

Evaluation of Overtopping on Rubble Mound Breakwaters using Numerical Modelling

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A Deus, ao meu País, a Casa.

*“Experience does not err; only your judgments err by
expecting from her what is not in her power.”*

Leonardo da Vinci

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I already did many things during my life but living abroad and experience what I experienced gave me another meaning to the most Portuguese word of all: *Saudade!*

RESUMO

Tendo em conta que no último século o nível médio da água do mar tem subido devido às alterações climáticas, e que uma parte considerável da população mundial vive em zonas costeiras, os perigos de perdas materiais e de vidas humanas são questões cada vez mais presentes para a sociedade civil.

Com o intuito de mitigar estes problemas, são necessários meios para a previsão de variados eventos relacionados com tempestades ou com a subida do nível médio da água do mar, para se delimitar zonas de não construção ou mesmo a construção de estruturas que permitam a defesa da linha costeira contra fenómenos marítimos mais agressivos. Os softwares de modelação numérica são ferramentas cada vez mais utilizadas neste tipo de situações. Programas deste género são bastante práticos para a simulação de fenómenos marítimos. Conseguem utilizar a informação recolhida por satélites ou voos de reconhecimento para definir a batimetria de um determinado local, tornando o estudo mais rápido, comparativamente com os modelos laboratoriais onde a sua construção é morosa e requer espaços propícios para a sua execução.

Um exemplo de uma empresa que desenvolve esse tipo de softwares é o *DHI*, instituto com décadas de experiência acumulada em hidráulica e softwares de modelação numérica. O *MIKE 3 Wave FM* é o software mais recente no mercado. Permite simular ondas num determinado local, recolhendo informação relativa às ondas geradas e às suas consequências no meio envolvente, como por exemplo galgamentos e zonas inundadas.

Uma vez que o *DHI* executou testes em modelos físicos para um estudo de reforço do dique *Afsluitdijk*, foi utilizada essa informação para se realizarem testes comparativos com o novo software *MIKE 3 Wave FM*. Contudo, a sua reprodução exata é complexa, devido a questões relacionadas com a não linearidade de alguns fenómenos.

O objetivo geral do trabalho foi realizar testes de sensibilidade do software e replicar os resultados obtidos pelo modelo físico. Ao longo do estudo foram realizadas séries de testes com o intuito de calibrar o modelo com parâmetros que melhor caracterizam os processos físicos presentes neste tipo de estudos. Inicialmente, foi feita uma análise para melhor definir a resolução do modelo, tanto horizontal como verticalmente, tentando nunca por em causa a velocidade de computação das simulações assim como os seus resultados. Foi também realizado um estudo para definir os valores de *Flood and Dry*, parâmetro do *software*. Nesse estudo foram experimentados vários valores que alteravam significativamente os resultados, optando-se pelos valores pré-definidos à escala do modelo. Posteriormente caracterizaram-se as ondas conforme o modelo físico e procedeu-se à sua calibração, devido aos assertos de espectros de ondas incidentes. A taxa relativa de galgamento de todos os testes realizados apresentou valores aceitáveis, tendo em conta outros estudos comparativos de modelos físicos com modelos numéricos. Contudo, os resultados obtidos não correspondem aos valores inicialmente esperados. O volume de água de cada galgamento é estatisticamente bastante superior ao verificado nos modelos físicos. Outro aspeto que ficou aquém do expectável, no modelo numérico, foi a distribuição das alturas de onda, ao longo do domínio simulado: as maiores ondas verificaram-se nas sondas hidrodinâmicas mais afastadas do gerador de onda, ao passo que, no modelo físico, as alturas de onda vão diminuindo com a distância.

Em suma, com os testes realizados ao longo deste trabalho, fica demonstrado que não se tem ainda o controlo total sobre o modelo numérico, embora a sua utilização seja fidedigna, com as devidas precauções.

PALAVRAS-CHAVE: Quebra-mar, Modelo Numérico, Modelo Físico, Galgamento, Engenharia Costeira.

ABSTRACT

Given that in the last century the mean sea level has risen due to climate change, and that a considerable part of the world's population lives in coastal zones, the dangers of material and human losses are issues more and more present to the civil society.

In order to mitigate these problems, means are needed to forecast various events related to storms or rise in sea level, to determine zones of non-construction or even the construction of structures that allow the defence of the coastline against more aggressive marine phenomena. Increasingly used tools in this type of problems consist in numerical modelling software. These programs are very practical for predicting marine events, and, because of the way they characterize the bathymetry of a given location, through satellites or reconnaissance flights, they make the study faster, compared to laboratory models where their construction is lagging and requires adequate facilities for their execution.

An example of a company that develops this type of software is *DHI*, an institute with decades of accumulated experience in hydraulic and numerical modelling software. *MIKE 3 Wave FM* is the latest software to be released. It allows the simulation of waves in a given location by collecting information about the generated waves and their consequences in the surrounding environment, such as overtopping events and flooded areas.

Since the *DHI* performed tests on physical models for the *Afsluitdijk* dike reinforcement study, that information was used to perform comparative tests with the new *MIKE 3 Wave FM* software. However, its exact replication is complex due to issues related with the non-linearity of some phenomena.

The general aim of the work was to perform software sensitivity tests and replicate the results obtained by the physical model. Throughout the study, a series of tests were carried out in order to calibrate the model with parameters that better characterize the physical processes present in this type of studies. Initially, an analysis was made to better define the resolution of the model, both horizontal and vertically, trying never to compromise the computational speed of the simulations as well as their results. A study was also performed to define the values of *Flood and Dry*. In that study, several values that significantly altered the results were experimented, opting for the pre-defined values divided by the scale of the model. Subsequently, the waves were characterized according to the physical model and their calibration was carried out, due to the correctness of incident wave spectra. The relative overtopping rate of all tests performed presented acceptable values, taking into account other comparative studies of physical with numerical models. Nevertheless, the results obtained do not correspond to the values initially expected. The volume of water from each overtopping event is statistically much higher than that of the physical models. Another aspect that fell short of what was expected, in the numerical model, was the distribution of wave heights along the simulated domain: the largest waves were found in the wave gauges farthest from the wave generator, while in the physical model, the wave heights decrease as the waves move away from the wave maker.

In short, with the tests carried out throughout this work, it is demonstrated that one does not have yet complete control over the numerical model, but its use is reliable, with due precautions.

KEYWORDS: Rubble Mound Breakwater, Numerical Model, Physical Model, Overtopping, Coastal Engineering.

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SYMBOLS, ACRONYMS AND ABBREVIATIONS

a	Laminar resistance coefficient (formula)
b	Turbulent resistance coefficient (formula)
d_{50}	Mean cumulative particle grain size
H	Wave height
h	Hour
$H_{1/3}$	Highest third of the waves
H_{m0}	Spectral significant wave height
L	Length
L_o	Wavelength
m_0	Zero order spectral momentum
R_c	Crest Freeboard
s_o	Wave Steepness
T	Oscillating period
$T_{1/3}$	Significant Wave Period
T_m	Average Wave Period
T_p	Peak Wave Period
$ u $	Magnitude of the flow velocity
u_i	Velocity components
u_m	Maximum oscillating velocity
x	Coordinate in the x-axis
y	Coordinate in the y-axis
α	Laminar Resistance Coefficient
α_i	Slope Angle
β	Turbulence Resistance Coefficient
γ	Non-dimensional peak shape parameter
Δx	Grid Spacing in the x-axis
Δy	Grid Spacing in the y-axis
ε	Kinetic Dissipation
η	Surface Elevation
θ	Direction of the wave
κ	Kinetic Turbulence
$\kappa\text{-}\varepsilon$	Kepsilon turbulence model
$\kappa\text{-}\omega$	Komega turbulence model
$\kappa\text{-}\omega$ SST	Komega SST turbulence model
ν	Molecular viscosity
λ	Scale
ρ	Fluid density
σ_a	Spectral width parameter a
σ_b	Spectral width parameter b
2D	Two dimensions
3D	Three dimensions
ASCII	American Standard Code for Information Interchange
AWACS	Airborne Warning and Control System
CFD	Computation fluid dynamics

CFL	Courant-Friedrich-Lévy
Corneel	Corneel werken aan een werelddijk
DHI	Dansk Hydraulisk Institut (Danish Hydraulic Institute)
HLLC	Harten, Lax, van Leer and Einfeldt
JONSWAP	Joint North Sea Wave Project
KC	Keulegan-Carpenter number
M3	MIKE 3 Wave FM
MPI	Message Passing Interface
NAP	Normaal Amsterdams Peil
NN	Neural Network
RANS	Reynolds Averaged Navier-Stokes
RAS	Reynolds Average Simulation
SST	Shear Stress Transport
TMA	Texel, Marsen, and Arsloe
US	United States
WL	Water level
WG	Wave gauge
WS	Wave Synthesizer

1

INTRODUCTION

1.1 CONTEXTUALIZATION

Sea level rise is linked to the increase of the global temperature, even with uncertainties about how much it can rise. It is almost assured that the rising of the sea level in this century and beyond will represent a growing challenge for coastal communities, infrastructures and ecosystems due to increased number of inundations, more frequent and extreme coastal flooding, erosion of coastal shoreline, and intrusion of saltwater into rivers and coastal aquifers (Grgić *et al.*, 2017). The change in sea level is driven by a variety of mechanisms that operate at different spatial and temporal scales (Kopp *et al.*, 2015). The global mean sea level rise is due to two main factors (Church *et al.*, 2013):

1. Increase of the volume of the seawater due to the ocean's thermal expansion as it warms up, Fig. 1.1;
2. Increase of the water mass in the ocean due to melting of ice from mountain glaciers and ice sheets in Antarctic and Greenland, Fig. 1.2.

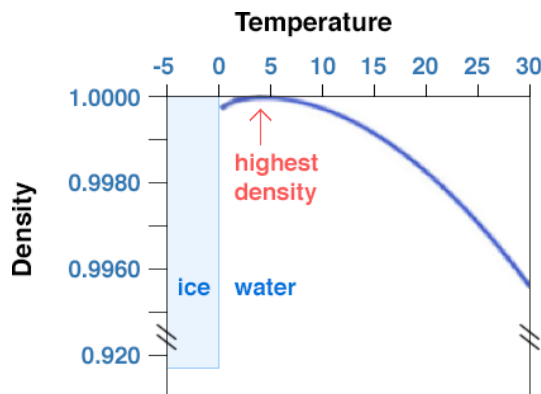


Fig. 1.1 - Water density and temperature graph (Saffer, 2017).

Sea level and its changes are not globally uniform for several reasons (Le Cozannet *et al.*, 2017). First, the atmosphere-ocean dynamics (driven by ocean circulation, winds and other factors), are associated with differences in sea surface level, as well as differences in density due to the distribution of heat and salinity in the ocean. Changes in any of these factors will affect the level of the sea surface. A weakening of Gulf Stream transport in the mid-2000s may have contributed to sea level rise in the oceanic environment extending to the northern United States (US) coast (Boon, 2012; Ezer, 2013; Sallenger *et al.*, 2012). A trend that many models expect that will continue in the future (Yin and Goddard, 2013).

Second, land melting sites and changes in the terrestrial water reservoir confer distinctive regional static equilibrium at sea level, based on the effects of gravitational, rotational and crustal deformation (Mitrovica *et al.*, 2011).

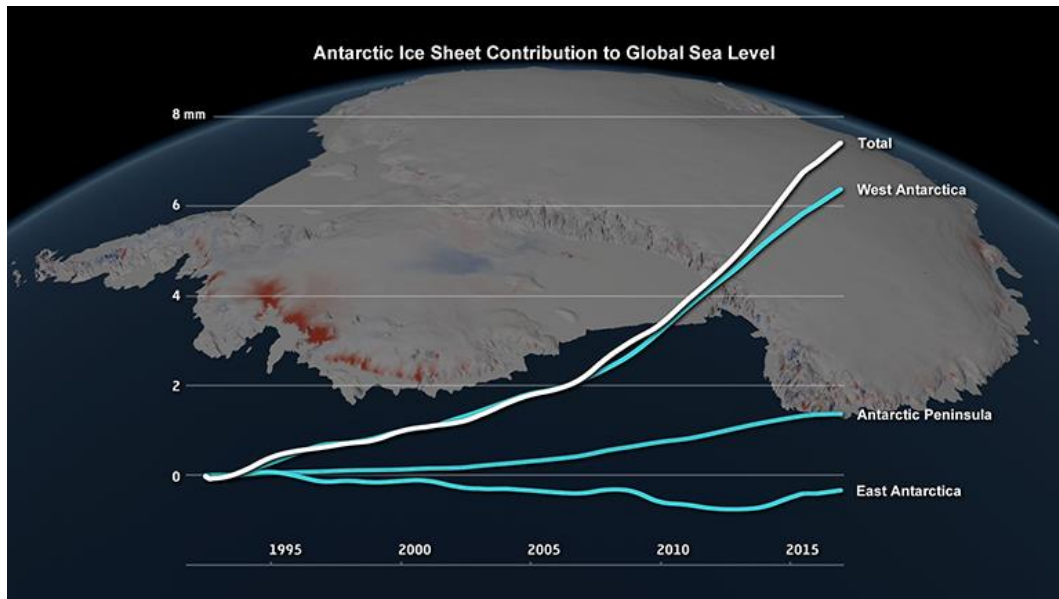


Fig. 1.2 - Changes in the Antarctic ice sheet's contribution to global sea level, 1992 to 2017 (Norton, 2018).

In the last two centuries, a global tide gauge network provides observations of local sea level rise, whereas satellite altimetry, starting in 1993, provides a wider coverage of sea surface level outside the polar regions. Global mean sea level trends in the period 1901-1990 vary slightly. Hay *et al.* (2015) 1.2 ± 0.2 mm/year, Church *et al.* (2016) 1.5 ± 0.2 mm/year. Thus, these results indicate about 11 to 14 cm of global mean sea level increase from 1901 to 1990. Tidal analysis indicates that global mean sea level has increased at a considerably faster rate of about 3 mm/year since 1993 (Church and White, 2011; Hay *et al.*, 2015), a result supported by satellite data indicating a trend of 3.4 ± 0.4 mm/year from 1993 to 2015 (Nerem *et al.*, 2010).

In addition to land ice, the mass-addition contribution also includes net changes in global land-water storage. This term varied in sign over the course of the last century, with human-induced changes in land-water storage being negative, -0.6 mm/year, during the period of massive dam construction in the middle of the last century (Wada *et al.*, 2017).

According to the United States Interagency Sea Level Rise Task Force, six scenarios of global mean sea level rise are expected (Sweet *et al.*, 2017) that can be used to risk-framing purposes. Numerous studies have estimated the probabilities of different quantities of global mean sea level in different pathways (Church *et al.*, 2013; Grinsted *et al.*, 2015; Jackson and Jevrejeva, 2016; Jevrejeva, *et al.*, 2014; Kopp *et al.*, 2014, 2016; Mengel *et al.*, 2016; Slangen *et al.*, 2014), using a variety of methods, including statistical and physical models. Most of these studies agree with the fact that the increase in global mean sea level by 2100 is very likely to be between 25-80 cm.

Summing up, although it is debatable how much the increase in global mean sea level will be, what is unquestionable is that coastal areas will be increasingly flooded by overtopping events. Taking into account that 600 million people (10% of the world's population) live in coastal areas that are less than 10 m above sea level (United Nations, 2017), the problem becomes even more evident.

1.2 AFSLUITDIJK CASE

Thanks to its geomorphology, since it is a topographically flat country and with an altitude very close to the sea level, the Netherlands has always had problems related to floods. Moreover, Netherlands means precisely lower countries, and that is because almost fifty percent of the country is less than one meter (m) above the sea level. Since the fifteenth century that the Dutch started to develop techniques to drain wet areas to agriculture: that new land reclaimed to water is called polder. The area conquered to the water through the years in the Netherlands is quite impressive, Fig. 1.3.

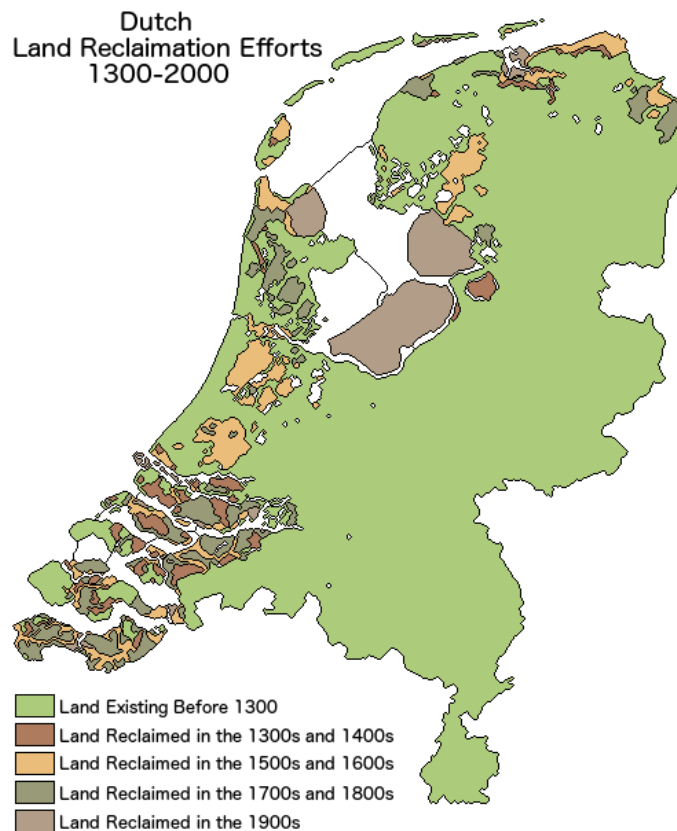


Fig. 1.3 - Evolution of land reclamation since the year 1300 (Hoeksema, 2007).

Over the years, due to these land reclaims and some storm events, some areas in the Netherlands were whipped for water flooding several times and caused thousands of dead and injured. Those events changed the way of thinking of an entire country and most recently brought the question of the defence of the land to a scale never seen before in the history of mankind. With the technological advances of the early twentieth century, the size of works and their complexity have increased exponentially, leading to the creation of even more ambitious projects. The great examples of this ambition are the projects of *Zuiderzee Works* and the *Delta Works*, the last being declared one of the Seven Wonders of the Modern World by the American Society of Civil Engineers.

In the year of 1927 began the construction of the *Afsluitdijk*, that means “Enclosure Dam” in Dutch. This dike links *Den Oever* on *Wieringen*, in North Holland province, to the village of *Zurich*, in Friesland province, Fig. 1.4.

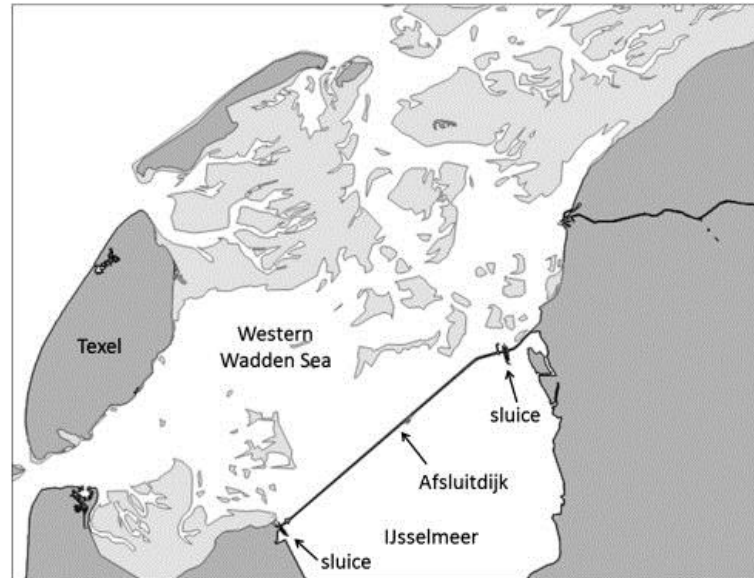


Fig. 1.4 - Location of the *Afsluitdijk* (Cervenci and Alvarez Fernandez, 2012).

The dike is developing over 32 km and with a width of 90 m. During the construction of the dike, thousands of workers and machinery were allocated. In Fig. 1.5 it can be seen some work activities at the construction of the *Afsluitdijk*. The complexity of the work is significant considering the resources available at the time. Even nowadays, with the current construction technology, this type of work would be complex to do. The main purpose of this project was to obtain flood protection against sea level rise and extreme weather conditions that will occur more frequently in the future, as well as water management.



Fig. 1.5 – Construction of the *Afsluitdijk* (Hakkenes 2017).

Since this structure has more than eighty years, it requires some maintenance, so the Dutch government saw it was a good opportunity to reinforce the dike. The idea was not to raise the dike but to give a sturdier top layer that would make it overflow-resistant. The reinforcement was projected with a risk of one-in-ten-thousand-years, that means that the *Afsluitdijk* will be able to resist to a coincidence of a spring tide and an extreme storm from north-westerly. In that situation, the dike must not break down.

At the request of *Corneel werken aan een werelddijk* (Corneel), tests were carried out at *Dansk Hydraulisk Institut* (DHI) physical modelling laboratories in Hørsholm, from June 26th to September 12th, 2017. The objective of these tests was to quantify the overtopping with different maritime conditions and dike's cross-sections.

After the tests were carried out and analysed, the information was assembled in a report, with all the information necessary for Corneel to make a decision regarding the project.

1.3 WORK STRUCTURE

With the intention of making the reading and comprehension of this work more intuitive, this report was divided in five chapters, in which the decisions taken are gradually understood throughout the process.

Chapter number two presents some key definitions and literature review about overtopping and coastal floods, based on studies of identical nature, using both physical and numerical models. With this chapter it is intended to justify the current state of knowledge.

In the third chapter it is explained how to work with the software MIKE 3 Wave FM (M3), in a user perspective. This chapter will not be about all the mathematical and computational processes and their challenges, but rather a detailed explanation, without being too exhaustive, in order to not deviate from the central objective of the work, although it is sometimes necessary to go into detail to justify certain decisions and problems. It will be also approached its graphic aspect as well as the functionalities and applications.

After the explanation of the software, in the fourth chapter the simulation process adopted to set the setup during the internship in the DHI will be explained. The idea is to go along with the decision-making process and the reasons why certain decisions were taken in detriment of others.

In the fifth and last chapter all the conclusions of the study will be summarized, and some considerations will be made that could guide a future and more accurate scientific development of the art.

2

STATE OF THE ART

2.1 INTRODUCTION

Being MIKE 3 Wave FM a software that has just been released, it is in the interest of DHI that it should be tested by a user who is not familiar with this type of software. This challenge would prove the software to be user-friendly to someone with the knowledge in the field of coastal engineering. At the same time, allows the software to be tested by opening the possibility to further system improvements.

However, the main goal of this thesis is to make a sensitivity analysis of the numerical model using the data collected in the physical model to compare the results. The software used was the brand new DHI software, M3. This software allows to simulate numerical models with accurate calculations and also with 3D view: it enables a precise representation of the porous structure, which simplifies the interpretation and understanding of the events.

To be possible to use these types of programs and expect good results, it is necessary to understand phenomena related with the behaviour of the sea. The dynamic developments that occur in the nearshore region are produced by a number of different phenomena that create a fluid motion of the water, as coastal currents, tides and tidal currents, internal and surface waves, storms surges and tsunamis (Horikawa, 1989).

The most relevant phenomena in the surf zone can be classified in four different types, according to (Sánchez-Arcilla *et al.*, 2009):

- Sediment transport and corresponding changes in morphology, with a characteristic time scale of 1 day to 1 month, and a spatial scale between 100 m and 1000 m;
- Currents (non-oscillatory flow), with time scales between 10 minutes (min) and 1 hour (h), and spatial scales similar to those of sediment transport;
- Organised oscillatory flows (*i.e.*, wind waves, infra-gravity waves), with time scales ranging from 10^{-1} seconds (s) to 10 min, and space scales from 1 to 100 m;
- Random oscillatory flow (turbulence), whose length scales are between 10^{-3} to 10^1 s, and with small (10^{-4} to 10^{-1} m) spatial scales.

In a wide-ranging overview, it can be said that the main features in coastal hydrodynamics are the wind waves, generated by the stress exerted on the ocean surface by the wind. As these waves travel from deep waters into shallower regions, they become more non-linear and dissipative, transferring energy from the peak of the spectrum to higher and lower frequencies (Fitri *et al.*, 2015). Eventually, the proximity of the sea bottom will induce the breaking of the waves, producing a severe increase in the marine turbulence level, and generating different types of currents, which may extend beyond the surf zone (Lozovatskyv *et al.*, 2012), Fig. 2.1.

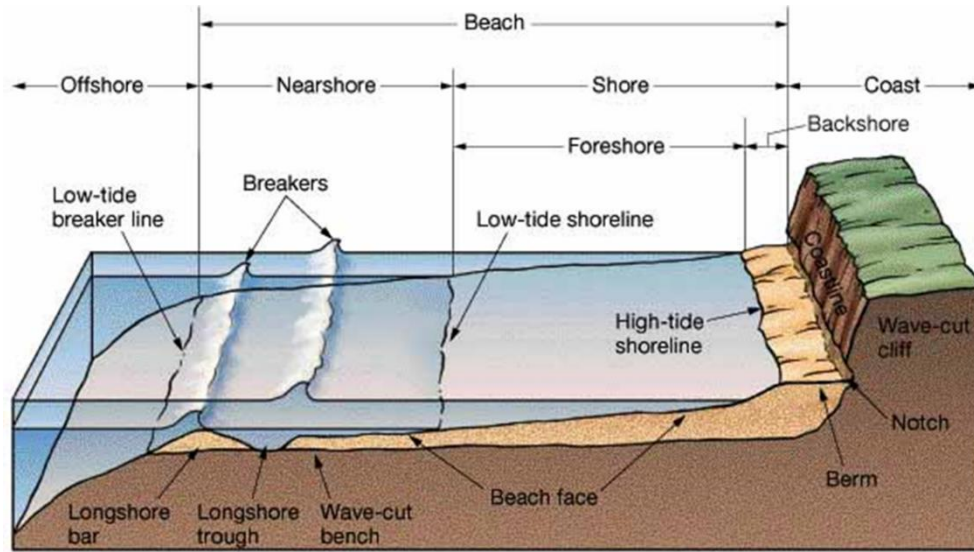


Fig. 2.1 - Beach zones (Thuy, 2018).

The basic theory of the generation of storm surges and waves by wind is in principle understood (Goring, 1979), although some details still need elucidation. For example, the wind stress is modelled empirically very successfully although the actual process of transfer of momentum and energy from the atmosphere to the sea via wind waves is very complex and not fully understood (Janssen, 2005). Meanwhile, there are some interactions between the mean circulation and the wind waves (Peregrine *et al.*, 1983).

2.2 DEFINITION OF KEY PARAMETERS

Usually, the wave used to calculate the wave run-up and overtopping is the incident significant wave height (H_{m0}), at the toe of the structure. It can also be called spectral wave height and it is defined with the m_0 (incident spectral momentum) as $H_{m0}=4(m_0)^{1/2}$ (Van der Meer *et al.*, 2016). Another way to define the significant wave height is the average of the highest third of the waves ($H_{1/3}$), but this definition is less accurate than the previous one, especially in situations of shallow waters. To calculate the wave height, it must be used the entire energy spectrum, including also long-waves.

Regarding the wave period, in the case of regular waves, it is very simple, however, when the waves are irregular, the situation is different. There are many types of periods, but the more common ones are:

- Peak period (T_p), gives the peak of the spectrum;
- Average period (T_m);
- Significant period, ($T_{1/3}$), that is the average of the highest 1/3 of the waves.

This parameter can be very important because it affects the transformation of the wave during the approximation to the coast.

The wave steepness (s_o) is a ratio of the significant wave height to offshore wavelength (L_o), $s_o=H_{m0}/L_o$. The purpose of this parameter is to explain some wave characteristics. For instance, $s_o=0.01$ means a swell sea, and a steepness between 0.04 and 0.06 a wind sea.

The breaker parameter or Iribarren number is defined as follows:

$$\xi_{m-1,0} = \frac{\tan \alpha_i}{\sqrt{\frac{H_{mo}}{L_{m-1,0}}}} \quad (2.1)$$

In this equation, α_i is the angle of the slope of the seaside of the structure and $L_{m-1,0}$ the deep water wave length (Van der Meer *et al.*, 2016). This parameter is very useful because it can identify certain types of wave breaking, Fig. 2.2.

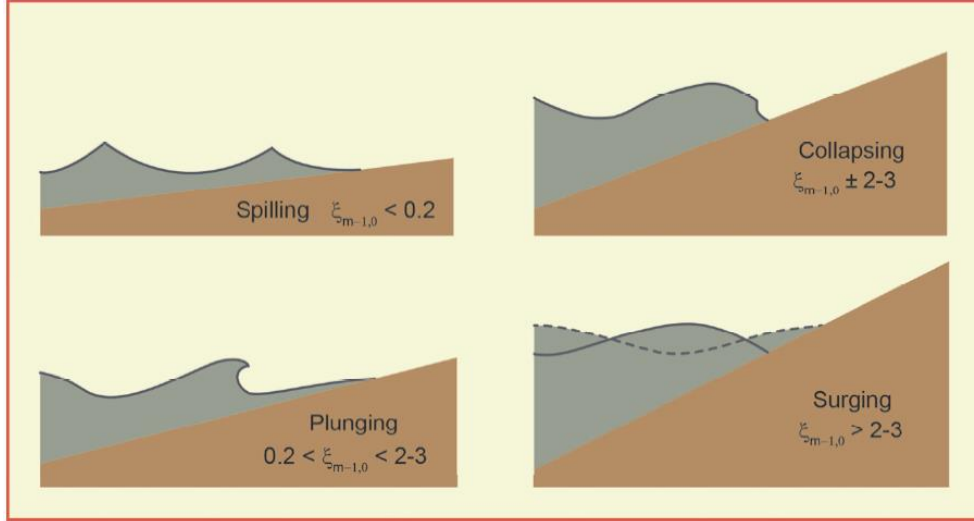


Fig. 2.2 - Wave breaking on a slope, types of wave breaking (Van der Meer *et al.*, 2016).

Once the sand moves during the different seasons and with big waves, the toe position may change. If the depth of the toe increases significantly, the increase of the wave height should be considered.

Three types of foreshore (area in front of the coastal structure) are defined: deep, shallow and very shallow. If the water is shallow or very shallow, the shoaling and wave depth limiting should be considered.

With the variation of the tidal cycles, seasonal cycles, storm events and human-induced changes to the coast, beach slope can also vary. Alongshore when the beach gets narrower or wider, the slope can also change (Doran *et al.*, 2015).

Berm is a kind of platform that serves to decrease the crest level of the structure. Its slope varies between horizontal and 1:15. The width of a berm cannot be greater than one quarter of a wavelength (Van der Meer *et al.*, 2016). It is frequently located on a dike to improve the wave defences' characteristics.

The crest is the highest point of the structure and when is related to the water level is defined as the crest freeboard (R_c). It is also the point on the structure where the water can no longer flow back to the seaside.

When the structure is vertical, a vertical up-rushing wave can produce some overtopping over the crest and partly fall back to the seaside. A good way to reduce overtopping is to build a return wall in the top of the vertical wall. This causes the up-rushing waves to reflect in the structure and return to the sea, decreasing overtopping.

The porosity is the empty space inside the structure in which the water can penetrate. It describes the fraction of voids in the material and therefore variates between 0 and 1. The porosity, geometric shape and the surface of the material can lead to different overtopping values (Yu, 1996). The roughness of

the slope is responsible for the energy loss of the wave during run-up. To increase energy losses, irregular surfaces should be selected.

Wave overtopping is a complex phenomenon to model. It involves wave shoal, wave breaking, wave reflection, turbulence and possibly wind effects on water spray (Salman and Raggal, 1999). This phenomenon is defined as the average discharge per meter of width in the crest of the dike. There is no constant discharge, since the process is actually very random in time, space and volume. As expected, higher waves will generate a larger volume of water over the crest.

2.3 TYPES OF COASTAL DEFENCE STRUCTURES

Coastal defence structures mitigate the erosion problem in a specific area but may aggravate or anticipate it in other places, once they can accumulate sediments in one side. Thus, these sediments remain in the updrift side of the defence structure reducing the amount of sediments, which may increase the incidence of erosion problems downdrift. Although, this is necessary to mitigate a serious problem that threatens populations, private and public property. These structures can be classified into four types:

- Cross works;
- Longitudinal works;
- Artificial nourishment and sand by-passing;
- Mixed solutions.

An example of a cross work defence is a groyne field, Fig. 2.3. A groyne field is an hydraulic structure placed crosswise to the shoreline, retaining greater or lesser percentage of sediments, due to longitudinal transport, as function of its length, height and constructive methodology. They can also be used to increase the width of the beach and are traditionally made of rock, concrete or timber, but in the last decades started to be constructed with geotextile sandbags.

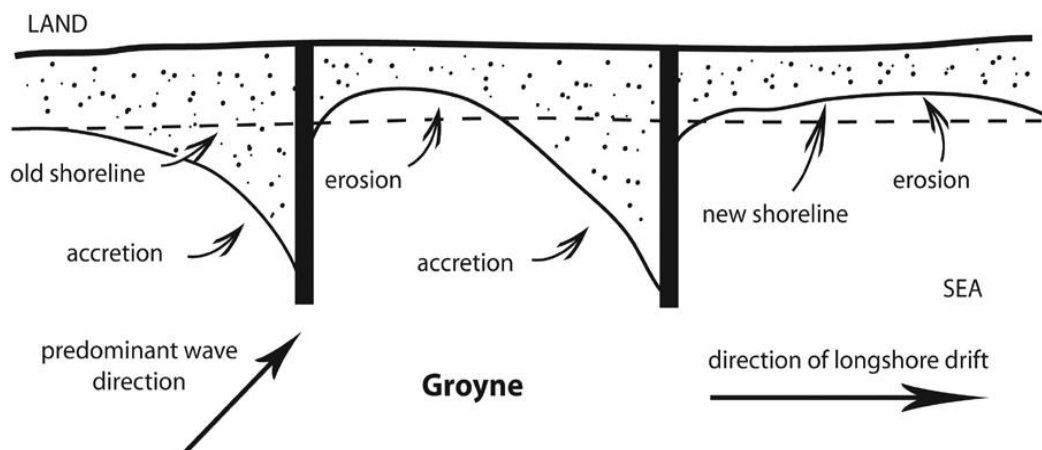


Fig. 2.3 - Groyne field and its effects (O'Keefe, 1978).

In the detached longitudinal works type, the structure is built offshore, usually parallel to the shoreline. This kind of coastal defences encourages deposition of sediments on the beach side of the structure. The sediments' deposition creates a coastal feature known as tombolo, Fig. 2.4.

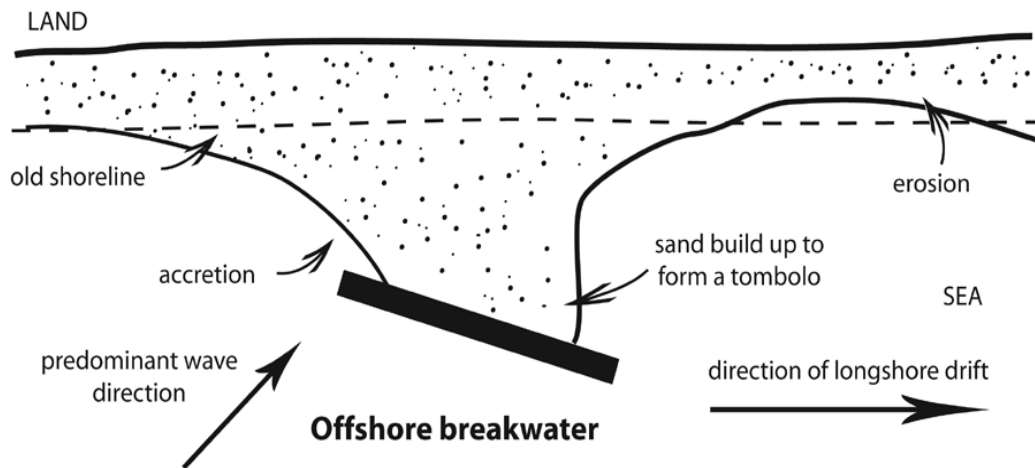


Fig. 2.4 - Offshore breakwater and its effects (O'Keeffe, 1978).

Artificial nourishment and sand by-passing are a type of defence that maintains the natural landscape. This kind of defence can be difficult to construct because the material used needs to match the local grain size and composition. If the material used is smaller than the natural one, it may be quickly eroded through wave action. However, to maintain the beach, a periodic re-nourishment is required, Fig. 2.5.



