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Overtopping over a real rubble mound breakwater calculated with SWASH

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TREBALL FINAL DE GRAU

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ABSTRACT

The main goal of this project is to verify the numerical model SWASH when dealing with overtopping prediction by comparison of full scale measurement results to results of numerical model testing. For that purpose, the harbour in Zeebrugge (Belgium) has been considered; there, several storm events have been measured in a rubble mound breakwater section. In order to get a proper reproduction of these, a stepwise process has been carried out:

First, wave propagation has been tested in SWASH along a flat flume. Results have shown good performance of the software.

The next step was to introduce a smooth impermeable dike on a shallow scenario. Also in this case, good performance has been observed; propagating waves reaching the dike are affected from reflection and they are able to run-up the slope, getting overtopping records in some cases.

Finally, to get a proper reproduction for the real case in Zeebrugge, porosity was added to the dike in the prior step. It has been concluded that SWASH is not able to model breakwaters only by means of a porous structure; therefore, an impermeable core is required to allow simulation of real phenomena.

After conducting some tests over an impermeable core with an outer porous layer, it has been proved the software is not able to reach reliable levels for waves running up the slope. Strong dissipation is taking place within the porous layer, so that SWASH is concluded to underestimate overtopping prediction for rubble mound breakwaters.

Key words: *Wave overtopping, SWASH model, Zeebrugge, breakwater, Eurotop, numerical wave modelling, run-up, porous structure, pore pressure attenuation, antifer blocks.*

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INTRODUCTION

Sea defence structures are built in order to protect the area behind them from incident waves coming from oceanic waters. However, this protection is not always enough and failure mechanisms can occur.
It has been observed in different port locations that water is sometimes crossing to the landward face of their breakwaters, so that overtopping discharges can be recorded during storm episodes. About this problem, research is still carried out in order to enhance design criteria for dikes and breakwaters in harbours.

Amongst several available tools, numerical models are used to study wave behaviour when approaching the shore and their effects when hitting different types of structures.

The present project is going to focus on the analysis of one of these numerical models, the so-called SWASH, which has been recently developed in the TU Delft. SWASH performance has already been verified for several simulation processes (wave propagation, breaking waves, wave run-up and overtopping over impermeable structures, etc.); however, when it comes to wave overtopping on partially permeable structures, the model has not shown proper results yet. Therefore, further investigation is to be done in order to find out what is the problem when dealing with that scenario.
The main goal of this study is to test how this software deals with overtopping discharge prediction in a specific partially permeable breakwater structure. By considering the real case of the Zeebrugge harbour, in Belgium, measured data collected in there is going to be compared to the output data obtained by SWASH through simulating the same scenarios. Thus, conclusions are going to be drawn according to the performance of the concerned model.

Along this document, prior knowledge of the reader about wave propagation and breaking principles is assumed. For further insight, the reader is referred to Holthuijsen (2007) [6].

The document is organised as follows:

- In chapter 1 the problem is introduced and described. The project objectives are defined.
- Study site and full scale measurements for validation of numerical models are described in chapter 2.
- Application of the SWASH model in the presented scenario is addressed in chapter 3. In there, a brief description on the basic computation principles for the model is given, followed by the validation against the corresponding real data of wave overtopping. The effect of reflected waves in overtopping discharges is also analysed.
- Finally, conclusions and recommendations are discussed in chapter 4.

1

PROBLEM APPROACH

Throughout history humans have always benefited from water in a broad sense, using it extensively whether for drinking, fishing or navigation, amongst others. It is for this precisely reason that nowadays, about half of the world's population is settled in areas lying very close to the shore. This fact, despite its obvious advantages in water accessibility, deals also with some risks.

Around the coastlines of Europe and elsewhere, the risk of flooding poses a threat to present and future socio-economic activities. Lowland lying areas, towns and transport infrastructure are often protected by defence structures against flooding or against erosion by waves and storm surges.

The Netherlands is one of the best places to exemplify such situation. In a country where about 60% of its surface is flood prone, dunes and dikes protect part of the land which is located below the sea level. Failure of these structures may lead to damage or even loss of life, so they have to be designed according to safety standards by law.

Although at present about a third of all flood defences do not comply with the current standards yet, dams, dikes and coastal defences in the Netherlands have never been stronger: the probability of encountering floods from rivers or from sea has substantially declined since the last flooding episodes. Research is still carried in order to enhance knowledge in the field.

Figure 1. Wave overtopping as a failure mechanism for sea defence structures

There are different failure mechanisms for such defence structures. When it comes to wave overtopping (Fig.1), there are still some gaps in knowledge and disagreements over the prediction of overtopping discharges, so that safety design standards have not been accorded yet. This kind of failure mechanism is of principal concern for structures constructed against flooding, structures that are termed sea defence.

Over the last years, there have been significant improvements in overtopping discharge predictions and a large amount of data is available from different research institutes and universities; however, since waves approaching the shore will behave in a very different way depending both on their own characteristics and on the coastal structure's features, it is tough to properly compile all the data.

In order to give a single design method, it is of major importance to fully understand the process. As an attempt to do that, researchers from The Netherlands, in cooperation with other Agencies from the UK and Germany have developed an Assessment Manual to deal with Wave Overtopping of Sea Defences and Related Structures; this manual is the so-called EurOtop (2007) [4]. On it, the concerned process is thoroughly introduced and different methods are presented to compute predicted values, as well as the uncertainties they deal with.

The present project is going to focus on the analysis of one of such computational methods, specifically on one model that has been recently developed in the Dutch university of TU Delft, the so-called SWASH. This software is going to be introduced and tested comparing with real data obtained from fieldwork in a real scenario.

1.1 WAVE OVERTOPPING

Wave overtopping is the average discharge per linear meter of width, q (in m3/s/m or in l/s/m). It occurs when waves come to the coast and their run-up level is higher than the crest level of the shoreline structure; then water crosses from the seaward to the landward face.
Wave run-up level, given by $R_{u2\%}$, is defined as the height exceeded only by the 2% of incident waves (Fig.2).

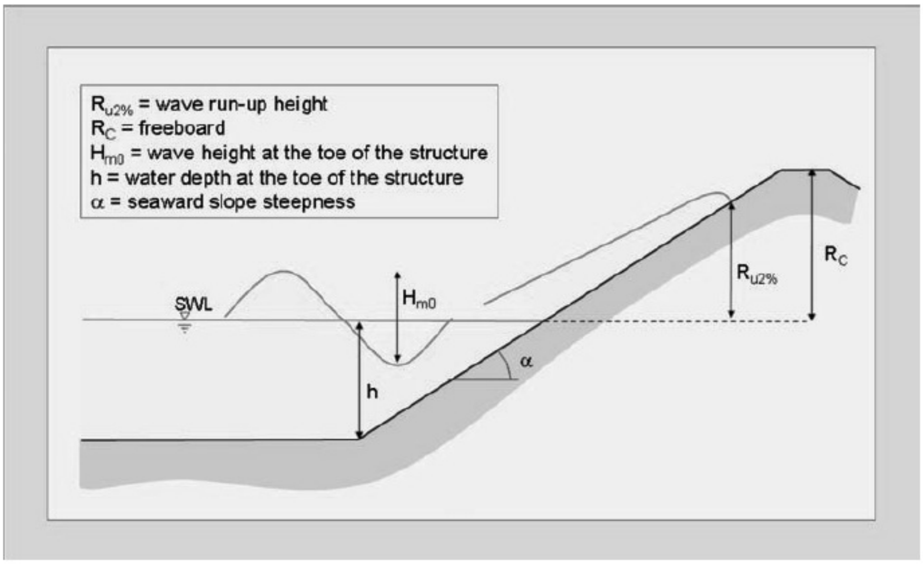


Figure 2 Wave run-up. Source: EurOtop

In fact, a distinction can be done between three different types of this process, according to the way the water passes over the structure:

On the first place, as stated before, water can pass through because of the high run-up level of the waves in the sea. Then it forms a continuous sheet of water over the crest, and it defines the “green water” overtopping case.

A second form takes place when the waves break on the seaward face of the structure, and it causes such a volume of splash that many droplets are then carried over the wall as a consequence of their own momentum or due to the onshore wind; this is termed “white water” or spray overtopping.

Last and less important, additional spray may also be generated by the wind blowing directly on wave crests. Although this spray is not even classed as overtopping and it is usually neglected when making prediction, its effects should be taken into account; even small volumes of spray can, for instance, substantially reduce visibility on highways near the shore. Spray volumes increase under strong winds.

Overtopping is not a constant process of water passing through the structure; it is instead a random process, both in volume and in time. Lower waves will not reach the crest height, so they are not going to produce any overtopping volume.

It can be seen below (Fig.3) an overtopping measurement recorded during a 200s period, showing this irregular nature. However, an averaged constant discharge is usually used (q) so that it allows easier measurement and classification.

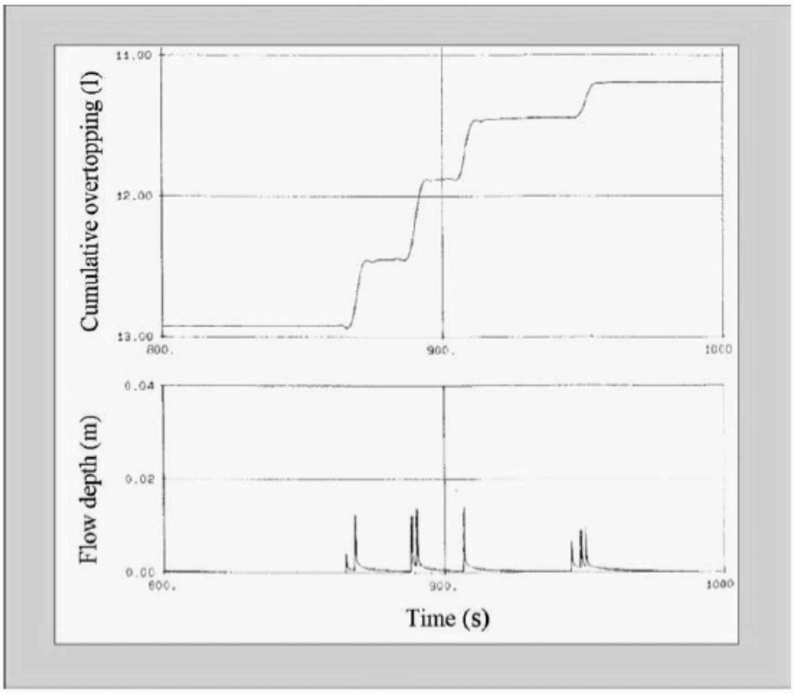


Figure 3. Wave overtopping measurement showing the random behaviour. Source: EurOtop

According to the mean value overtopped different consequences can be predicted. Nevertheless, this constant discharge does not describe the real behaviour of wave overtopping, so it does not give us information enough when determining the possible damage to a certain structure. In order to evaluate that, we should know the number of waves that will overtop, and the volume pushed landward by each.

To deal with this whole prediction it is necessary then to take into account the volume distribution per overtopping wave, and this is why different methods have been developed in order to present guidance for the assessment of structure design.

1.2 COMPUTATIONAL METHODS

In the EurOtop manual such computational methods are presented.

Different behaviours are going to be observed from waves depending on their own characteristic and the type of structure they are facing to. They will not have close responses; therefore, it is important to be aware of the situation to deal with when studying overtopping and when choosing the working methodology.

1.2.1 Empirical models

On the first hand, a compilation of formulae is presented based on empirical data. Very often this empirical formulae is only applicable for principal type of structures, such as smooth slopes, rubble mound structures or walls. When there is need to cover less simple situations, alternative methods should be used.

1.2.2 Neural Network tool

This method, also empirical, requires a large amount of data to get a reliable prediction. Its basis consists on introducing a number of input parameters according to the desired modelling scenario and from them the network checks its previously registered information looking for similar situations; the obtained output values are based on that previous registry. The large amount of possibilities in combining input parameters allows handling with a wider array of structure configurations than empirical models do. Therefore, the more collected data, the more accurate the model is, and this was precisely the main reason that led the European CLASH project starting.

CLASH project was released to help uncertainties in analysis and to improve prediction methods for use of coastal engineers when it comes to wave overtopping; it mainly consists on the performance of an extensive amount of wave overtopping tests resulting in a broad database of measured values, allowing this way the creation of the CLASH network.

Empirical based data turns out to be not sufficient sometimes due its limitation to a relatively small number of structure configurations. Hence, when it is not possible to provide empirical data or reliable results, other methods are also available, albeit less simple.

1.2.3 Physical modelling

It allows determining overtopping discharges for arbitrary coastal structural geometries through the construction of a scaled model simulating the real scenario. The obtained results should not be taken as accurate values, but they rather provide an order of magnitude to assess discharges.
Special care should be taken in scale effects when reproducing the real situation to avoid incorrect prototypes leading to wrong results.

