the water flow should be diverted into overtopping when the waves impact the structure. Probably, this would be the most difficult recommendation to implement in SWASH. Mellink already thought about improving the interphase, but the problem is more evident and important when studying wave overtopping, because it can not be simply overcome by calibrating the reflection and transmission coefficients of the Forchheimer formula as he did.

- Related to the previous one, the structure geometry should act as a boundary bottom. Despite being a permeable body that allows waves to propagate through, the breakwater slope must induce wave breaking on it, because it limits water depth. The porous structure should be modelled in a closer way to an impermeable bottom condition to deal with the water flow outside it.

- In order to be able to describe wave interaction with rubble mound breakwaters, SWASH should be able to model multilayer porous structures. It could be done by superposing two (or more) grids of the commands POROSITY, PSIZE and HSTRUCTURE. For example, the first one would represent the core and the second one, the armour layer. Another alternative could be developing the idea that was previously tried without success, which consisted on placing a porous structure over an impermeable core described by a bottom variation. However, first it would be necessary to solve the problems that have appeared and model water flow through an emerged porous structure. On one hand, the first idea would need to further develop the modelling of water flow through a porous media following the previously given recommendations. On the other hand, the second one could be a way to overcome some of the problems faced in SWASH without building a more complicated model, provided that the global performance was found to be good enough.

- SWASH can only measure discharge through the complete cross-section, but not on a part of it. Then, the software could not measure wave overtopping on rubble mound breakwaters in case the found problems with porosity were solved. For this reason, it would be a good idea to create a new output parameter that is able to measure discharge over a certain specified height.

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Appendices

The thesis report includes two appendices:

- Appendix A. Summary of the SWASH scripts
- Appendix B. Summary of the Matlab scripts

The appendix A is a summary of the majority of the SWASH simulations performed during this work. The input parameters and results are commented in detail. As a consequence, this appendix is very long and it has 108 pages. For this reason it has its own table of contents and lists of tables and figures.

The summary of SWASH scripts was written as a laboratory notebook at the same time tests were run. Therefore, the main purpose was to collect and analyse the results as soon as the simulations were performed and give a useful tool to anyone starting working with SWASH, so he could benefit from the acquired experience. The partial conclusions written there are nothing more than ideas and thoughts and should not be considered as strong arguments. All the valid explanations and conclusions are commented in chapter 3.

The codes of 3 SWASH scripts are also included at the end, but the input files are not.

In appendix B the most important used Matlab programmes are commented. They are either scripts or functions that analyse the wave records obtained with SWASH or compute wave overtopping to speed up calculations.