Fig. 2.5 - Beach nourishment (Martin County Florida).

The name is self-explicative: the mixed solution is the combination of artificial nourishment with sand and other works, cross or longitudinal. This coastal defence work is more complete but at the same time more expensive.

2.4 PHYSICAL MODELLING

Although the present work has used numerical modelling, some considerations must be made about physical models, because it is the predecessor and complementary process. The physical modelling in coastal engineering is a good approximation of the maritime phenomena in port and coastal structures. It can be used as a support of maritime work's projects and also to calibrate or validate numerical models.

Physical models as a physical experiment is an important research procedure because of strong points such as (Van Os *et al.*, 2004):

- Observability: investigate mechanisms that are difficult to observe in nature;
- Measurability: isolate phenomena that are difficult to measure in nature;
- Repeatability: investigate the range of inherent uncertainty of phenomena;
- Input control: various realistic input time series, to assess the predictive capability of numerical models;
- Process control: simple situations in order to validate the results of theoretical behaviour analyses;
- Overview: spatial pattern formation and requiring synoptic measurements;
- Robustness: modelling of processes that cannot be modelled numerically.

Depending on the type of studies, physical modelling can be two-dimensional or three-dimensional. The first case is made in a wave flume and a profile of a structure is reproduced. Fig. 2.6 shows the flume of DHI facilities in Denmark where such simulations are performed.



Fig. 2.6 - Wave flume at DHI facilities.

In the second case, the tests are made in a wave tank with the bathymetry and the geometry of the structures that will be studied. The shoreline in the downward side of the defence structures is also covered in this study, Fig. 2.7.



Fig. 2.7 - Wave tank at DHI facilities.

In both cases, the final goal is the behavioural analysis of the structures under specific sea conditions. The waves are the central factor of any coastal study, so, after the construction of the bathymetry and structures, tests are made in a way that replicate the wave conditions of the area.

To understand the impact of the waves in a maritime structure or in the coast line due to natural causes as well as human interventions, physical models play an important role. For the physical model tests, some characteristics need to be selected, like:

- Type of waves;
- Model geometric scale ratio;
- Representative sea states;
- Duration of the time series;
- Free and bound long waves;
- Two or three-dimensional waves.

One disadvantage of physical modelling is the price and the time spent to make a full study. It is necessary to have proper facilities that allow to make this type of tests, what turns to be very exclusive to companies with economic power.

2.5 NUMERICAL MODELS

Due to the improvement of the increasingly powerful computing machines and the development of numerical models, the use of this type of tools begins to be even more attractive. Numerical simulations are useful tools for designing coastal structures as well as for understanding natural hydrodynamic processes in the field of coastal engineering (Hieu and Tanimoto, 2006).

The motion of an incompressible Newtonian fluid can be described accurately by the Navier-Stokes equations (Van Gent, 1995). These equations were created from applying Isaac Newton's second law to fluid motion, with the assumption that the stress in the fluid is the sum of a diffusing viscous term and a pressure term (Fefferman, 2006).

Navier-Stokes equations are accurate to determine the velocity and pressure field but at the same time very computational demanding. That is why the Reynolds Averaged Navier-Stokes (RANS) solver is commonly used, also known as Reynolds Average Simulation (RAS). This kind of models are non-linear and dispersive, the non-linearity of wave interaction, such as their bursting are implicit characteristics in these equations. In this way, these models allow the modelling of the interaction of the waves with the coastal structure (Lara *et al.*, 2013).

As it can be seen in Fig. 2.8, in the RANS equation method, the turbulence is not simulated but modelled, thanks to models that close the systems of equations as kEpsilon (κ - ϵ), the kOmega (κ - ω) and the kOmega SST (κ - ω SST). The most common, and used in M3, is the κ - ϵ that allows the simulation, in average terms, of the characteristics of a turbulent flow, considering two variables only: kinetic turbulence (κ) and kinetic dissipation (ϵ) of κ (Versteeg and Malalasekera, 2007). This kind of models is more adequate for situations of relatively small pressure gradients, having a low performance in the case of higher pressure gradients, such as confined flows (Higuera, 2015)

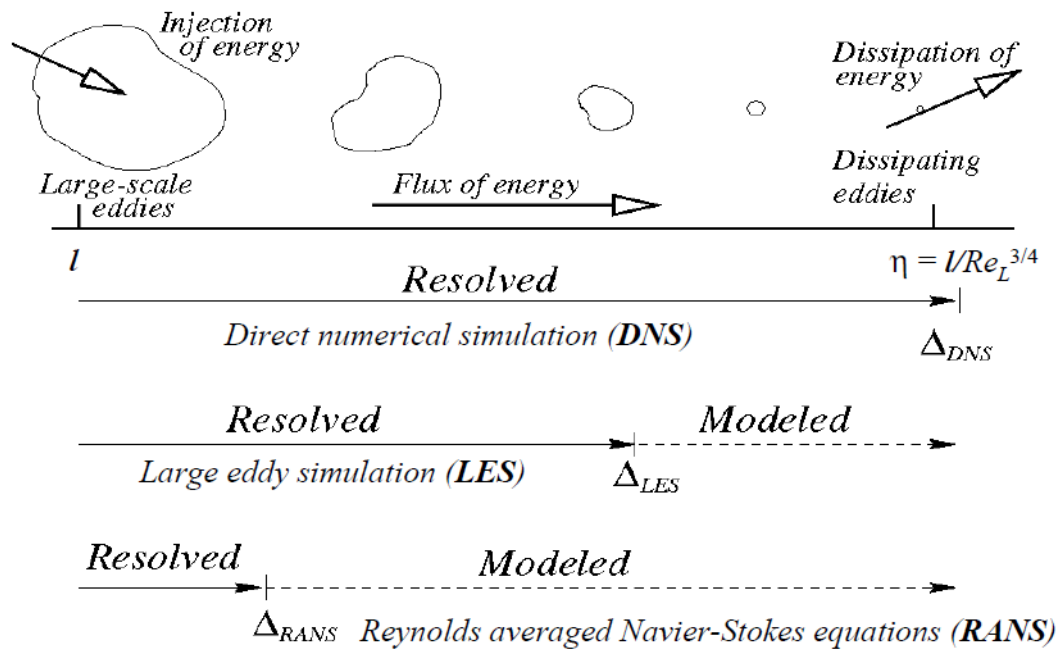


Fig. 2.8 - Turbulence simulation methods using RANS solver (Bakker, 2006).

Barotropic tides and surges are usually modelled using shallow-water equations since they have very long wavelengths compared to the water depth (Goring, 1979). Heaps (1983), Flather (1981) and Pugh (1988) reviewed earlier work on numerical modelling of the storm surges. Small area models are subjected to errors in boundary conditions and tidal models may omit frequencies, local effects of the tide generation forces. Shum (1997) reviewed ocean tidal models, concluding that the models agree well in the deep ocean.

In the numerical models based on Boussinesq-type equations, adding a new term to the depth-integrated momentum equation parameterizes the wave breaking process. While Zelt (1991), Karambas and Koutitas (1992) and Chen *et al.* (2000) used the eddy viscosity model, Brocchini *et al.* (1994) and Schäffer *et al.* (1993) employed a more complex roller model based on the surface roller concept for spilling breakers.

2.6 VALIDATION RESULTS

Before any release of a numerical model software, it is necessary to compare the results produced by the software together with the results obtained with physical model tests acquired through the development of the state of the art. Therefore, to ensure that M3 is reliable, capable of producing accurate and precise results, a validation report was made. Due to the ongoing software development, this document is still confidential. In this validation report, many cases of study that focus in different aspects were presented, such as:

- Standing Wave in a Deep Basin;
- Propagation of Regular waves over a Submerged Bar;
- Non-linear Refraction-Diffraction of Regular Waves over a Semi-circular Shoal;
- Wave Runup on a Gently Sloping Beach;
- Shoaling and Breaking of Regular Waves on a Gently Sloping Beach;
- Rip Channel;
- Porous Dam-break;
- Regular Waves Interacting with Vertical Porous Breakwater;
- Wave Breaking over a Submerged Porous Breakwater;
- Breakwater Overtopping;
- Submerged landslide.

In order not to disperse to many features of M3 software, only cases that resemble the *Afsluitdijk* case will be considered.

2.6.1 PROPAGATION OF REGULAR WAVES OVER A SUBMERGED BAR

The first case to be analysed is the propagation of regular waves over a submerged bar. Otta *et al.* 2015, performed a series of tests to compare between experimental measurements and numerical results showing that the model can be used for satisfactory prediction of nonlinear transformation of non-breaking waves over varying depth. The flume test selected to replicate was the case for regular nonbreaking waves with a wave period of 2.02 s and a wave height of 0.02 m. The bathymetry of the submerged bar has a slope of 1/20 in the upward side and 1/40 in the downward side, and a constant depth of 0.4 m in both sides of the bar and a 0.1 m water depth on the top of the bar, Fig. 2.9.

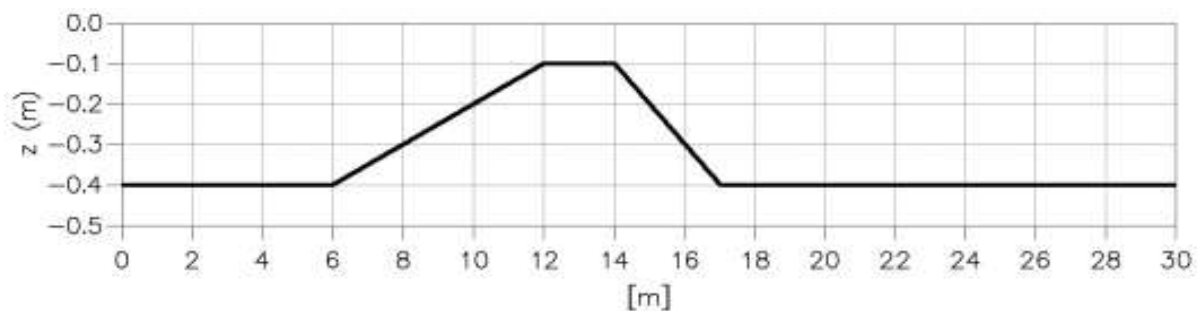


Fig. 2.9 - Bar bathymetry (DHI, 2019a).

The flow in this case study is one-dimensional, therefore, the width of the channel was defined as a single element in the numerical simulation. To the horizontal discretization a quadrangular mesh with $\Delta x = 0.02$ m of size was used. To the vertical 3, 5 and 10 non-equidistant sigma layers were used. At the same time 3 equidistant sigma layers were also simulated. To generate the waves a relaxation zone defined by a line from $(x,y) = (2.0 \text{ m}, 0.05 \text{ m})$ to $(x,y) = (2.0 \text{ m}, 0.0)$ was used and the width of the ramp-up zone was 1.8 m in length. A Stokes 5th order wave theory was used to create the waves. In the

downward side of the bar was created a 4 m sponge layer to absorb the waves' energy. During the simulations, the HLLC Riemann solver and the horizontal and vertical eddy viscosity was turned off.

The goal of this test was to show the good correlation between the surface elevation in the experimental data and the numerical results obtained using M3. The position where the surface elevation was measured was $x=13.5$ m, $x=17.3$ m, $x=19.0$ m and $x=21.0$ m, respectively.

The simulation was performed with 10 non-equidistant sigma layers and the results are very similar, Fig. 2.10. This chart illustrates the surface elevation on the top of the submerged bar.

The surface elevation is slightly different in Fig. 2.11, but the accuracy remains fairly similar. This test was also carried out with 10 non-equidistant sigma layers. The position of these wave gauges was measured in the end of the downward slope.

In Fig. 2.12 and Fig. 2.13, respectively $x=19.0$ m and $x=21.0$ m, the biggest differences are in some troughs but even so the accuracy of the surface elevation is substantial. Similarly to the other tests, 10 non-equidistant sigma layers were used here.

Also, in the $x=21.0$ m an analysis to compare the different number of layers with the experimental data was made. The first thing to notice is that the different number of layers is very similar but at the same time, when the number of layers increases, the results are more accurate, Fig. 2.14.

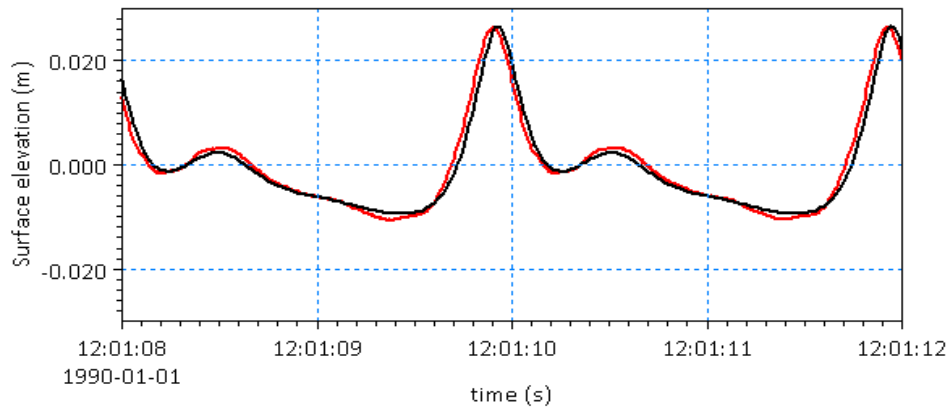


Fig. 2.10 - Surface elevation at $x=13.5$ m. Black line: M3, Red line: Experimental data (DHI, 2019a).

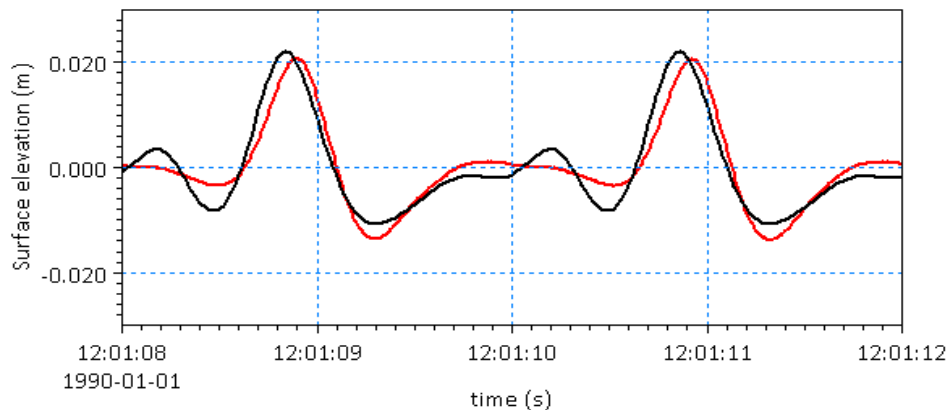


Fig. 2.11 - Surface elevation at $x=17.3$ m. Black line: M3, Red line: Experimental data (DHI, 2019a).

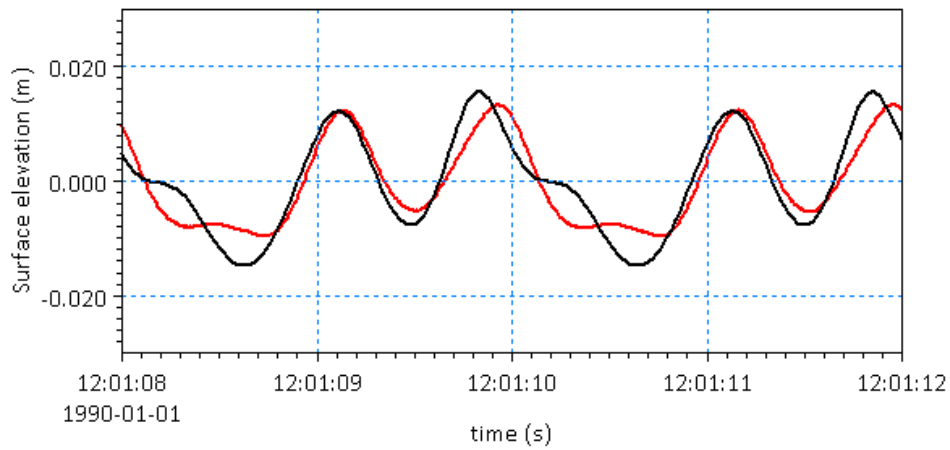


Fig. 2.12 - Surface elevation at $x=19.0$ m. Black line: M3, Red line: Experimental data (DHI, 2019a).

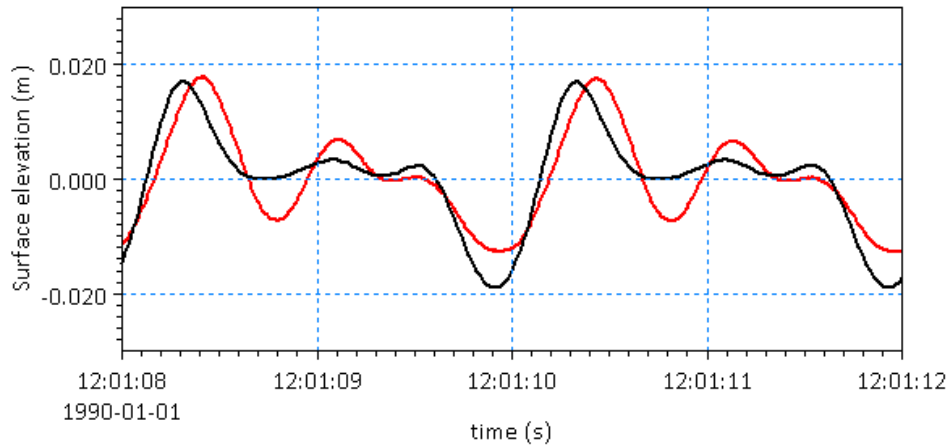


Fig. 2.13 - Surface elevation at $x=21.0$ m. Black line: M3, Red line: Experimental data (DHI, 2019a).

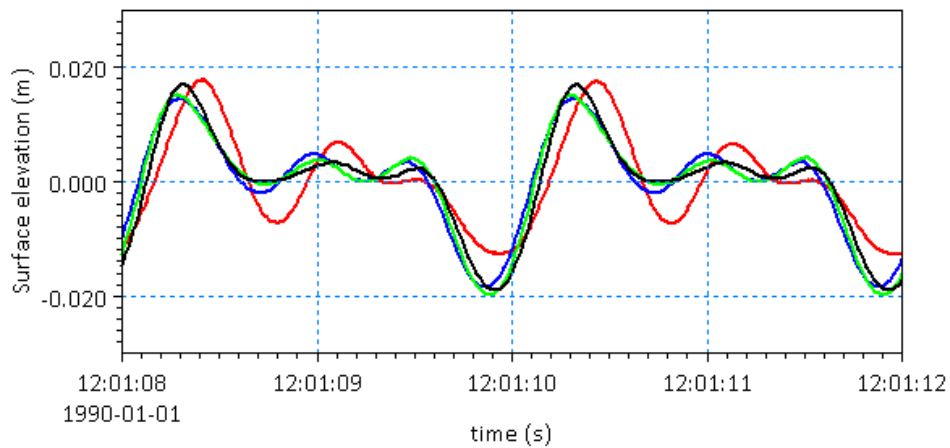


Fig. 2.14 - Surface elevation at $x=21.0$ m between 3 (green), 5 (blue) and 10 (black) non-equidistant layers with the experimental data (red) (DHI, 2019a).

2.6.2 REGULAR WAVES INTERACTING WITH VERTICAL POROUS BREAKWATER

Lara *et al.* (2012) studied the interaction of waves with porous coastal structures and the data was used to validate the interaction between waves and a porous breakwater.

The dimensions of the flume where the tests were conducted are: 22 m long, 0.585 m wide and 0.78 m high. A porous structure was placed such that the centre of the structure is located 11.519 m from the mean position of the wave maker, Fig. 2.15.

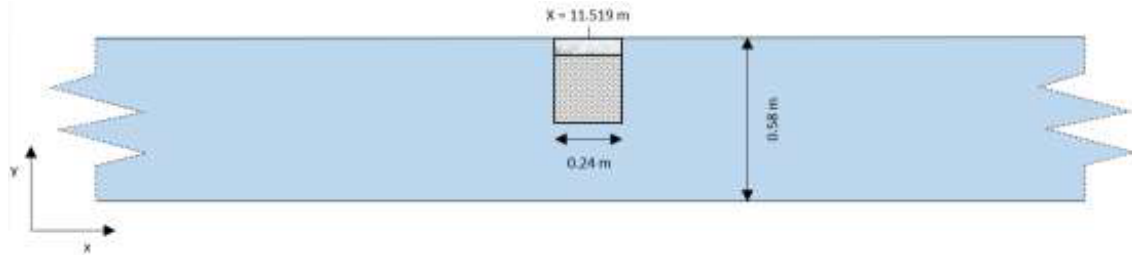


Fig. 2.15 - Position of the porous structure and respective dimensions (DHI, 2019a).

The flume was also equipped with 12 wave gauges to measure the surface elevation in some specific points, Fig. 2.16.

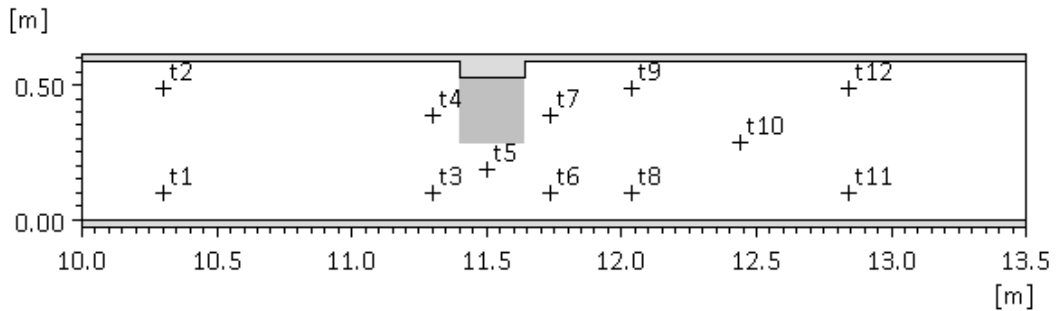


Fig. 2.16 - Wave gauge location (DHI, 2019a).

Regular waves were used, with a period of 2 s, a height of 0.06 m and a water depth of 0.25 m. The numerical experiments were configured with quadrangular elements with a grid spacing in the range $\Delta x = [0.02 \text{ m}, 0.04 \text{ m}]$ and $\Delta y = 0.015 \text{ m}$, prefacing a 2D mesh with 23460 elements. The area near the porous structure had a higher density of cells because it is the area of interest, Fig. 2.17. The vertical discretization was made using 10 equidistant sigma layers, and therefore, the total number of cells in the numerical simulation is 234600.

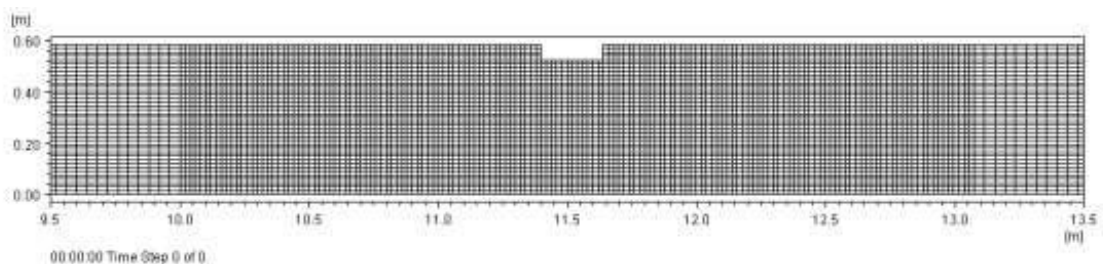


Fig. 2.17 - Computational mesh (DHI, 2019a).

The waves were generated using a stream function wave and the relaxation zone was positioned at $x=1$ m with a width of 0.8 m and using the values of the linear and non-linear friction parameters set to $\alpha=500$ and $\beta=2$. Simulations have been running with HLLC solver turned on. κ - ϵ formulation was used in the horizontal and vertical eddy viscosity (DHI, 2019a).

When comparing the results obtained by Lara *et al.* (2012) with those obtained with M3, the agreement is very good, Fig. 2.18. Although, in the end of the time series, some problems occurred due to wave reflection at the end of the flume in the wave gauge area.

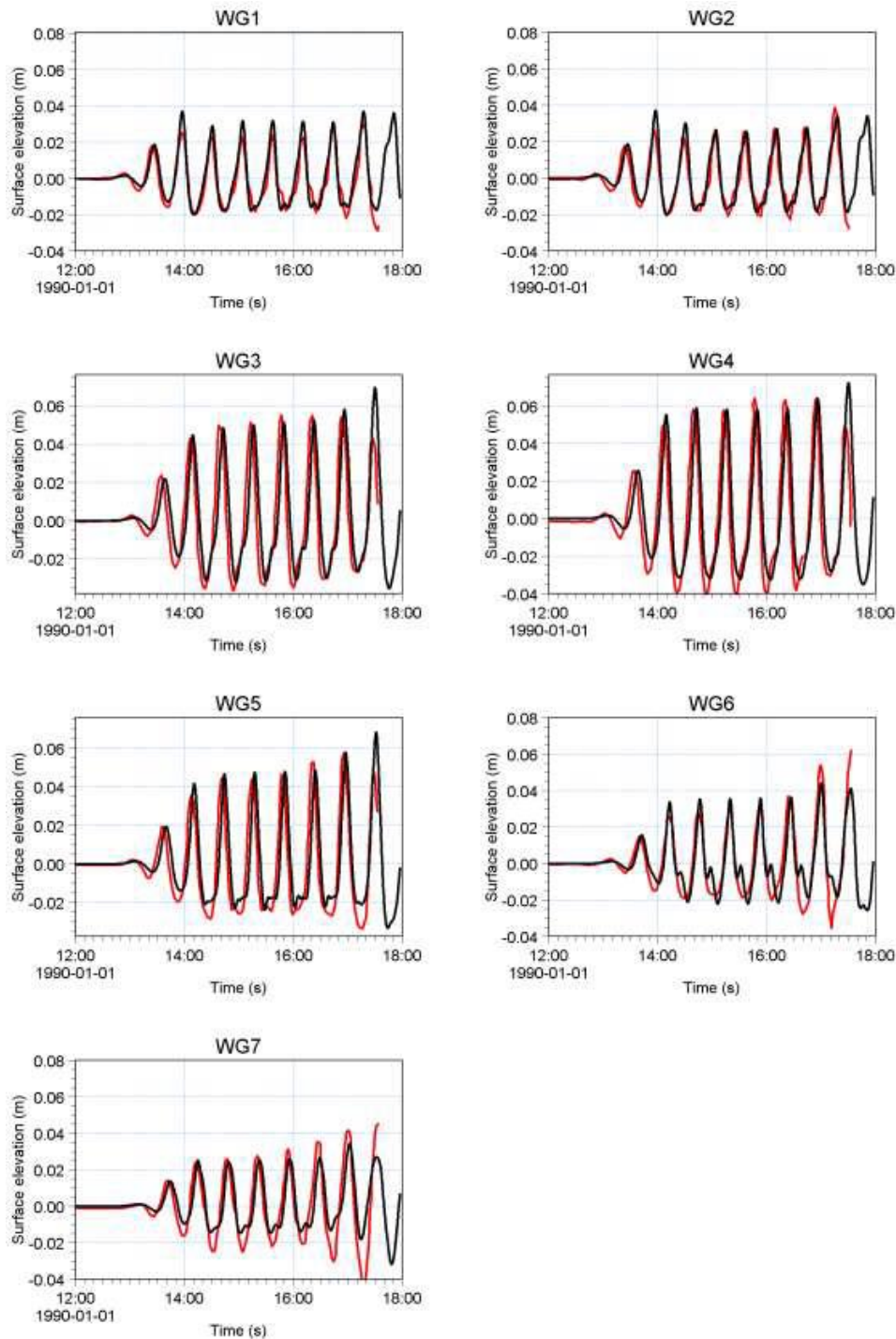


Fig. 2.18 - Surface elevation at WG1-WG7. M3 (black), experimental data (red) (DHI, 2019a).

2.6.3 WAVE BREAKING OVER A SUBMERGED POROUS BREAKWATER

The experimental data of this study was presented by Hieu and Tanimoto (2006). The dimensions of the flume where the tests were developed are: 18 m long, 0.4 m wide and 0.7 m high. The submerged porous breakwater is made of stones with 0.025 m of diameter, which results in a porosity of 0.45. The breakwater dimensions are presented in Fig. 2.19.

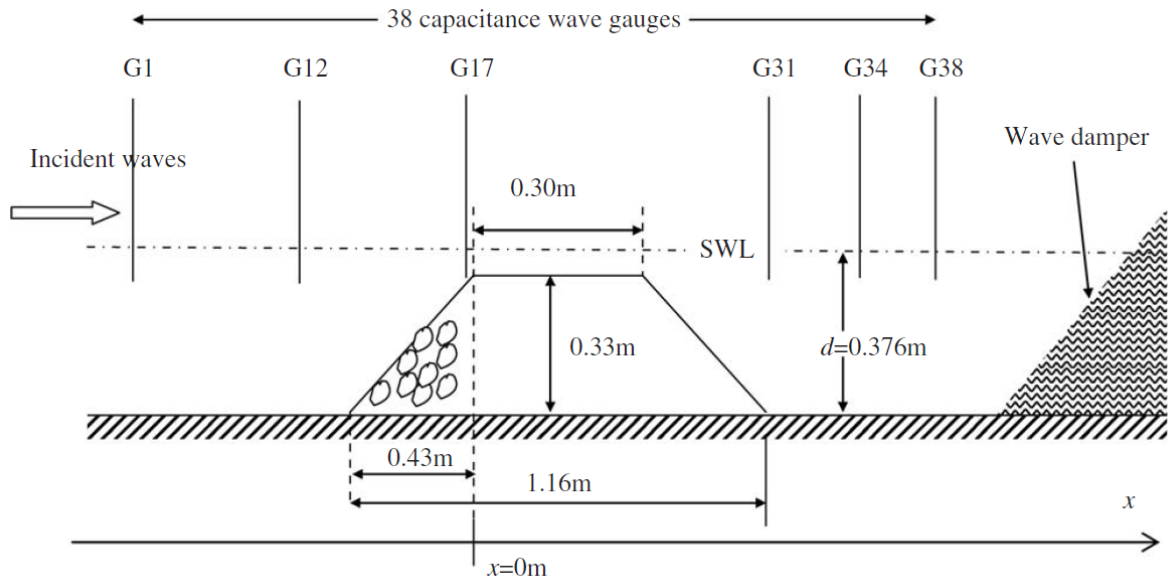


Fig. 2.19 - Sketch of the submerged porous breakwater model (Hieu and Tanimoto, 2006).

The generated waves had a period of 1.6 s and a wave height of 0.092 m. To avoid wave reflection at the end of the flume, a wave damper was placed. The origin of the coordinates system is in the bottom of the flume lined up with the end of the slope.

The numerical experiments were prepared in a flume of 20x0.4 m. The horizontal discretization was set as $\Delta x=0.02$ m and $\Delta y=0.4$ m. Once more, the width of the flume and the total number of elements is 1000 quadrangular cells. Vertically it is defined with 40 equidistant sigma layers, which results in a total of 40000 cells. The porosity structure was set as shown in Fig. 2.20.

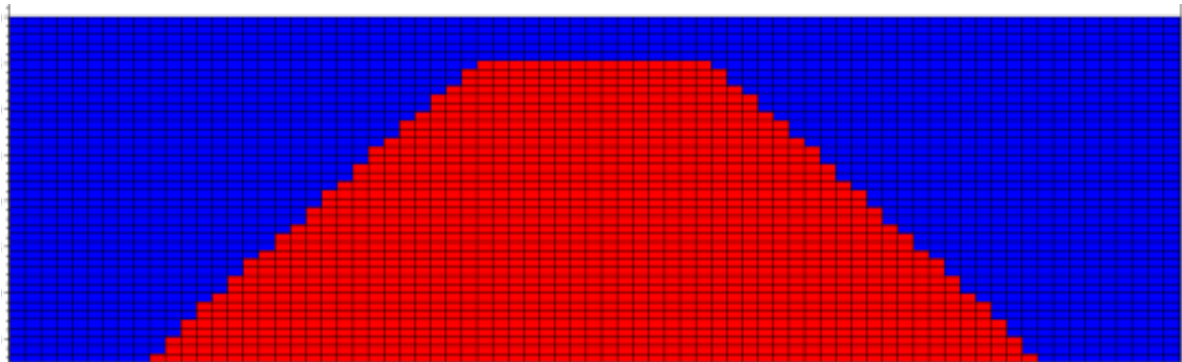


Fig. 2.20 - Porosity map (DHI, 2019a).

As can be seen in the following figures (Fig. 2.21 to Fig. 2.26), when overlapping the data obtained by Hieu and Tanimoto (2006) with that obtained with M3, the results are very good in the different wave gauge's positions.

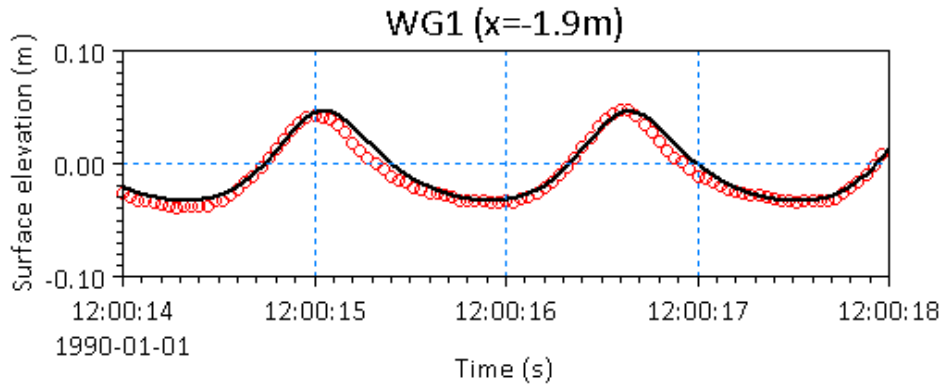


Fig. 2.21 - Surface elevation at $x=-1.9$ m. M3 (black), Experimental data (red) (DHI, 2019a).

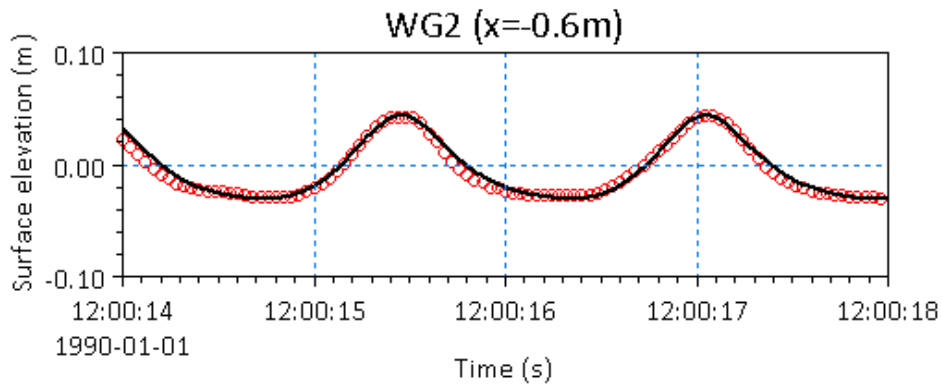


Fig. 2.22 - Surface elevation at $x=-0.6$ m. M3 (black), Experimental data (red) (DHI, 2019a).

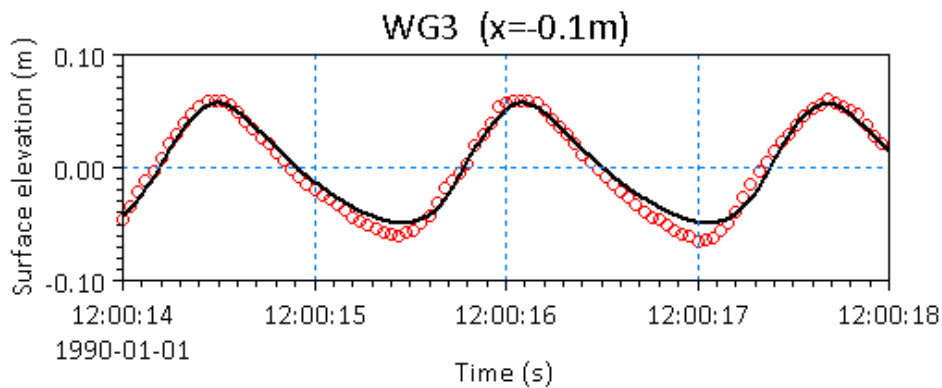


Fig. 2.23 - Surface elevation at $x=-0.1$ m. M3 (black), Experimental data (red) (DHI, 2019a).