1.2.4 Numerical models

As aforementioned, empirical data is sometimes restrictive and it doesn't cover every existing scenario. Apart from the already stated physical modelling, other alternative methods have to do with numerical models. Those models, in theory, can be configured for any realistic situation; they are able to simulate wave behaviour in accordance to their physics, as well as their interaction when hitting coastal defence, whatever their features are.
In order to perform these modelling accurately, lots of physical processes should be accounted for, which makes it very hard nowadays to develop a single model meeting the whole criteria in a computationally effective or economical way. However, some model types have been developed meeting part of the overall criteria, falling essentially in two principal categories:

- Models based on solving Navier-Stokes equations

- Models based on solving the nonlinear shallow water equations (NLSW), which are derived from the Navier-Stokes equations and simplify the mathematical problem considerably.

It is in this very last category where SWASH, the model in which this project focuses, can be classified. Its principles and applicability will be further discussed below in this document.

None of the above methods is going to provide us with an exact result; it is important to have in mind that they will deal with uncertainty instead.

1.3 SWASH

Starting from the Navier-Stokes governing equations in fluid mechanics it is possible to get the simplified nonlinear shallow water (NLSW) equations by integrating the first ones over the flow depth; NLSW models are therefore more efficient, albeit less sophisticated.
NLSW equations do not have an exact solution yet, so that numerical methods are used in order to approximate them. To do that, several models have been developed, each with its own characteristics and applicability.
It is in this context where SWASH model has been created.

SWASH (an acronym of Simulating Waves till Shore) is a non-hydrostatic wave-flow model that has been recently developed in Delft University by Zijlema. It is intended to be a numerical tool for simulating and predicting transformation of waves from offshore to the shore in order to study the phenomena occurring in coastal waters, where rapidly changing conditions take place within the scale of a wave length. Hence, SWASH is suitable for a wide range of simulation scenarios in shallow and intermediate water.

As said, the governing equations for this model are the nonlinear shallow water equations. Moreover, as a non-hydrostatic model, the software also includes a non-hydrostatic pressure term in the momentum equation in order to relax the effects of the pressure simplifications when integrating. Therefore, the simulation of broken waves and wave run-up amounts to the solution of the NLWS equations for free-surface flow in a depth-integrated form.

Such models are still being explored, refined and validated, but they are gaining recognition because of their numerical robustness, simplicity, ease of use and economy, at the same time that presenting the same capabilities from other more complex methods.

For further insight in the physics and mathematics of the model, the reader is referred to the SWASH website [13] or to the paper by Zijlema *et al*. (2011) [17].

1.3.1 Main features

The current version of SWASH is 2.00. This software is on ongoing development to keep enhancing its skills and performance. So far, the model accounts for a large amount of physical phenomena such as frequency dispersion, shoaling, nonlinear wave-wave interaction, wave breaking or wave run-up amongst others, and it is able to predict numerous outputs as surface elevation, velocities, pressures or discharges.

More details about how the software works are going to be shown along this document, as the model is applied to different scenarios.

1.4 PROJECT OBJECTIVES

As stated before, SWASH is in current development, so that it still needs to be validated for some situations and phenomena.

Martínez Pés in his thesis (2013) [8] compared the values obtained when computing overtopping discharges in SWASH to the ones resulting from the EurOtop empirical formulae. The main goal was to test the performance of that new software in order to verify its reliability in determining such volumes, focusing that study in rubble mound breakwaters

The present project is also aimed to work in a similar direction; the main objective is also testing the performance of SWASH when solving overtopping problems; in this case, however, the validation is going to be based on comparisons with real data collected within the framework of the already mentioned CLASH project.

To do that, a real harbour location is going to be considered. In there, CLASH tests have been carried out on full-scale measurements so that real data is available about overtopping discharge volumes. The regarded location is provided with rubble mound breakwaters; hence, the present study is going to be focused in this type of sea defence structures as well.

It has to be said that SWASH is provided as an open source code, therefore it is a free software which can be downloaded from its own website (http://swash.sourceforge.net). That makes it more accessible than other similar models at the same time it encourages everyone to further improve the science and configuration of the model.

2

CLASH PROJECT: STUDY SITE AND COLLECTED REAL DATA

The CLASH project (Crest Level Assessment of Coastal Structures by Full Scale Monitoring, Neural Network Prediction and Hazard Analysis on Permissible Wave Overtopping) was founded by several partners from EU member states in order to investigate and assess wave overtopping. As stated before, one of the main reasons that boosted this project was the lack of widely applicable and safe prediction methods for sea defence design.

Within the CLASH project, more than 10,000 overtopping tests were performed from January 2002 until December 2004, whose results were collected in the so-called CLASH database.

The present document is going to refer to the full-scale measurements carried out within the CLASH framework at the breakwater of the Zeebrugge harbour (Belgium). The overall information about the concerned study site has been taken from the original report within the CLASH project, by Geeraerts and Boone (2004) [5].

2.1 RUBBLE MOUND BREAKWATERS

As stated, this project is going to focus on the overtopping in a specific type of sea defence structures, the so-called rubble mound breakwaters. Therefore, as a starter, it would be wise to introduce such kind of construction:

Figure 4. Rubble mound breakwater protecting a harbour from incident waves. Source: www.baird.com

Rubble mound breakwaters consist of layers of quarried rock fill protected by rock or concrete armour units designed to resist wave action without significant displacement. That forms rough porous slopes which dissipate a proportion of the incident wave energy in breaking and friction. This disposition is more commonly used in areas where harder rock is available, since their cost of transportation is high.

Rubble mound revetments can also be used to protect embankment; thus, it leads to make a distinction when it comes to the permeability of the core. The armour rubble mound slope is very permeable, so that waves will easily penetrate within the layers and, as aforementioned, it will dissipate energy. However, a different behaviour is observed when dealing with a non-permeable core; then up-rushing waves cannot infiltrate into the structure, making the water to accumulate in the core layer and consequently causing an increase of the wave run-up level.

In the kind of structures where an impermeable core is encountered, it has to be highlighted that wave overtopping has been often more important than wave run-up, so that the crest height has been designed in order to assure allowable levels rather than to prevent them.

2.2. ZEEBRUGGE

2.2.1 Layout and measurement station

- Field site

The Zeebrugge harbour is located on the eastern part of the Belgian Coast. Figure 5 sketches the layout of that port, which is sheltered by two rubble mound breakwaters on the outer part. In the figure, the location of the CLASH measurements is indicated with an arrow.

The slope of the breakwater is 1:1.5 (1:1.4 where the measurements take place) and it has an armour layer consisting of 25 ton grooved cubes which are somewhat flattened (Height/Width=0.85). The core consists of quarry run 2-300 kg and a filter layer made of rock 1-3 ton.

The breakwater is 20m high, with the crest level at Z=12.4 (theoretical design level). The design conditions are a significant wave height Hs = 6.20 m, maximum peak period Tp= 10 s, and water level Z + 6.76.

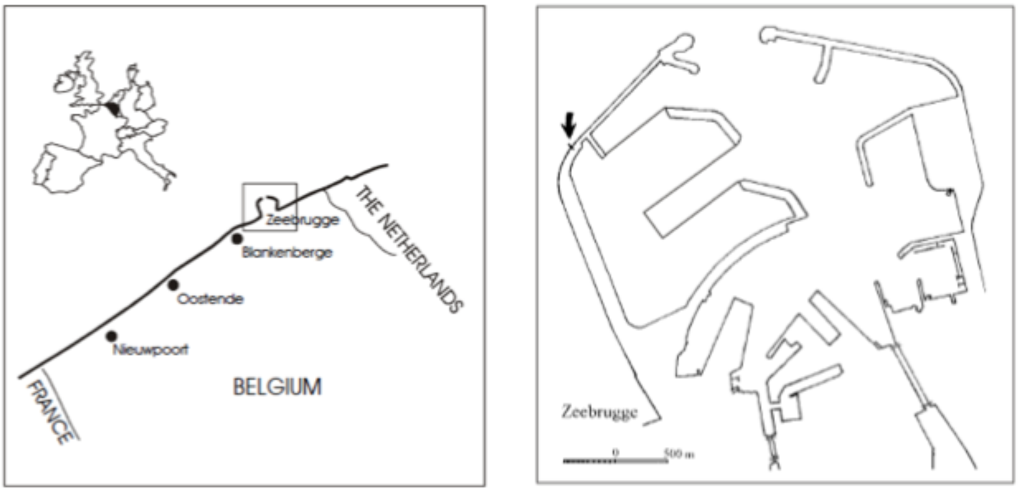


Figure 5. Location of the field site at the Zeebrugge harbour, in the Belgian North Sea Coast

Around the arrow area over the western breakwater, two cross-sections are to be considered, the ones that are instrumented. They are spaced 140m from each other (Fig.6).

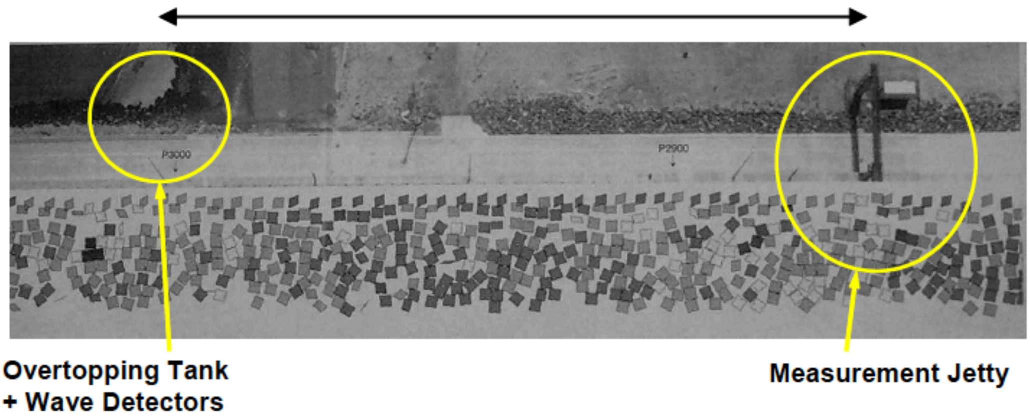


Figure 6. Instrumented cross-section on the field site

On the right, a measurement jetty is going to register wave conditions through several instruments placed in there, whereas some wave detectors and a wave overtopping tank on the left cross-section are going to inform about the overtopped volumes beyond the structure.

- Cross section 1: measurement jetty

The jetty is 60m long, constructed on the top of the breakwater it is supported by a steel pile on the seaside and several concrete columns. The required instruments are placed over the cross section (Fig.7).

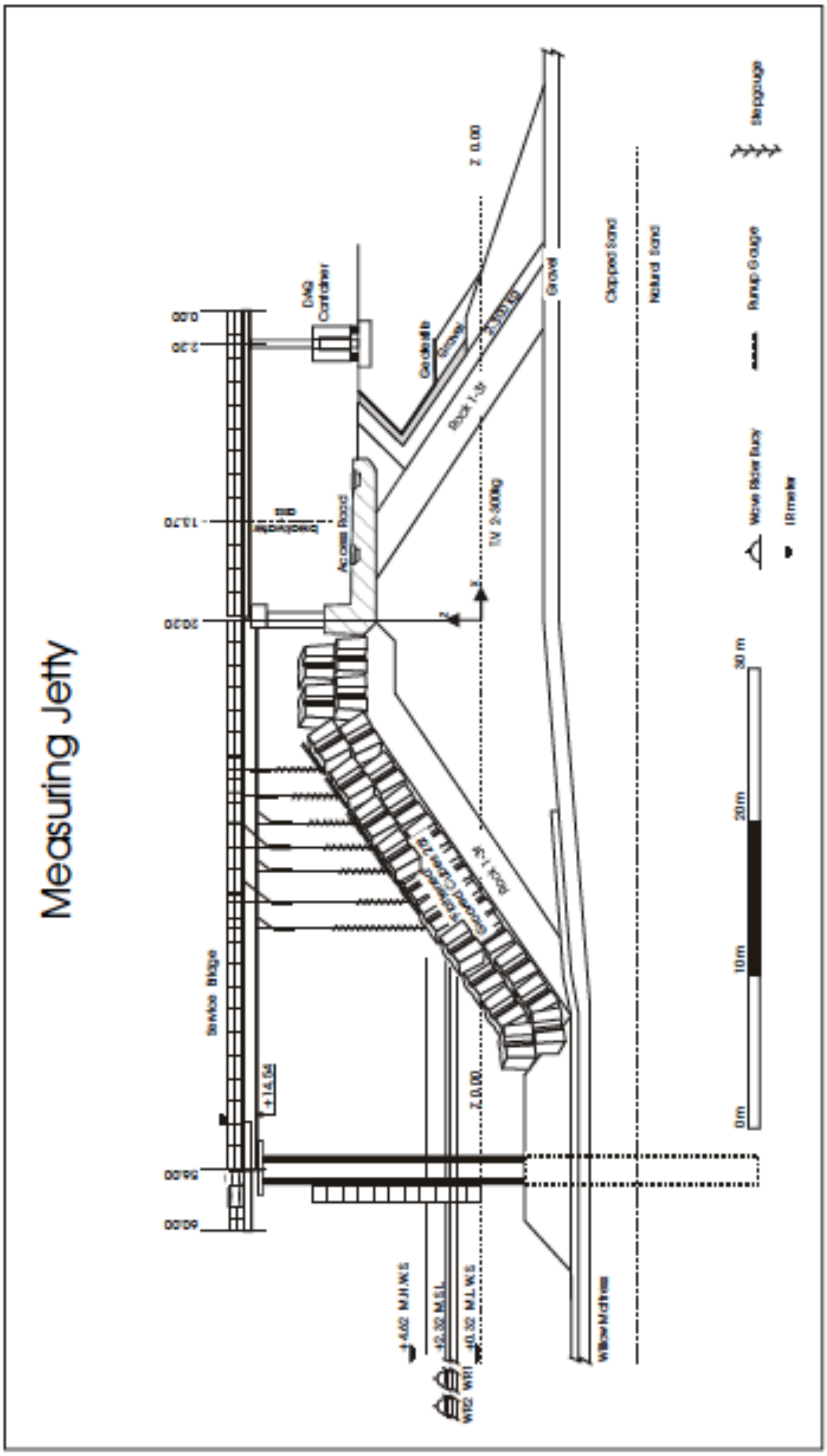


Figure 7. Cross-section of the Zeebrugge breakwater with measurement jetty and locations of the instruments