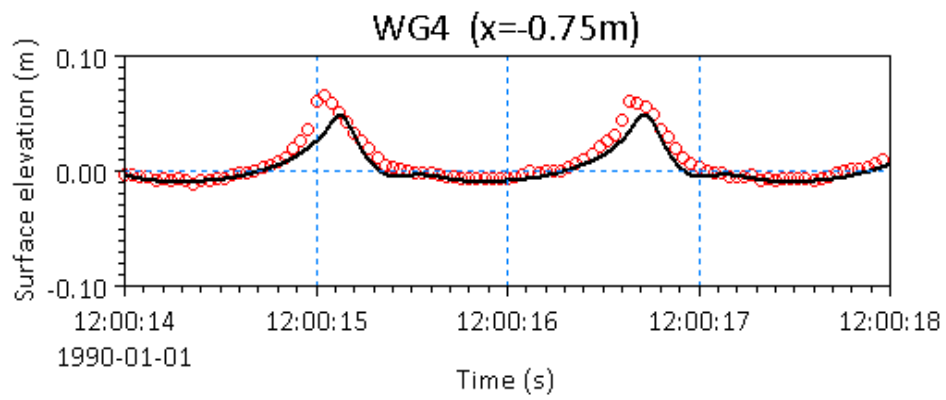


Fig. 2.24 - Surface elevation at $x=-0.75$ m. M3 (black), Experimental data (red) (DHI, 2019a).

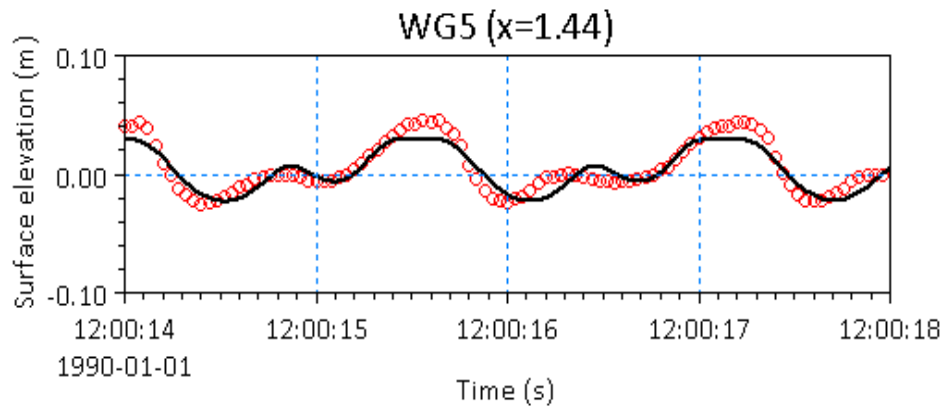


Fig. 2.25 - Surface elevation at $x=1.44$ m. M3 (black), Experimental data (red) (DHI, 2019a).

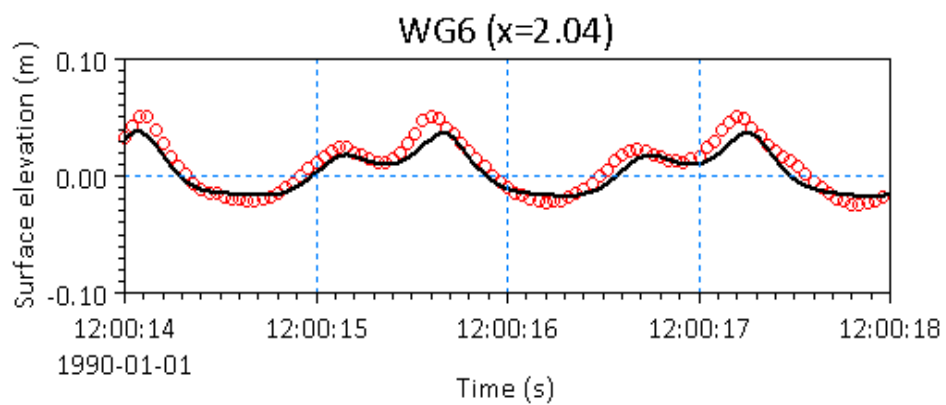


Fig. 2.26 - Surface elevation at $x=2.04$ m. M3 (black), Experimental data (red) (DHI, 2019a).

2.6.4 BREAKWATER OVERTOPPING

Bruce *et al.* (2009) performed a program of small-scale physical model tests of wave overtopping over breakwater structures. The purpose of the study was to better establish the influence of armour type and configuration on overtopping. The program consisted in determining the relative difference in overtopping behaviour for 13 different types and armour configurations.

The dimensions of the flume used were 16x1 m (Bruce *et al.*, 2009). As the flow of this simulation is considered as 2D, $\Delta y=1$ m (one element width) and $\Delta x=0.025$ m. Vertically, it is defined with 30 layers. The breakwater crest level is at 0.2812 m, the toe of the breakwater is placed 10 m from the wave generator and the slope of both sides is 1/1.5. In the physical model test, the breakwater is constructed with three materials: core, filter layer and armour layer, Fig. 2.27.

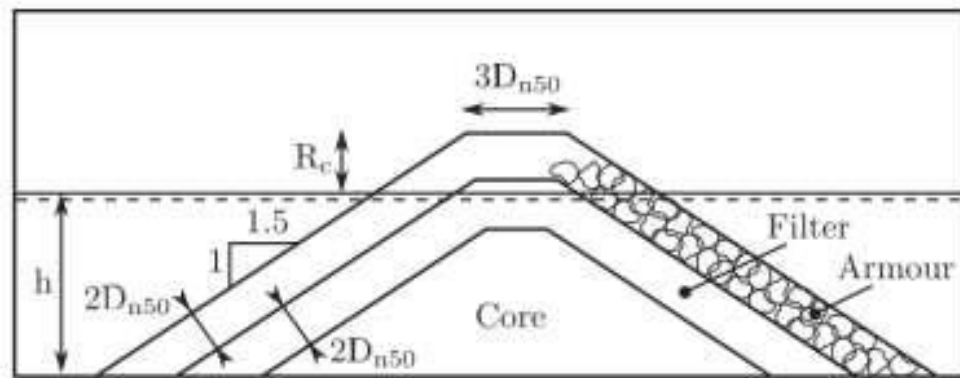


Fig. 2.27 - Sketch of the porous breakwater (DHI, 2019a).

Each layer is defined by the material and, therefore, by its thickness. The armour layer is made of natural rocks with a diameter of $d_{50}=0.03$ m, the grain diameter of the filter is 0.014 m and the core is 0.007 m. Laminar and turbulent coefficients were set respectively as $\alpha=500$ and $\beta=2$, and the porosity 0.4 for all three zones. The porosity map used in this numerical simulation is showed in Fig. 2.28.

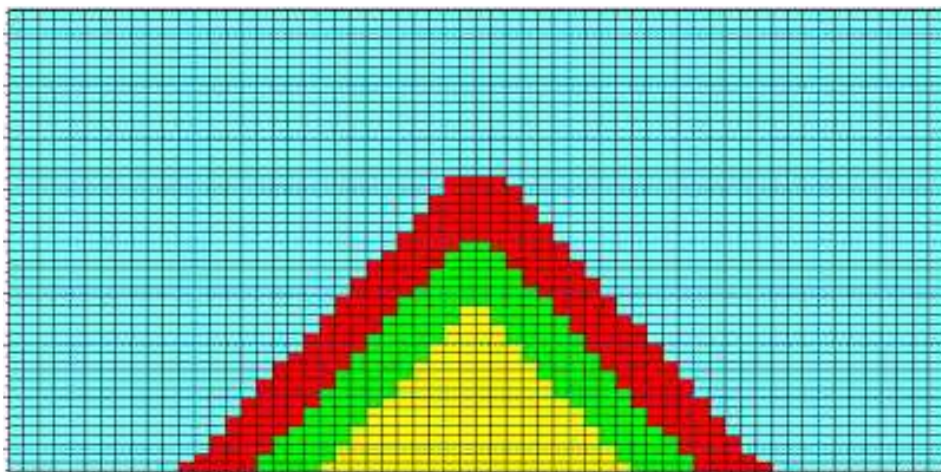


Fig. 2.28 - Porosity map. Armour layer (red), filter (green) and core (yellow) (DHI, 2019a)

According to Bruce *et al.* (2009), two different water depths were used in the experiments, $h=0.185$ m and $h=0.222$ m. Once the breakwater crest level is at 0.2812 m, the freeboard is respectively $R_c=0.0962$ m and $R_c=0.0592$ m. The wave generation zone is defined by the relaxation zone: Line from $(x,y)=(3.0$ m, 0.0 m) to $(x,y)=(3.0$ m, 1.0 m) and the width of the ramp-up zone is 3.0 m. They are generated as irregular waves based on the standard JONSWAP spectrum with a peak enhancement factor of $\gamma=3.3$. In the present example, $H_{m0}=0.111$ m, $h=0.222$ m and $T_p=1.56$ s. During the simulations, the HLLC Riemann solver and the horizontal and vertical eddy viscosity were turned on.

To measure the discharge in the crest of the breakwater it was necessary to insert a discharge line output on top of it. This allows to measure the flow and the accumulated discharge where the line was set.

Instead of using the accumulated discharge, to perform this kind of analyses usually it is used the overtopping rate. This parameter is obtained by dividing the accumulated discharge by the length of the simulations and making non-dimensional by the gravitational acceleration and significant wave height. The overtopping is presented as a function of the non-dimensional freeboard.

Fig. 2.29 presents three different sources of results: empirical, experimental and numerical. Empirical and experimental results are both presented in Bruce *et al.* (2009) and the numerical one is from M3. This chart is related with the impermeable breakwater and the agreement is very good. Nevertheless, with the increasing values of the dimensionless freeboard, the numerical results start to disperse, taking into account the empirical results. In Fig. 2.30, only one case of the numerical values is presented, but the result is near of the experimental values. This is the case of a porous breakwater.

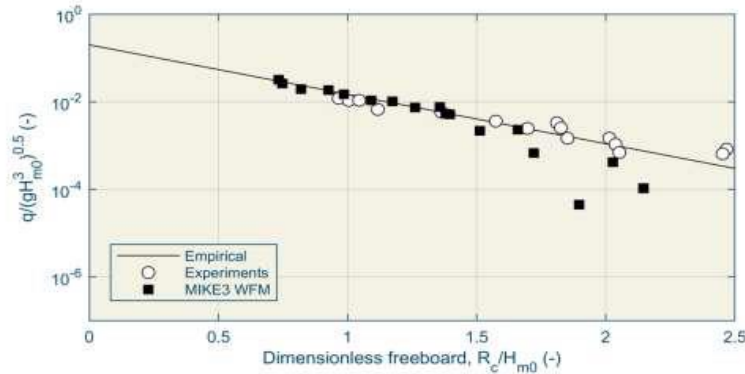


Fig. 2.29 - Comparison of the empirical, experimental and M3 overtopping for the impermeable breakwater (DHI, 2019a).

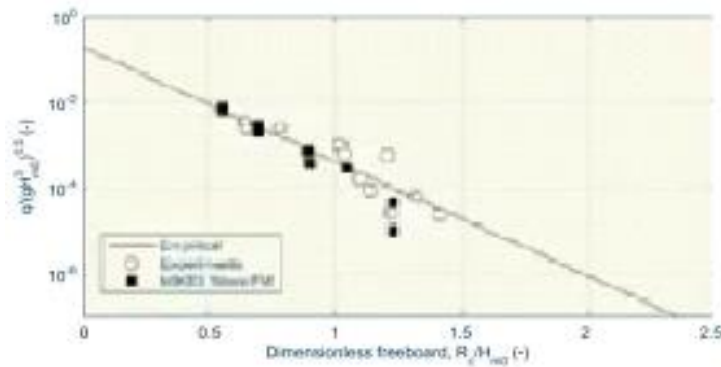


Fig. 2.30 - Comparison of the empirical, experimental and M3 overtopping for the porous breakwater (DHI, 2019a).

2.7 FINAL CONSIDERATIONS

Bearing in mind what was concluded in each study, there are sufficient conditions to state that numerical models in general, but M3 in particular, are trustable to assess physical processes and environmental impacts. This is a huge step in coastal engineering, since it opens the possibility to predict some extreme events due to adverse meteorological conditions, such as 50 or 100 year storm. Another use of numerical modelling software is the simulation of water level in coastal structures or the definition of hazard zones in communities near the coastline.

Another software used in this kind of numerical models' study is the SWASH software, created by Deltares. The governing equations are similar to those used in M3. It would be interesting to compare the two models, using the data provided. Pes (2013) study the accuracy of the SWASH model to predict the mean wave overtopping discharge in rubble mound breakwaters.

After the presentation of the previous cases, the conditions are fulfilled to proceed to the explanation of the software and consequent analysis of results in the following chapters.

3

MIKE 3 Wave FM

3.1 SOFTWARE INTRODUCTION

The DHI is a software engineering and development consulting firm, with a worldwide presence. They are responsible for the development of the M3 FM (Flexible Mesh), used in this thesis for the numerical modelling and consequent data analysis. The software is a non-hydrostatic wave model that can simulate complex wave processes from deep water to nearshore. This model increases the versatility and applicability of the MIKE software in offshore, coastal and port engineering. It is a phase-resolving wave model based on the 3D Navier-Stokes equations and with the free surface described by a height function. Since the surface elevation can only have a single value, the tracking of surface elevations are considered as an height function. The numerical techniques applied are based on an unstructured (flexible) mesh approach.

MIKE Zero is the base system for the M3 software, working as a Graphical User Interface common to all modelling systems. In addition to pre- and post-processing functions, presentation and visualization of results, it has included fundamental modules for the generation of basic data necessary for the application of the other modelling systems, such as M3. The modules Bathymetry Editor and Mesh Generator, provide an environment for creating, editing and presenting a two-dimensional model of bathymetry. Although they have the same function they are distinguished essentially by the type of mesh that is used for the generation of bathymetry. The Bathymetry Editor generates bathymetry on a rectangular grid with fixed horizontal and vertical spacing, while Mesh Generator generates bathymetry in flexible mesh, usually with triangular elements, although it also allows the application of a rectangular grid of variable spacing.

In the Bathymetry Editor, a projection system, an origin for the grid, the orientation and the extension of the grid must be defined. There is also defined an equidistant spacing for the grid in both the x-direction and the y-direction, thus giving rise to a grid of rectangular elements.

In Mesh Generator a projection system and polygons must be defined that constitute areas of the model where the triangular mesh is generated. The operation of the Mesh Generator is based on arcs, nodes and vertices. An arc is a set of straight segments closed by two nodes or enclosed in itself in the node that gave rise to it. The vertices are the vertices of the arc segments. Nodes are distinguished from vertices also by having assigned a code value.

Since this chapter is intended to explain the software from a user perspective, the guide document will be the M3 User Guide, available on the company's website, and the experience as a software user, (DHI, 2019b). Once the process of numerical modelling is a complex process, it is important to plan the entire

operation to minimize the errors and have reliable results that can be used to understand the physical development. The check list proposed by DHI consists in these six tasks:

1. What are the wave's characteristics?
2. Collect data related with the problem;
3. Set-up up the model;
4. Calibrate and verify the model;
5. Run the simulations produced;
6. Present the results.

The setup editor of M3 has three different windows, Fig. 3.1. In the left part of the monitor it is the navigation tree. That shows the structure of the model setup file and is used to navigate through the different sections of the file. When an item is selected in the navigation tree, the corresponding editor pops up in the central window.

The central window shows the editor window. This editor is where the user inserts all the parameters in the software. The content of this window is specific for the selected item and contains different pages. The last window, the validation window, is in the bottom of the editor and shows the possible errors that may occur during the simulation. Once is dynamically updated, it can be analysed instantly by double-clicking on the error. This action will take the user to where the error occurs.

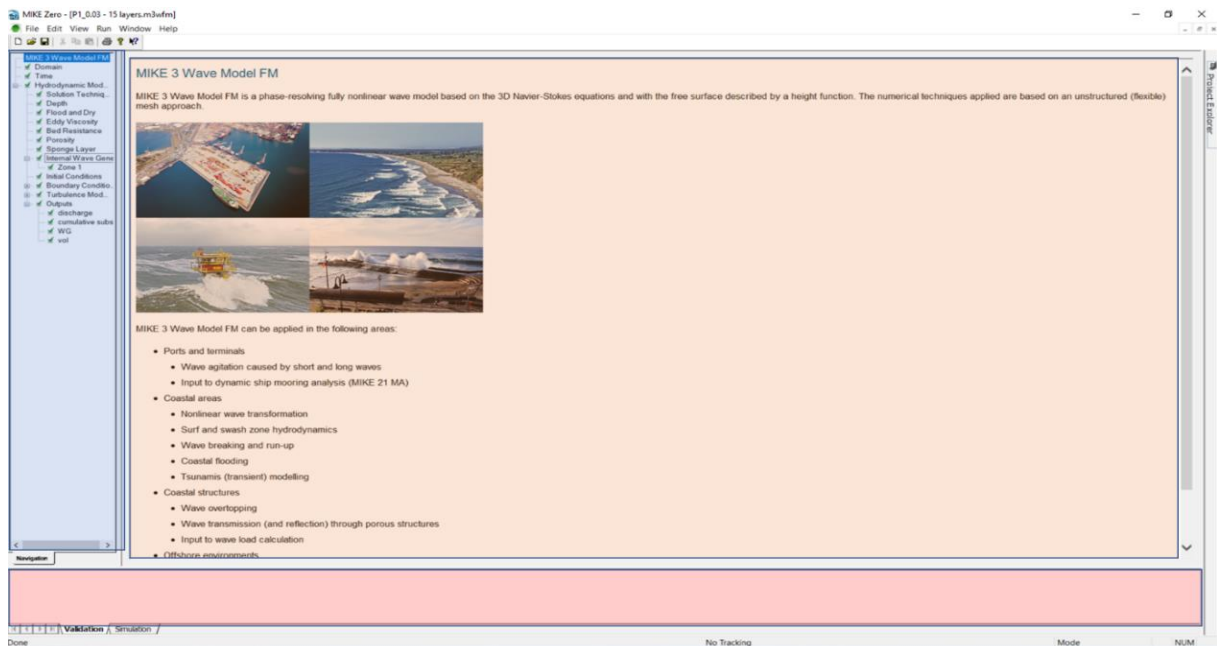


Fig. 3.1 - M3 home screen. The navigation tree (blue), the editor window (orange) and the validation window (red).

One of the tools that the software has is the Online Help. When the user press F1 in the keyboard, the online help is activated and opens an internet page where it is explained what is necessary to introduce and is often accompanied by a mathematical demonstration. This tool is very useful because it provides justifications that help the user to choose between different parameters.

Once the objects of study are the waves and their behaviour, naturally the first step proposed was not the one who was immediately analysed. The first step of this study was to select and interpret the data present in the Corneel Report and pick the information that it will be needed to proceed the simulation.

To configure the model it is necessary to understand world phenomena and its events and have some theoretical background in coastal hydraulics and oceanography.

3.2 INSIDE MIKE 3 WAVE FM

3.2.1 DOMAIN

In the navigation window, the first item in the list is the Domain. It is here where the mesh file is inserted, the vertical mesh is defined, as layers, and some specifications related with the domain and boundary conditions.

Providing the program with appropriate mesh and bathymetry is one of the most important steps to have reliable results. To define the horizontal domain, two dimensions are required: length (dx) and width (dy). When the user is creating a mesh, one of the initial questions is which resolution should be used to define it. A layered mesh is used, and both horizontal and vertical domains have a structured mesh. In Fig. 3.2 an example of an unstructured mesh with sigma coordinates can be seen.

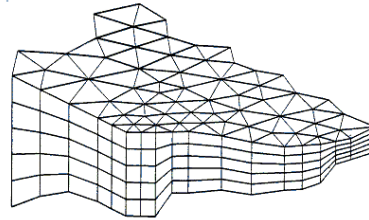


Fig. 3.2 - Example of an unstructured 3D mesh (DHI, 2019b).

In the case of studying a coastal stretch, the file with the information of the aerial survey done in the zone is inserted as an American Standard Code for Information Interchange (ASCII) file in MIKE Zero Mesh Generator. This tool is used for the generation and handling of unstructured meshes, including the definition and editing of boundaries, Fig. 3.3.

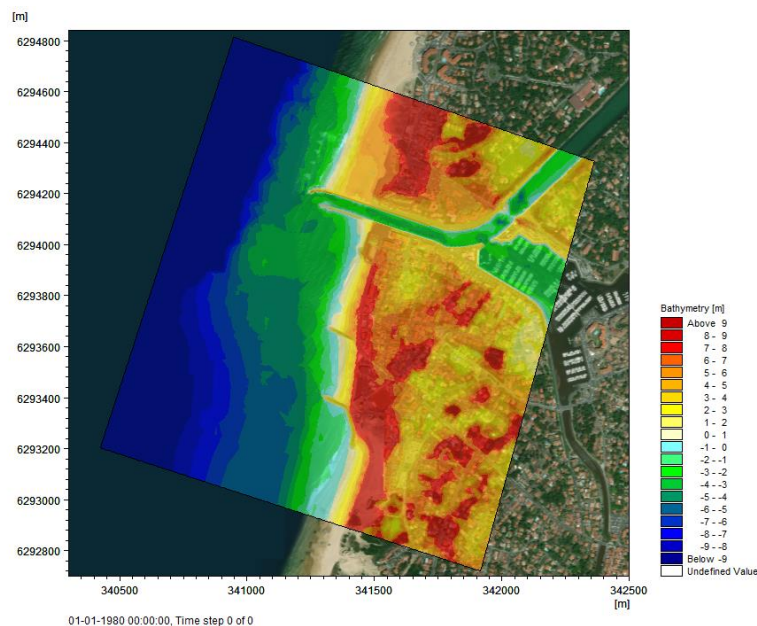


Fig. 3.3 - Domain and bathymetry of a coastal stretch (DHI, 2019b).

In its turn, the vertical mesh is defined either as sigma-coordinates or combined sigma/z-level coordinates. This mesh is responsible for the waves discretisation. To have better results sigma coordinates were used to the vertical discretization because they represent the bathymetry accurately and the results near the bottom of the flume are more consistent. There are three ways to specify the sigma layers, Fig. 3.4:

- Equidistant;
- Layer thickness;
- Variable.

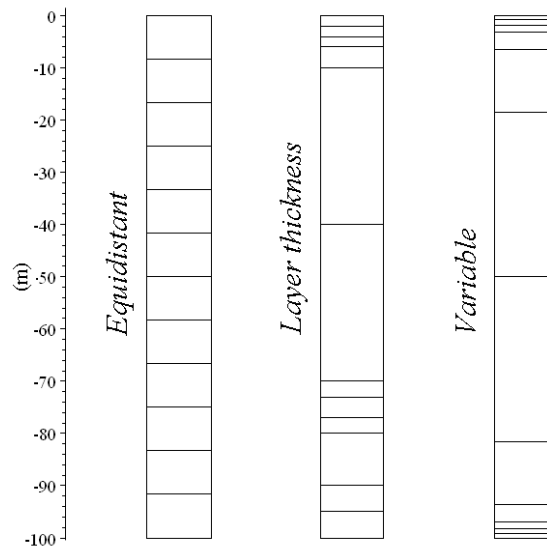


Fig. 3.4 - Example of a vertical mesh with 12 layers for the three options (DHI, 2019b).

Still within the domain item it is necessary to define the water depth. This is one of the most important parameters in the modelling process. Even if the bathymetry and the mesh are well defined, if the water depth is poorly defined from the start, it can lead to great delays in calibrating the model in the future.

3.2.2 TIME

After the Domain the next item is the Time. In this dialog the user can choose how long to simulate a particular event in real time, that means the period to be covered by the simulation. Three parameters must be defined: the simulation starting date, the number of time steps and the time step interval. The simulation starting date is the day and hour that will be considered the time step 0. In its turn, the number of time steps is the amount of “jumps” that the software will make, and the time step interval is the “size of the jump”. Those parameters are used to determine the frequency for which the output can be obtained. It must be stressed that the time steps for the hydrodynamic calculations are dynamic and determined to conciliate the stability criteria.

3.2.3 HYDRODYNAMIC MODULE

The third and last item is the Hydrodynamic Module. This is the item that aggregates all subitems responsible for the flow and all the different obstacles and wave conditions.

3.2.3.1 Solution Technique

The first subitem is the Solution Technique, where are defined some Navier-Stokes equations and numerical parameters related with the Riemann solver. In the Navier-Stokes parameters it is needed to define the minimum and maximum time steps, that are the smallest and biggest steps that the equation will take to be able to complete the calculation. Other parameter that must be defined is the Critical Courant-Friedrich-Lévy (CFL) number that it is a stability criterion for explicit schemes. As the explanation present in the user guide is concise, this small excerpt is transcribed: “The Navier-Stokes equations are discretized using a finite volume method with a shock-capturing scheme where the interface fluxes are calculated using an approximate Riemann solver. The shock-capturing scheme enables robust and stable simulation of flows involving shocks or discontinuities such as bores and hydraulic jumps. This is essential for modelling of waves in the breaking zone or porous structures. The numerical dissipation accounts for the dissipation in the breaking waves.”. In the numerical parameters, two types of Riemann solver parameters are chosen: No Riemann solver and HLLC (Toro, 1994). If the Riemann solver (HLLC) is selected, the Riemann factor must be specified. The factor should be in the range of [0,1], in which the value 1 corresponds to a total use of HLLC and the value 0 is the same as no Riemann solver.

3.2.3.2 Depth

Depth is the following subitem and is used to make some corrections in the bathymetry. Two options can be selected: no depth correction and specified bed level change. In the tests that were performed, the no depth correction option was used. The specified bed level change option is used in cases such as earthquakes and consequent tsunamis. Inside this option, two formats can be selected: varying in space but constant in time and varying in time and domain. In both cases, a data file with that information must be inserted.

3.2.3.3 Flood and Dry

The next subitem is Flood and Dry and two options can be selected: No Flood and Dry, and Flood and Dry. This action disables or enable the parameter. The main purpose of Flood and Dry is to respond to the moving boundaries problematic based on the work by Zhao (1994) and Sleight *et al.* (1998). When the water depths are small, the problematic is reformulated and when the depths are very thin, the elements are removed from the calculation process. This reformulation is put into practice with the introduction of a momentum flux to zero and only consider the mass fluxes. This ensure the continuity of flow calculation when the water is in the run-up and run-down zone at a dike.

As it is explained in the general description of the guide: “A non-physical flow across the face will be introduced for a flooded face when the surface elevation in the wet element on one side of the face is lower than the bed level in the partially wet element on the other side. To overcome this problem the face will be treated as a closed face. In case the water depth become negative, the water depth is set to zero and the water is subtracted from the adjacent elements to maintain mass balance. When mass is subtracted from the adjacent elements the water depth at these elements may become negative.”

This is due to the fact that two consecutive cells can have different flow information, that can be dry and wet successively. When Flood and Dry is turned on, we must define the drying depth and wetting depth, and this last has to be higher than the drying value.

3.2.3.4 Eddy Viscosity

To predict the effects of turbulence that will occur in the simulation, the viscosity is divided in horizontal and vertical eddy viscosity. Both cases can be selected in three different ways:

- No eddy;
- Constant eddy formulation;
- $k-\varepsilon$ formulation (Rodi, 1980).

In the horizontal parameter the Smagorinsky formulation can be also selected, where it need to be specify the Smagorinsky coefficient.

3.2.3.5 Bed Resistance

This parameter defines the roughness of the bottom of the case study, once the friction between the water and the bottom is related with the bed resistance. This parameter can be specified in three different ways:

- No bed resistance;
- Quadratic drag coefficient;
- Roughness height.

Both the drag coefficient and roughness height can be defined in one of three ways:

- Constant in time and domain;
- Constant in time and varying in domain;
- Varying in time and domain.

When the values vary in domain, a data file containing the drag coefficient, the roughness height or the grain diameter must be prepared. If the value is constant in time and domain, the only thing that is needed to define is the grain size.

3.2.3.6 Porosity

Once one of the objects of study is a rubble mound breakwater, it is needed to define it. The easiest way to do it is to export a porous map from the output, without any porous information. That way, the file can be edited with the accurate position and information. In Fig. 3.5, a submerged breakwater in red can be seen. It is defined layer by layer as a different porous zone. In this case the blue area is the water, zone 0, and the red zone is the zone 1.

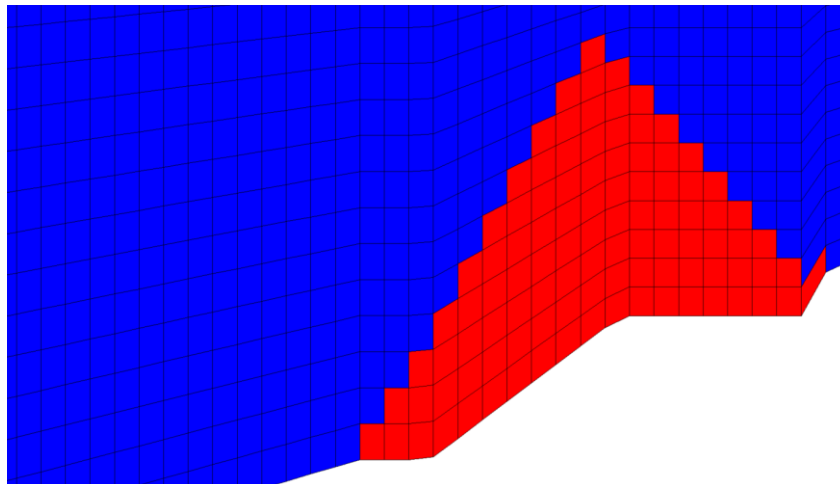


Fig. 3.5 - Submerged breakwater defined as a porous map.

After that, the map must be uploaded in M3. Now it is possible to define the porosity, grain diameter (d_{50}), laminar resistance coefficient (α), turbulent resistance coefficient (β) and oscillating period (T) of

the zone 1 (red zone). The water resistance inside the porous structure is described by the linear and non-linear resistance forces expressed as:

$$F_i = \rho \cdot a \cdot u_i + \rho \cdot b \cdot |u| \cdot u_i \quad (3.1)$$

Where ρ is the fluid density a is the laminar resistance coefficient, b is the turbulent resistance coefficient, u_i is the velocity components, ν is the molecular viscosity and $|u|$ the magnitude of the flow velocity. The terms a and b are determined by the empirical expressions formulated by van Gent (1995) and Liu (1999):

$$a = \alpha \frac{(1-n)^2}{n^3} \frac{\nu}{(d_{50})^2} \quad (3.2)$$

$$b = \beta \left(1 + \frac{7.5}{KC} \right) \frac{(1-n)}{n^2} \frac{1}{d_{50}} \quad (3.3)$$

KC is the Keulegan-Carpenter number defined as:

$$KC = \frac{u_m \cdot T}{n \cdot d_{50}} \quad (3.4)$$

Where u_m is the maximum oscillating velocity and T is the period of the oscillation.

With that information, the breakwater can be simulated in the most accurate way, similar to what it is in real scale.

3.2.3.7 Sponge Layer

As it is explicit in the name of this parameter, it is a numerical wave absorber. It is the responsible for absorbing the waves energy where it is placed in the model area. This parameter is used to stop the propagation of waves and prevent them to reflect in the boundaries of the model. The boundary code should be selected where the sponge should be.

3.2.3.8 Internal Wave Generation

Since this is a numerical wave simulation program, this subitem is the most important of all. All other items can be utopically well defined, but if the waves are poorly defined, the model does not serve the purpose.

To create a wave, the first thing to do is to define a relaxation zone that is responsible for the wave generation and absorption. That allows the model to generate the wave without the overlap of waves in the generating zone.

After defining the relaxation zone, the wave conditions must be set. There are four different options of waves:

- Regular waves;

- Irregular waves;
- Wave spectrum from file;
- Wave components from file.

Inside the regular wave option there are three different theories:

- Stokes 1st order;
- Stokes 5th order;
- Stream function.

For both Stokes theories, it is necessary to stipulate for the wave:

- Height, H ;
- Period, T ;
- Direction, θ ;

For the Stream function theory, it can be chosen between defining the wave trough, the wave period or the wave length, plus the same as the Stokes theories and the following ones:

- Length, L ;
- Number of components (Fenton 1988).

The last parameter to define in all theories of regular waves is the angle of convention. The options are meteorological or scientific convention, Fig. 3.6.

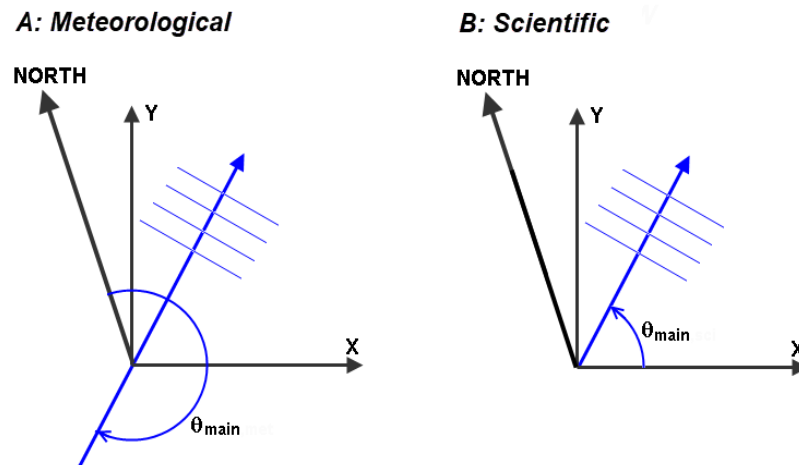


Fig. 3.6 - Types of wave angle conventions (DHI, 2019b).

When the option is irregular waves, the degree of complexity increases considerably. In this case it is necessary to choose the frequency spectrum and the directional distribution. The following list give us the types of spectra and its parameters:

- Pierson-Moskowitz:
 - Significant wave height, H_{m0} ;
 - Peak wave period, T_p .
- JONSWAP and Texel, Marsen, and Arsløe spectrum (TMA) (Bouws *et al.*, 1985; Kitaigorodskii, 1975):
 - Significant wave height, H_{m0} ;
 - Non-dimensional peak shape parameter, γ ;

- Spectral width parameter, σ_a ;
- Spectral width parameter, σ_b .

To specify the directional distribution the user has different options:

- Unidirectional;
- Cosine;
- Normal;
- Uniform;
- Frequency dependent.

Still in the wave data window it is necessary to define the minimum and maximum cut-off frequency for the frequency spectrum, and the number of Fourier components. To accelerate the calculation process, it can be selected an option of rescale truncated spectrum, that allows the software to cut the spectrum and increase the significant wave height and mean wave periods but maintaining the energy, Fig. 3.7.

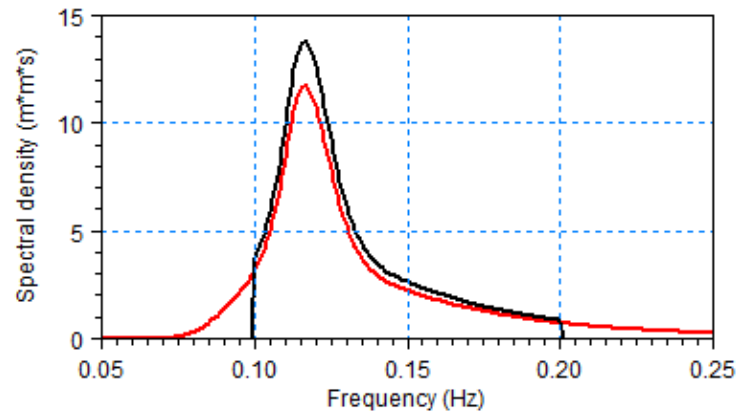


Fig. 3.7 - Original and rescaled truncated spectrum (DHI, 2019b).

Finally, the last two options of waves, the wave spectrum from file and wave component from file. In those options the user needs to input the file with the waves' information. These options have not been used in this work and therefore will not be the subject of a more exhaustive explanation.

3.2.3.9 Initial Conditions

When it is necessary to simulate many consecutive and related models, we have to define the initial conditions for a follow-up of the end of one simulation and the beginning of another. Since in this study the simulations are independent, this parameter does not have great applicability, so all the used values are the default ones.

3.2.3.10 Boundary Conditions

To limit the area that the software needs to simulate, some solid boundaries are created. These specify where there is no velocity after that vertical surface. Usually these parameters are already defined because the Mesh Generator and the MATLAB script automatically identify the land boundary. Each limit has a code name, but that name can be changed to whatever the user decides.

3.2.3.11 Turbulence Module

This parameter has four subitems that allow us to define it:

- Equation;
- Solution Technique;
- Dispersion;
- Initial Conditions.

Those parameters are very complex and since default values were used, they will not be further analysed. The turbulence model used is the K-epsilon ($k-\epsilon$), which is the most common model used for turbulent flow conditions in computation fluid dynamics (CFD).

3.2.3.12 Outputs

This last subitem is the one responsible for the selection of the information that the user wants to analyse after the simulation. The first thing to do is to create a new output. After that, the field type must be selected, and the options are:

- 2D flow parameters;
- 3D flow parameters;
- Mass budget;
- Discharge through a cross section;
- Cumulative statistics 2D;
- Subseries statistics 2D;
- Cumulative statistics 3D;
- Subseries statistics 3D.

There are two different flow parameters: 2D and 3D. If 2D flow parameters are selected, cumulative statistics 2D or subseries statistics 2D, the user can choose the following formats:

- Point series;
- Lines series;
- Area series.

If it is selected 3D flow parameters, cumulative statistics 3D or subseries statistics 3D, the following formats can be chosen:

- Point series;
- Lines series;
- Volume series.

If mass budget is selected for the field type, the domain where the mass budget should be calculated must be defined. If the discharge option is selected for the field type, you must specify the cross section through where the discharge should be calculated.

4

CASE STUDY

4.1 PHYSICAL MODEL SETUP

As explained in the first chapter, the main purpose of this study is to make a sensitivity analysis of the numerical model, using the M3, and compare the results with the experimental data of the physical model tests performed at the DHI test facilities in Hørsholm, Denmark.

Fifty-seven different profiles were tested, however, the present study only analyses three cases: the profiles P-1, H1-4 and H4d-5. P-1 is the profile before the reinforcement as it was in the original stage, built in the XX century, Fig. 4.1. The profile of the dike starts with a flat zone from the beginning of the flume to 16.77 m. The dike itself has a 1:7.7 slope that ends when the rubble mound breakwater begins. After that, another slope of 1:3.2 and 1:4.5 begins, finishing with a flat platform. The datum used in these tests is the *Normaal Amsterdams Peil* (NAP), used in large parts of Western Europe. In the following schemes, the drawn line is the NAP reference level, not the water level tested.

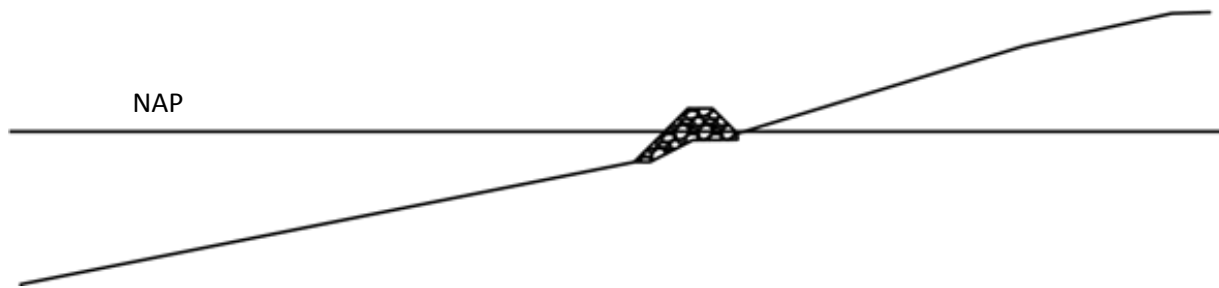


Fig. 4.1 - Scheme of the profile P-1 (Hansen and Schlutter, 2018).

Both H1-4 and H4d-5 profiles have the same standard foreshore from the initial wave generation until the dike toe. The slopes and dimensions of the initial part of the profile measured in prototype scale are presented in Fig. 4.2.

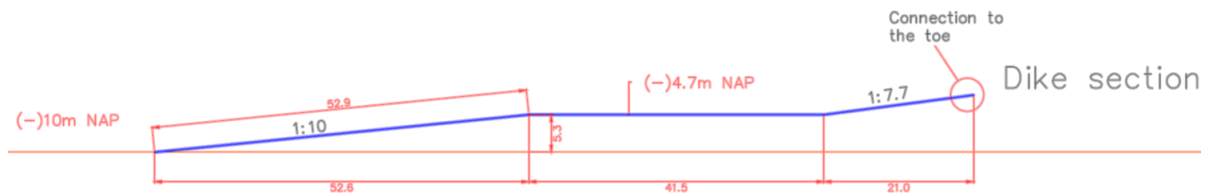


Fig. 4.2 - "Standard" foreshore in prototype scale (Hansen and Schlutter, 2018).

H1-4, Fig. 4.3, is the more complex profile of these three, because it has two different submerged breakwaters with different characteristics, and six ribs that will increase the energy loss in the overtopping process.

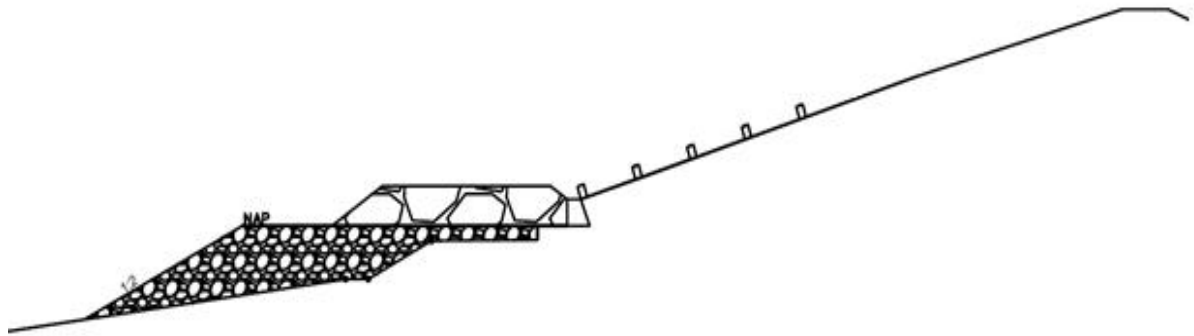


Fig. 4.3 - Scheme of the profile H1-4 (Hansen and Schlutter, 2018).

The rib configuration in H1-4 is constructed as shown in Fig. 4.4. This configuration induces energy loss to the waves during the run-up process, and allow the water to run back through the gaps in the ribs. In this case, the rib spacing is 2.55 m and the height is 0.5 m.

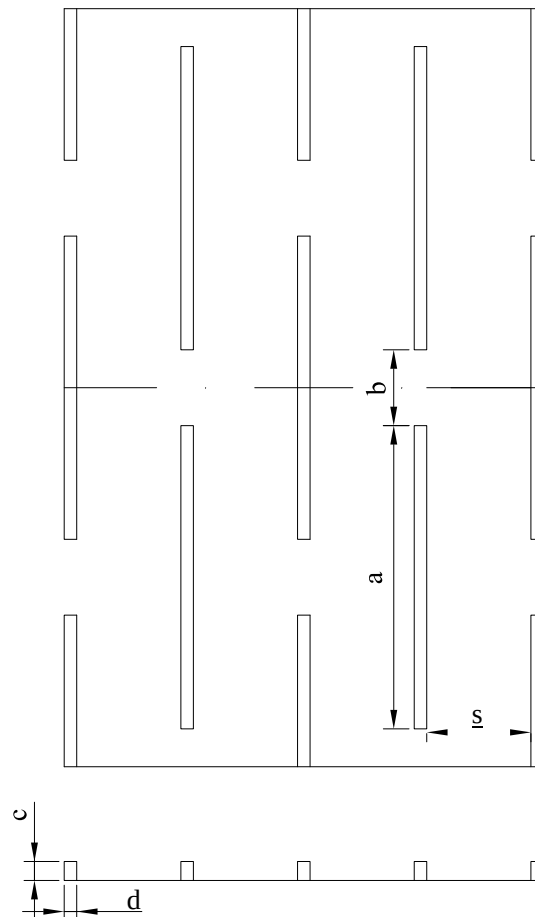


Fig. 4.4 - Configuration of the ribs: s corresponds to the spacing between ribs; a to the length of the rib; b to the gap width; c correspond to the rib height and d to the rib width. Upper and side view (Hansen and Schlutter, 2018).

H4d-5, after the initial standard foreshore, has a submerged breakwater and two slopes of 1:3 and 1:2.5 with a wave return structure at the top of the final slope, Fig. 4.5.

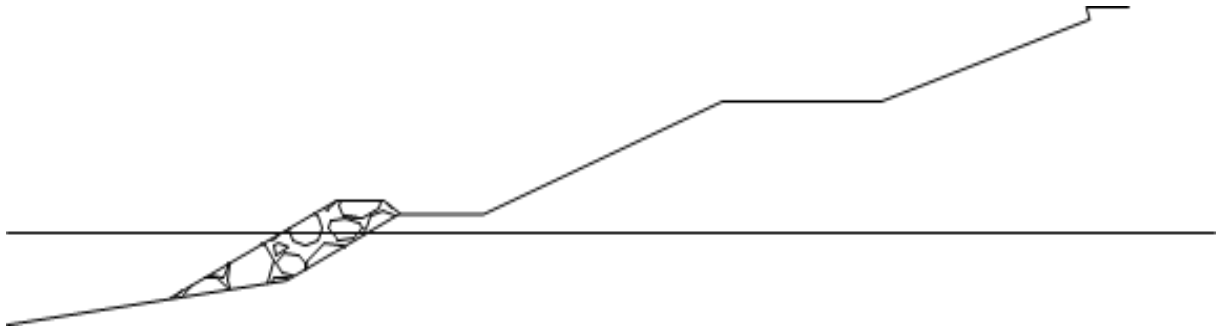


Fig. 4.5 - Scheme of the H4d-5 profile (Hansen and Schlutter, 2018).

The physical model tests were performed on a 28 m long, 0.74 m wide and 1.2 m deep wave flume. As stated in the report, the flume is equipped with an electric-driven wave generator at one end, for random and regular wave generation, Fig. 4.6. The wave generator is controlled by DHI's Wave Synthesiser software package. The wavemaker stroke length is 0.8 m with maximum paddle speed of 1.0 m/s. It is also equipped with DHI's Airborne Warning and Control System (AWACS) state-of-the art wave reflection compensation module. This module allows the wave generator to absorb the waves which are reflected from the breakwater, while generating the specified incident waves.



Fig. 4.6 - Wave maker and wave gauges at DHI facilities in Hørsholm (Hansen and Schlutter, 2018).

The physical model tests were made using a geometric scale, λ_L , of 1:18 in both vertical and horizontal directions. The Froude model law was also used for time, mass and discharge:

- Length, $1/\lambda$;
- Time, $1/\sqrt{\lambda}$;
- Mass, $1/\lambda^3$;
- Discharge, $1/\lambda^{5/2}$.

The flume is made of transparent glass to allow the visualization of the wave propagation and other events during the test. It has two arrays of five wave gauges each. These arrays allow to do an analytical spectral separation of the incoming and reflected waves. The first array is positioned near the wave maker and the second group of wave gauges is located at the end of the foreshore. The dike profile was constructed in plywood covered with sand. The porous breakwater was made with rocks that were Froude scaled to guarantee the correct representation of permeability and secured by a wire net to ensure that the rocks do not move during the test.

The test flume is also equipped with two groups of five wave gauges (WG) each: the first group is WG1-WG5, the second one is WG6-WG10, Fig. 4.7. These wave gauges measured the surface elevation, which permitted to make an analytical spectral separation of the incoming and reflected waves.

Table 4.1 - Wave gauge positions from the wave maker paddle.

	Position [m]
WG1	2.00
WG2	2.40
WG3	2.75
WG4	3.00
WG5	3.10
WG6	15.50
WG7	15.90
WG8	16.25
WG9	16.50
WG10	16.60



Fig. 4.7 - Scheme of the wave gauge location.

To measure overtopping, a method that consists in a chute on the top of the crest of the dike profile was adopted to guide the water to a container behind the dike. A screen to prevent water loss during the test was also inserted. To avoid buoyancy effects in the overtopping container, this structure was inserted in a dry box on a force gauge, Fig. 4.8.

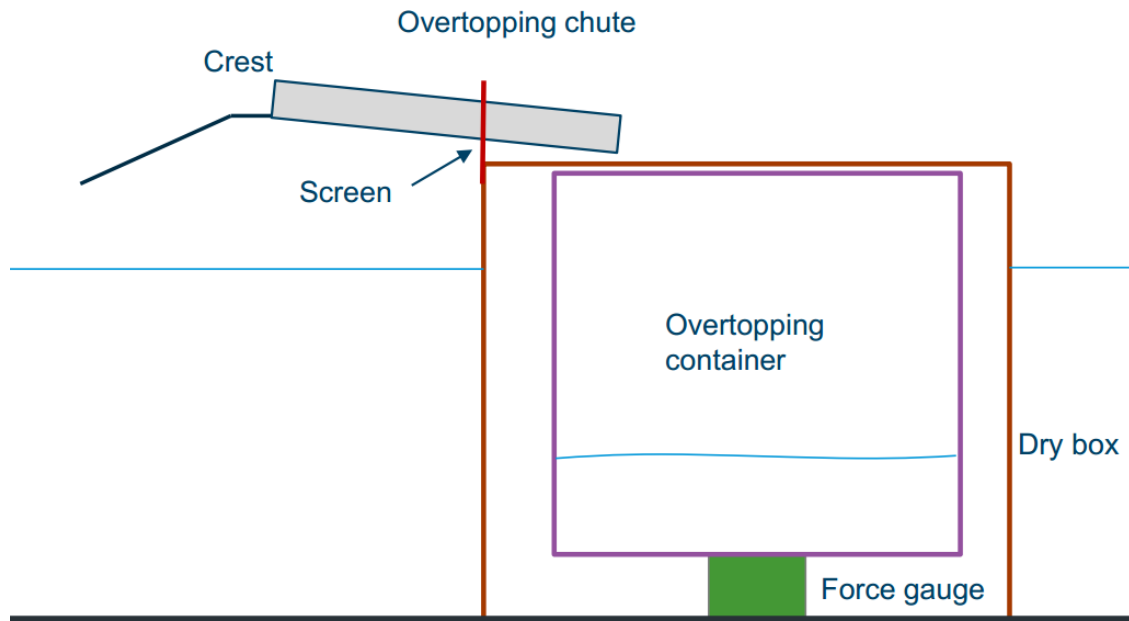


Fig. 4.8 - Scheme of the overtopping measurement system (Hansen and Schlutter, 2018).

With the weight of the water in the container and knowing the width of the chute, it is possible to calculate the overtopping rate, q [L/m/s]. Once it is necessary to maintain the water level during the test, it was added a pump that compensates the water that entered in the container.

Since the goal of the Corneel report was to define a profile that ensures less overtopping of the *Afsluitdijk* dike, the wave generation is one of the most important factors of the whole process. Due to the maritime characteristics of the Northern Sea, the waves are generated as irregular JONSWAP wave spectra with standard parameters $\gamma=3.3$, $\sigma_a=0.07$ and $\sigma_b=0.09$.

For each profile, a series of three tests was executed, as presented in Table 4.2. The water level (WL) reference is the NAP.

Table 4.2 - Test series of the physical model, in real scale

Test no.	Profile	Description	H_{m0} [m]	T_p [s]	WL [m]	Duration [hh:mm:ss]
5	P-1	Design waves	4.27	7.69	5.21	00:30:00
16		Design waves + 10%	4.70	8.07		00:20:00
36		Design waves -10%	3.84	7.29		01:00:00
85	H1-4	Design waves -10%	3.84	7.29		01:00:00
86		Design waves	4.27	7.69		01:00:00
87		Design waves + 10%	4.70	8.07		01:00:00
316	H4d-5	Design waves -10%	3.84	7.29		01:00:00
317		Design waves	4.27	7.69		01:00:00
318		Design waves + 10%	4.70	8.07		01:00:00

After an in-depth study of the aforementioned report, it was possible to collect the essential information for the elaboration of the study of the same project but this time through numerical models.

4.2 NUMERICAL MODEL SETUP

4.2.1 RESOLUTION OF THE MODEL

Since the physical models tests were performed in a wave flume, the easiest way to do the mesh is using a MATLAB script, Fig. 4.9, created by DHI specifically to generate quadratic meshes. First of all, it is necessary to assemble the coordinates of the bathymetry of the flume. That information is gathered as an AutoCAD file. In that way is possible to provide information that will be used to create the mesh. The scale used in the numerical model test was the same used in the physical model, so the dimensions in both models are exactly the same. The lines 3 and 4 in the script are used to define the discretization in the x and y direction. In its turn, the lines 5 and 6 define the boundaries of the flume. The 7th line is used to express the depth, so it is a sort of vertical boundary. The command lines in positions 8, 9 and 10 create vectors with the “jumps” that each direction will take. The bathymetry information is introduced in the lines 11 and 12, respectively in the x and z directions. In the 14th line, using the MATLAB function `interp1(xp,zp,xx)`, being xp the coordinates in the x axis, zp the coordinates in z axis and xx the vector input to generate the quadratic mesh, is possible to interpolate the points with a straight line. To represent the bathymetry of the flume, the information is plotted in the 16th line. In the 18th line there is a command that calls a function for the mesh generation located in the DHI Matlab Toolbox. To select the projection type exists the line 20, where it is possible to choose the map-projection. Lastly, lines 22 and 24 serve, respectively, to write the file name and then create the mesh file.

```

1 - clearvars, clc,
2 - %% ----- This script generates quadriangular rectangular meshes -----
3 - dx          = [0.03];                % dx [m]
4 - dy          = [1];                  % dy [m]
5 - LBasinX     = 28;                   % x-length of the basin
6 - LBasinY     = 1;                   % y-length of the basin
7 - Depth       = 0.556;               % z-depth of the basin
8 - xx          = [0:dx:LBasinX];       % vector input to GenerateQuadMesh
9 - yy          = [0:dy:LBasinY];       % vertical mesh of the basin
10 - zz         = xx ;                  % bathymetry of the domain, use negative values.
11 - xp=[0;16.775;19;19.057;19.213;19.375;19.376;20.413;20.948;21.085;26;28]
12 - zp=[-0.556;-0.556;-0.111;-0.111;-0.031;-0.031;-0.008;0.311;0.429;0.433;-0.556;-0.556]
13
14 - zz = interp1(xp,zp,xx);
15
16 - plot(xp,zp,'o',xx,zz,'-')
17
18 - [Elmts, Nodes] = GenerateQuadMesh(xx,yy,[zz;zz]); % call function for mesh generation
19
20 - proj          = 'NON-UTM';          % choose map-projection
21
22 - filename{1}   = 'P-1_0.03';        % filename of the mesh
23
24 - mzWriteMesh(filename{1},Elmts,Nodes,proj); % write file

```

Fig. 4.9 - MATLAB script used to create a quadratic mesh.

Once in this case the propagation of the waves is perpendicular to the lateral limits of the flume, the thickness of the mesh is the width of the flume itself. In Fig. 4.10 it can be seen an example of a mesh with $dx=0.03$ m and dy equal to the width. If the wave propagation, instead of a perpendicular propagation, had some deviation, dy would have to be defined with several numbers of elements in order to properly characterise the wave.

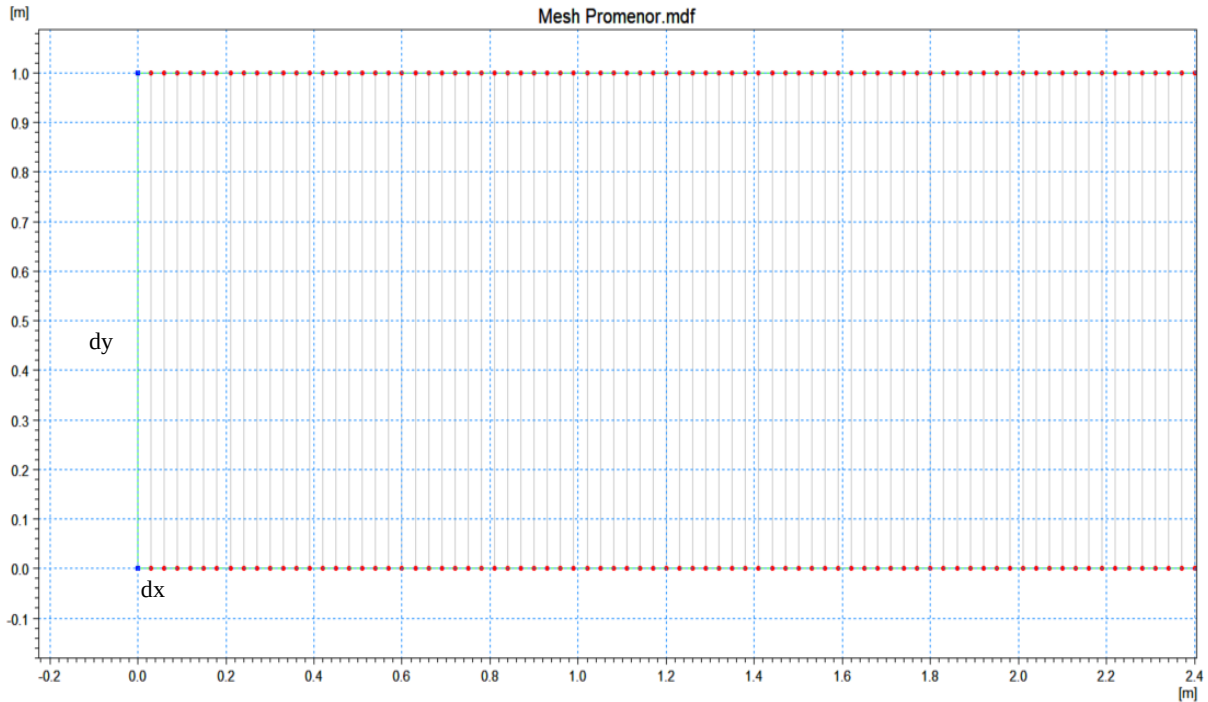


Fig. 4.10 - Horizontal mesh with $dx=0.03$ m and $dy=1$ m.

The first step in numerical modelling process is the generation of the mesh. This mesh must be well defined and adequate to the results that are object of study in the simulation, in this case the overtopping on the crest of the dike and the wave behaviour. However, it is necessary to find a balance between the mesh resolution and the computational process. Here, the first dilemma appears: the smaller the mesh, the more precise the results will be, but more time the simulation will take, once M3 calculates element-by-element. A good way to make it easier to decide the dx discretization is to calculate the number of points per wave length.

This kind of study was already made by many researchers and the range of values can variate between 50 and 500 points per wavelength. These range of values depends on the problem and its size. The following list present the range of points per wavelength in different literature:

- 50-100 (Vanneste and Troch, 2015);
- 300-400 (Contestabile *et al.*, 2017);
- 400-500 (Garcia *et al.*, 2004);
- 200-300 (Lara *et al.*, 2011a);
- 320-450 (Garcia *et al.*, 2004);
- 260-380 (Lara *et al.*, 2011).

With the purpose of creating a model with a resolution that allows accurate results and a reasonable speed, a simulation plan was made. The plan consisted in fixing all M3 parameters and changing the type and number of layers and the size of the mesh/points per wavelength. Since the main goal of these tests is the definition of an horizontal and vertical resolution, instead of simulating irregular waves as in the physical models, regular waves were used. This decision was made to be easier to understand the results.

The profile used in these tests was the P-1, because is the simplest and is sufficient for this study. The mesh values tested were: 0.01, 0.02, 0.03, 0.04, 0.05 and 0.06 m to the horizontal discretization. Once the wavelength in these tests was 4.52 m, the values of points per wavelength are respectively: 452, 226, 151, 113, 90 and 75. For the vertical direction were defined 5, 10, 12, 15, 17 and 20 variable and equidistant layers.

However, to carry out this study, 72 different simulations were required, Fig. 4.11, which would take several weeks to complete if it had been run one by one. So, to speed up the process, a scrip in Notepad++ which runs the simulations sequentially with a single order was created, instead of running simulation by simulation. The only division made was to separate the simulations by the different mesh sizes.

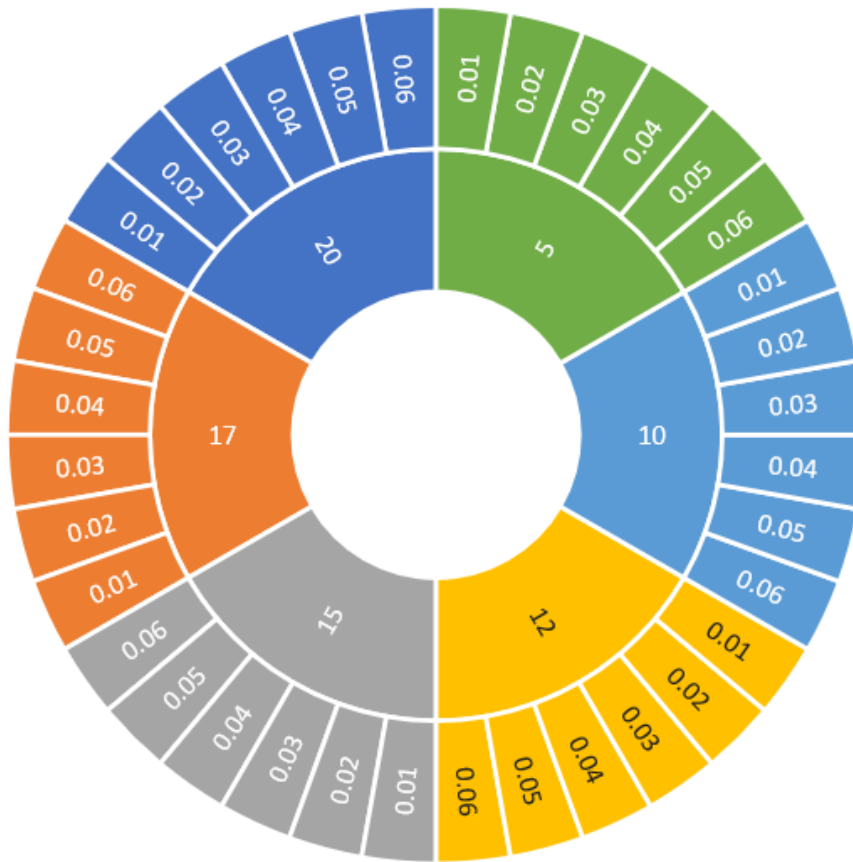


Fig. 4.11 - Summary of the tests performed for each layer type, variable and equidistant.

Fig. 4.12 presents is an example of a scrip to run the simulations with $dx=0.06$ m. For each mesh size, 12 tests were performed corresponding to the 6 different numbers of variable and equidistant layers. To be easier to work with the results, the name of each file contains the information of the mesh (006 is the size of the grid that is equal to 0.06 m), number of layers (5, 10, 12, 15, 17 and 20) and type (var for variable and eq to equidistant layers). In the script it can be also defined the number of cores that will work to run it.

```

1 set engine="C:\Program Files (x86)\DHI\2019\bin\x64\FemEngineHD"
2 set OMP_NUM_THREADS=1
3 set numberMPI=4
4 echo [%DATE% %TIME%]
5 mpiexec -n %numberMPI% -localonly %engine% 006_5_eq_layers.m3wfm
6 echo [%DATE% %TIME%]
7 echo [%DATE% %TIME%]
8 mpiexec -n %numberMPI% -localonly %engine% 006_5_var_layers.m3wfm
9 echo [%DATE% %TIME%]
10 echo [%DATE% %TIME%]
11 mpiexec -n %numberMPI% -localonly %engine% 006_10_eq_layers.m3wfm
12 echo [%DATE% %TIME%]
13 echo [%DATE% %TIME%]
14 mpiexec -n %numberMPI% -localonly %engine% 006_10_var_layers.m3wfm
15 echo [%DATE% %TIME%]
16 echo [%DATE% %TIME%]
17 mpiexec -n %numberMPI% -localonly %engine% 006_12_eq_layers.m3wfm
18 echo [%DATE% %TIME%]
19 echo [%DATE% %TIME%]
20 mpiexec -n %numberMPI% -localonly %engine% 006_12_var_layers.m3wfm
21 echo [%DATE% %TIME%]
22 echo [%DATE% %TIME%]
23 mpiexec -n %numberMPI% -localonly %engine% 006_15_eq_layers.m3wfm
24 echo [%DATE% %TIME%]
25 echo [%DATE% %TIME%]
26 mpiexec -n %numberMPI% -localonly %engine% 006_15_var_layers.m3wfm
27 echo [%DATE% %TIME%]
28 echo [%DATE% %TIME%]
29 mpiexec -n %numberMPI% -localonly %engine% 006_17_eq_layers.m3wfm
30 echo [%DATE% %TIME%]
31 echo [%DATE% %TIME%]
32 mpiexec -n %numberMPI% -localonly %engine% 006_17_var_layers.m3wfm
33 echo [%DATE% %TIME%]
34 echo [%DATE% %TIME%]
35 mpiexec -n %numberMPI% -localonly %engine% 006_20_eq_layers.m3wfm
36 echo [%DATE% %TIME%]
37 echo [%DATE% %TIME%]
38 mpiexec -n %numberMPI% -localonly %engine% 006_20_var_layers.m3wfm
39 echo [%DATE% %TIME%]
40 pause

```

Fig. 4.12 - Notepad++ script that run the simulations with $dx=0.06$ m.

After performing this battery of tests, a data analysis was done, focusing on the different results in the different resolutions. The parameters analysed to compare the different resolutions were the overflow rate (q_d) and the maximum surface elevation (η). In the first stage the influence of the type of layer was analysed: Equidistant Layers vs Variable Layers. In this analysis, the same number of layers was compared within each mesh size and the ratio between Equidistant Layers and Variable Layers was calculated.

The 1st mesh size analysed was the $dx=0.01$ m, which was the mesh with the higher resolution simulated in those tests, Fig. 4.13.

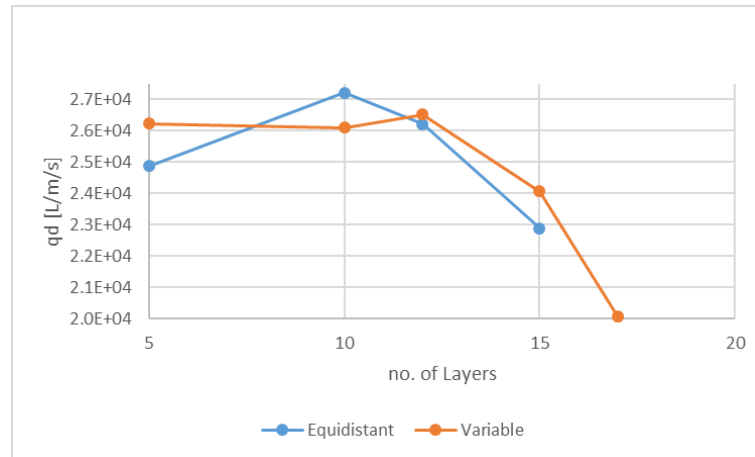


Fig. 4.13 - Overflow rate in $dx=0.01$ m.

The biggest difference in this test was observed in the resolution with 5 layers and the smallest one was obtained with 12 layers. The difference between equidistant and variable layers was 5.2% in the worst case and 1.1% in the best case. The tendency of these results was, in both cases, the decrease of the overflow with the increment of resolution. As it is possible to verify in the graph, the values of 17 and 20 Equidistant layers and 20 Variable layers are missing. This is due to the great resolution of the cells that causes a blow-up in the system. Turning now to the maximum surface elevation, Fig. 4.14, the greater difference, 3.6%, was verified with 15 layers. In turn, the smallest difference, 2.0%, was found when using a resolution of 5 layers.

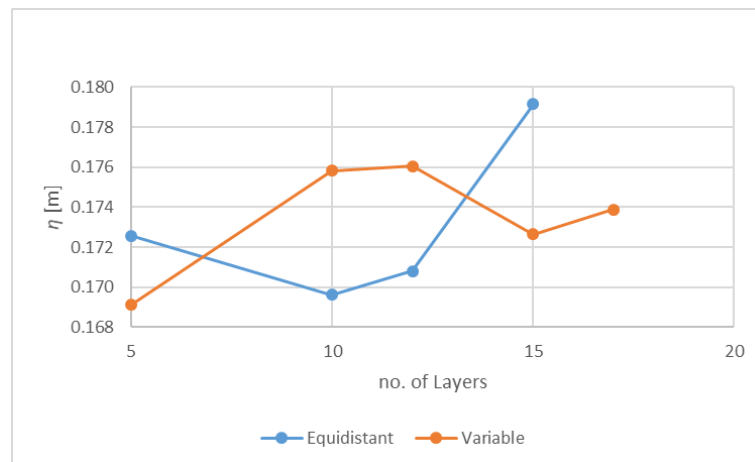


Fig. 4.14 - Maximum surface elevation in $dx=0.01$ m.

In the model with a mesh size of 0.02 m, Fig. 4.15, the resolution with the main difference is obtained with 10 layers with a ratio of 4.2%. The best case occurs with 17 layers and the ratio between variable and equidistant layers is 0.2%. However, the trend is to attain more stable results with 12 layers.

In the maximum surface elevation, Fig. 4.16, the poorest vertical resolution is obtained with 5 layers with a ratio of 2.2%, and the best one with 15 or 17 layers, both with a ratio of 0.2%. Once again, the higher the resolution, the better the correlation of the equidistant and variable layers.

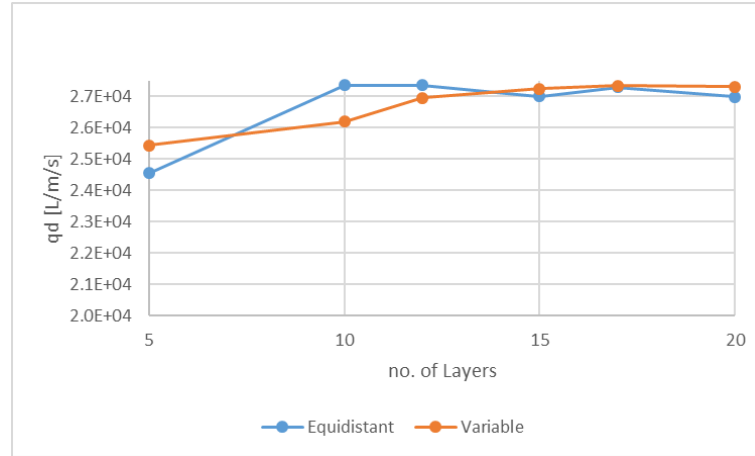


Fig. 4.15 - Overflow rate in $dx=0.02$ m.

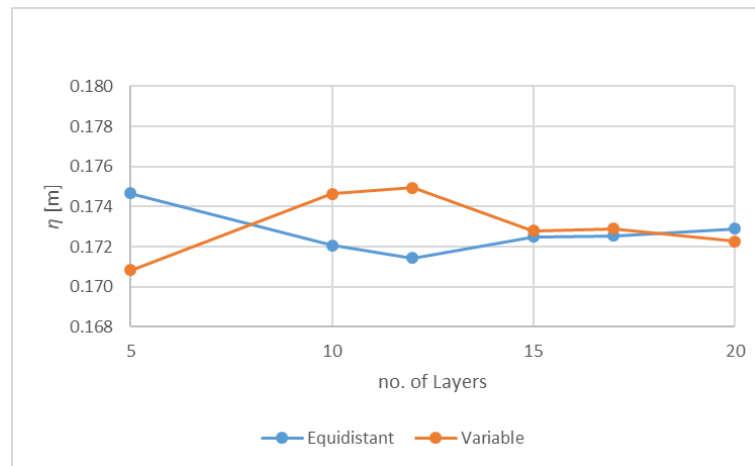


Fig. 4.16 - Maximum surface elevation in $dx=0.02$ m.

The third mesh size analysed was the $dx=0.03$ m. The results of the overflow rate, Fig. 4.17, show that the number of layers with the worst ratio, 3.5%, is with 10 layers and the best one, 0.1%, is obtained with 17 layers.