Figure 8. Measurement jetty on the Zeebrugge breakwater

- Cross section 2: overtopping measurements

Although not being exactly the same, Figure 7 can be used as a sketch for both regarded cross-sections.

A concrete tank placed just behind the crest of the breakwater handles the collection of the overtopped water (Fig.9). Water volumes inside are measured by means of pressure.

At the same time, six wave detectors near the crest armour are going to facilitate information about the number of overtopping waves (Fig.10).

Figure 9. Detail of the overtopping tank (on the left) and wave detector, WD (on the right)

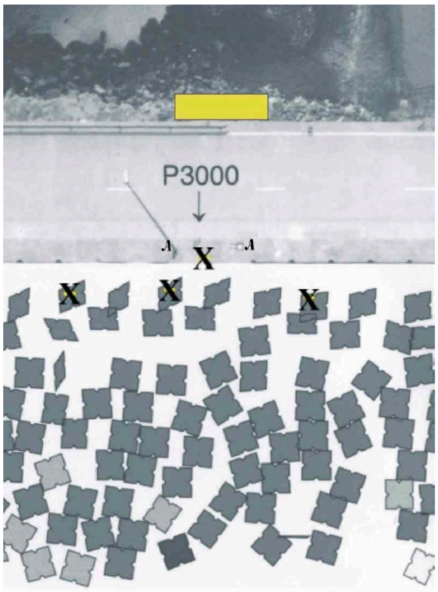


Figure 10. Overtopping tank and wave detectors disposition over the breakwater

This way it is possible to obtain values from an overtopping event.

Further information about the instrumentation and how the measurements work is available in the original CLASH report.

2.2.2 Collected data

Wave overtopping has been measured at the Zeebrugge breakwater during nine storm events (Table1)

Storm No.	Date	Time	Duration (s)
1	6 November 1999	11h30 – 13h30	7200
2	6-7 November 1999	23h45 – 01h45	7200
3	8 November 2001	16h15 – 18h15	7200
4	26 February 2002	12h30 – 14h30	7200
5a	27 October 2002	17h00 – 18h00	3600
5b	27 October 2002	18h00 – 19h00	3600
5c	27 October 2002	19h00 – 20h15	4500
6	29 January 2003	10h00 – 12h00	7200
7	7 October 2003	12h00 – 14h00	7200
8	22 December 2003	00h00 – 02h00	7200
9	8 February 2004	14h45 – 16h45	7200

Table 1. Storms measured in Zeebrugge

Thanks to the wave rider buoys it has been possible to obtain the wave characteristics for the different recorded storms (Table 2). Non-directional buoys were used from event number 1 until number 6, whether from this last one on, directional instruments collected additional wave data; they show wave attack being mainly perpendicular to the breakwater in all cases.

Storm No.	H_{m0} (m)	H_s (m)	$T_{m-1,0}$ (s)	T_p (s)	T_m (s)	ξ_0 (-)	SWL (m Z)
1	3.04	2.89	6.88	7.34	5.70	3.52	5.28
2	2.60	2.44	6.93	9.3	5.36	3.88	5.11
3	3.47	3.31	8.41	10.28	6.35	4.05	5.01
4	2.63	2.52	6.49	7.91	5.32	3.68	4.21
5a	3.74	3.61	7.50	8.57	6.21	3.46	4.40
5b	3.86	3.71	7.64	8.57	6.35	3.47	4.60
5c	3.71	3.55	7.98	8.57	6.45	3.70	4.35
6	3.16	3.03	7.28	7.91	5.94	3.66	4.71
7	3.23	3.08	7.00	7.91	5.84	3.47	4.77
8	3.03	2.88	7.33	8.57	5.85	3.76	5.26
9	3.59	3.41	7.37	8.57	6.14	3.47	5.32

Table 2. Wave characteristics and water level for the storms measured in Zeebrugge

From the whole available data gathered through the measurement instrumentation, it has been possible to compute wave overtopping discharges in each storm. Results have been summarized in Table 3.

Storm No.	q_{ceq} (l/sm)	q_{Vi} (l/sm)	$q_{\Delta h}$ (l/sm)	N_{ov} (-)	N_{ov} / hour (-)
1	3.161E-02	5.709E-02	4.677E-02	10	5
2	2.299E-02	2.211E-02	1.842E-02	3	1.5
3	2.825E-01	3.310E-01	3.588E-01	29	14.5
4	3.919E-03	1.010E-02	9.031E-03	1	0.5
5a	4.037E-01	5.158E-01	4.404E-01	19	19
5b	5.919E-01	8.585E-01	5.963E-01	30	30
5c	6.296E-01	7.036E-01	6.780E-01	31	24.8
6	8.479E-02	9.620E-02	8.646E-02	9	4.5
7	6.410E-02	8.920E-02	7.280E-02	9	4.5
8	2.900E-02	6.680E-02	5.590E-02	2	1
9	2.200E-01	5.910E-01	5.630E-01	16	8

Table 3. Average overtopping rates calculated using 3 different methods for all the storms, respectively with the number of overtopping events

Different mean discharge per meter structure width (*q*) are displayed in the table above; they are calculated according to three different methodologies:

- Method 1: Using the continuity equation q_{ceq}
- Method 2: Using individual overtopping volumes q_{Vi}
- Method 3: Using water depth jumps q_{Ah}

Figure 11 illustrates the results obtained in Table 3.

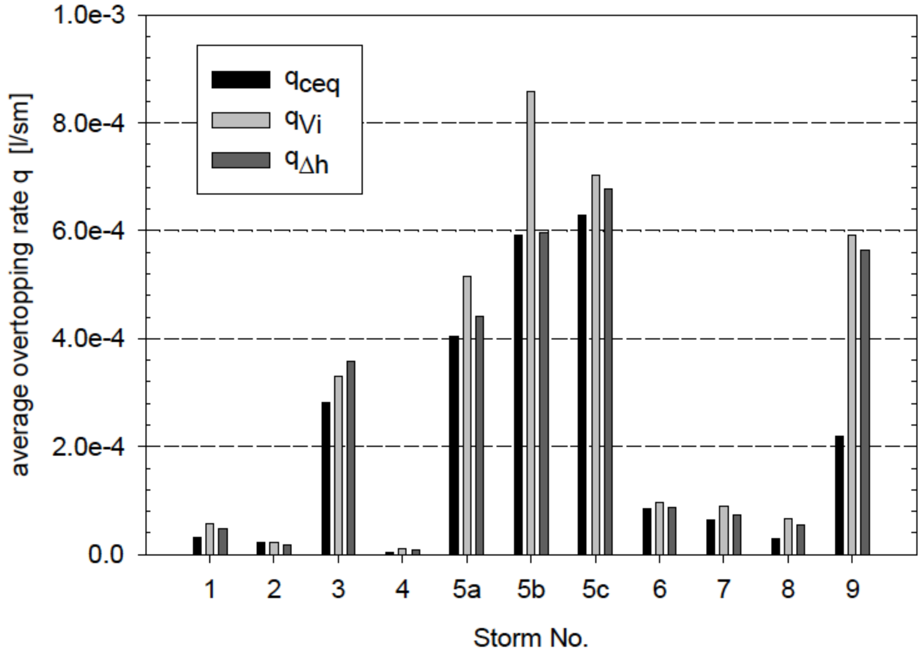


Figure 11 Graphical illustration of the average overtopping rates for three different calculation methods, based on Table 3

Results have to be discussed and compared with values obtained in SWASH; individual overtopping volumes (q_{Vi}) are thought to be the more appropriate when performing such comparison.

It must be pointed out that measured overtopping discharges in Zeebrugge result in very low values in comparison to the maximum allowable design standards, which can amount to order of magnitudes around 50 to 100 L/sm.

3

SWASH MODELLING

As aforementioned, the presented harbour location on chapter 2 has to be modelled in SWASH in order to simulate the recorded storms occurred during the concerned season. Regarding simplified 1D scenarios, wave overtopping volumes are going to be computed.

A stepwise process is going to be followed to test the real structure. Simpler models defined before are going to let the study of wave behaviour and check if all the commands have been correctly introduced.

To start with, a deep water flume is going to be considered in order the bottom does not affect propagating waves. The second step will be to generate waves in shallow water, where nonlinearities take place. Afterwards, an impermeable dike is going to be placed at the end of the flume and, finally, porosity will be included in the structure simulating the real breakwater.

Both regular and spectrum waves are going to be tested in the above scenarios.

3.1. INTRODUCTION FOR USE

SWASH is based on an explicit, second order finite difference method for staggered grids whereby mass and momentum are strictly conserved at a discrete level. Both spatial grids and time windows need to be defined by the user.

The spatial resolution should be enough to resolve relevant details of the wave field and, since SWASH is based on explicit schemes, time integration requires strict conformity of stability; the computational time step is limited by a Courant stability criterion. The condition for 1D problems is given by:

$$Cr = \frac{\Delta t(\sqrt{gd} + |u|)}{\Delta x} \leq 1$$

with Δx the mesh width, Δt the time step, u the flow velocity, and Cr the Courant number.
SWASH accounts for an automatic time step control doubling or halving Δt depending on the prescribed range for Courant. Note that a maximum for this number is advised to be of 0.5 in case of high waves, nonlinearities (e.g. wave breaking, wave-wave interactions) and wave interaction with steep structures.

Another important feature of this model is that it can be run either in depth-averaged mode or multi-layered mode. In the latter, the computational grid is divided into a fixed number of vertical layers. Thus, instead of increasing the frequency dispersion by increasing the order of derivatives of the dependent variables like Boussinesq-type, SWASH accounts for it by increasing the number of vertical layers. It contains at most second order spatial derivatives.

Also the output grid has to be specified by the user. About its resolution, it must be pointed out that the information on such grid is obtained from the computational grid by bi-linear interpolation; there is no interpolation in time; output time is shifted to the nearest computational time level. This implies some inaccuracies are introduced; such errors can be reduced by taking all input, computational and output grids and windows as much equal to one another as possible.

SWASH expects all given quantities to be expressed in S.I units. Consequently the water level and depths are in m, velocities in m/ etc. SWASH operates either in a Cartesian or in a spherical coordinate system. In the current project simulations, the first one is going to be used.

3.2. SIMULATION TESTS

This section is an overview of the whole modelling process, which can be found entirely developed in *Appendix A*.

3.2.1 Wave propagation in a deep water flume

To start with, regular waves were generated in order to observe their behaviour more easily than it would be in case of spectrum waves. The used wave parameters were the ones reported from "Storm 1", this way the obtained results are going to fall within the same order of magnitude as in the real case. Therefore, the tests were performed in a 40m depth flume with a significant wave height of 3.04m and a mean period of 6.88sec. At first, albeit some points still needed to be fixed or properly explained, the obtained results were close to the expected ones; sinusoidal pattern propagation with no setup and constant height was more or less observed.

The following step was to deal with irregular waves in the same defined scenario. A JONSWAP spectrum was introduced using the same wave input parameters as in the case above with regular waves.

By applying both a wave per wave analysis and spectral analysis to the results in SWASH, output can be better studied. Hence, Matlab codes were taken and checked from Martínez Pés (2013) [8]. Since the spectral analysis script presented some trouble, a different code was considered and corrected after finding existing errors. The latter Matlab script can be found in *Appendix B*.

After running the mentioned analysis on the performed simulations, it was observed, on the one hand, both regular and spectrum waves presented decreasing heights along the flume and, on the other, period in spectrum waves where also varying; Tm increased and Tp deceased instead.

Some advice and recommendations led new simulations being performed in order to prove the causes of the observed behaviour; from that the following conclusions came up:

- Height decreasing, which implies loss of energy along the propagation, can be explained by the software accounting for the presence of high harmonics and therefore, by wave-wave interaction taking place. Hence, even rough hand-calculation was previously done to design a scenario free from nonlinearities, SWASH seems to be accounting for them anyway.

That means the water depth was not large enough for the intended purpose, but the simulation has been performed properly and the results can be taken as correct.

Moreover, another issue has to be regarded when it comes to spectrum waves. A first reduction of the height value is occurring since SWASH has to turn the input spectrum data into water elevation data. In this process the programme computes the Fourier transformations without accounting for the fact that waves travel in group. That can be corrected by adding the bound wave effect through the command ADDBOUND in the created script.

- The reduction in the peak period (Tp) has been observed to remain stable along the performed tests. This behaviour makes completely sense, since nonlinear wave-wave interactions occurring will imply the production of higher frequencies.

- Mean period increase has been proved to be caused, up to some extent, by the already mentioned nonlinearities. Though, in the conducted tests the strongest variation seems to be triggered by the limitation of the software accounting for high frequencies when fixing a small number of vertical layers in the computational grid. The celerity of such high frequencies is not properly taken and therefore, errors arise.

Also in this scenario some misleading output appeared due to the fact the output mesh was not wisely selected. It is important to be always aware of the meshes you are working with.

3.2.2 Wave propagation in a shallow water flume

The same procedure as in deep water was applied in the shallow water flume. Generated waves were also defined with the same parameters as the ones in the previous case.

It has been seen that SWASH already accounted for nonlinear effects in the intended deep water scenario, so that part of the observed behaviour in the shallow scenario, where nonlinearities are obviously present, is going to be explained by the same reasons as before.

- About regular waves propagation, the main easily-noticed difference with respect to deep water was the elevation of water levels along the flume. That was seen to be caused by nonlinearities, since the apparition of higher harmonics involve a skewed signal. Note that such effect does not imply the variation of the mean water level value. Even in a less clear way, spectrum waves also show water levels moving up.

- As in deep water, H diminishes over distance, both in regular and spectrum waves.

- Here, unlike in deep water, no significant period differences occur. That could be explained by the use of 3 layers in a water depth around 12m, which might present less high frequency limitation.

It can be assumed therefore that SWASH performs well when it comes to wave propagation, both in deep and shallow waters.

3.2.3 Impermeable smooth dike

To end up dealing with the real situation, the following step was to introduce the dike. That addition consisted simply on creating a new bathymetry where a sloped bottom was to be considered.

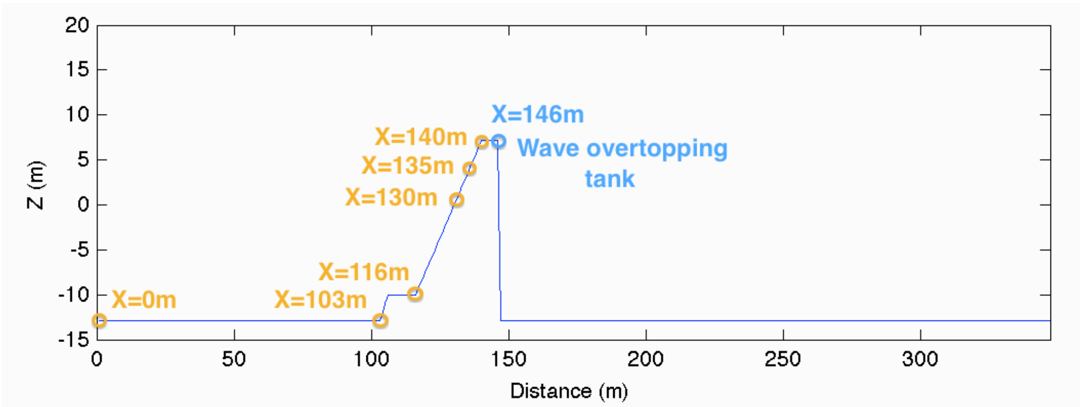


Figure 12. Bathymetry considered for the Zeebrugge cross-section (1)

Figure 12 simulates the reported bathymetry in Zeebrugge, starting at the point where the buoys are recording the available information for waves. Just behind the dike a sponge layer is located in order to absorb any water energy triggered by overtopping volumes.

Both regular and irregular waves were tested using the same definition as in former scenarios, namely significant wave height of 3.04m and mean period of 6.88sec.
The presence of a dike at the end of the flume induced to large instabilities at first, obtaining several error messages from SWASH. After some attempts, it was possible to find out how to solve such problems. Reducing the computational time step or activating some commands as BREAK or DISC MOM for the momentum conservation was imperative. The type of momentum discretisation scheme and the number of vertical layers also play a role in stability; the less vertical layers, the more stable is the model. Finally, when dealing with spectrum waves, what was decisive to get stability was limiting the Courant number.

As mentioned in section 3.1 on this document, SWASH accounts for an automatic time step control doubling or halving Δt depending on the prescribed range for Courant. Therefore, as the used values for Δx and Δt were 0.5 and 0.001 respectively, according to the Courant expression, it was fundamental to apply the command TIMEI 0.01 0.25 in order to allow the software start the simulation.

By analysing the results, the following could be stated:

- Reflection effects are clearly present. On the one hand, the amount of energy in the spectrum varies according to the reflected wave travelling on the opposite direction, which implies that its amplitude will be added to the incident one. On the other hand, it has to be considered the fact that long waves tend to reflect rather than short waves, which are more likely to break. Therefore, energy in the spectrum will increase mainly focused on longer periods (Fig.13).
Those effects should be taken into account since they must be related somehow with run-up levels and wave overtopping. This point is analysed below on the current document (Section 3.3)

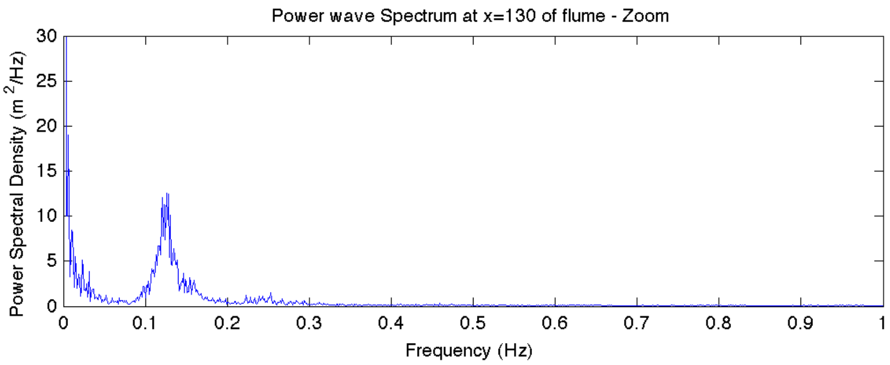


Figure 13. Wave spectrum at X=130m for wave propagation through an impermeable smooth dike during Storm 1 (1)

- Command CYCLE in the definition of the spectrum should be shifted from 30 MIN to 90 MIN in order to improve the statistics of the obtained discharge output.

- Obtained discharges at the tank location (X=146m) are much higher than the values in the real section. Whilst in the real section q_{Vi}=5.709E-02 L/ms, SWASH computes q=12.49L/ms. That behaviour can be explained by the fact that smooth impermeable structures allow larger run-up than breakwaters, where pores not only lead to energy dissipation but also allow water transmission through them.

Therefore, and since the number of overtopped waves was very similar to the real case (11 computed events whereas in the reality 10 waves crossed), that led to think the designed model was ready to be tested for the other storm episodes in Zeebrugge contained within the CLASH project.

Tests were conducted for each storm according to the wave parameters reported on Table 2 and the respective time duration, on Table 1.

The model applied to Storm 2 showed no overtopping rates. At first the result looked confusing, but some aspects of SWASH should be taken into account: the software is not able to account for spray-volumes generated by the impact of the waves through the structure, it can only consider the amount of water running up the slope.

Since in the CLASH report only 3 waves are reported to have crossed landwards, it can be thought they were just three events where some droplets were carried due to their own momentum or due to the onshore wind.

However, when working on Storm 3, totally inexplicable results came across. Unexpected high volumes of water were recorded to cross the structure at X=146m, volumes which moreover, were not recorded 6 meters before, at X=140m. (Fig.14)

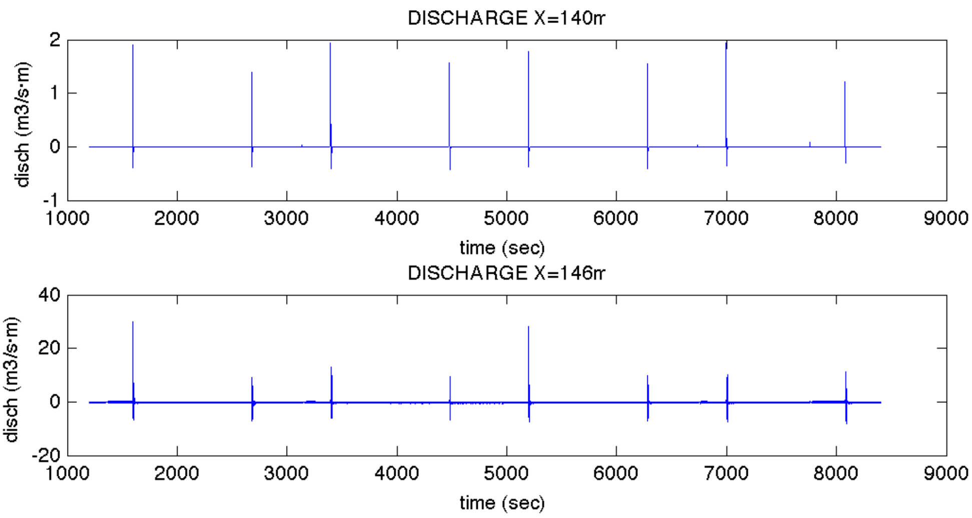


Figure 14. Computed wave overtopping at X=140m and at X=146m (wave overtopping tank) over an impermeable smooth dike during Storm 3 (1)

Something was clearly wrong.

Nevertheless, by following given recommendation, it was possible to overcome such problem; to make a smoother slope behind the dike (Fig.15) was enough to solve it. The main conclusion drawn from these circumstances was that SWASH is not able to handle properly with very steep slopes, as the one introduced in the first considered bathymetry (Fig.12).

Other advises such as to erase the sponge layer (since it is not developing any function) or to add some friction (even a small value, since friction 0 is too much unrealistic) were also regarded for the following simulations.

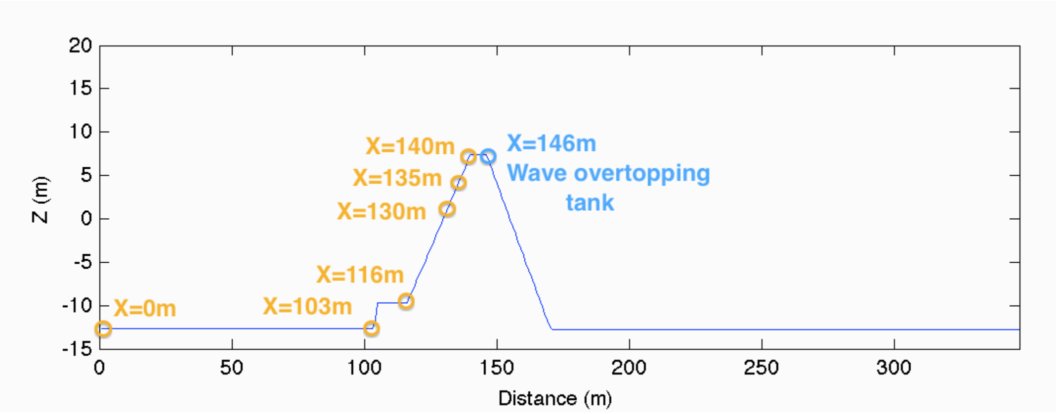


Figure 15. Bathymetry considered for the Zeebrugge cross-section (2)

In order to keep with the study of an impermeable smooth dike, a small Manning coefficient was introduced; FRIC 0.01 can be representative for very smooth asphalt, for instance.