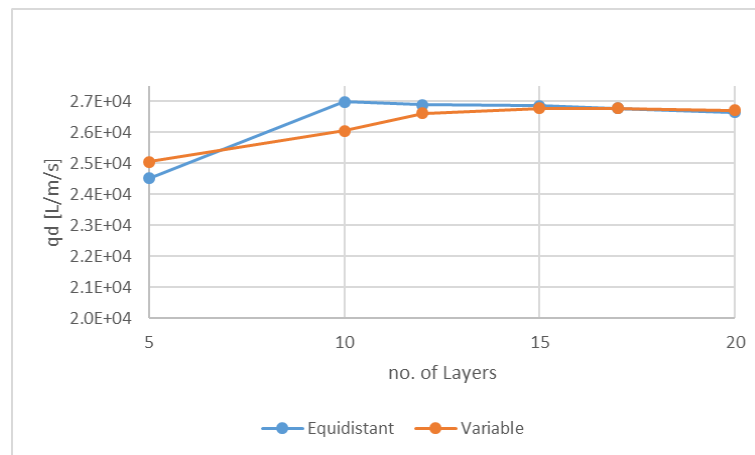


Fig. 4.17 - Overflow rate in $dx=0.03$ m.

Analysing the maximum surface elevation, Fig. 4.18, it can be easily understood that the 12 layers option is the one with less stable results, 2.2%. In its turn, with 15 layers, the results are very similar, the ratios vary between 0.2% and 0.3%.

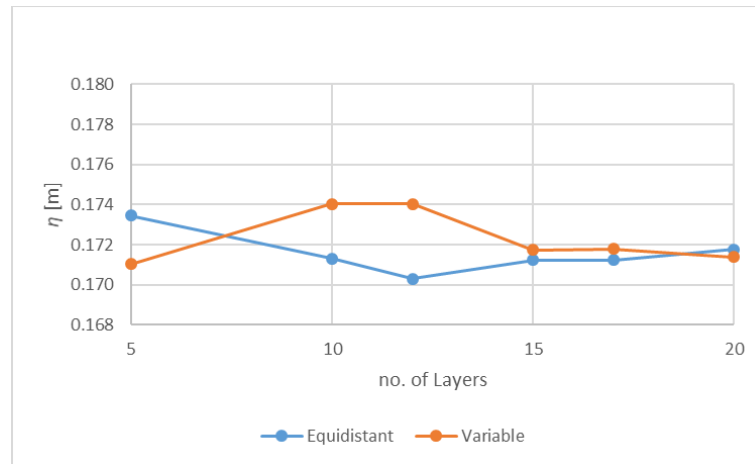


Fig. 4.18 - Maximum surface elevation in $dx=0.03$ m.

Analysing the mesh with the horizontal resolution of 0.04 m, it is verified that the result that gives less guaranties in the overflow rate, Fig. 4.19, is achieved with 17 layers with a ratio of 3.1%, and the one with a more stable result with 5 layers and a 0.1% ratio.

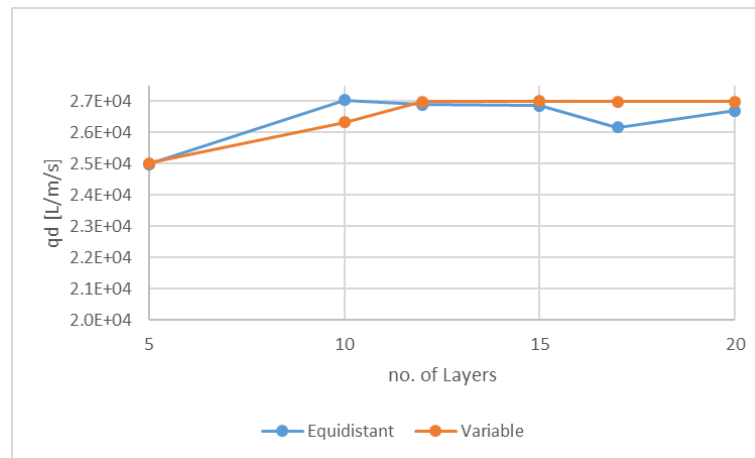
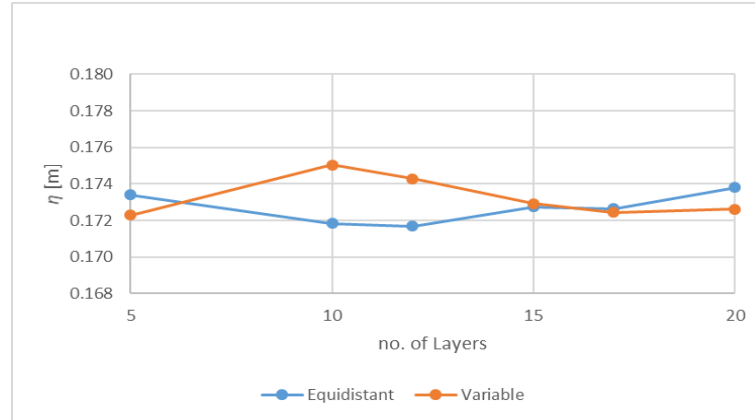
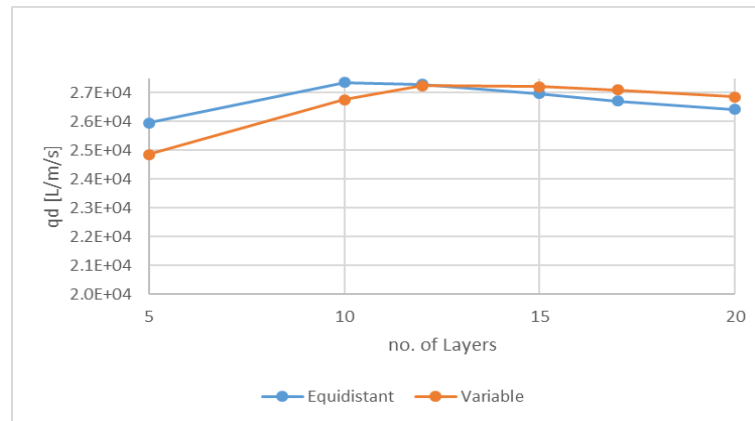


Fig. 4.19 - Overflow rate in $dx=0.04$ m.

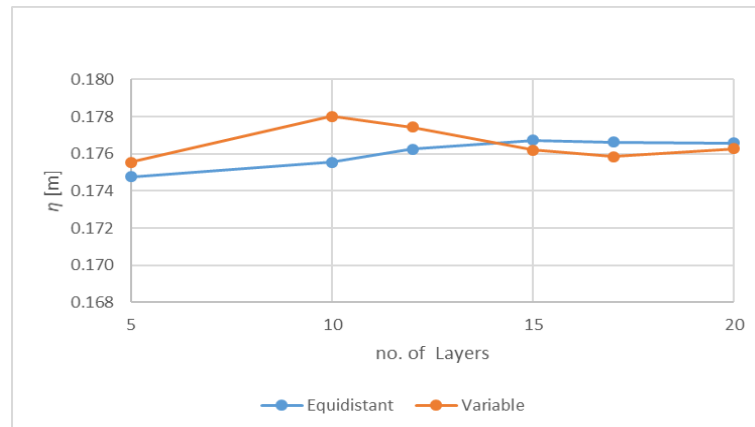
For the maximum surface elevation, Fig. 4.20, the worst scenario is obtained with the 10 layers model that had a ratio of 1.8%, and the best ones are with 15 and 17 layers with a relation of 0.1%.


 Fig. 4.20 - Maximum surface elevation in $dx=0.04$ m.

Passing now to the 5th case, $dx=0.05$ m, in the overflow ratio analyses, Fig. 4.21, it can be seen that using 10 layers leads to the worst result with a ratio of 2.2%. The best ratio is found between 12 equidistant and variable sigma layers, with a ratio of 0.2%.


 Fig. 4.21 - Overflow rate in $dx=0.05$ m.

Taking into account the maximum surface elevation, Fig. 4.22, the best relation is obtained with 20 layers, with a ratio of 0.2%, and the worst one with 1.4% for the 10 layers option.


 Fig. 4.22 - Maximum surface elevation in $dx=0.05$ m.

The final case is the $dx=0.06$ m. In this case, the overflow rate shows that the best relation is found with 12 layers and a ratio of 0.5% and the worst with 20 layers and a ratio of 3.2%, Fig. 4.23.

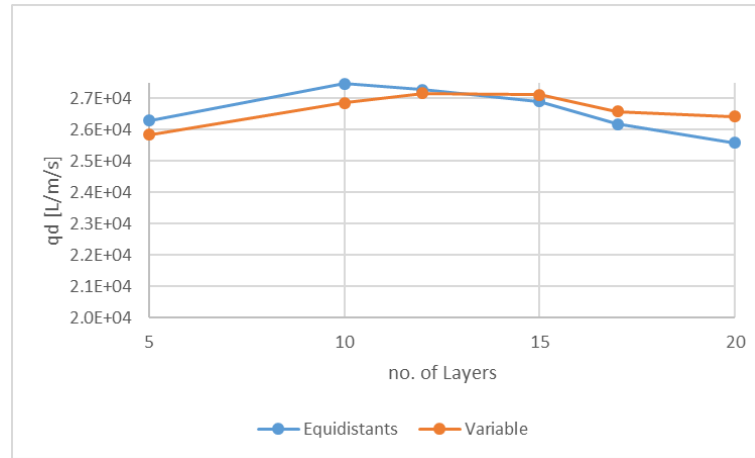


Fig. 4.23 - Overflow rate in $dx=0.06$ m.

Finally, for the maximum surface elevation with $dx=0.06$ m, the best relation is obtained with 10 layers with a ratio of 0.1%, and the worst is for 20 layers with a ratio of 1.2%, Fig. 4.24.

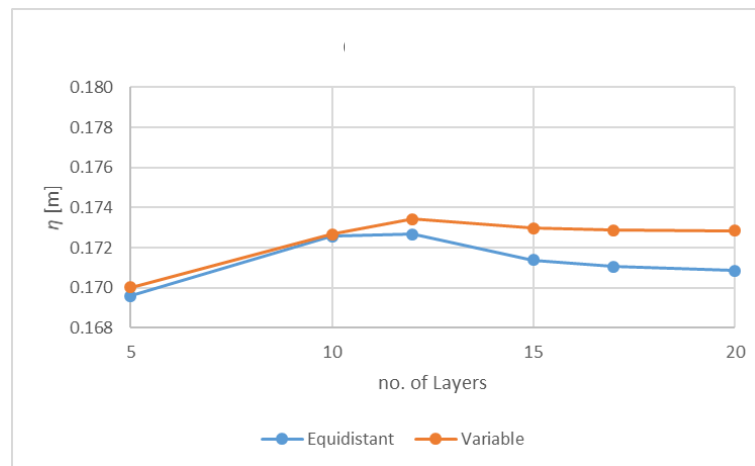


Fig. 4.24 - Maximum surface elevation in $dx=0.06$ m.

The main purpose of this analysis was to compare the variable sigma layers with equivalent sigma layers, in order to set a vertical and horizontal discretization in the numerical model. Tendentially, these tests showed that the best relations are verified with the increase of the resolution. However, if the resolution is higher, the numerical process is consequently slower because the number of cells is bigger. Thus, to exist an equilibrium that improves the computing speed without compromising the accuracy of the results, it was selected the mesh with $dx=0.03$ m and 15 variable sigma layers. The option variable sigma layer was chosen instead of equidistant layer because the density of layers in the bottom and surface is higher and the physical phenomena are better replicated.

4.2.2 FLOOD AND DRY

This parameter is not a physical feature in the sense that it does not represent any physical phenomenon. To determine the values of the drying and wetting depth a group of simulations was performed, considering the default values of the software. The conclusion of the study is that the values that grant more stable results were the default values divided by the scale used in the model, Fig. 4.25.

Drying depth	0.000277 [m]
Wetting depth	0.000694 [m]

Fig. 4.25 - Flood and Dry

4.2.3 WAVE GENERATION

Since this is a study related to the sea and its interaction with the coast, the most important aspects to consider are the waves and its physical characteristics. So, to be able to have a trustworthy model, the wave parameters must be very accurate in order to replicate the reality.

As it was explained in the chapter 3, the first thing to define in the wave generation is the relaxation zone. This is defined by the introduction of the coordinates of the boundary points, set as a Cartesian coordinates system, Fig. 4.26.

Coord No.	Easting [m]	Northing [m]
1	4.6	1
2	4.6	0

Zone width 4.5 [m]

Fig. 4.26 - Coordinates of relaxation zone. Obtained in M3.

The length of the relaxation zone must be, at least, one wavelength, which in this case is 4.5 m. The value 4.6 in the easting coordinate, Fig. 4.26, was defined on purpose. Once the area is set from the points to the beginning of the flume, and the width is 4.5 m, this will create a space of 0.1 m that reduces the water friction in the wavemaker, Fig. 4.27. This gap should be two or three cells of width to prevent that.

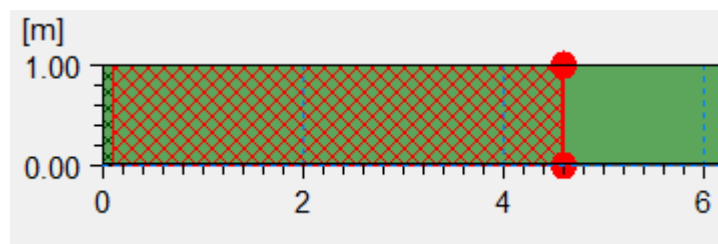
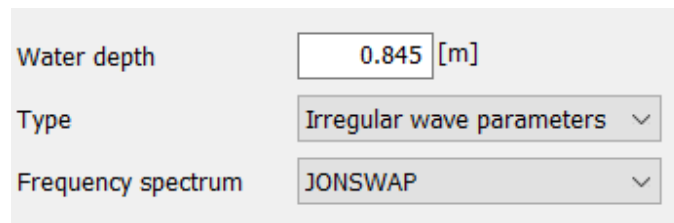


Fig. 4.27 - Detail of the relaxation zone. Red points are the coordinates defined previously.

For the purpose of characterizing the wave, some parameters must be defined. First of all, the type of wave. The physical model was set as irregular wave with a JONSWAP spectrum, the numerical model must be defined that way as well, Fig. 4.28.

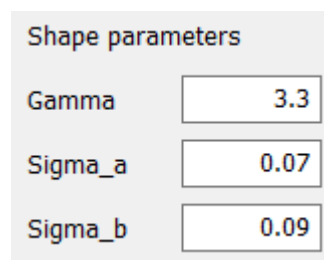


Water depth	0.845 [m]
Type	Irregular wave parameters ▾
Frequency spectrum	JONSWAP ▾

Fig. 4.28 - Wave data. Type and spectrum frequency.

Considering that the bottom of the flume is at -10 m (NAP), and the water level is + 5.21 m (NAP), the water depth in real scale is 15.21 m and 0.845 m in model scale.

The definition of the shape parameters, Fig. 4.29, is the same used in the wave generator in the physical model setup.

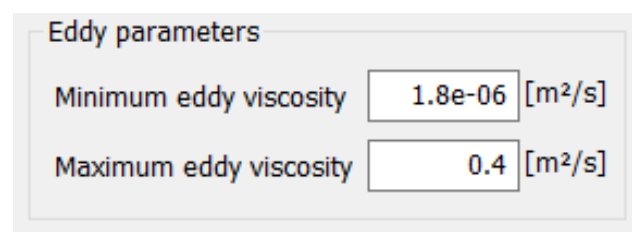


Shape parameters	
Gamma	3.3
Sigma_a	0.07
Sigma_b	0.09

Fig. 4.29 - Shape parameters of the JONSWAP spectrum.

4.2.4 PHYSICAL FEATURES

In this section, the physical features of each profile, such as bed resistance, eddy viscosity and porosity will be presented and explained. For each profile, the value of eddy viscosity and bed resistance is the same. The vertical and horizontal eddy viscosity have the default values, Fig. 4.30.



Eddy parameters	
Minimum eddy viscosity	1.8e-06 [m ² /s]
Maximum eddy viscosity	0.4 [m ² /s]

Fig. 4.30 - Vertical and horizontal eddy viscosity.

The bottom of the flume in the physical model was covered with grains with 0.02 m of diameter. So, in that way, the roughness height was set with a constant value of 0.02 m in the numerical model, Fig. 4.31.

Roughness height data

Format Constant

Constant value 0.02 [m]

Fig. 4.31 - Bed resistance.

To create the porosity map it is mandatory to know the dimensions of the rubble mound breakwater to define the limits of the porous zone. However, since the resolution is dependent on the number and type of sigma layers in the vertical axis, and the mesh size in the horizontal axis, the cells' dimensions are not refined enough to precisely reproduce the breakwater.

4.2.4.1 P-1 Porosity

In the P-1 profile the rubble mound breakwater was defined with a porous map, and the boundaries of it were set as it is displayed in Fig. 4.32.

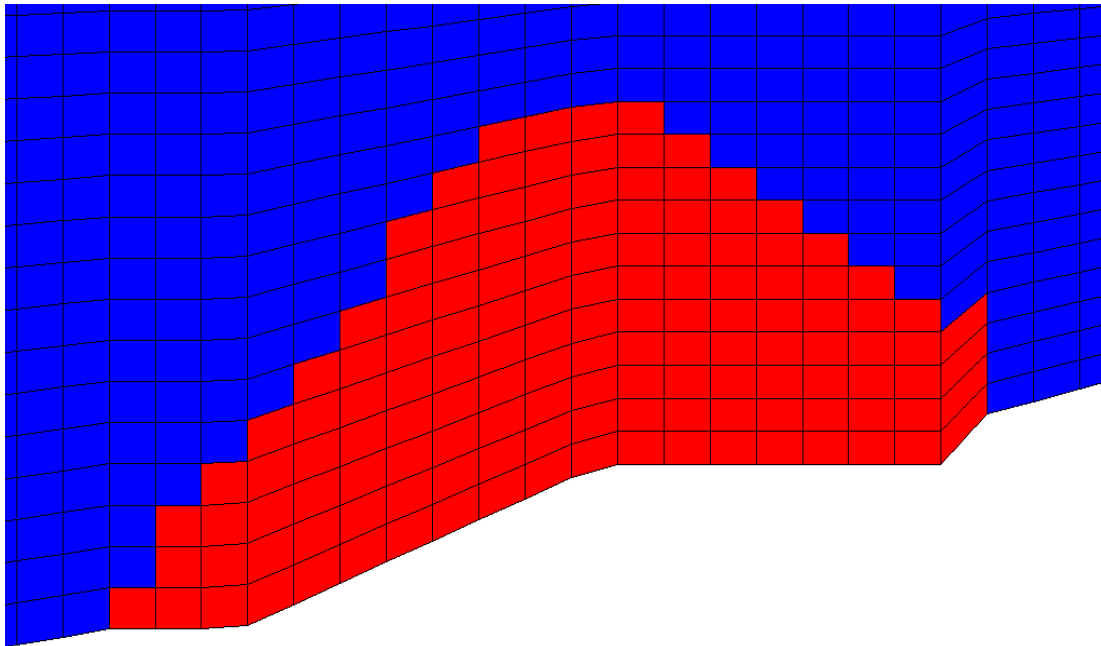


Fig. 4.32 - P-1 rubble mound breakwater porosity map.

The porosity parameters used in this profile were grain diameter, $d_{50}=0.005$ m, laminar resistance coefficient, $\alpha=1000$, turbulent resistance coefficient, $\beta=2.8$, oscillating period, $T=8$ s and the porosity equal to 0.4. During the tests, several values of the grain diameter were used, but after the analysis of the physical model simulation videos (made available by DHI), the conclusion was that 0.005 m is the one that better represents the reality.

4.2.4.2 H1-4 Porosity

This structure is composed of two different rubble mound breakwaters on top of each other. So, in this case, since their physical characteristics are different, they were defined as different porous zones to be possible to set the parameters, Fig. 4.33. In this figure, in the upper right corner, the question of resolution can be perceptible. Due to the high slope in that section, the definition of the light green area does not match the prototype. This happens because each sigma layer is parallel to the bottom of the flume.

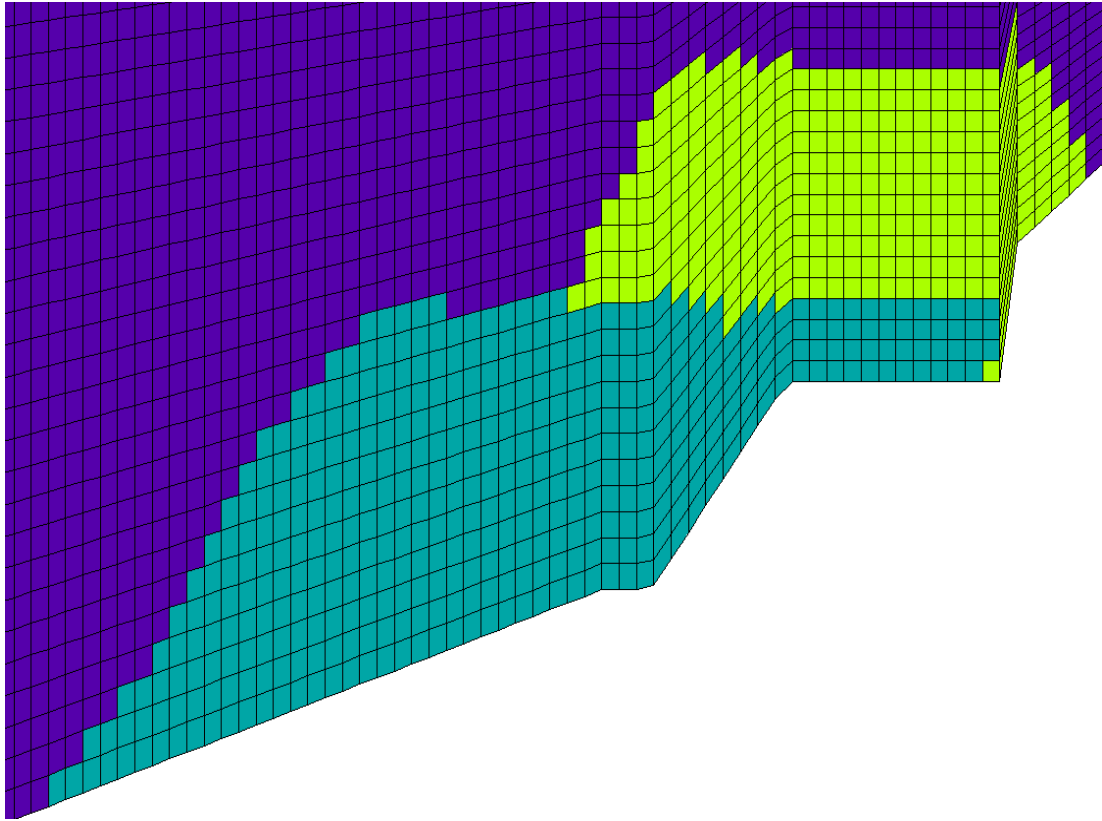


Fig. 4.33 - H4-1 rubble mound breakwater porosity map.

The parameters used to define the lower rubble mound breakwater, the dark green one, were grain diameter, $d_{50}=0.0026$ m, laminar resistance coefficient, $\alpha=1000$, turbulent resistance coefficient, $\beta=2.8$, oscillating period, $T=8$ s, and the porosity equal to 0.4. The upper one with the light green colour, was defined with the same parameters, except for the grain diameter that was set as 0.005 m.

Another feature of this profile is the presence of ribs after the rubble mound breakwater. This characteristic was not defined, as it should be, as a bathymetry. This is due to the angle of the ribs, because the angle between the slope and the rib wall is 90° , Fig. 4.34.

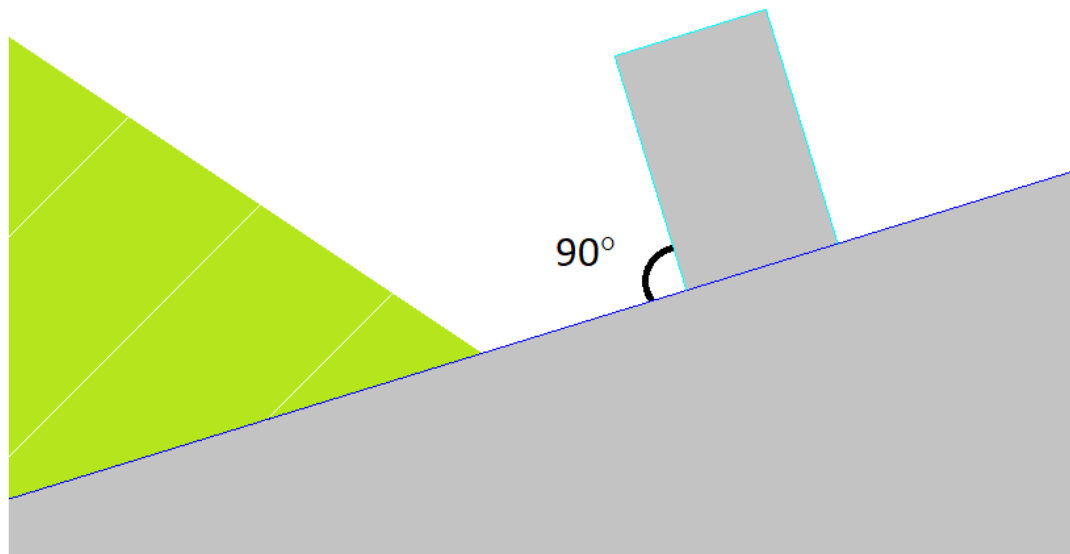


Fig. 4.34 - End of the upper breakwater (light green) and detail of the first rib. Side view.

Once the definition of the bathymetry is made with vertices connected by lines, the bottom rib vertex is further ahead, in the x axis, than the top one, what makes it mathematically impossible to define. So, the process adopted to create the ribs was to make them as a porous zone, with the porosity parameters that simulate that it is an impermeable object, Fig. 4.35.

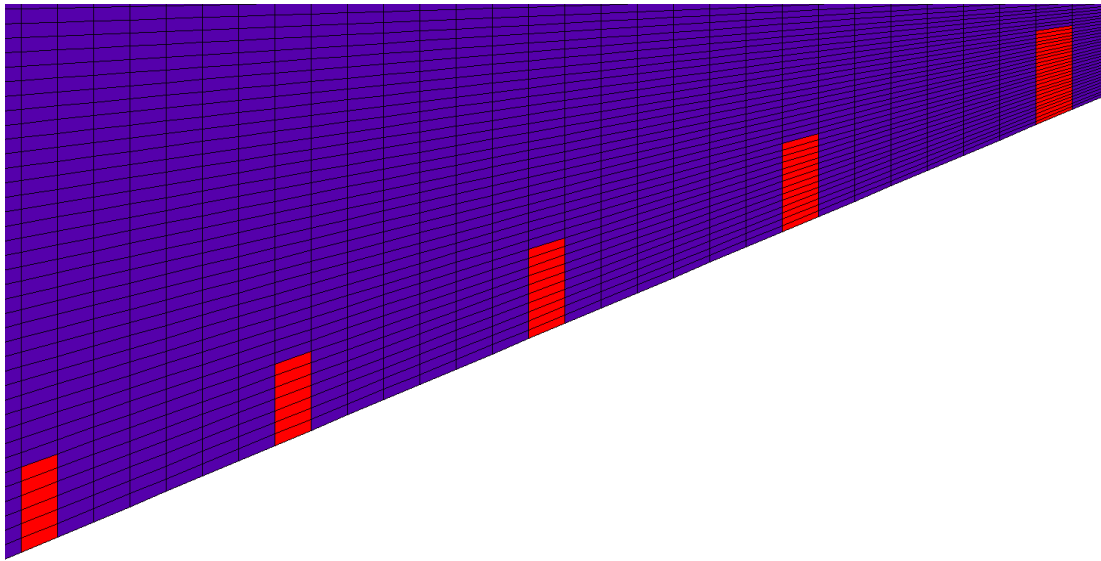


Fig. 4.35 - Ribs made as a porous object.

However, defining the ribs this way, two adversities occur. The first one is, as explained earlier in the Physical model setup sub-chapter, the ribs' pattern has some transversal gaps, Fig. 4.4, and once the numerical model only has one cell width, is impossible to create these gaps. The definition of the ribs would also be impossible to do as a bathymetry since the bathymetry is the same along the width of the flume.

Secondly, the angle between the slope and the rib wall was not 90° as it should be, because of the resolution of the model, Fig. 4.35. That can be easily understood when the image is analysed: the

columns of cells are always vertical. The only way to overcome this obstacle is to change the vertical and horizontal resolution to a very small discretization and, that way, it would be possible to create a porosity map with approximately the same slope. But this option would take a long time to be realized. So, the objects defined in the profile H1-4 as porous maps were the lower and upper rubble mound breakwaters, and the ribs. The porosity parameters used in the ribs were grain diameter, $d_{50}=0.005$ m, laminar resistance coefficient, $\alpha=1000$, turbulent resistance coefficient, $\beta=580000000$, oscillating period, $T=8$ s, and the porosity equal to 0.2. The parameters that deserve more attention in this case are the turbulent resistance coefficient and the porosity. The turbulence resistance coefficient is a parameter responsible for the resistance between the grains and the water. The higher this value the greater the resistance. This value is usually between 1 and 3, but in this case, to create the false impermeabilization, the value is high as 580000000. The other parameter is the porosity that was set with the value of 0.2. A lower value of porosity would be better to increase the resistance of the water, but during the calibration process some lower values were used and the simulations stopped due to blow-ups.

4.2.4.3 H4d-5 Porosity

The third and last profile was set with two porous zones although it only has one rubble mound breakwater, Fig. 4.36. The parameters used to define the breakwater porosity were: grain diameter, $d_{50}=0.05$ m, laminar resistance coefficient, $\alpha=1000$, turbulent resistance coefficient, $\beta=2.8$, oscillating period, $T=8$ s, and the porosity equal to 0.4.

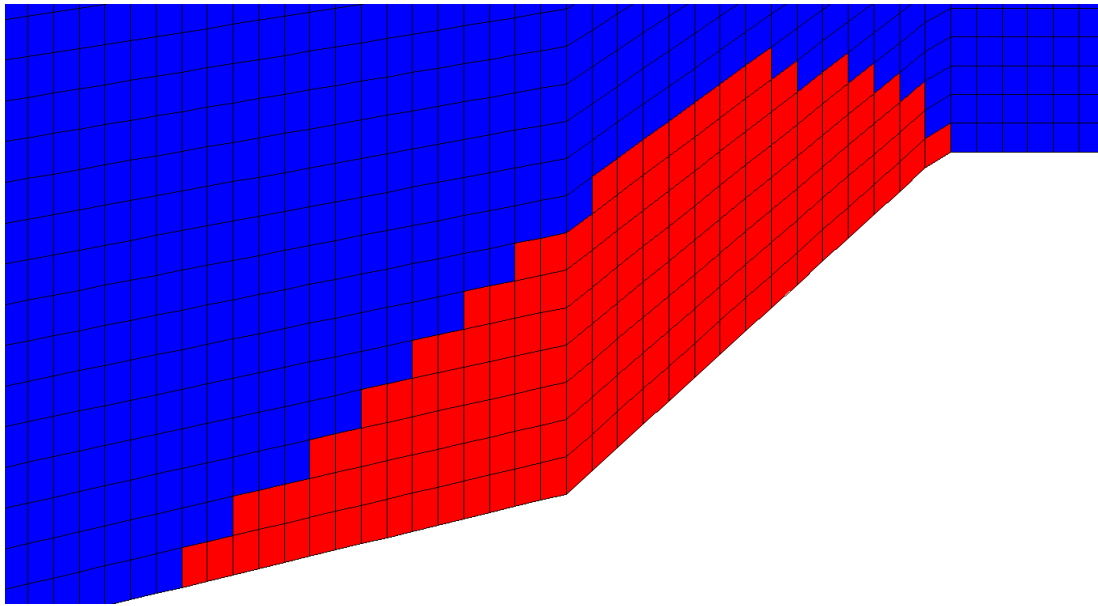


Fig. 4.36 - H4d-5 rubble mound breakwater porosity map.

The second porous zone is due to the presence of a bullnose in the crest of the dike, Fig. 4.37. Since the angle between the slope and the bullnose is 90° , the process to create this bullnose was the same as in the previous case, made as a porous zone.

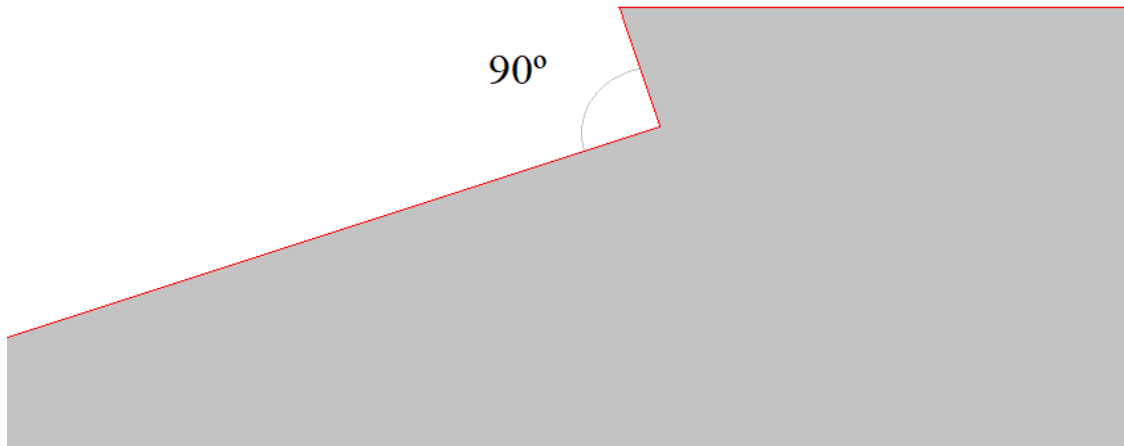


Fig. 4.37 - Bullnose detail.

The parameters used to define the porosity were: grain diameter, $d_{50}=0.005$ m, laminar resistance coefficient, $\alpha=1000$, turbulent resistance coefficient, $\beta=580000000000$, oscillating period, $T=8$ s, and the porosity equal to 0.1. In this porosity map did not occur any blow-up because the model is simpler than the previous case.

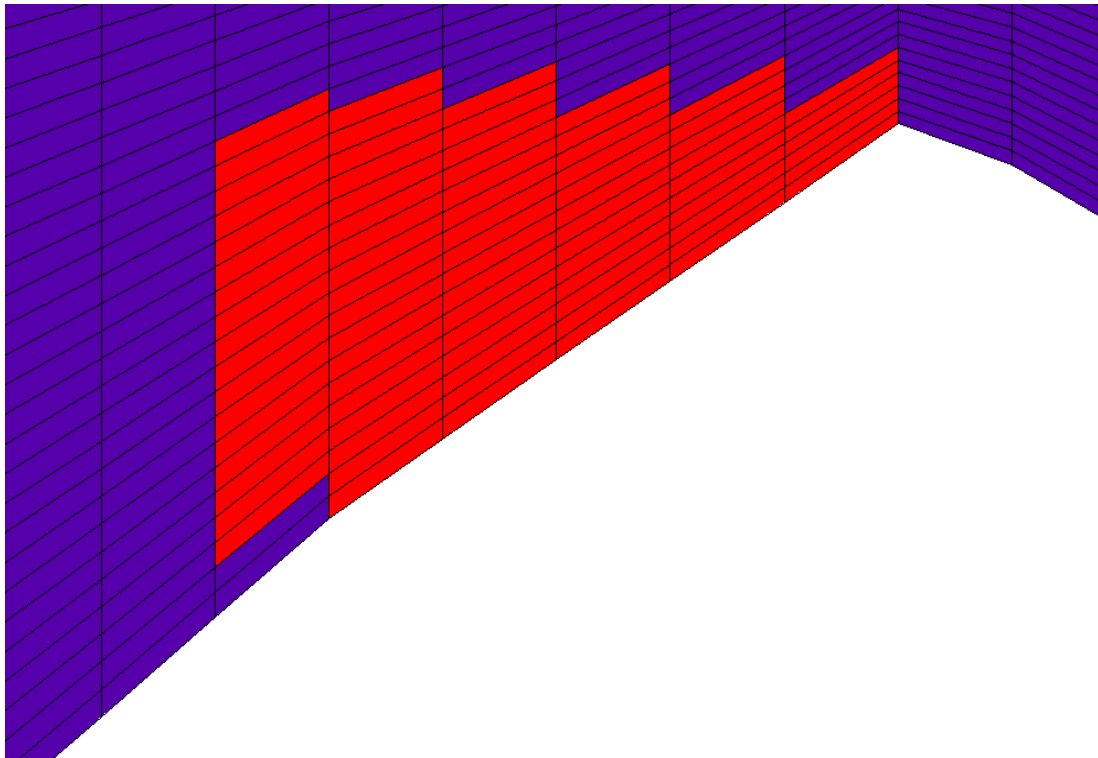


Fig. 4.38 - Bullnose made as a porous object.

4.3 NUMERICAL MODEL RESULTS

During the calibration tests, on average, 20 simulations were performed for each profile, because in this process only one parameter can be changed each time to determine its influence in the final results. Once the only differences between tests in the same profile are the peak period and the incident significant wave height, the first thing to do was to combine the energy of the wave spectrum from the numerical model with the physical model. That will grant that, statistically, the waves in both cases are the same. To do this analysis, a tool from the MIKE Zero was used: Wave Synthesizer (WS) Reflection Analyses. This tool allows to determine the wave spectrum through the measurements of the waves recorded by the wave gauges.

Having available sufficient data, from the physical model tests, to compare two incident waves spectrum analyses were performed: the first wave spectrum was defined by the wave gauges 1 to 5, Fig. 4.39, and the second one by wave gauges 6 to 10, Fig. 4.40.

The screenshot shows the 'Wave Gauges' section of the WS Reflection Analyses tool. It contains a table with 5 rows and 4 columns: Gauge no., Name, Elevation, and Position. The table lists gauges WG1 through WG5 with their respective elevations and positions. To the right of the table are four buttons: a grid icon, a red X icon, an up arrow icon, and a down arrow icon. To the right of the table is an 'Output' section with five checkboxes: 'Auto spectra', 'Average auto spectrum', 'Incident spectrum' (checked), 'Reflected spectrum', and 'Reflection function'. At the bottom of the 'Output' section is a checked checkbox for 'Spectrum parameters'.

Gauge no.	Name	Elevation	Position
1	WG1	1 (WG1: Surface elevatio	2.000
2	WG2	2 (WG2: Surface elevatio	2.400
3	WG3	3 (WG3: Surface elevatio	2.750
4	WG4	4 (WG4: Surface elevatio	3.000
5	WG5	5 (WG5: Surface elevatio	3.100

Fig. 4.39 - WS Reflection Analyses tool. Selection of the wave gauges 1 to 5.

The screenshot shows the 'Wave Gauges' section of the WS Reflection Analyses tool. It contains a table with 5 rows and 4 columns: Gauge no., Name, Elevation, and Position. The table lists gauges WG6 through WG10 with their respective elevations and positions. To the right of the table are four buttons: a grid icon, a red X icon, an up arrow icon, and a down arrow icon. To the right of the table is an 'Output' section with five checkboxes: 'Auto spectra', 'Average auto spectrum', 'Incident spectrum' (checked), 'Reflected spectrum', and 'Reflection function'. At the bottom of the 'Output' section is a checked checkbox for 'Spectrum parameters'.

Gauge no.	Name	Elevation	Position
1	WG6	6 (WG6: Surface elevatio	15.500
2	WG7	7 (WG7: Surface elevatio	15.900
3	WG8	8 (WG8: Surface elevatio	16.250
4	WG9	9 (WG9: Surface elevatio	16.500
5	WG10	10 (WG10: Surface eleva	16.600

Fig. 4.40 - WS Reflection Analyses tool. Selection of the wave gauges 6 to 10.

Even though this tool gives the incident wave spectrum, it cannot determine the value of the H_{m0} . Therefore, a MATLAB script was created where the incident spectrum is uploaded and the H_{m0} is calculated by doing the area's integral, m_0 , and then multiply four times by its square root ($H_{m0}=4(m_0)^{1/2}$).

Another analysis done was the calculation of the wave heights in different positions of the flume. That allows the comparison of the numerical model with the physical model and the prediction of the waves'

behaviour. This analysis was also performed with another tool from MIKE Zero: WS Crossing Analyses. This tool equally uses the output information from the wave gauges and makes a peak-to-peak analysis.

To evaluate the discharge in the crest of the dike it was necessary to create a MATLAB script, Fig. 4.41, to manipulate the data from the M3, because the discharge output gives the results in the form of time steps instead of overtopping events.

```

1 - clearvars,
2
3 - load('time_series_0316.txt');
4 - t_diff=diff(time_series_0316,1,1)
5 - [idd,~] =dat2crossind(t_diff,1e-12,'d');
6 - [idu,~] =dat2crossind(t_diff,1e-12,'u');
7 - figure,
8 - plot(t_diff,'b.-'); hold on
9 - plot(idd,t_diff(idd),'r.',idu,t_diff(idu),'k.');
```

```

% load data
% transform cdf to peaks
% find zero-downcrossings
% find zero-upcrossings
```

```

10
11 - figure,
12 - plot(time_series_0316); hold on
13 - plot(idd,time_series_0316(idd),'r.',idu,time_series_0316(idu),'r.')
14 - % idu=idu+1
15 - %idu(end)=[];
16 - idtot =[idu,idd];
17 - % plot results
18
19 - % calculate the discharge on an event basis
20 - Q_events =time_series_0316(idd)-time_series_0316(idu);
21 - figure,
22 - histogram(Q_events);
```

```

% maybe correct upcrossings DETERMINE
% indices
```

Fig. 4.41 - MATLAB script to determine each overtopping event.

To run this script is necessary to create a .txt file with the time series discharges from the M3 output. This scrip identifies the when the discharge is zero in the time series and create indexes. The first index is before the discharge and the second index is after. This way each overtopping event is isolated from the continuous time series. So, the volume of each event is the charted area between the indexes.

In the end of each simulation is created a log file where is summed up all the information regarding the model. In Annex A is presented the log files of the final simulations. Only the input information is presented because the extension of each file is considerable.

4.3.1 RESULTS OF P-1

4.3.1.1 Test 5

The test number 5 was performed with the design wave height characterized by $H_{m0}=4.27$ m and a $T_p=7.69$ s. As expected, from the firsts attempts, the spectrum of the wave gauges 1 to 5 did not have the same behaviour as in the physical model so, some calibrations were made to match it. This equality was achieved with the alteration of the significant wave height parameter.

The criterion assumed was to match the incident spectra of the first array of wave gauges. Thus, the wave generation has the same starting point, relatively to the wave energy. This makes the comparative analysis possible, since the characteristics are the same *a priori*.

Although the peak of the incident spectrum is greater in the numerical model, Fig. 4.42, the significant wave height is practically the same. In the numerical model the H_{m0} is 4.55 m, and in the physical one is 4.56 m, values measured in the first array of wave gauges

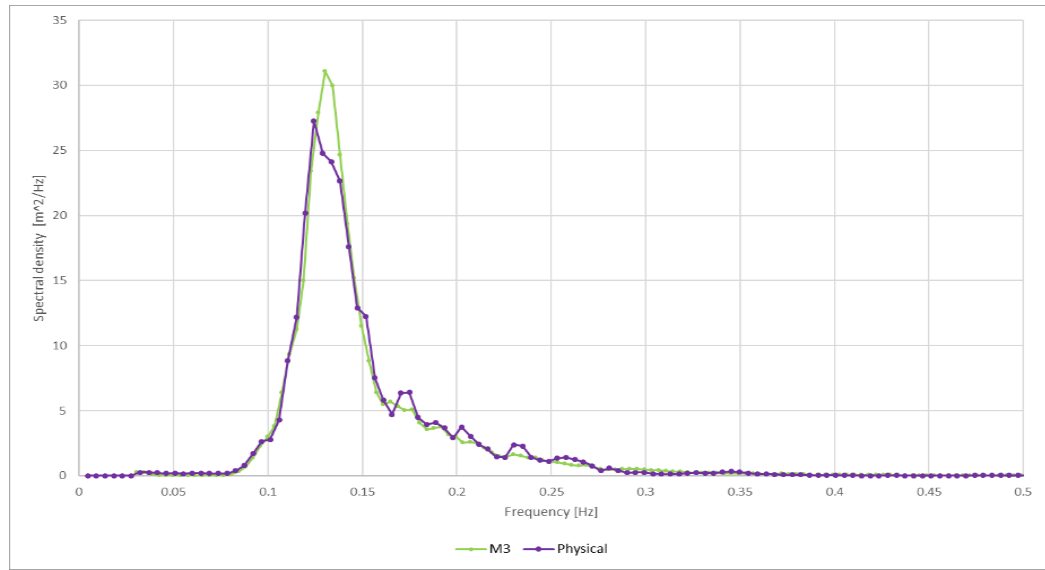


Fig. 4.42 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 5.

In its turn, in the second array of wave gauges, WG6 to WG10, although the spectra seems more identical, Fig. 4.43, the fact is that the H_{m0} of the numerical model is 4.04 m and in the physical model is 4.32 m. It is verified that between arrays the numerical model loses more wave energy than the physical model.

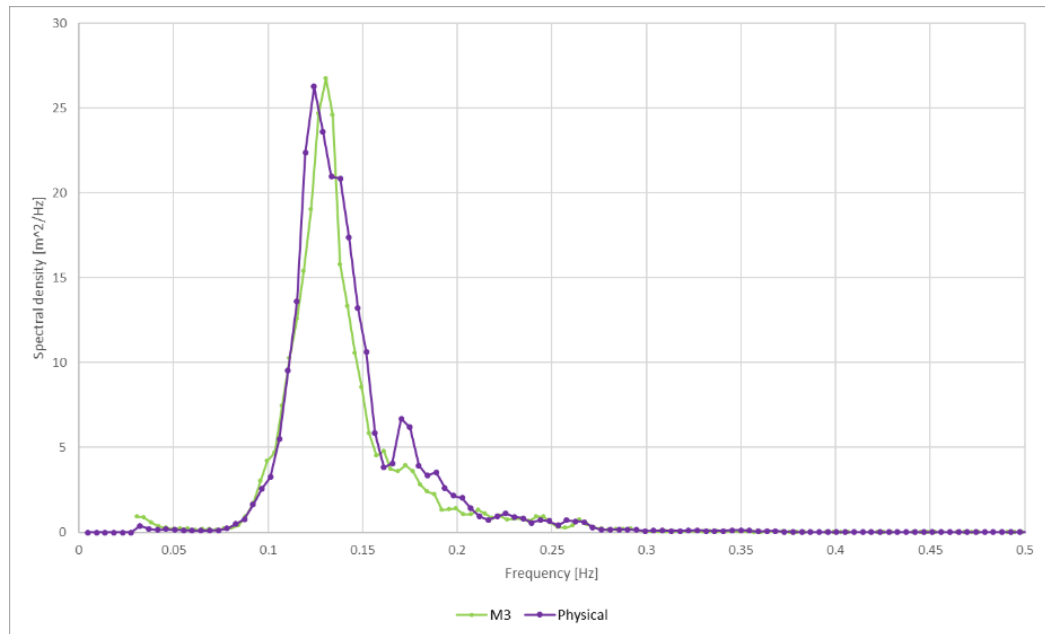


Fig. 4.43 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 5.

In the physical model test, the distribution of the individual wave height behaviours as expected: the further away the wave gauges are from the wave generator, the smaller the wave heights are, Fig. 4.44.

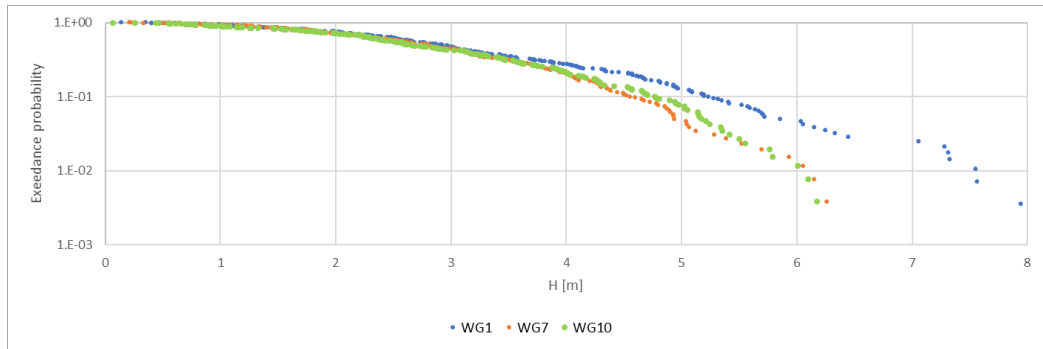


Fig. 4.44 - Wave height distribution of the physical model. Test no. 5.

In turn, the numerical model presents an identical behaviour identically to the physical model, Fig. 4.45.

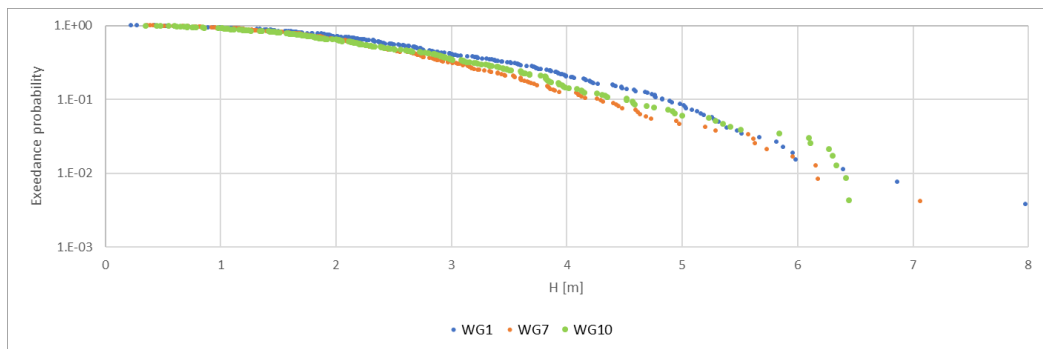


Fig. 4.45 - Wave height distribution of the M3 model. Test no. 5.

Considering the position of each wave gauge, an analysis was made comparing the wave height distribution in each position. In the WG1, the probability of occurring waves smaller than 4 m is practically the same in both models, Fig. 4.46. However, in the physical model, there is a higher probability of occurring waves with an height higher than 4 m.

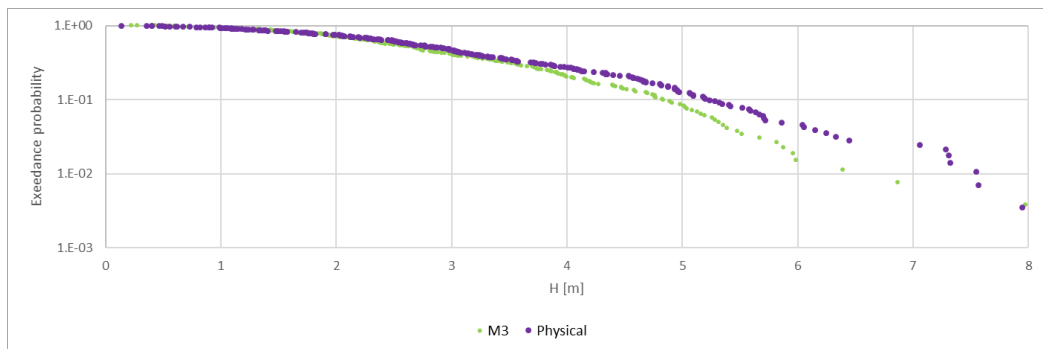


Fig. 4.46 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 5.

In the WG10, at the toe of the dike, waves smaller than 2 m have similar probability to happen, either in the physical as in the numerical model. Between 2 m and 5 m of wave height, the physical model is the one that generates higher waves. In its turn, in the numerical model it is found a higher probability of measuring waves higher than 5 m, Fig. 4.47.

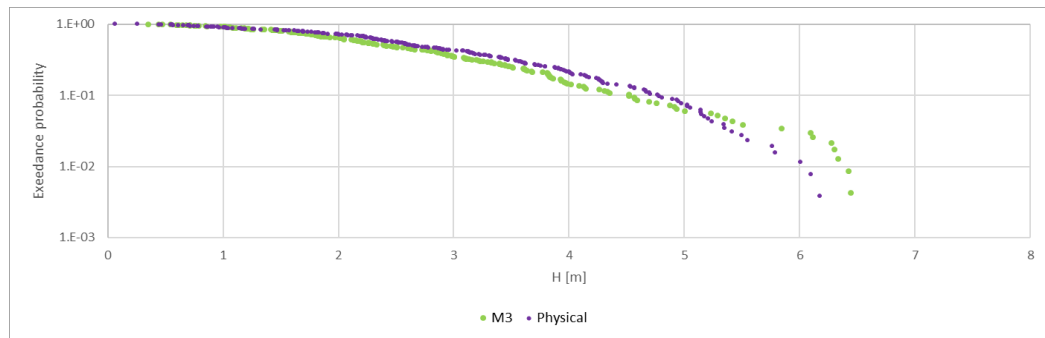


Fig. 4.47 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 5.

Considering the volume of water that passes over the crest of the dike in each overtopping event, observing the graphic it is easily understood that each discharge event has a larger amount of water in the numerical model than in the physical one, Fig. 4.48. Other aspect to take into account is that the physical model had 137 overtopping events while the numerical model had almost half that value, 79.

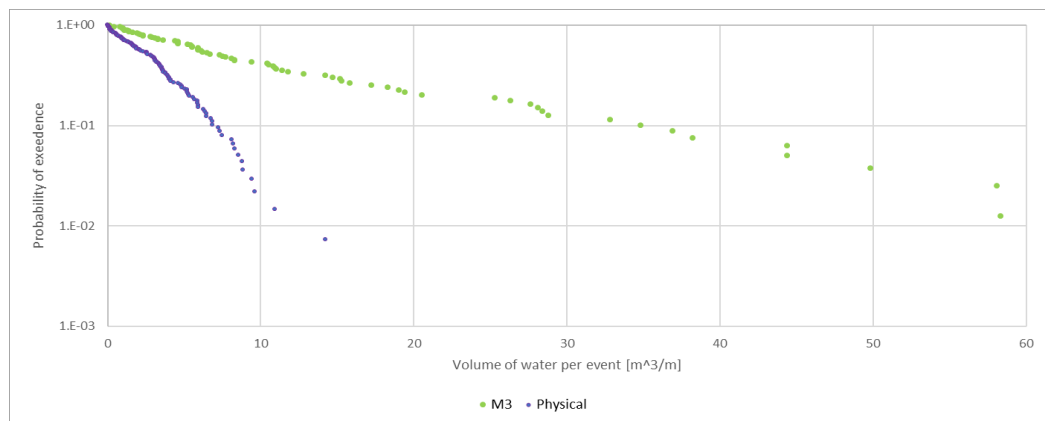


Fig. 4.48 - Event Volume distribution. Test no. 5.

Finally, the analysis of the accumulated overtopping volume over the simulation time, Fig. 4.49, shows a considerable difference between both models in the final accumulated overtopping volume. This disparity is due to a major factor: the volume of water of each overtopping event.

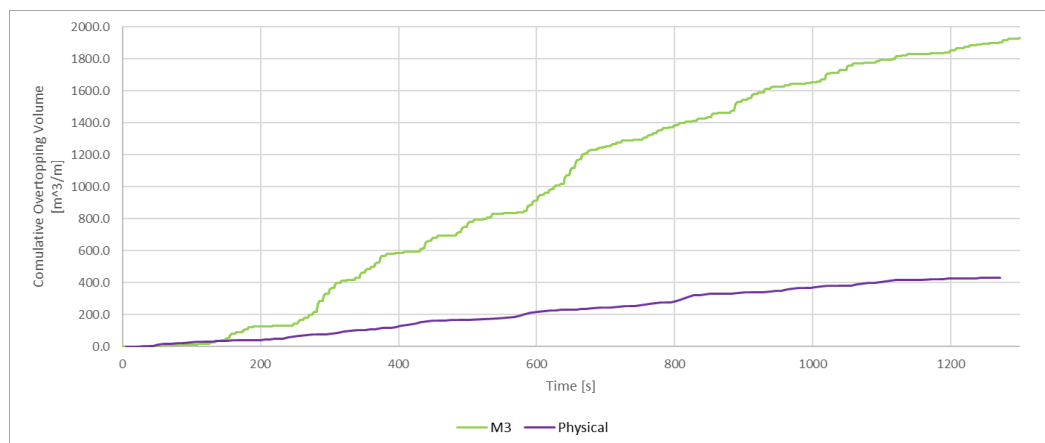


Fig. 4.49 - Time series of the accumulated overtopping volume. Test no. 5.

4.3.1.2 Test 16

In the test number 16, the second test of the P-1 profile, the wave conditions that are the design wave +10%, were set with a $H_{m0}=4.70$ m and a $T_p=8.07$ s. In the first array of wave gauges the H_{m0} of both models was identical, 5.14 m to the numerical model and 5.16 m to the physical. The incident spectra of the models match despite some peak differences, Fig. 4.50.

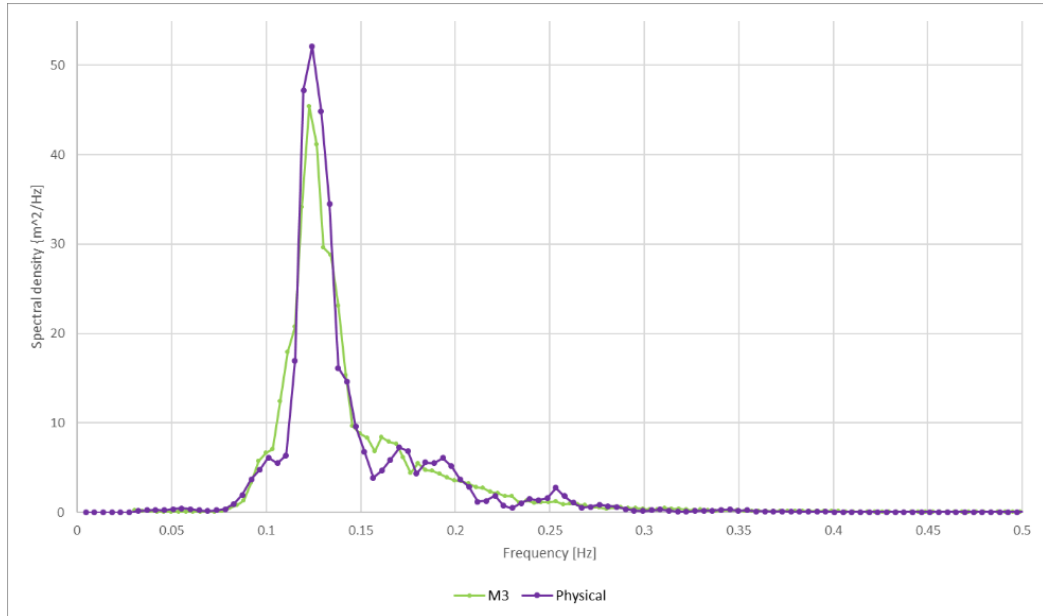


Fig. 4.50 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 16.

In the second array, the incident spectra (numerical and physical) had significantly perceptible differences, Fig. 4.51. The H_{m0} was 4.40 m in the M3 model and 4.82 m in the physical model. Again, the waves of the numerical model lost more energy than physical model ones.

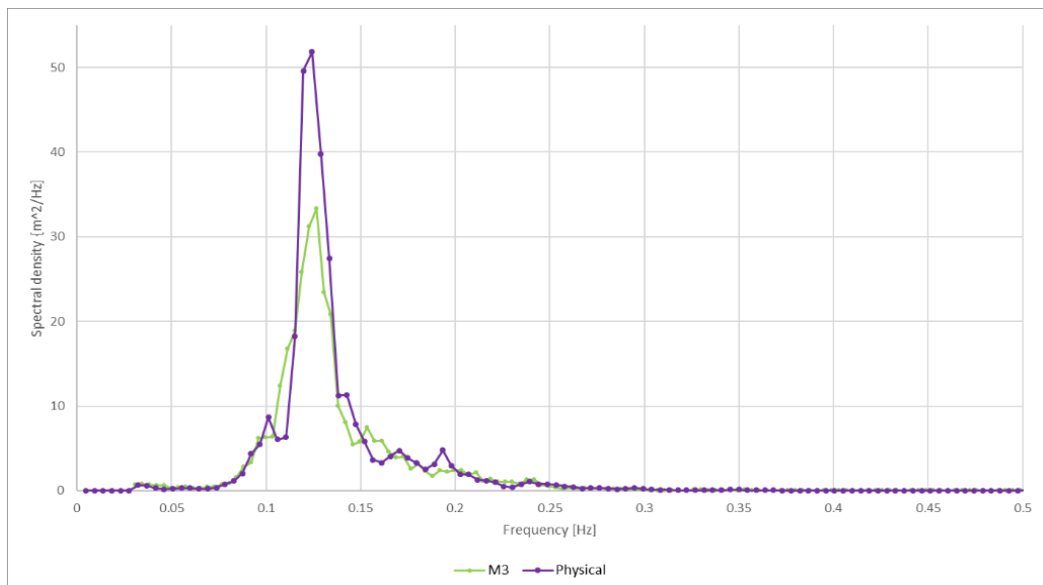


Fig. 4.51 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 16.

The waves behaviour as anticipated, the further away from the wave generator, the smaller the waves are, Fig. 4.52. It is more likely to have bigger waves near the wave maker than close to the dike, because the waves have more energy in the beginning and, as expected, loose it with the progression in the flume.

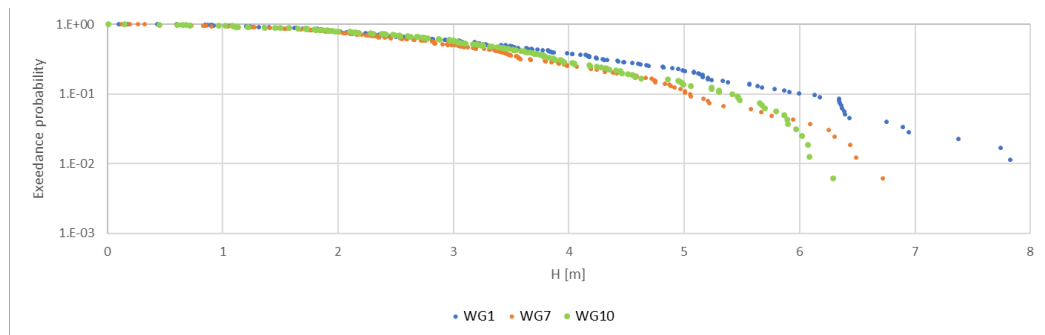


Fig. 4.52 - Wave height distribution of the physical model. Test no. 16.

The wave heights in the numerical model had some unexpected values. The WG1 and WG10 had similar values despite being the first and the last gauge, Fig. 4.53. Mainly considering that the WG7 is very close to the WG10 and the values are smaller.

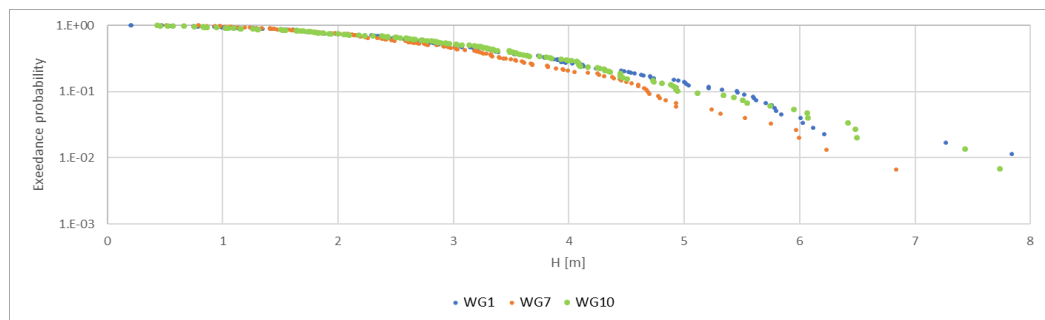


Fig. 4.53 - Wave height distribution of the M3 model. Test no. 16.

The wave heights in the first wave gauges, were bigger in the physical model, except to waves lower than 3 m, where the probability of occurrence is the same, Fig. 4.54.

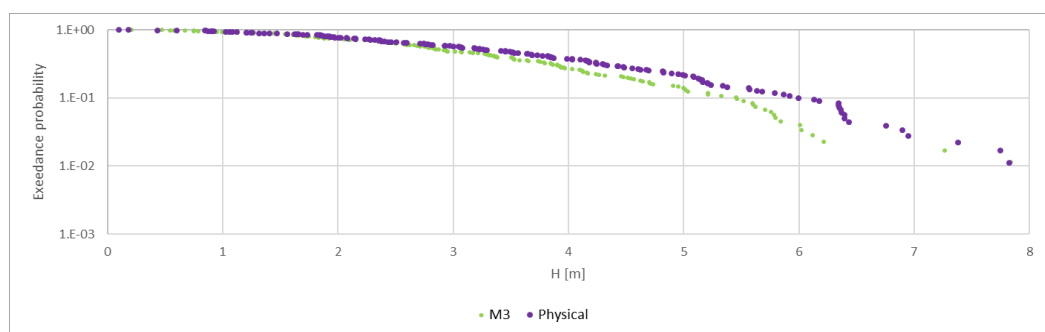


Fig. 4.54 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 16.

Waves with less than 6 m have the same probability of occurring in both models, but the waves bigger than that are more likely to be observed in M3, Fig. 4.55. It is noteworthy that there is a group of waves greater than 6.5 m in the numerical model, which is not verified in the physical model.

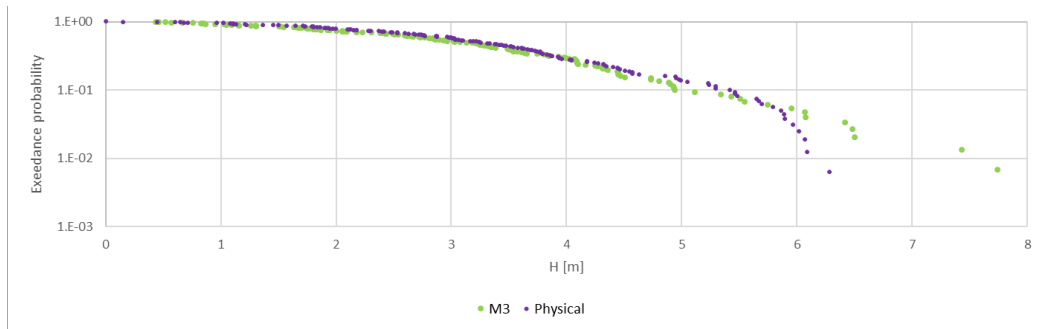


Fig. 4.55 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 16.

The overtopping discharges in this test had strange values. It is again verified that, in the numerical model, each overtopping event has a greater amount of water, Fig. 4.56. The largest volume discharged by the physical model was 12 m^3 while the largest volume of the numerical model was 75 m^3 . The number of events is also discrepant, the physical model had 114 overtopping events while the numerical model had 159.

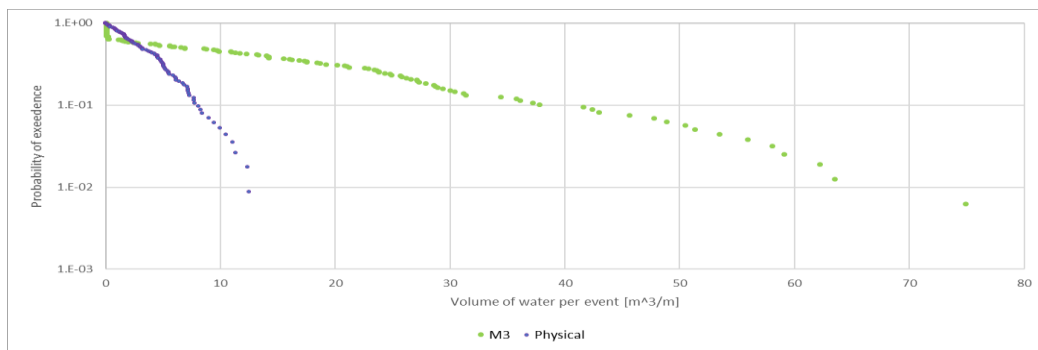


Fig. 4.56 - Event Volume distribution. Test no. 16.

Regarding the time series of the accumulated discharge, the numerical model presented a larger volume of water passing over than the physical model, Fig. 4.57. This can be explained by three factors: bigger waves near the dike, more overtopping events and the bigger volume of water of each event.

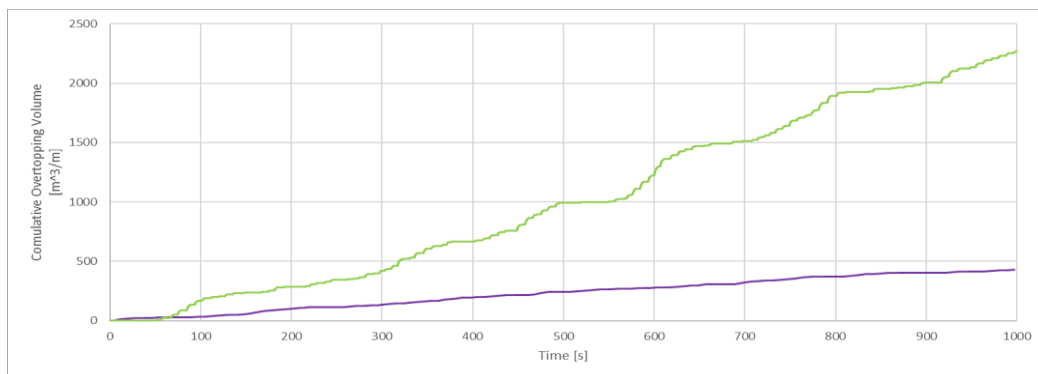


Fig. 4.57 - Time series of the accumulated overtopping volume. Test no. 16.

4.3.1.3 Test 36

In the last test in the present profile, P-1, the wave conditions were established as the wave design -10%, with a $H_{m0}=3.84$ m and a $T_p=7.29$ s. The overlap of the spectra is almost total, except the peak where it is slightly higher in the physical model, Fig. 4.58. The H_{m0} of the M3 is 4.13 m and in the physical one is 4.14 m.

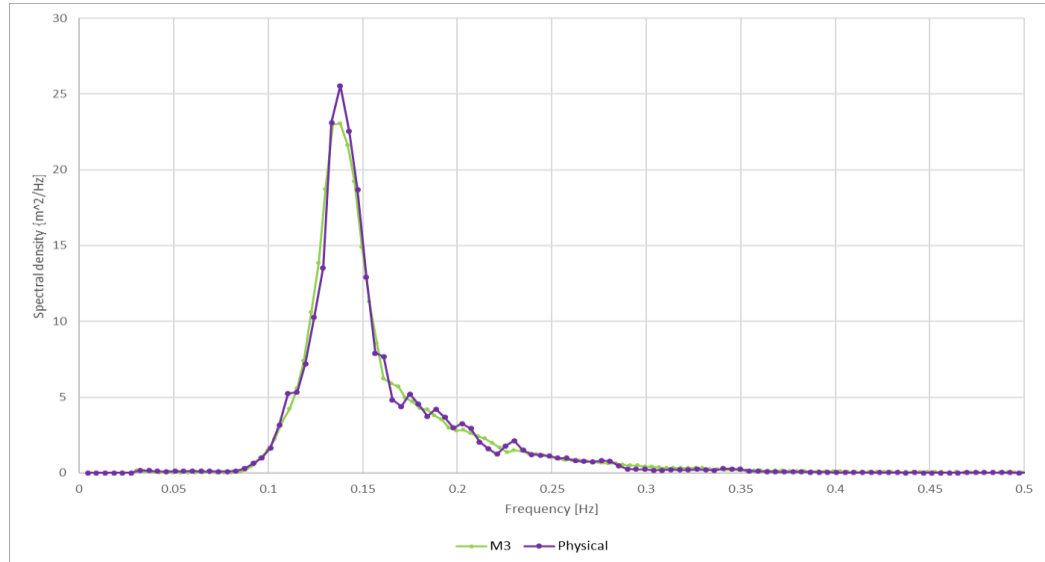


Fig. 4.58 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 36.

The H_{m0} calculated with the second array, WG6 to WG10, was 3.65 m in the numerical model and 3.95 m in the other model. This difference can be noticed in Fig. 4.59, the incident spectrum of the physical model is bigger, resulting in greater H_{m0} .