As the scenario has was changed, it turned into a much more stable situation. This means that simulations are allowed for a higher time step, and consequently, for higher Courant numbers. Hence, a new time step of 0.005 sec was used instead of 0.001 sec, since it implies a shortening of the time needed to run each simulation. Courant was therefore adjusted through the command TIMEI 0.02 0.3.

Thus, accounting for all the above-mentioned changes, the new computation led to reasonable output (Fig.16).

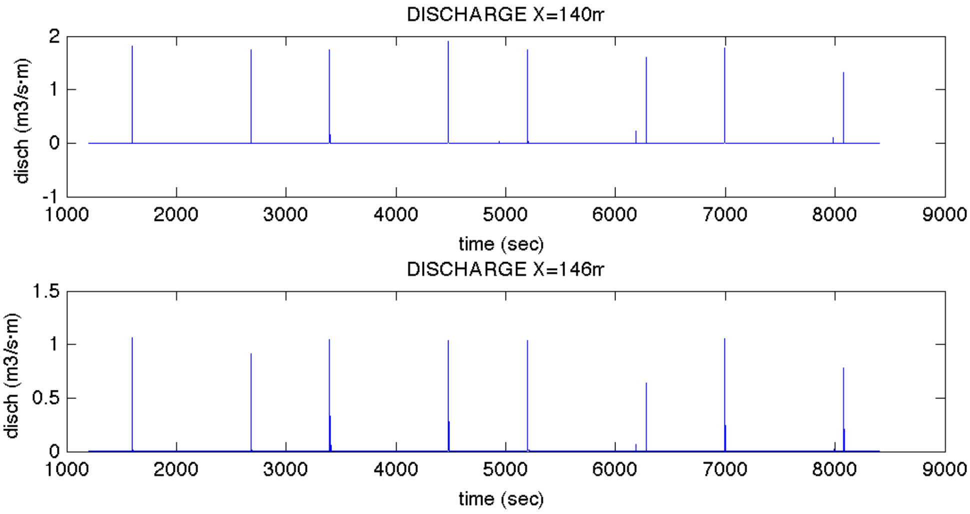


Figure 16. Computed wave overtopping at X=140m and at X=146m (wave overtopping tank) over an impermeable smooth dike during Storm 3 (2) – Corrected scenario

It was also proved that the model is stable even for a time step of 0.01 sec, but then the overtopping volumes increased largely. Such observation can be attributed to the fact that SWASH has to interpolate to get the intermediate values, so that it introduces greater error to the output.

Even though the former scenario with a steep slope was wrong, it can be stated that for the new scenario, wave behaviour propagation follows the same principles as described before. Figure 17 computed for the current scenario shows the same tendency for the spectrum as the observed in Figure 13: reflection takes place and it would mean long waves focusing close to the dike location.

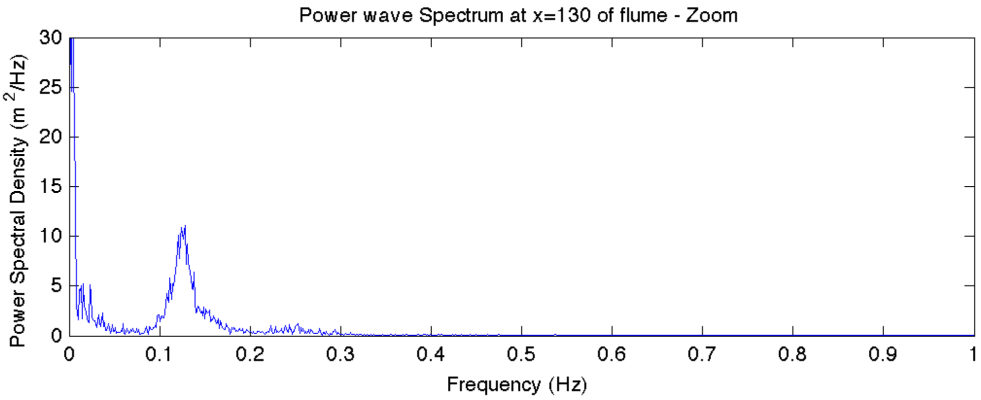


Figure 17. Wave spectrum at X=130m for wave propagation through an impermeable

Hence, it could be considered that the latter model was already suitable to be applied for all the regarded storms in Zeebrugge. Table 4 shows the obtained results. Command CYCLE in the spectrum definition has been changed in each simulation according to its real storm duration.

Storm No.	Hm0 (m)	Tm-1,0 (s)	ξo (-)	Rc (m)	Nov (-)	Nov SWASH (-)	q_{Vi} (L/ms)	q_{SWASH} (L/ms)
1	3.04	6.88	3.52	7.12	10	10	5.709E-02	29.51
2	2.60	6.93	3.88	7.29	3	0	2.211E-0.2	0
3	3.47	8.41	4.05	7.39	29	18	3.310E-01	35.4
4	2.63	6.49	3.68	8.19	1	0	1.010E-02	0
5a	3.74	7.50	3.46	8	19	1	5.158E-01	21.12
5b	3.86	7.64	3.47	7.8	30	7	8.585E-01	26.05
5c	3.71	7.98	3.70	8.05	31	5	7.036E-01	26.52
6	3.16	7.28	3.66	7.69	9	2	9.620E-02	15.73
7	3.23	7.00	3.47	7.63	9	6	8.920E-02	15.58
8	3.03	7.33	3.76	7.14	2	10	6.680E-02	23.86
9	3.59	7.37	3.47	7.08	16	33	5.910E-01	87.32

Table 4. Overtopping rates computed by SWASH over an impermeable smooth dike and comparison with the real values measured in the Zeebrugge breakwater during the storms reported within the CLASH project.

The latest version of the used SWASH code for impermeable smooth dikes, which has been used to compute values from Table 4, can be found in *Appendix B*.

Since the two compared scenarios, a dike and a breakwater, lead to very different wave behaviour, it makes it difficult to compare the obtained results in each storm. That is why, instead of comparing event per event, it will be wise to compare the general tendency in each circumstance.

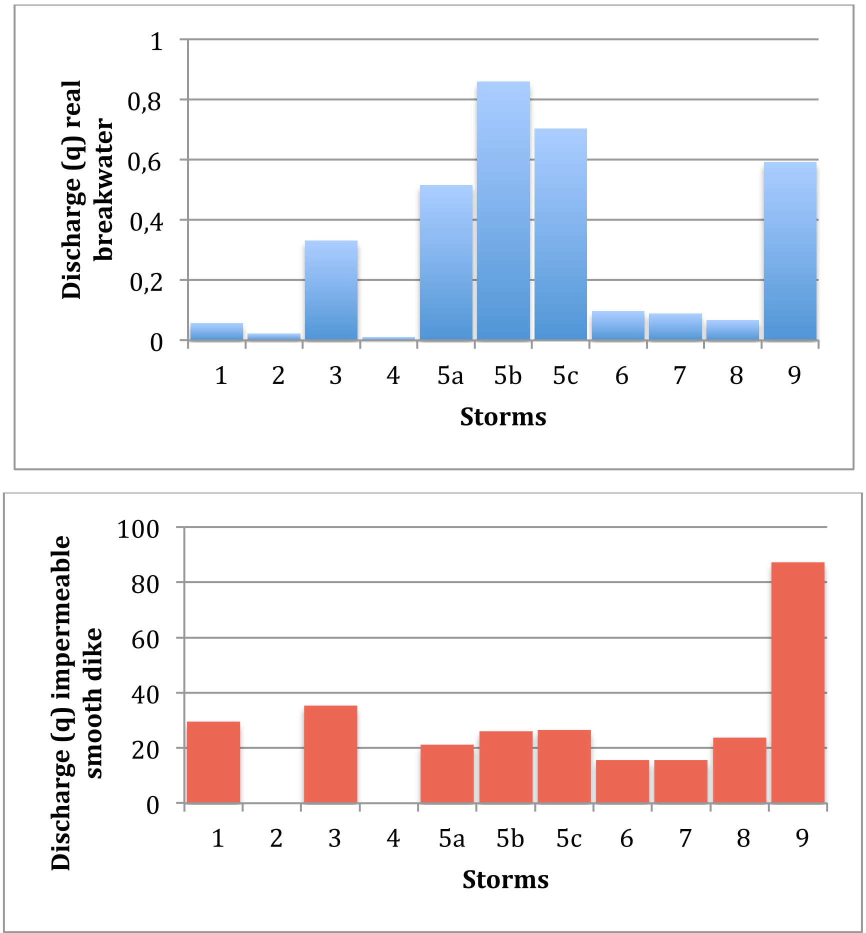


Figure 18. Discharge volumes observed at the real breakwater and computed over the dike during the concerned storm events

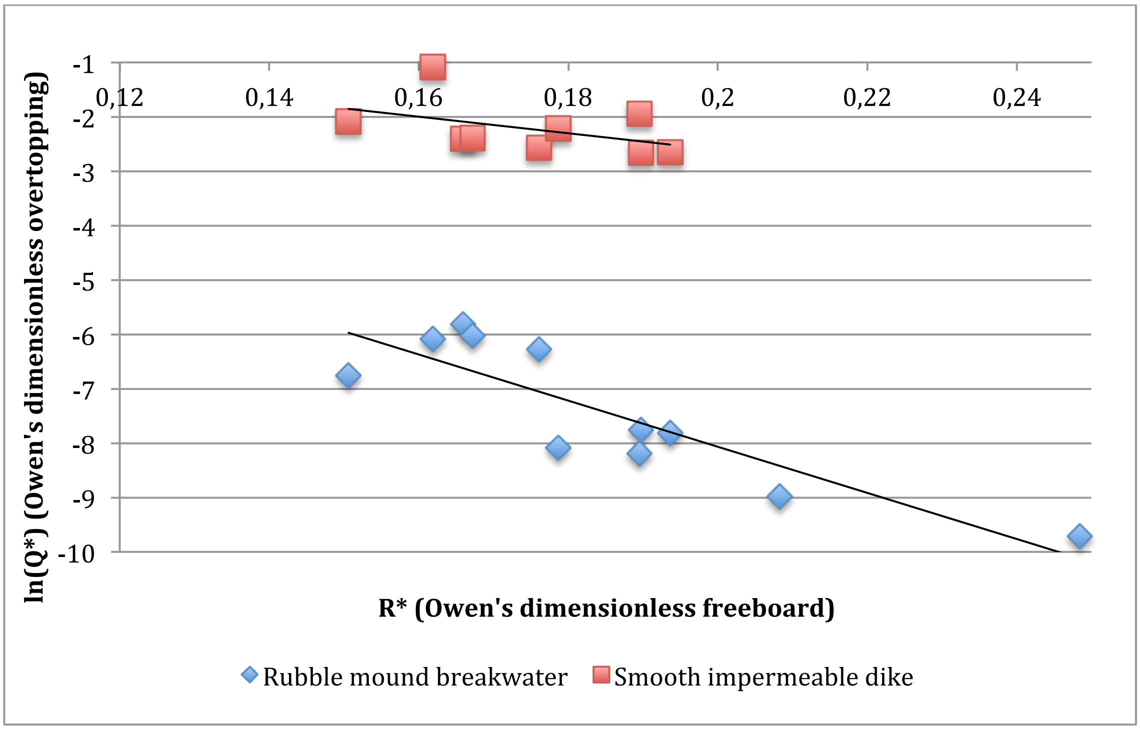


Figure 19. Dispersion graph comparing the obtained discharge in SWASH for a smooth impermeable dike and the measured discharge in a real breakwater scenario

Two points have been removed from the above graph for the smooth impermeable dike, since they had a 0-value that has to be erased in order to obtain good statistics.

Conclusions can be drawn from the above charts (Fig.18 and Fig.19):

- On the one hand, it is easily noticed that the dike allows much higher discharges than the breakwater does. This result makes completely sense since the dike is composed of a smooth impermeable wall with very low friction, which leads to almost no energy dissipation and, therefore, to higher run-up levels with larger volumes carried on each wave.

- Nevertheless, contrary at what it should be expected from the just mentioned behaviour, the number of waves that cross the structure is mostly higher on the breakwater scenario. In order to explain such results, it should be taken into account the fact that SWASH does not account for the “white water”, which appears due to the impact of waves into the structure; that means droplets are carried over the wall as a consequence of their own momentum or due to the onshore wind. Hence, more cases of overtopping are recorded in the real case probably regarding those splash volumes.

The latter exposed behaviour can be also guessed from the fact that, as the freeboard gets higher, no discharge volumes are recorded anymore for the case of the dike (Fig.19). In those situations, if waves do not reach the crest when running up the slope, no overtopping will take place, whilst on the breakwater, even lower run-up, droplets are going to cross anyway.