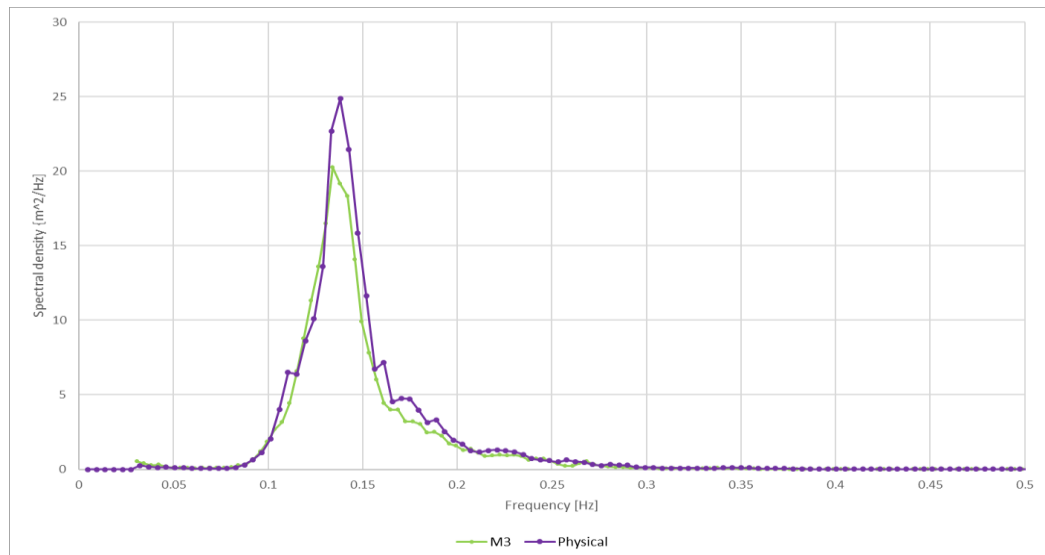


Fig. 4.59 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 36.

In the physical model occurs the same as in the previous cases: the wave gauges closest to the generator have higher wave heights, Fig. 4.60.

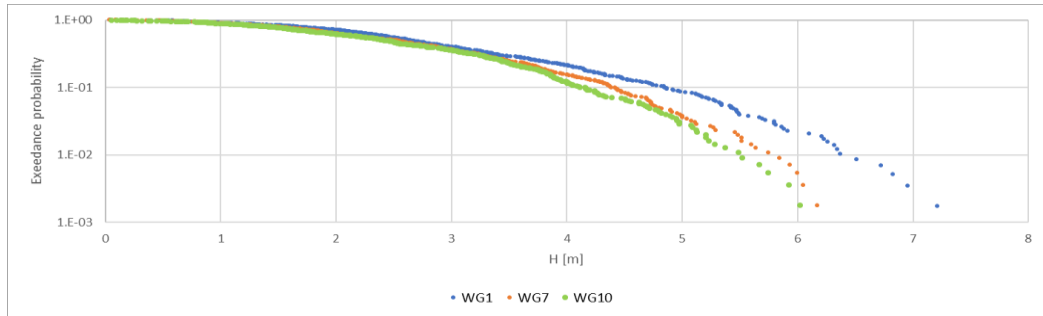


Fig. 4.60 - Wave height distribution of the physical model. Test no. 36.

The waves generated in M3 had some curious values. In WG10 there is a higher probability of occurring waves bigger than 5 m than the in the WG7, Fig. 4.61. That is an unexpected result since both wave gauges are in the flat part of the flume and the WG7 is near to the wavemaker.

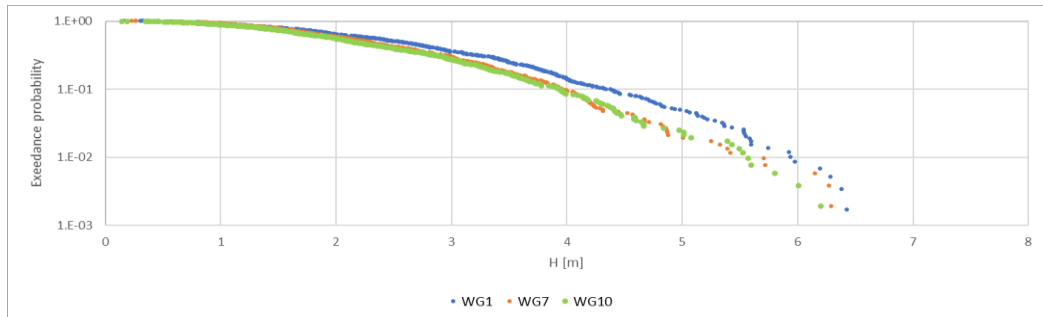


Fig. 4.61 - Wave height distribution of the M3 model. Test no. 36.

In the position of $x=2$ m, WG1, the waves until 3.5 m have identical probability of happening in both models. However, there is a higher chance of occurring waves bigger than 3.5 m in the physical model, Fig. 4.62.

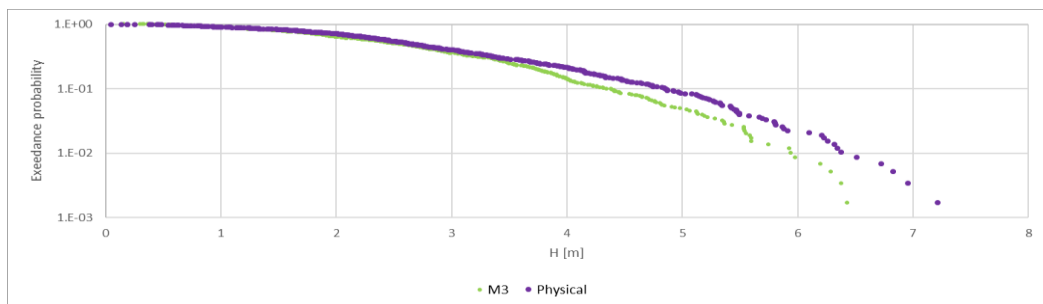


Fig. 4.62 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 36.

In the WG10, the wave heights are very similar in the two models, Fig. 4.63. There is a small variance in the wave heights but it is residual.

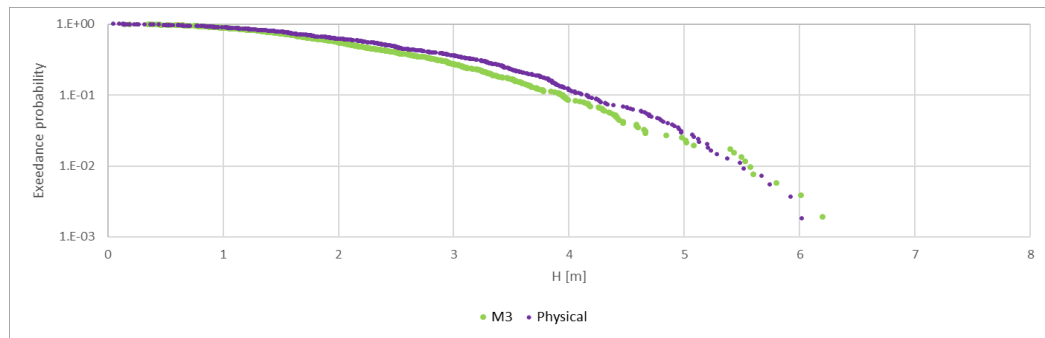


Fig. 4.63 - WG10 ($x=16.60$ m) Wave height distribution. Test no. 36.

Comparing the discharge of the models, the physical model had 205 overtopping events while the numerical model had 283. Other concerning aspect is the volume of each event. The largest volume discharged by the physical model was 9 m^3 while the largest volume of the numerical model was more than 6 times this value, 60 m^3 , Fig. 4.64.

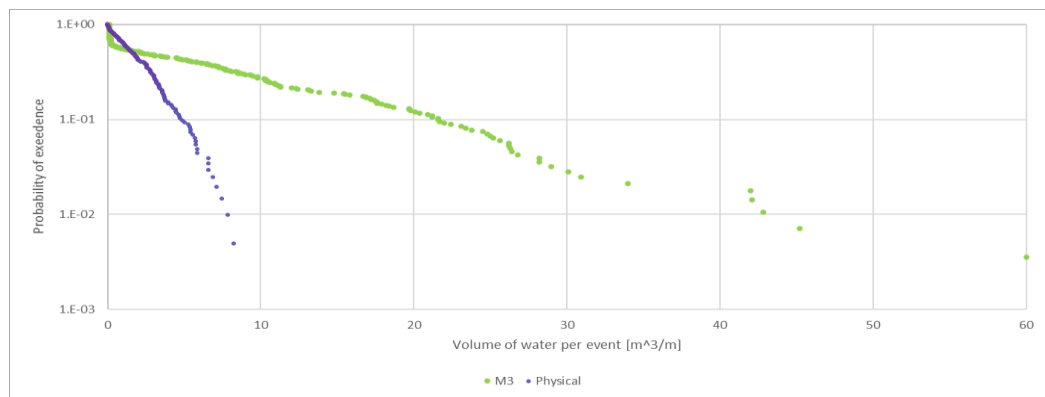


Fig. 4.64 - Event Volume distribution. Test no. 36.

The time series of the accumulated discharge shows what was foreseeable due to the values analysed above: a big difference between the number of discharges and the volume of each of these events, Fig. 4.65. However, since the wave heights in the WG10 are quite similar, these discharges are surprising.

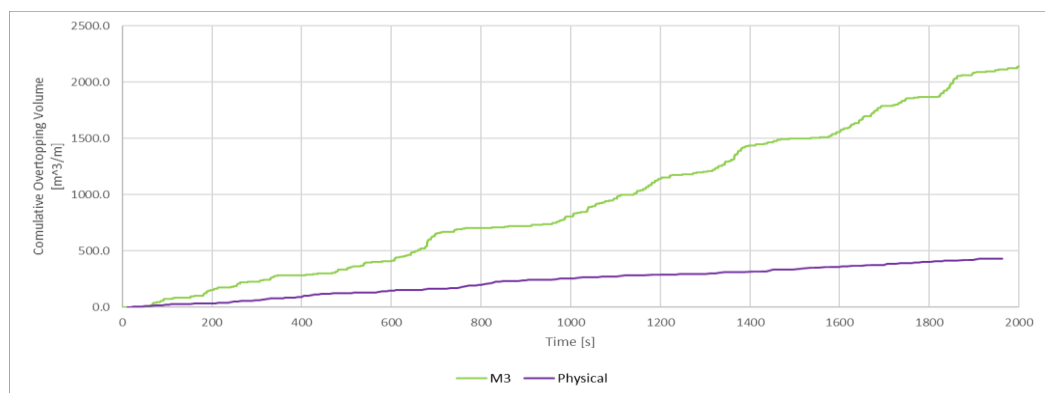


Fig. 4.65 - Time series of the accumulated overtopping volume. Test no. 36.

4.3.1.4 P-1 Synthesis

Overall, the correlation of the M3 model with the physical model is good. The relative freeboard is almost the same and the difference in the relative overtopping rate is acceptable, considering that the exact result is very difficult to achieve, Fig. 4.66. The relative overtopping rate is often used to judge allowable overtopping.

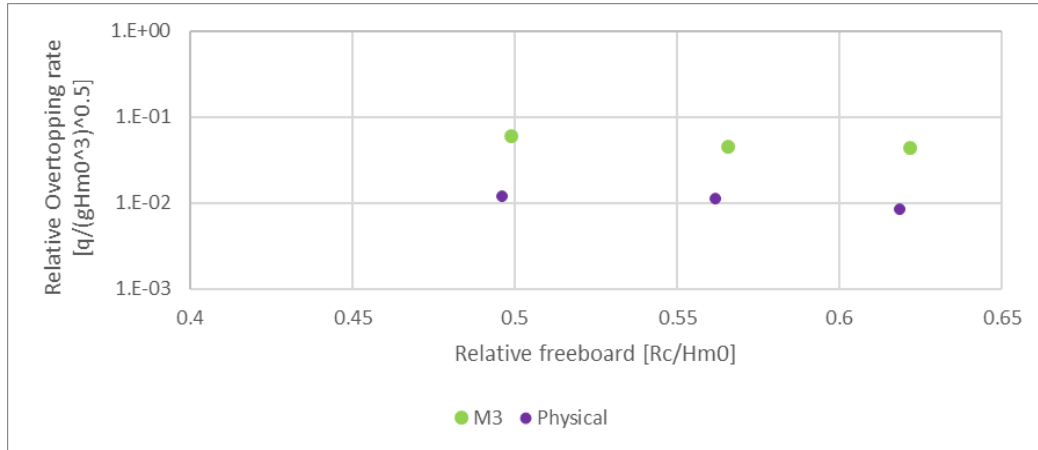


Fig. 4.66 - Comparison of the overtopping in the physical model and the M3 model, profile P-1.

Summing up, the results show that the numerical model lost more energy than the physical between the two arrays of wave gauges, and that the number of overtopping events is higher in M3 model, except the test 5, Table 4.3.

Table 4.3 - Synthesis of P-1 tests.

Profile	Test	H _{m0} [m]				no. of overtopping events	
		WG1-WG5		WG6-WG10			
		M3	Physical	M3	Physical	M3	Physical
P-1	5	4.55	4.56	4.04	4.32	79	137
	16	5.14	5.16	4.4	4.82	159	114
	36	4.13	4.14	3.65	3.95	283	205

4.3.2 RESULTS OF H1-4

4.3.2.1 Test 85

The test number 85 is the first test of the profile H1-4. The test was performed with a wave design - 10%, with a H_{m0}=3.84 m and a T_p=7.29 s. Although the spectra do not appear to coincide, the H_{m0} values associated with them are exactly the same, 3.85 m, Fig. 4.67.

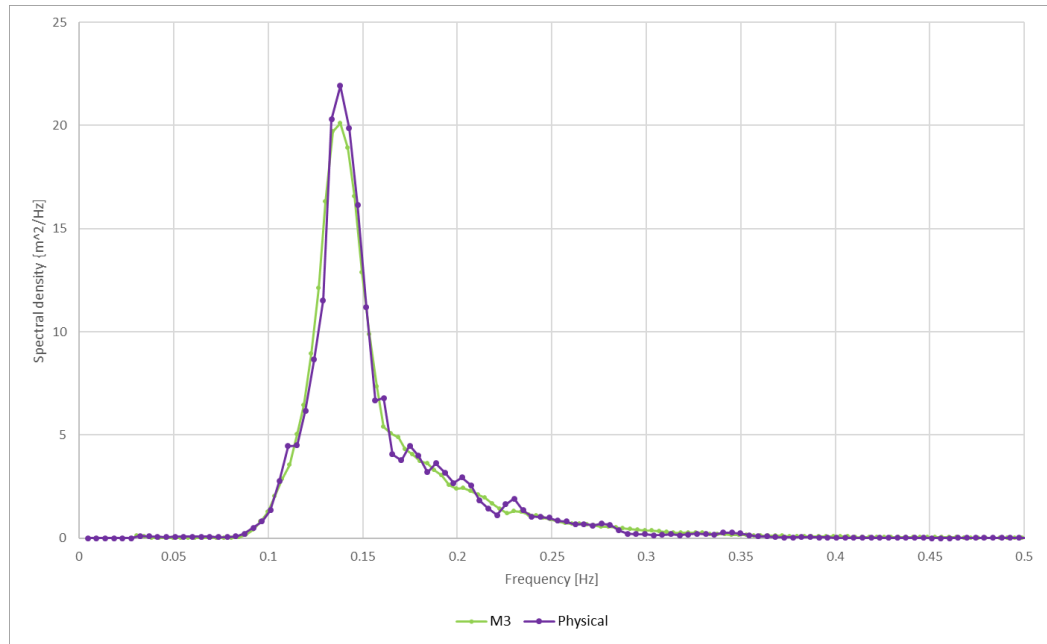


Fig. 4.67 - Incident spectrum of the M3 (green) and the physical model (purple), from WG1 to WG5. Test no. 85.

In the second array, the overlap in the spectra is considerable and the H_{m0} results reflect that, 3.45 m in the numerical model and 3.47 m in the physical model, Fig. 4.68.

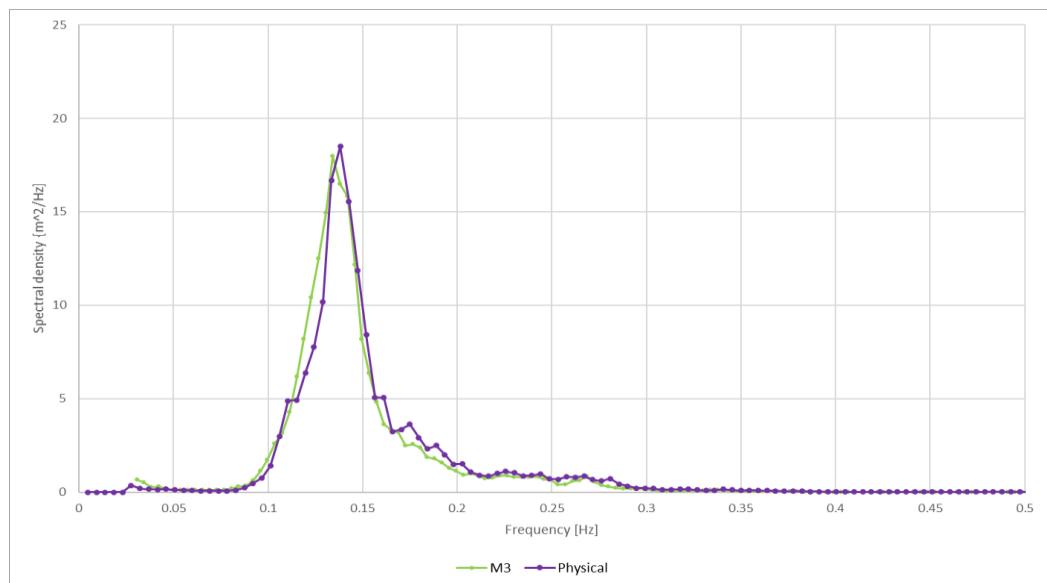


Fig. 4.68 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 85.

In the physical model, the wave gauges that are close to the wave generator have bigger waves than the most distant ones, Fig. 4.69. Since the WG7 and WG10 gauges are close to each other and both are in the flat zone of the flume, the results are identical. Although those of WG10 are smaller because in the distance between wave gauges the waves loose energy.

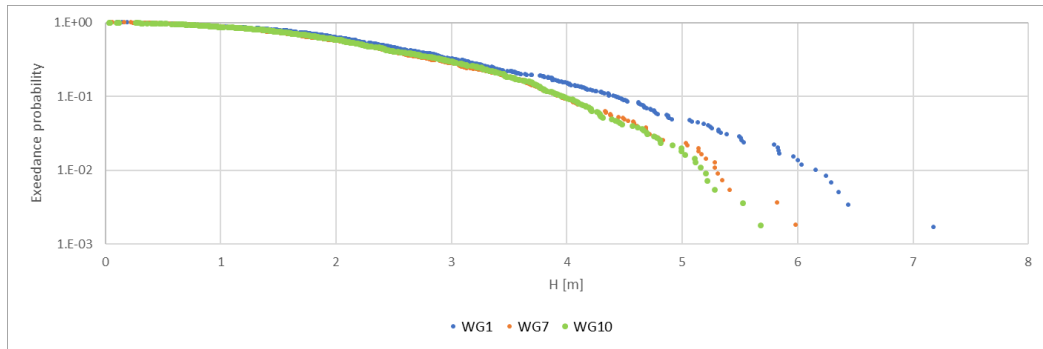


Fig. 4.69 - Wave height distribution of the physical model. Test no. 85.

The waves in the first and in the last wave gauges are very similar, which, again, would not be the most expected results, Fig. 4.70.

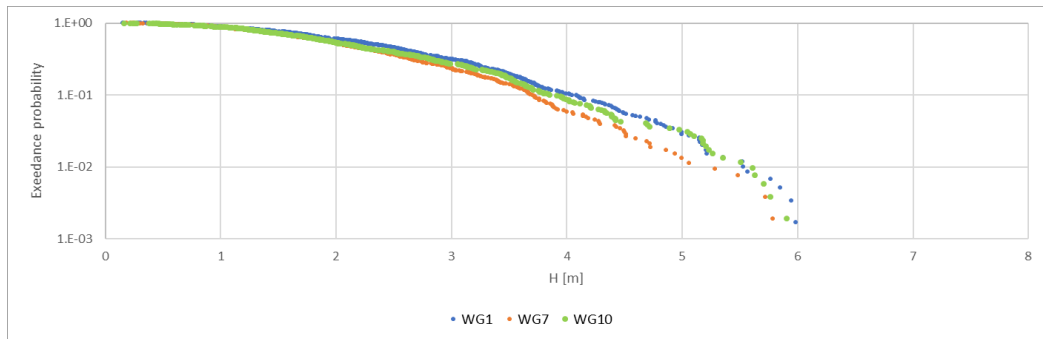


Fig. 4.70 - Wave height distribution of the M3 model. Test no. 85.

The probability to observe waves smaller than 3.5 m is identical in both models but waves higher than that are more likely to occur in the physical model, Fig. 4.71.

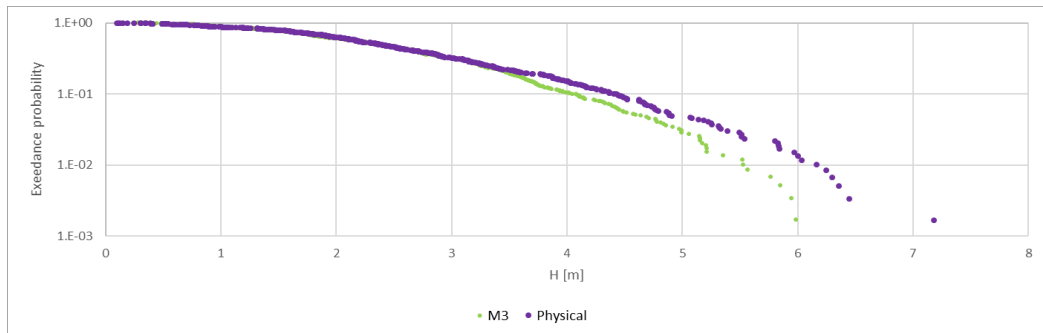


Fig. 4.71 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 85.

The situation reverses when the WG10 is analysed. After a set of wave heights where the probability of occurring is identical, it is found that the wave heights in the numerical model are bigger and more likely to occur, Fig. 4.72.

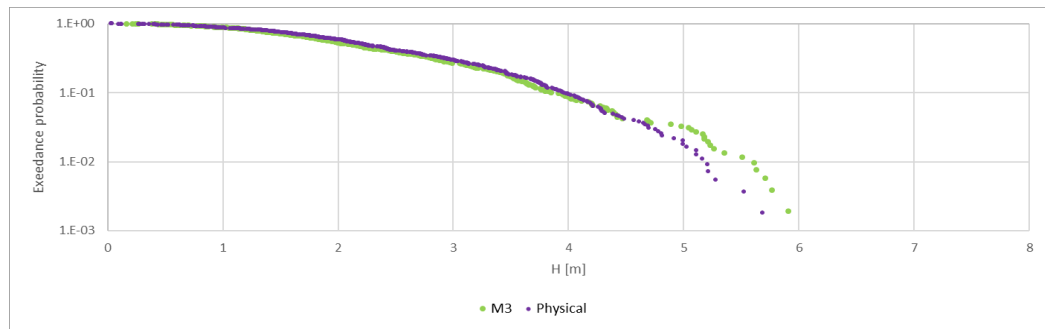


Fig. 4.72 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 85.

Yet, even with these results, the discharge did not present a good performance, Fig. 4.73. The physical model had 110 overtopping events while the numerical model had 97 events and each one with a bigger volume of water.

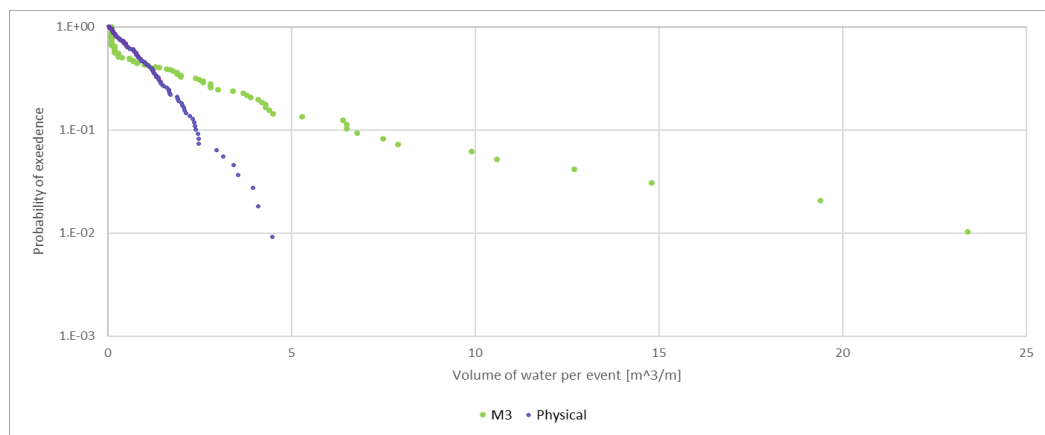


Fig. 4.73 - Event Volume distribution. Test no. 85.

Globally, this numerical model had an accumulated discharge similar to the physical, Fig. 4.74. This gap can be due to the difference in the volume of each overtopping event.

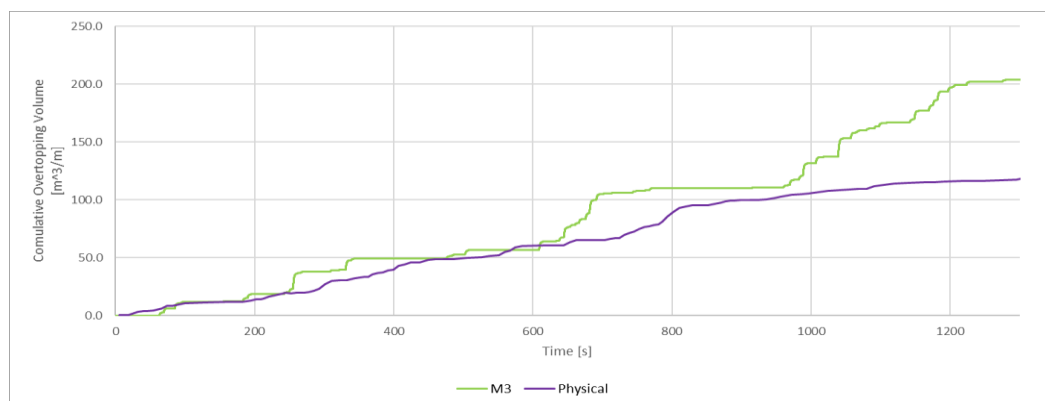


Fig. 4.74 - Time series of the accumulated overtopping volume. Test no. 85.

4.3.2.2 Test 86

In the second test of the H1-4 profile, test number 86, the wave characteristics were set as wave design, with an $H_{m0}=4.27$ m and a $T_p=7.69$ s. The spectra overlay is significant but the H_{m0} is slightly different, Fig. 4.75. The H_{m0} of the numerical model is 4.34 m and in the physical model is 4.32 m.

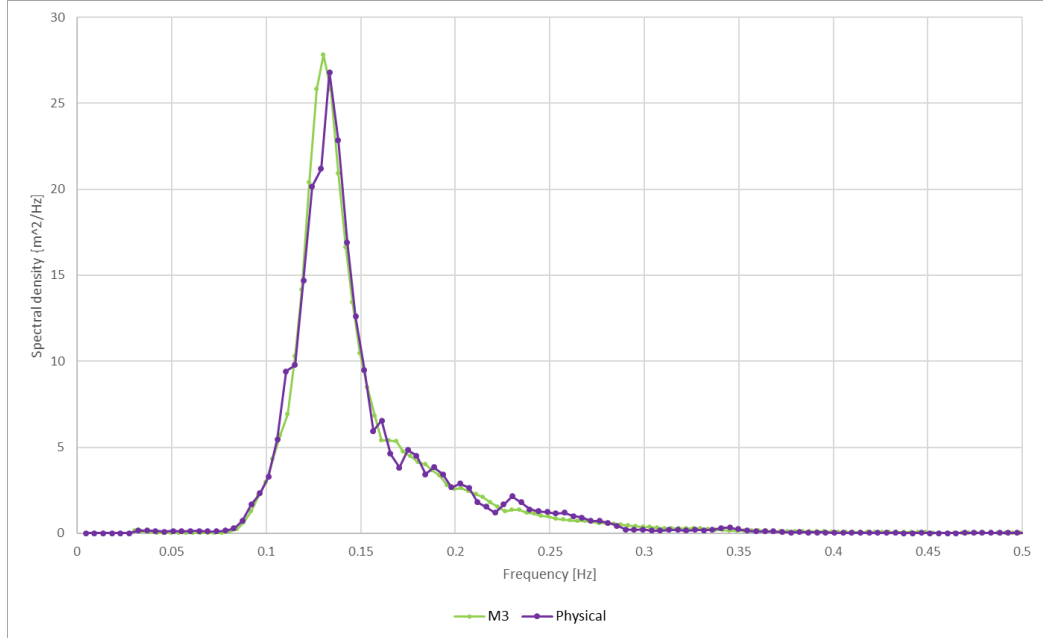


Fig. 4.75 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 86.

In the second group of wave gauges, the numerical spectrum is bigger and that is reflected in the values of the H_{m0} , 3.91 m in M3 and 3.87 m in the physical model, Fig. 4.76.

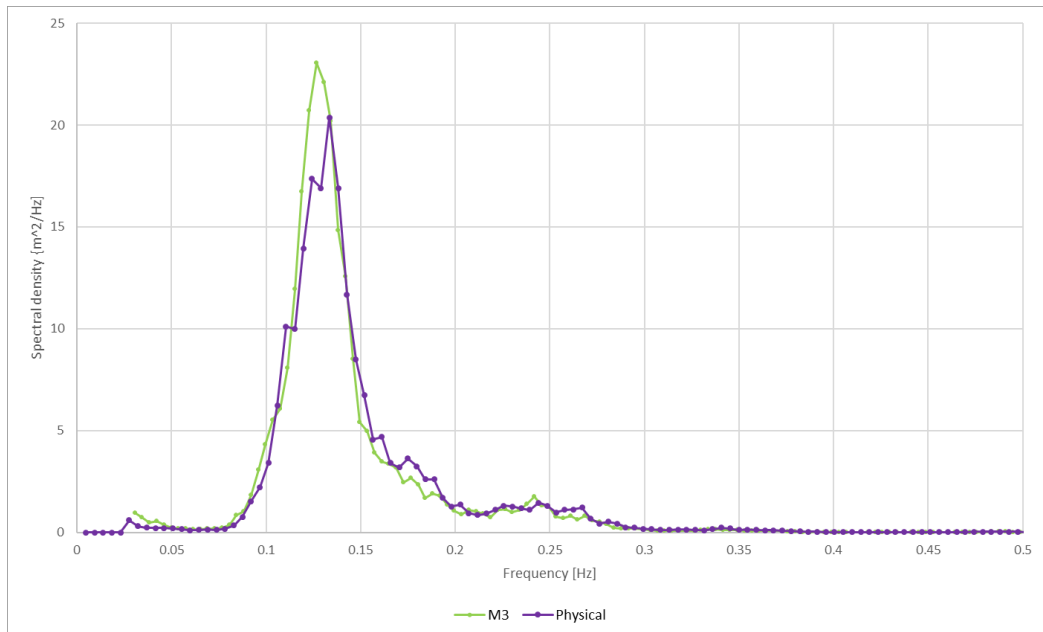


Fig. 4.76 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 86.

The behaviour of the waves in this model is something that has been repeated in all physical models, the waves decrease in intensity with increasing distance of the wave generator, Fig. 4.77.

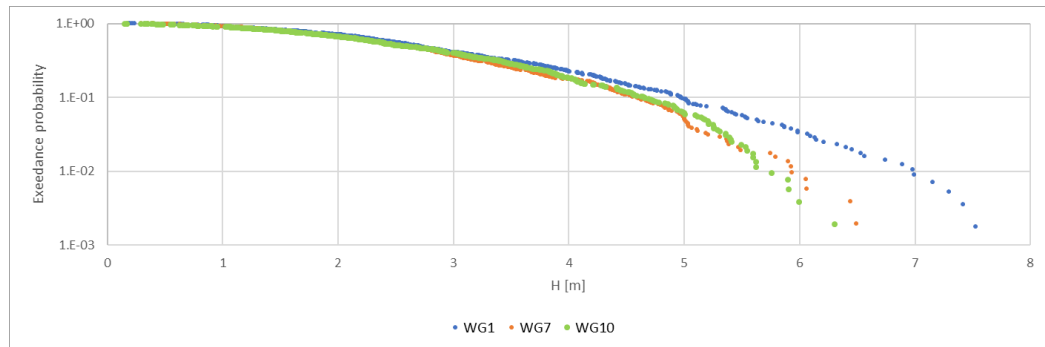


Fig. 4.77 - Wave height distribution of the physical model. Test no. 86.

The waves height in the numerical model had the same behaviour than in the other tests, since the first and last wave gauge have similar wave heights, Fig. 4.78.

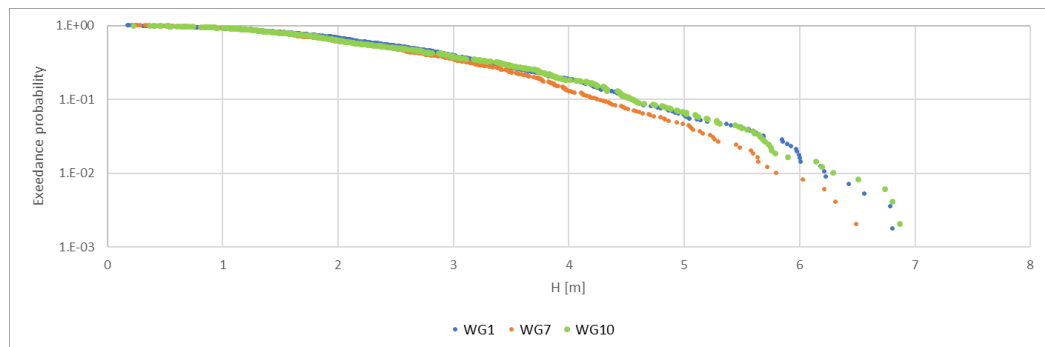


Fig. 4.78 - Wave height distribution of the M3 model. Test no. 86.

In the beginning of the flume, $x=2.00$ m, the waves smaller than 3.5 m had the same probability to occur in both models, but waves bigger than that were more likely to happen in the physical model, Fig. 4.79.

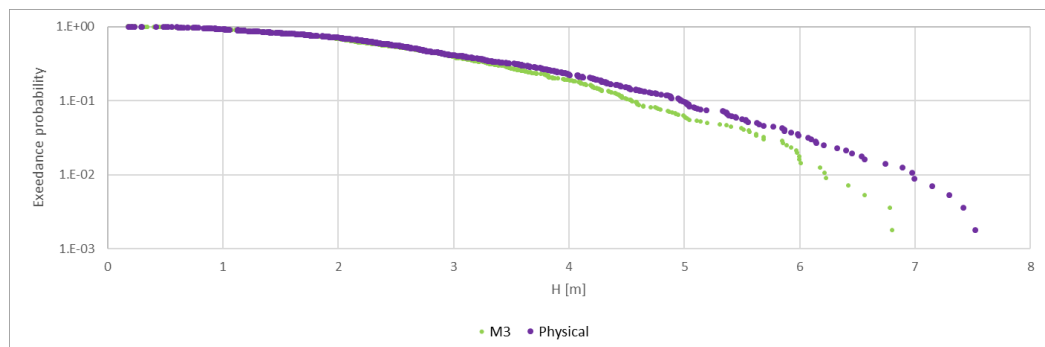


Fig. 4.79 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 86.

The waves smaller than 5 m had the same probability to occur but the scenario changes to waves bigger than that, where the M3 model has more chance of producing bigger waves, Fig. 4.80.

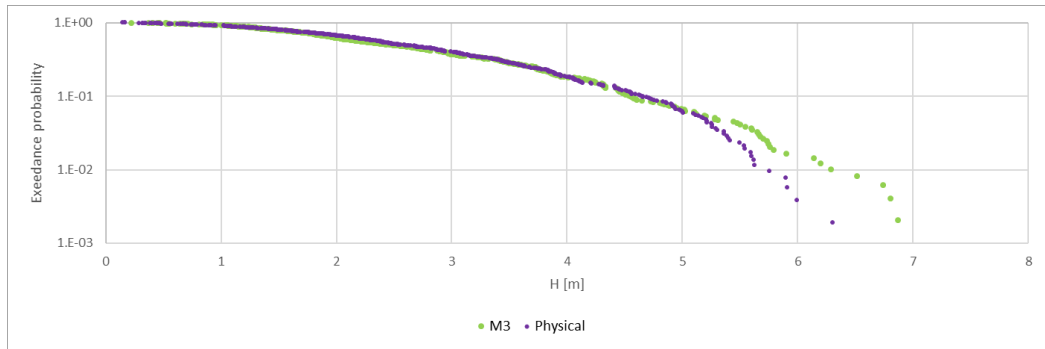


Fig. 4.80 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 86.