- It can also be said from Figure 19 that overtopping over breakwaters is more susceptible to changes on the freeboard magnitude than over dikes. The higher the freeboard is the more the waves are affected by friction and energy dissipation processes. In dikes instead, mainly the inertia is playing a role, so that overtopping, if occurring, does not reduce that much.

- Apart from the effect of the dike roughness and the existing freeboard, it can be concluded from the above results that overtopping volumes are also related with the incoming waves. In a same scenario, some events can lead to higher overtopping rates than others even not having the lowest freeboards. That is explained through smooth waves (high Iribarren number) reflecting more than steep waves when reaching the structure; more energy reflected back seawards will imply lower run-up and therefore, lower overtopping rates. Hence, wave steepness also plays a role in determining the amount of water that will cross landwards. This behaviour is further explained in section 3.3.

3.2.4 Rubble mound breakwater

The real situation in Zeebrugge has to be dealt with by introducing porosity to the defined structure in SWASH, as it is important to account for the physical processes occurring within the pores.

As a first approach, the whole breakwater profile was defined by means of a porous structure. However, the obtained results were not successful.

- It has to be regarded the impossibility to deal with small porosities in SWASH for the intended purpose; $n \leq 0$ is taken as a fully impermeable vertical wall by the software.

Once the mentioned aspect was accounted for, the simulation in Figure 20 could be conducted. For that test, porosity was defined as $n=0.45$. Even though it was possible to observe both reflection and transmission, obtained results did not show set-up or run-up processes over the slope.

Martínez Pés (2013) concluded that when introducing a porous structure in SWASH, it behaves as a kind of dissipation box able to dissipate and to reflect incoming energy. The structure height is only considered as a weight parameter to compute an equivalent porosity to be applied in the total water depth, and the structure itself is ignored.
Hence, as several test observations led to the same conclusions, such explanation is taken as valid. Consequently, it is concluded that SWASH is not able to model breakwaters by only defining a porous structure. Some other methodology must be followed.

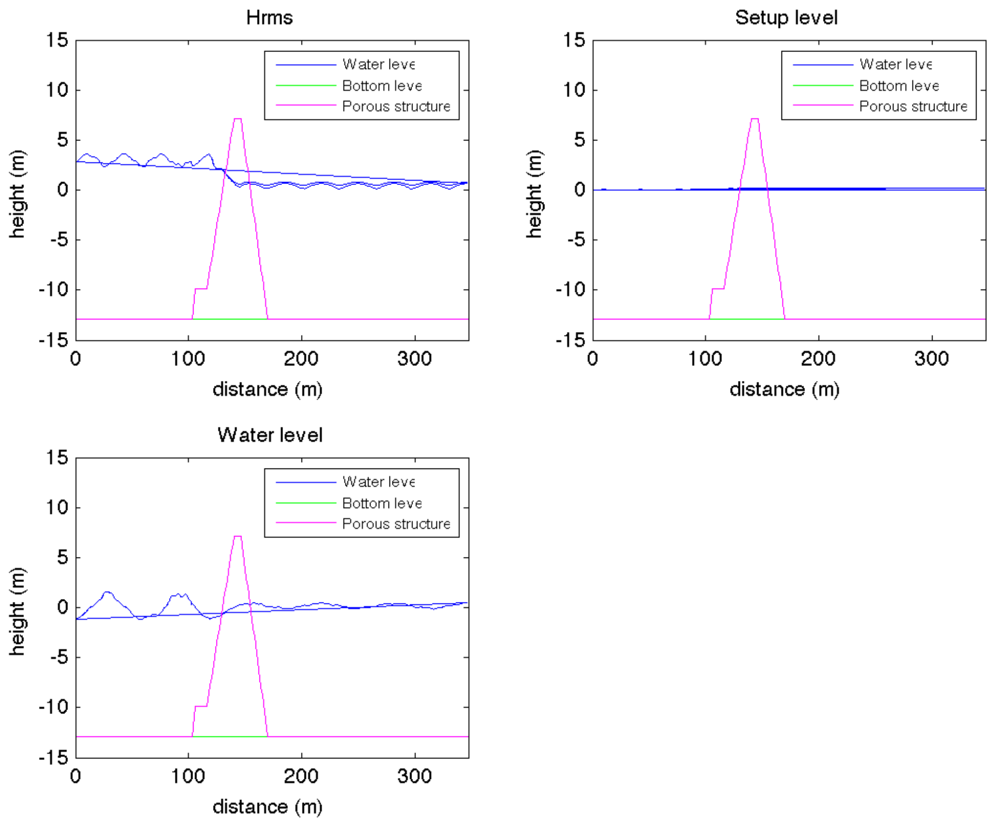


Figure 20. Wave propagation results for a rubble mound breakwater modelled by a porous structure (n=0.45)

3.2.4 Rubble mound breakwater with an impermeable core

Below, the modelling of the real breakwater in Zeebrugge is going to be regarded by the definition of an impermeable core provided with a porous structure above it (Fig.21). The porosity layer is thought to undertake the physical phenomena occurring within the armour layer for the real case.

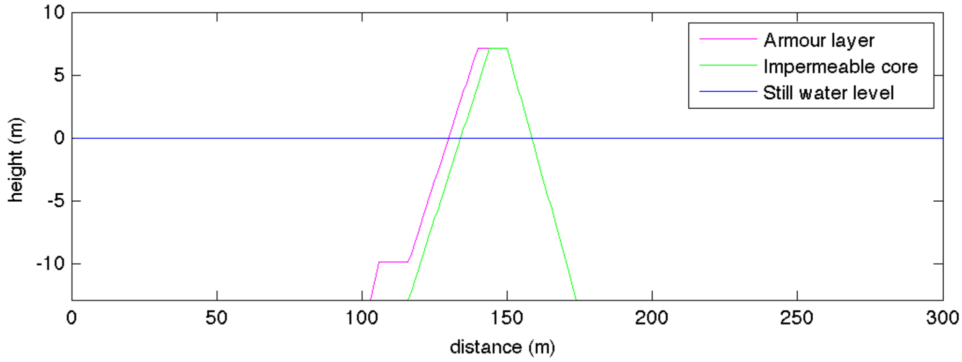


Figure 21. Layout of the used scenario for an impermeable core breakwater

Several tests were performed attempting with different porosity rates and particle sizes. Though, none of the trials led to run-up processes reaching the upper stretches of the slope; energy was fully dissipated before in all cases.

To avoid wave dissipation from the lower part of the porosity, a new scenario was defined shortening the armour layer as depicted in Figure 22. Even in that case, in order to obtain some overtopping values at the crest, the thickness of armour layer needed to be reduced, and also porosity needed to be settled at n=0.98. It was observed that by settling n=0.95, water was not able to reach the crest level, so that small changes in the porosity value seem to produce significant differences in energy dissipation along the breakwater.

Therefore, applying a porosity n=0.98 and defining both the height and the particle size at 1.4m for the armour layer, overtopping volumes were computed at X=146m during the concerned storm events in Zeebrugge. Nevertheless, it has to be borne in mind that the geometry in the defined scenario does not fit the real case; consequently, obtained results were expected to differ somehow from the reported values in the real breakwater.

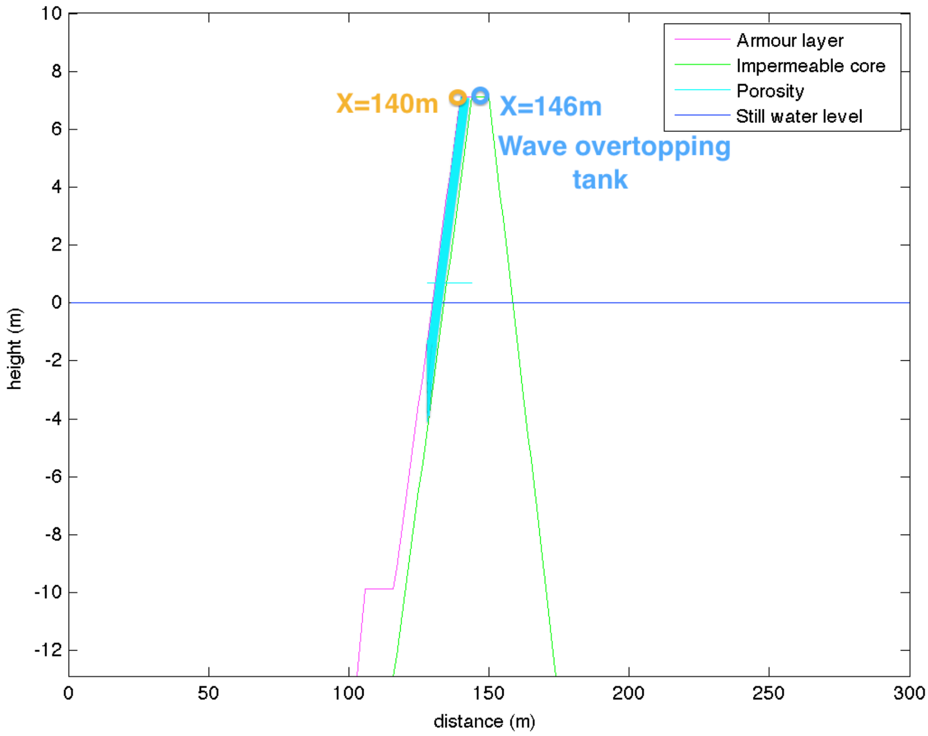


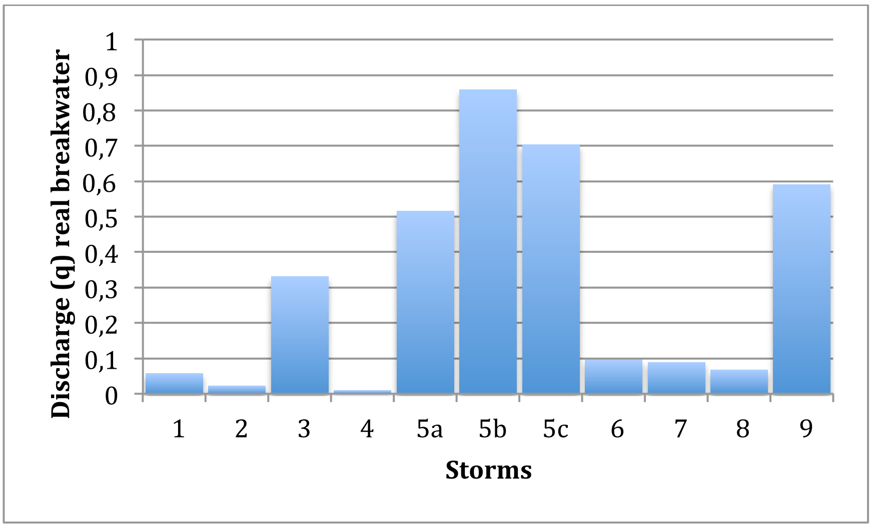
Figure 22. Layout of the used scenario for an impermeable core breakwater reducing the longitude of the porous armour layer

Presented values in Table 5 refer to the obtained results from the exposed conditions above.

Storm No.	Hm0 (m)	Tm-1,0 (s)	ξo (-)	Rc (m)	Nov (-)	Nov SWASH (-)	q_{Vi} (L/ms)	q_{SWASH} (L/ms)
1	3.04	6.88	3.52	7.12	10	7	5.709E-02	10.96
2	2.60	6.93	3.88	7.29	3	1	2.211E-0.2	1.44
3	3.47	8.41	4.05	7.39	29	12	3.310E-01	16.62
4	2.63	6.49	3.68	8.19	1	0	1.010E-02	0
5a	3.74	7.50	3.46	8	19	2	5.158E-01	6.6
5b	3.86	7.64	3.47	7.8	30	6	8.585E-01	9.64
5c	3.71	7.98	3.70	8.05	31	5	7.036E-01	13.6

6	3.16	7.28	3.66	7.69	9	6	9.620E-02	8.68
7	3.23	7.00	3.47	7.63	9	7	8.920E-02	16.65
8	3.03	7.33	3.76	7.14	2	5	6.680E-02	6.34
9	3.59	7.37	3.47	7.08	16	19	5.910E-01	45.08

Table 5. Overtopping rates computed by SWASH over an impermeable core breakwater and comparison with the real values measured in the Zeebrugge breakwater during the storms reported within the CLASH project.



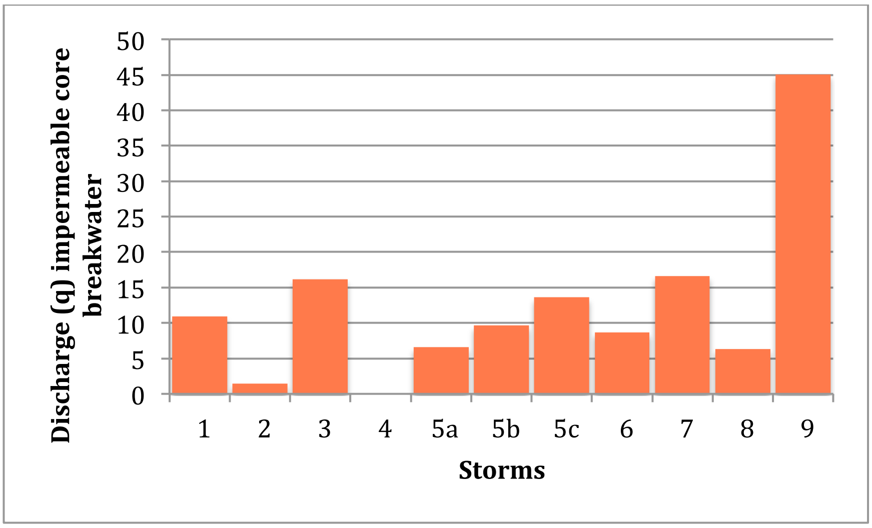


Figure 23. Discharge volumes observed at the real breakwater and computed over the impermeable core breakwater during the concerned storm events

As in the case of a smooth impermeable dike, results for the model of an impermeable core breakwater were plotted on a dispersion graph (Fig.24).

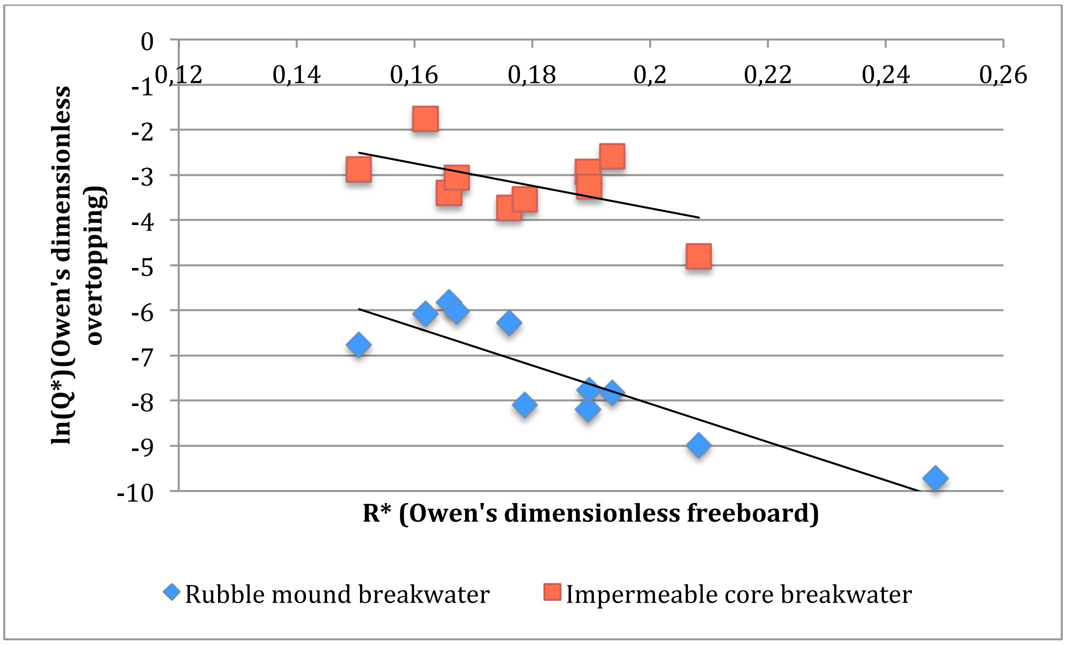


Figure 24. Dispersion graph comparing the obtained discharge in SWASH for an impermeable core breakwater and the measured discharge in a real breakwater scenario

Conclusions can be drawn from the acquired information above.

- First, it is easily noticed a reduction of overtopping volumes in comparison with the smooth impermeable dike. Since outer porosity has been placed above the impermeable slope, energy dissipation processes are taking place in this case. Though, real values have not been reached by the designed model yet.

- It was stated before that overtopping over breakwaters was more susceptible to changes on the freeboard magnitude than over dikes, as porosity acts dissipating energy along the slope. This effect can be confirmed by the results on Figure 24, where the tendency line for the impermeable core breakwater shows higher steepness than in case of the smooth slope in the dike (Fig.19). Nevertheless, the tendency slope does not equal the real one yet, and that shows a lower dissipation capacity of the designed armour layer with respect to the real case. This behaviour makes sense, as porosity in SWASH has been placed over a completely impermeable structure and limited to a certain thickness; such a limitation does not exist in reality.

- Also in these circumstances, as in smooth impermeable dikes, when the freeboard reaches a certain longitude, no more volumes are detected to cross over the structure. As already discussed, this might be a consequence of SWASH not accounting for splash volumes.

- Moreover, as concluded before, it is also explained by splash volumes the fact that the number of overtopped waves is generally higher in real storms than in SWASH records. However, this is not the case for all the situations, so that in order to analyse this behaviour, the number of overtopping events has been plotted against the feeboard height in Figure 25.

The graph in Figure 25 does not show a clear tendency, but still it can be noticed that mainly the computed amount of overtopping waves by SWASH exceeds the real value in situations with lower freeboards. In fact, as a rough approach, 3 different tendencies could be distinguished from the above chart:
For larger freeboards, the reported values for the real situation are much larger than the computed ones from the model since, as aforementioned, large part of the events might be caused by droplets instead of by run-up. As the freeboard becomes lower, the difference on the number of overtopped waves is observed to decrease (but still real values remain higher). However, up to a point, the mentioned tendency inverts and computed values by SWASH become bigger than the real ones.

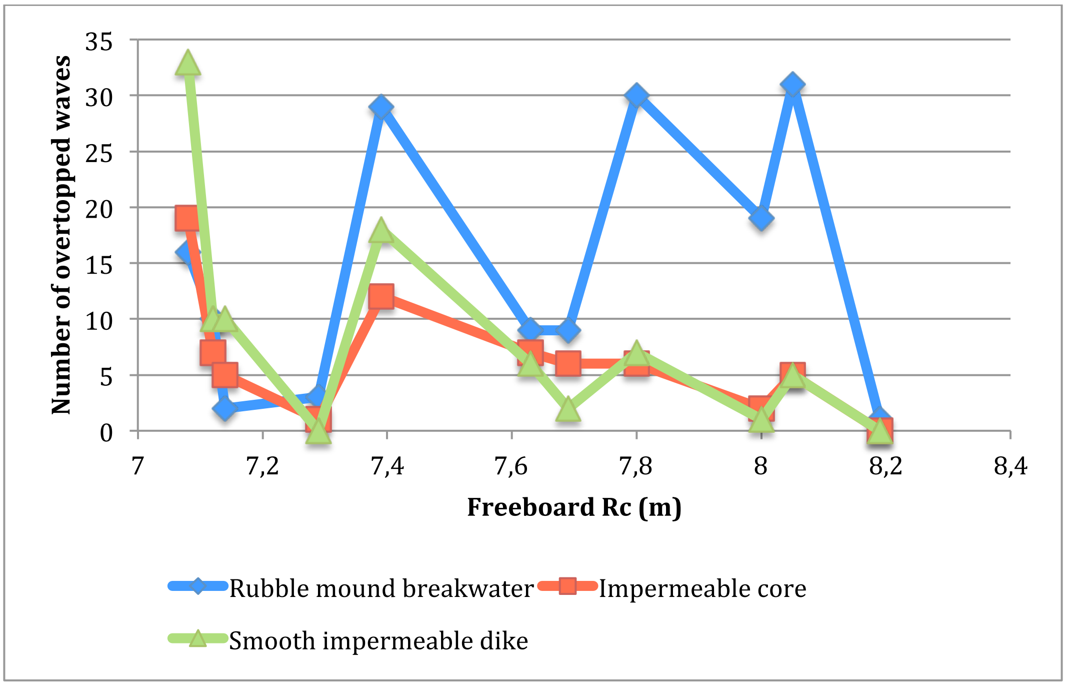


Figure 25. Comparison of the number of overtopped waves against the freeboard for the real case of the rubble mound breakwater in Zeebrugge and for both the smooth impermeable dike and for the impermeable core breakwater modelled in SWASH.

The observed behaviour might be explained by the fact that SWASH, in both designed models (smooth impermeable dike and impermeable core breakwater) has been proved to be provided by less dissipative surfaces. That enhances higher run-up levels and thus, even disregarding splash events, larger amount of waves will be able to cross lower freeboard levels

It might be attributed to the Iribarren number, which plays also an important role in run-up, the fact of not reaching a clearer tendency when plotting the number of overtopped waves accoring to the available freeboard (see Section 3.3).

Moreover, since the expected tendency should be a decreasing number of waves as Rc becomes larger, the Iribarren number could be an explanation for the observed irregular pattern as well. Also a variation of the sea water level along the storm record could produce such effect in the graph tendency, but this might be a minor cause in the regarded case as SWL has been reported to be almost constant during the indicated time spans.

Although the latter results allowed an analysis of how SWASH deals with an impermable core covered with a porous layer, the main goal was not achieved thorugh that; it looked like the real breakwater situation was not possible to be properly modelled by the software.
From the carried simulations so far, it could be concluded that the problem relates to energy dissipation within porosity, which was observed to be too strong in all tests.

After some recommendations about how to calibrate dissipation processes within porosity in SWASH, new tests were performed decreasing the parameter accounting for turbulent friction loss (β), which is a default value of β=2.8 in the software. This coefficient can be defined as $1.8 \leq \beta \leq 3.6$.
Hence, to analyse the effect of β on the obtained results, the parameter was fixed as β=1.8 in the following tests.

Indeed, in later tests it could be observed that overtopping values rose after lowering the effect of dissipation processes through the armour layer. However, this variation turned out to be not successful enough to get a proper approach for the real scenario. That would mean that SWASH underestimates waves reaching the crest of breakwaters so that it is not possible to model the desired situation.

3.3. EFFECT OF THE REFLECTED WAVES

Waves propagating through a dike or a breakwater are going to be reflected back up to some extent when reaching the structure, depending on the amount of energy that has dissipated before (due to breaking or some other mechanisms). The effect of the reflected waves will affect on the observed behaviour when it comes to run-up over the slope, and consequently, it will also affect overtopping.

In order to estimate the amount of energy that is going to reflect on a slope, the Iribarren number ξ is used

$$\xi = \frac{\tan\ \alpha}{\sqrt{H/L_O}}$$

with α being the slope angle of the structure, H the incident significant wave height and L_O is the deep-water wave length, often based on Tp.
According to the value of Iribarren, waves can be predicted to dissipate or rather reflect when meeting with the regarded slope, being more prone to reflection in case of reaching high values.

3.3.1 Effect of the reflected waves on impermeable smooth dikes

Reflected waves travelling opposite to the incoming waves will affect the resulting spectrum; that must have some implication in water levels running up the structure.

Based on Ahrens' experiments (1981), guidance for irregular wave run-up design on smooth, impermeable slopes is given in EM-1110-1100, Coastal Engineering Manual (CEM). From Ahrens' data, Figure 26 can be depicted.

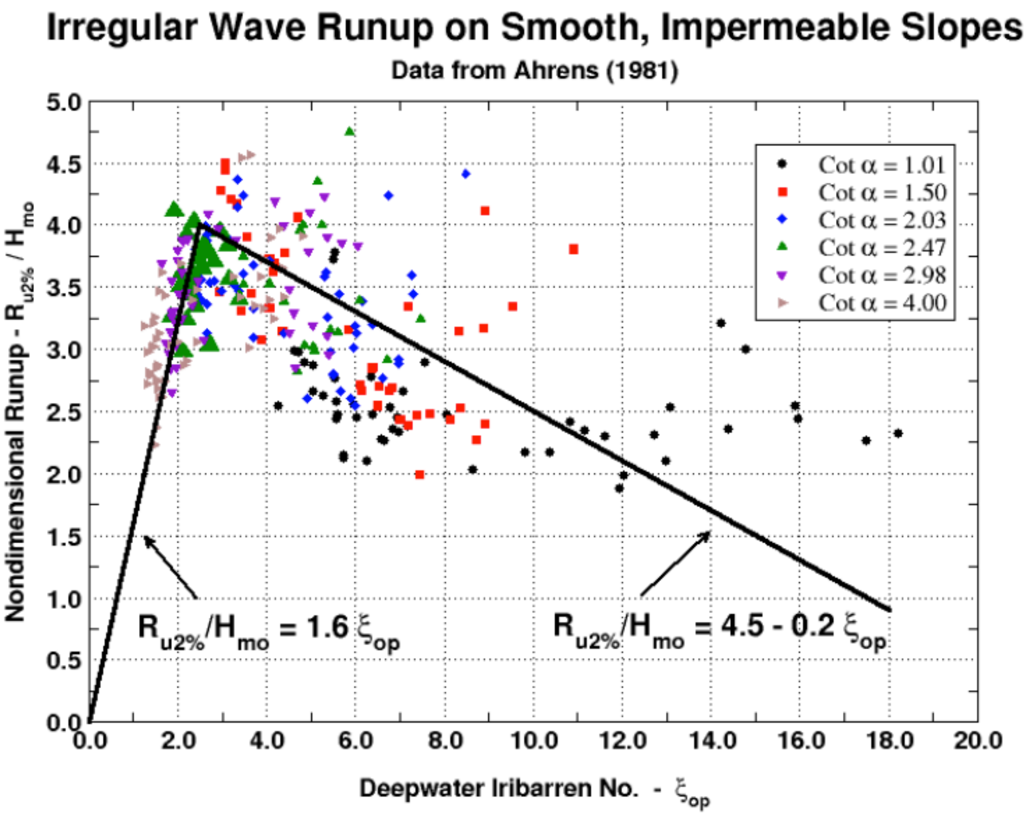


Figure 26. Irregular wave run-up on smooth, impermeable slopes. Data from Ahrens (1981)

From that information we can foretell that, up to a point, the more reflection the more run-up will occur. However, from ξ=2.5 onwards, the opposite tendency is observed; high reflection will imply diminishing run-up levels, and therefore, the reduction of overtopped water volumes.

This behaviour can be roughly explained on the one hand by steeper waves breaking before reaching the slope and therefore dissipating part of their energy in front of the structure (low ξ) and on the other, when the effect of reflected waves become stronger, their interaction with incident waves may counteract the incoming energy turning out into a reduction of the run-up level.

3.3.2 Effect of the reflected waves on impermeable dikes with porosity

Due to porosity, incident waves will be affected by dissipation processes; these will yield to a lower run-up, since energy is going to be declined along the armour layer. This effect will be dependant on the type of material used in the armour construction (particle size and porosity). Part of the wave energy will be also reflected back, but reflection will be less than in smooth impermeable slopes.

To take into account the roughness of the slope, some researchers suggested the application of a reduction factor to wave run-up formulae valid for smooth impermeable slopes. However, this approach has been proved not to be a good estimation; it might be correct only in case of breaking waves ($\xi \leq \sim 2.5$), as dimensionless run-up increases linearly with increasing Iribarren numbers. When $2 < \xi < 5$, the difference between smooth and rough slopes is more clear, whilst when $\xi > 5$ wave run-up becomes independent on the slope roughnes.

There is a vast array of formulas in literature trying to compute the run-up level in breakwaters. However, the most important equation which describes wave run-up on a breakwater attacked by irregular waves is the formula of van der Meer and Stam (1992):

$$\frac{Ru_{2\%}}{Hs} = 0.96 \cdot \xi_{om} \qquad\qquad for\ \xi_{om} \leq 1.5$$
$$\frac{Ru_{2\%}}{Hs} = 1.17 \cdot {\xi_{om}}^{0.46} \qquad\qquad for\ \xi_{om} > 1.5$$

It is therefore determined an increasing run-up as long as ξ_{om} raises.

Assuming a direct relation between R2% and the overtopping rate (q), results from the formula of van der Meer and Stam can be compared with the obtained data for overtopping.

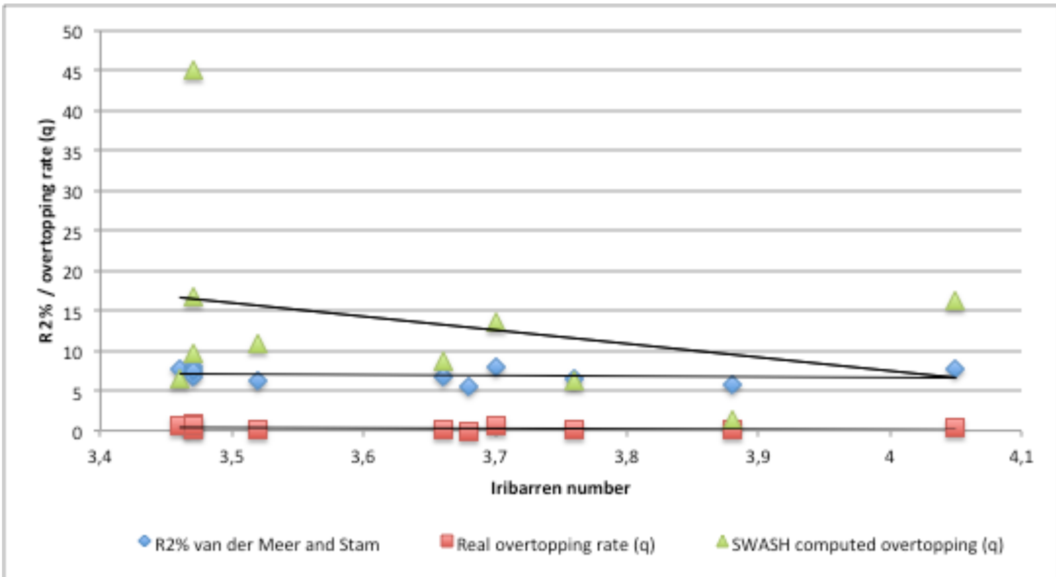


Figure 27. Comparison between de R2% value obtained by the equation of van der Meer and Stam (1992) and the overtopping values (q) both in reality and computed by SWASH according to the Iribarren number.

Figure 27 shows how the real measured values for overtopping fit the presented equation above. However, SWASH results exhibit a different tendency. The explanation for such behaviour might be that, as simulations performed in SWASH for the storm events did not fit the real geometry definition, obtained data differs from reality in a way that the effect from the dike below can be perceived. When increasing the Iribarren number, wave run-up is expected to diminish in case of smooth impermeable dikes, as presented before (Fig.26).

It is not possible to plot the data above for more realistic approaches, since no overtopping values have been computed in such cases.

4

CONCLUSIONS AND RECOMMENDATIONS

1. From the first simulations dealing with flat flumes, both in deep and shallow water, it can be stated that SWASH performs properly in wave propagation. The software is able to account for existing nonlinear processes, so that it adjusts well to reality.

2. SWASH looks suitable to model run-up and overtopping processes over smooth impermeable dikes. Reflection is considered at the dike wall and its effects are taken into account. However, obtained results should be compared with real data for a similar case. It is important to be aware of some aspects:

- Special care has to be taken when defining input in the software in order to fulfil stability requirements. The Courant number plays an important role in stability.
- Very steep slopes must be avoided since the software is not able to properly deal with them.
- When computing overtopping, SWASH limits these volumes to the continuous sheet of run-up water crossing landwards. It does not take into account droplets coming from the impact of waves into the structure. Hence, the recorded amount of water by the software is expected to be smaller than in the real case.

3. Breakwaters have to be modelled by means of an impermeable core provided with outer porosity; otherwise it is not possible to account for run-up phenomena. It seems that by only defining a porous structure, SWASH accounts for it just as a box able to dissipate and also to reflect incoming energy. The

structure height is only considered as a weight parameter to compute an equivalent porosity in the total water depth; the structure itself is ignored.
Moreover, porosities $n \leq 0.1$ are regarded as fully impermeable vertical walls by SWASH.

4. Strong dissipation has been reported within porous layers in breakwaters. Such effect makes not possible to get overtopping volumes for a realistic geometrical approach of the Zeebrugge breakwater.

The porosity is accounted in SWASH by including the Forchheimer relation in the porous momentum equation by means of two extra terms f_l and f_t.

$$\frac{1}{n}\frac{\partial u}{\partial t} + \frac{1}{n^2}\frac{\partial u^2}{\partial x} + \cdots + f_l u + f_t u|u| = 0$$

$$f_l = \alpha \frac{(1-n)^3}{n^2}\frac{\upsilon}{d^2} \quad , \quad f_t = \beta \frac{1-n}{n^3}\frac{1}{d}$$

where f_l stands for the laminar flow and f_t for the turbulent one.
Constants α and β have to be experimentally determined. The default values in SWASH for those coefficients are $\alpha_0 = 1000$ and $\beta_0 = 2.8$.

As mentioned, in the present study the default value for the turbulent friction ($\beta_0 = 2.8$) seemed to induce to an extremely high dissipation within the porous structures. In order to model real situations, these coefficients should be calibrated. However, even it improved somehow the results, lowering this parameter to its allowed minimum turned out to be not sufficient to get a proper model for the real case.

Therefore, it can be concluded that SWASH underestimates run-up and overtopping in breakwaters.

5. When using a small and thin porous layer over an impermeable core, the obtained values for overtopping have proved to be much influenced by the dike below. Hence, these cannot be considered for reliable prediction.

It must be highlighted that the obtained conclusions about dissipation in porosity are valid for the analysed geometry. For different structures further research should be carried, since other studies have concluded different behaviours. Mellink (2012), for instance, observed that SWASH tends to underestimate reflection while overestimates transmission when dealing with a vertical wall.

Mellink (2012) discussed some possible explanations for his results. The most likely reason is suggested by Van Gent (2012). He argues that besides the correct representation of porous flow, also a good numerical solution is needed for the interface between water and the porous medium; there, vertical accelerations differ due to the porous structure hindering the vertical motion. Mellink states that SWASH only uses horizontal porous resistance terms, so that vertical acceleration is calculated as a very rough estimation in pores. That led, in his case, to unrealistic high values for transmission, which means low vertical resistance; however, such behaviour does not fit the observed one in the current project.

Hence, the reason for the strong dissipation obtained in the conducted tests is still unknown. As Van Gent (2012) suggested, the problem might be related to an incorrect modelling of discontinuities on the transition from free surface to the porous medium. Note that in a slope scenario, up-rushing waves are affected longer for this transition zone; that might be one of the reasons for the observed difference in the behaviour with respect to a vertical wall.

Since only a few simulations were tested in the present study, it is recommended future research to be carried in order to find out the occurring phenomena and to prove if it actually relates to the interphase between water and porosity. As already recommended in other literature, adding porous friction to the vertical momentum balance could be attempted in order to prove whether it enhances the obtained results.

4.1 FUTURE WORK

To sum up, SWASH is not suitable yet to be applied for overtopping prediction in rubble mound breakwaters:

- Impossibility to model the real porosity within the structure; permeable cores are not allowed.
- Impossibility to properly account for the pore pressure attenuation, leading to an underestimation of the waves run-up and, consequently, to an underestimation of wave overtopping.
- Impossibility to account for overtopping volumes caused by droplets generated during the impact of the waves into the structure.

Nevertheless, as SWASH consists on a numerical model which is still on ongoing development, further development is expected. Future modifications on the mentioned aspects would probably allow the model to overcome the encountered problem, enhancing the software performance when it comes to overtopping prediction.

4.1.1 SWASH model for the Catalan coast

In the present study, the model has only been tested for overtopping prediction in a single location; however, conducted tests could be extrapolated to other scenarios. As the current project has been developed for the Universitat Politècnica de Catalunya, it would be interesting to analyse the applicability of SWASH to the existing structures around the Catalan coast.

As an example, the Port of Blanes has been reported to suffer from overtopping events, yielding problematic situations for a proper activity development. Alabart Llinàs in his thesis (2013) proved SWASH to be suitable for wave propagation into the port domains. Hence, good performance of the model in overtopping prediction could make it a strongly valuable tool.

That said, once the mentioned issues in the numerical model can be fixed, it is suggested to conduct an applicability study for the Catalan case.

Meanwhile, SWASH seems to be a potential tool for other purposes apart from the studied case of wave overtopping prediction on partially permeable structures.

Nowadays, wave prediction within the Spanish port domains is carried by the numerical model SWAN, which allows to simulate waves approaching the coast by using a phase average model. The working methodology of that model differs from the one in SWASH, since the latter consists on a phase solving model instead. Phase average models have been reported not to be adequate in places with rapidly varying properties. Besides, these models are not able to reproduce reliably simulations neither for diffraction and reflection effects, nor for other nonlinearities, all of which play a major role in harbour domains.

As SWASH have been proved to account for these effects, it is additionally suggested to perform further analysis about the applicability of this type of numerical modelling along structures in the Catalan coast, as Alabart Llinàs (2013) did in the Port of Blanes. The main drawback of these phase solving models is that they take much more computational time, and that is why for the moment they are only used in small domains or for the study around structures where they are strictly needed. However, it is thought that SWASH can become an important tool in future port planning and design.

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APPENDICES:

A. Simulation tests - SWASH
B. SWASH and Matlab scripts