The number of discharges in the physical model was 76 while the numerical model presented 53, Fig. 4.81. The volume of each discharge is also to be noticed, the overtopping in the numerical model is more voluminous.

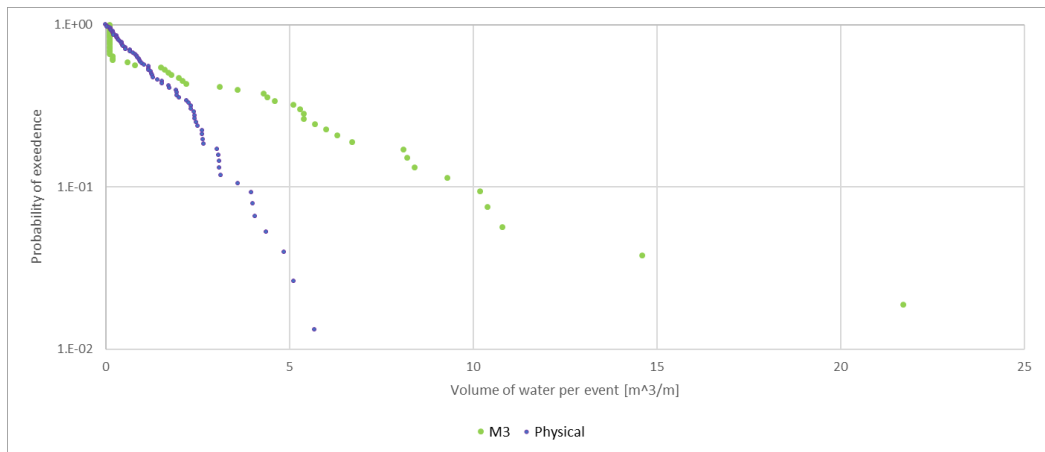


Fig. 4.81 - Event Volume distribution. Test no. 86.

The accumulated discharge is similar during almost all the tests but from 650 s the numerical model increased considerably the accumulated volume, Fig. 4.82. This is related to the fact that each discharge has a greater amount of water.

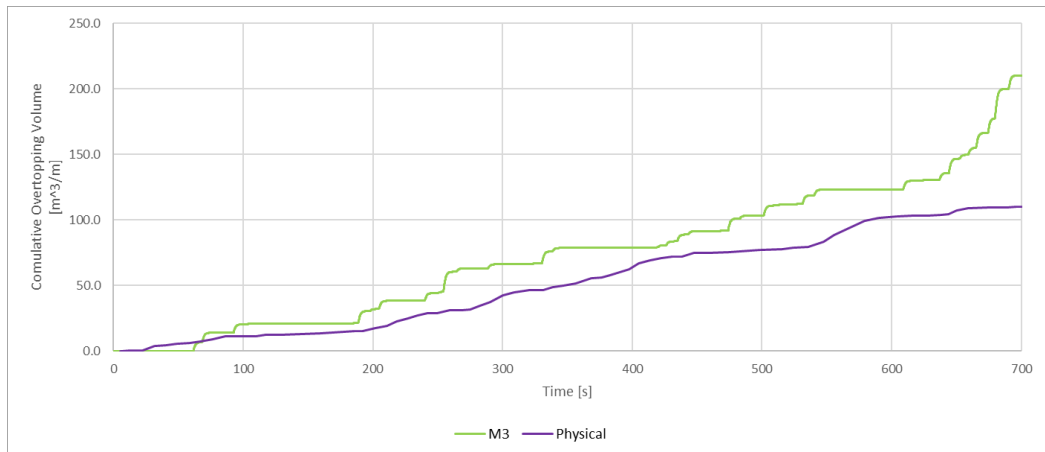


Fig. 4.82 - Time series of the accumulated overtopping volume. Test no. 86.

4.3.2.3 Test 87

In the last test of the H1-4 profile, the established wave conditions are $H_{m0}=4.70$ m and a $T_p=8.07$ s, wave design +10%. The incident spectra of the first group of wave gauges have some differences but the most perceptible is that the peak of the numerical model is bigger, Fig. 4.83. The H_{m0} of the biggest spectrum is 4.73 m while the other is 4.72 m.

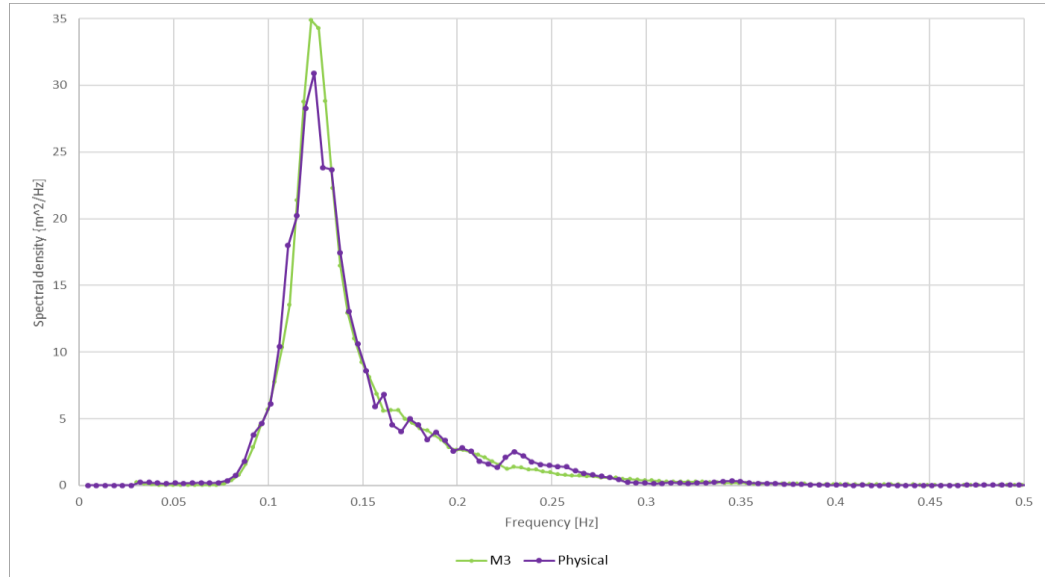


Fig. 4.83 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 87.

The incident spectra are similar, but the M3 model has an higher peak, Fig. 4.84. The values of the H_{m0} are 4.28 m in the numerical model and 4.18 m in the physical one.

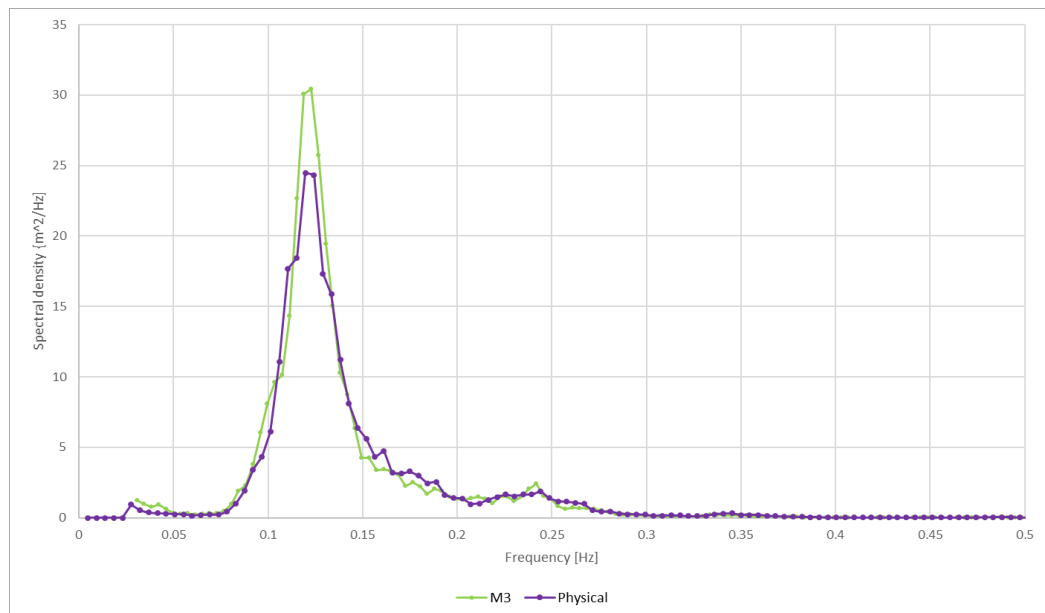


Fig. 4.84 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 87.

The performance of the waves in the physical model was more predictable, the wave gauges with bigger waves are close to the wave generator, Fig. 4.85.

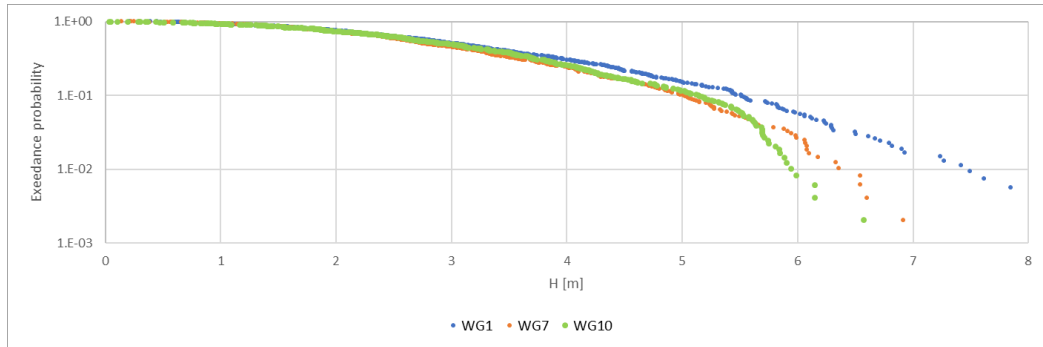


Fig. 4.85 - Wave height distribution of the physical model. Test no. 87.

In its turn, the waves heights values in the numerical simulations were identical to other numerical tests, the biggest waves occurred in the last wave gauge, WG10, Fig. 4.86.

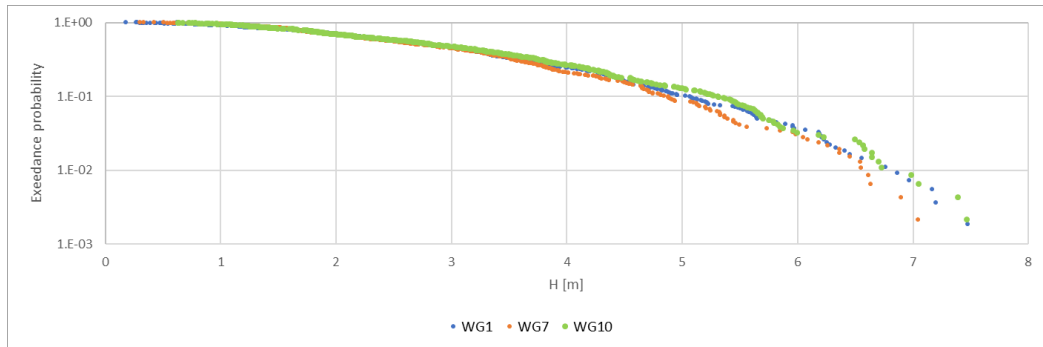


Fig. 4.86 - Wave height distribution of the M3 model. Test no. 87.

The measurements in the WG1 show that the physical model produces bigger waves than the numerical one, Fig. 4.87.

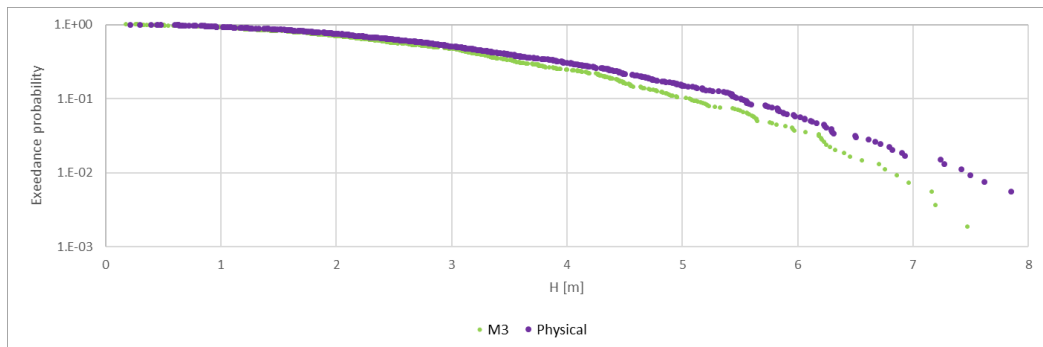


Fig. 4.87 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 87.

However, at the toe of the dike, the waves of the numerical model were bigger comparing with the ones of the physical model, Fig. 4.88.

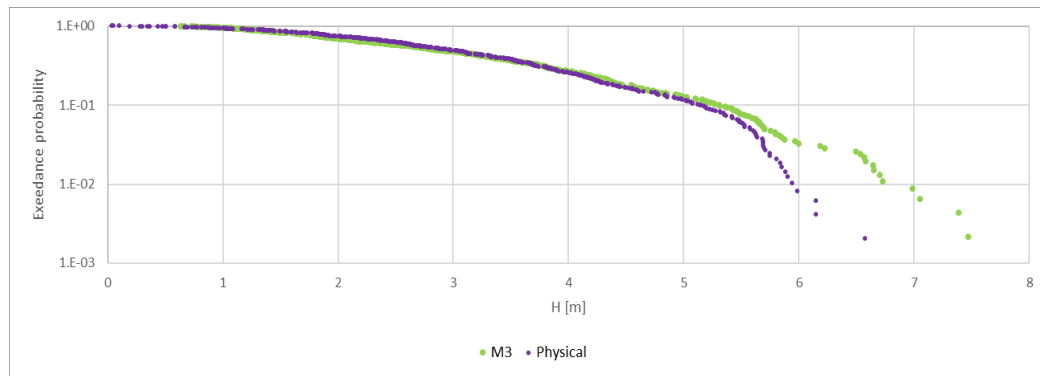


Fig. 4.88 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 87.

The volume discharge of both models is similar, but the numerical model has 16 overtopping events where the volume is bigger than 6 m^3 while the physical model only has two, Fig. 4.89. The physical model had 57 discharges while the numerical had 56, in the same time series.

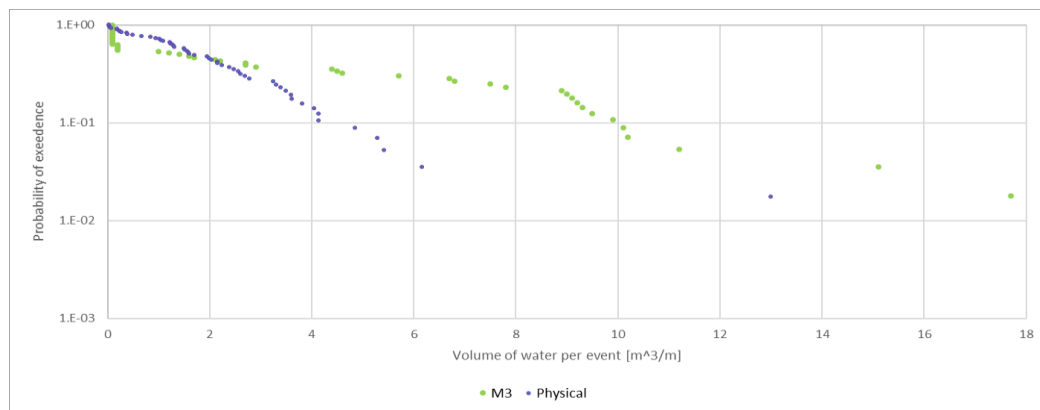


Fig. 4.89 - Event Volume distribution. Test no. 87.

The time series show that the models behaved identically even though the accumulated discharge in the numerical model is twice the physical one, Fig. 4.90. This fact can be explained due to the volume of each discharge.

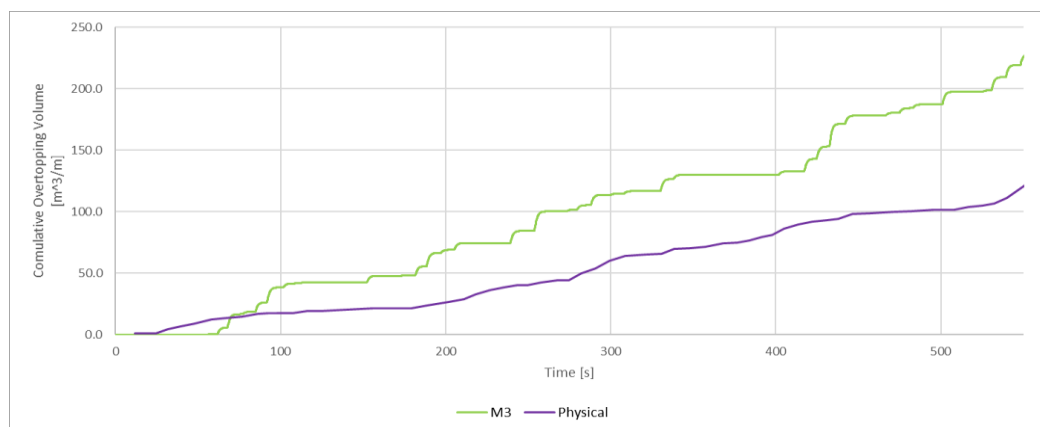


Fig. 4.90 - Time series of the accumulated overtopping volume. Test no. 87.

4.3.2.4 H1-4 Synthesis

Globally, the correlation of the M3 model with the physical model is quite accurate. The relative freeboard is almost the same and the difference in the relative overtopping rate is acceptable, Fig. 4.91.

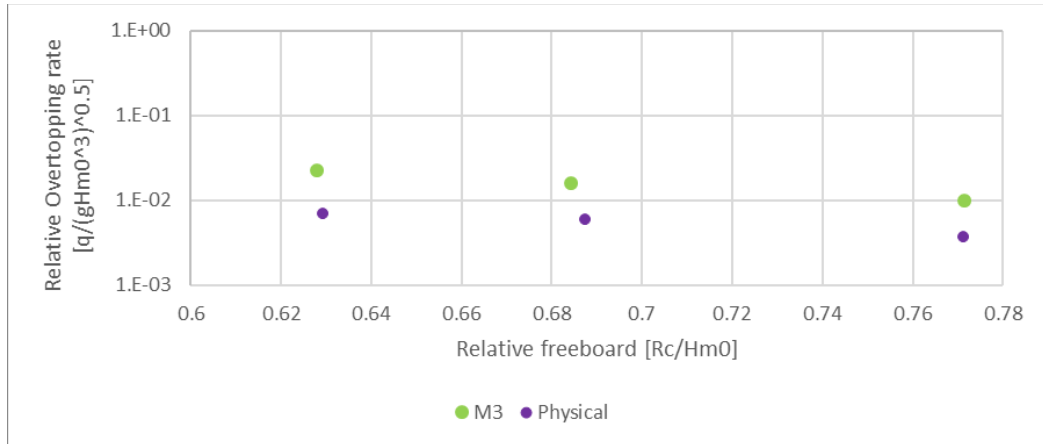


Fig. 4.91 - Comparison of the overtopping in the physical model and the M3 model, profile H1-4.

Summing up, the results show that the physical model lost more energy than the numerical model between the two arrays of wave gauges, and that the number of overtopping events is lower in M3 model, Table 4.4.

Table 4.4 - Synthesis of H1-4 tests.

Profile	Test	H _{m0} [m]				no. of overtopping events	
		WG1-WG5		WG6-WG10			
		M3	Physical	M3	Physical	M3	Physical
H1-4	85	3.85	3.85	3.45	3.47	97	110
	86	4.34	4.32	3.91	3.87	53	76
	87	4.73	4.72	4.28	4.18	56	57

4.3.3 RESULTS OF H4D-5

4.3.3.1 Test 316

The test number 316 is the first test of the profile H4d-5 and it was performed with the design wave - 10%, where $H_{m0}=3.84$ m and a $T_p=7.29$ s. The overlap of the spectra is almost total, and this is verified when confronted with the results of the H_{m0} , 3.70 m in the numerical model and 3.72 m in the physical model, Fig. 4.92.

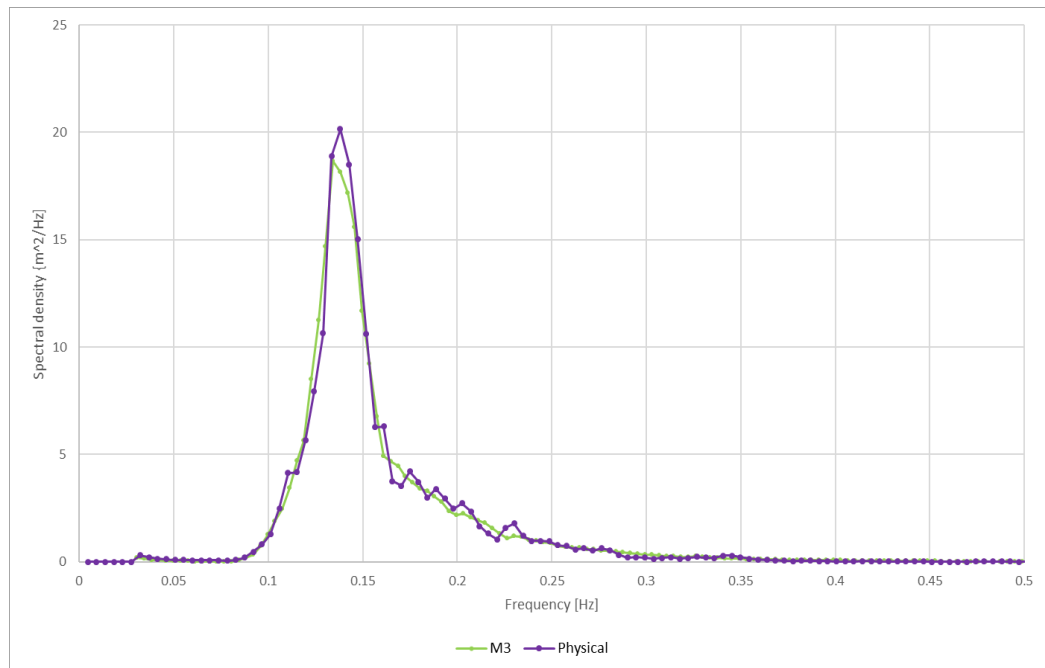


Fig. 4.92 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 316.

In the second group of wave gauges the difference is bigger. The waves of the numerical model lost more energy than in the physical model, Fig. 4.93. The H_{m0} of the greater spectrum is 3.61 m and the smaller is 3.39 m.

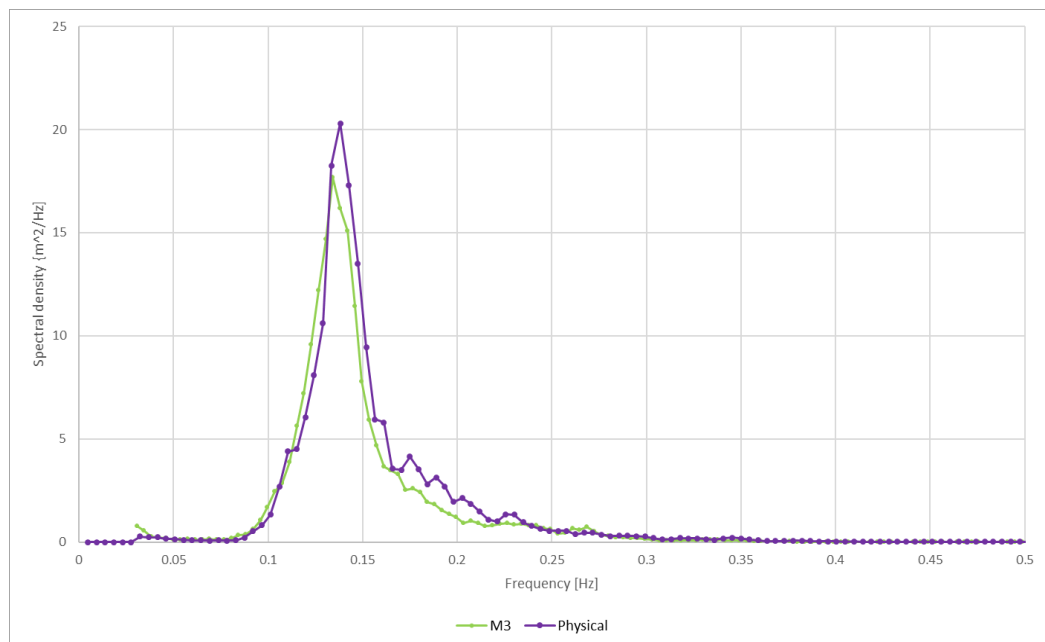


Fig. 4.93 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 316.

In Fig. 4.94, it is possible to see that the wave gauges near the wave maker measured bigger waves.

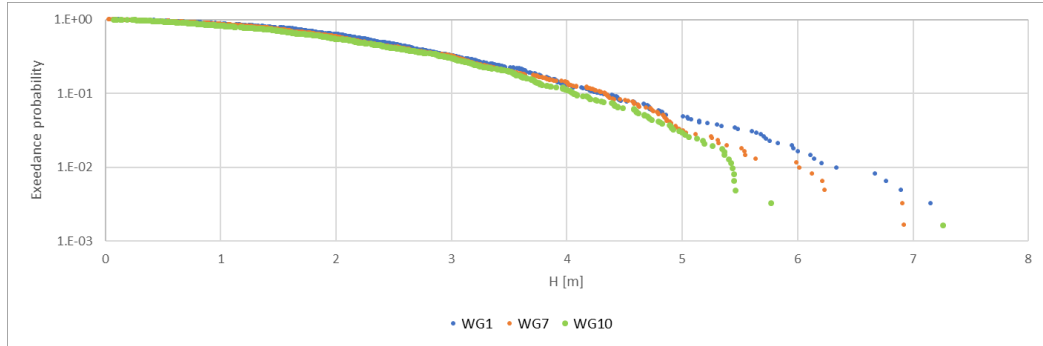


Fig. 4.94 - Wave height distribution of the physical model. Test no. 316.

The wave heights of this model are illogical, such as in previous cases where the largest waves are observed in the wave gauge furthest from the wave generator, Fig. 4.95.

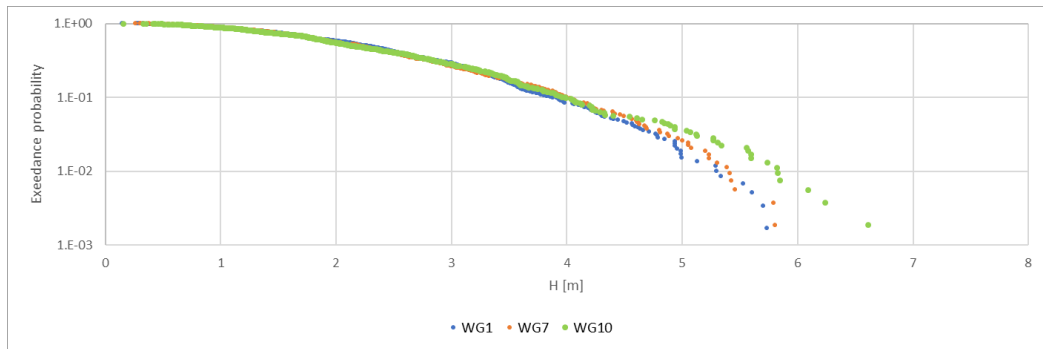


Fig. 4.95 - Wave height distribution of the M3 model. Test no. 316.

The probability of occurring waves smaller than 3 m is practically the same in both models, Fig. 4.96. However, in the physical model, there is a higher probability of occurring waves whose height is superior to 3 m.

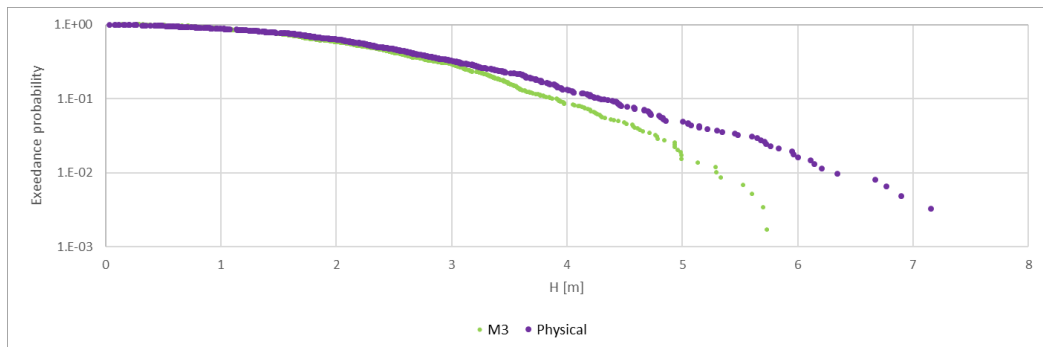


Fig. 4.96 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 316.

In the WG10, the position at the toe of the dike, waves smaller than 4.5 m have similar probability of occurrence, either in the physical model as in the numerical model. In its turn, in the numerical model it is found a higher probability of measuring waves superior than 4.5 m, Fig. 4.97.

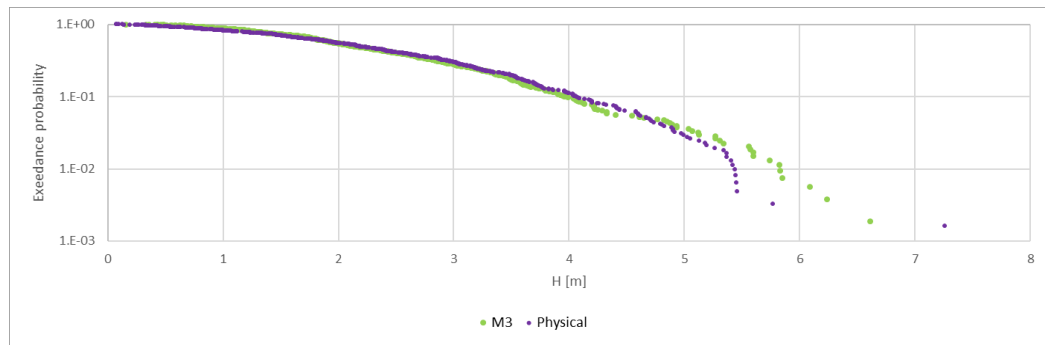


Fig. 4.97 - Wave height distribution at WG10 (x=16.60 m). Test no. 316.

Comparing the discharge of the models, the physical model had 100 overtopping events while the numerical model had 121. Other concerning aspect is the volume of each event. The largest volume discharged by the physical model was 1.6 m^3 while the largest volume of the numerical model was more than 13 times this value, 22 m^3 , Fig. 4.98.

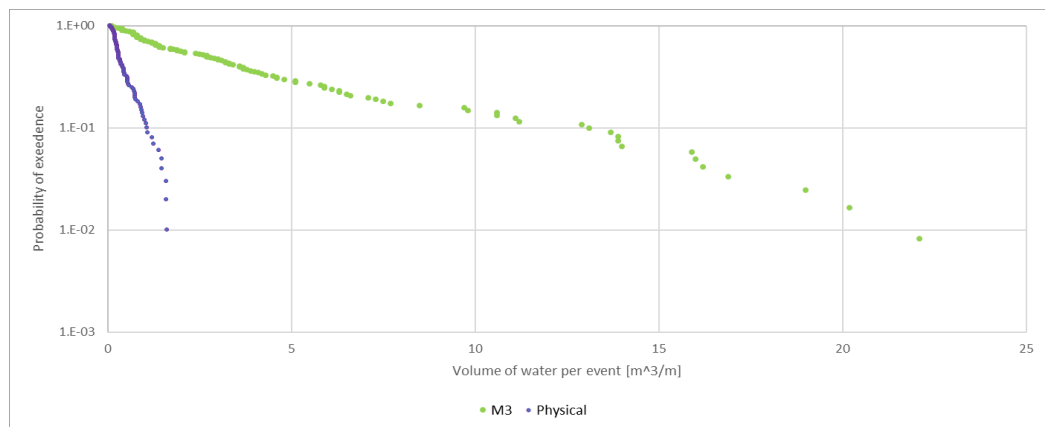


Fig. 4.98 - Event volume distribution. Test no. 316.

In this time series it is possible to see a considerable difference between the two models, Fig. 4.99. This contrast is due to the significant difference in each overtopping event. In fact, the volume of water that goes over the crest of the dike in the numerical model is far superior.

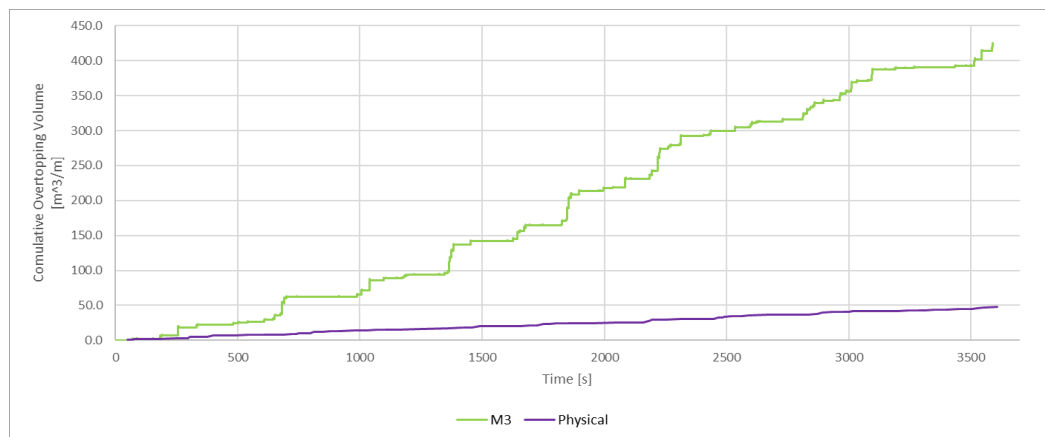


Fig. 4.99 - Time series of the accumulated overtopping volume. Test no. 316.

4.3.3.2 Test 317

In the second test of the profile H4d-5, test number 317, the characteristics of the design wave were $H_{m0}=4.27$ m and $T_p=7.69$ s. The spectra overlay is significant but the H_{m0} is slightly different, Fig. 4.100. The H_{m0} of the numerical model is 4.03 m and in the physical model is 4.05 m.

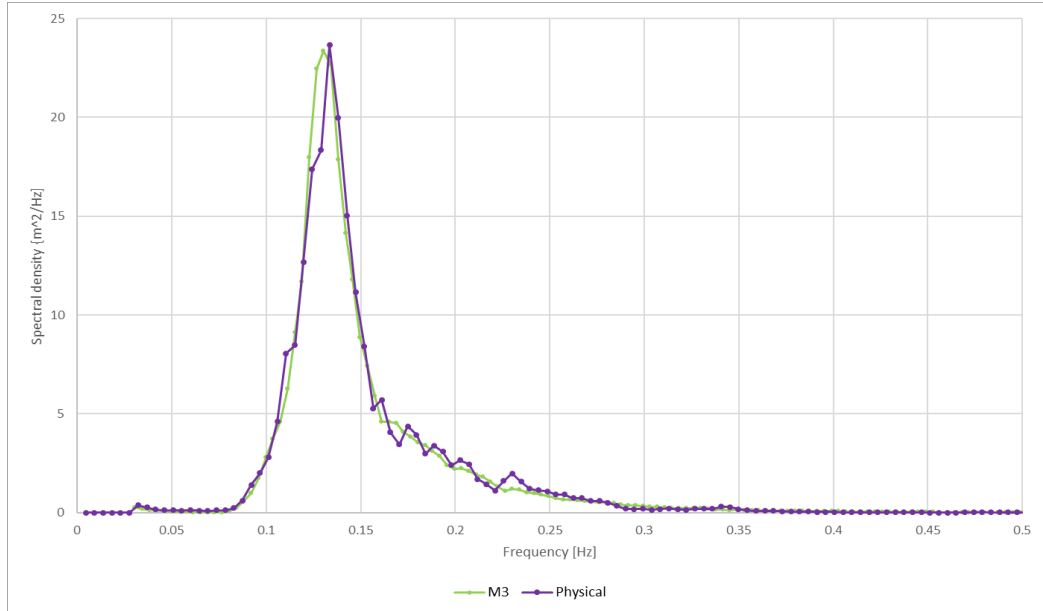


Fig. 4.100 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 317.

The H_{m0} measured in the second array, WG6 to WG10, was 3.75 m in the numerical model and 3.93 m in the physical model. This difference can be noticed in Fig. 4.101, since the incident spectrum of the physical model is bigger, resulting in a bigger H_{m0} .

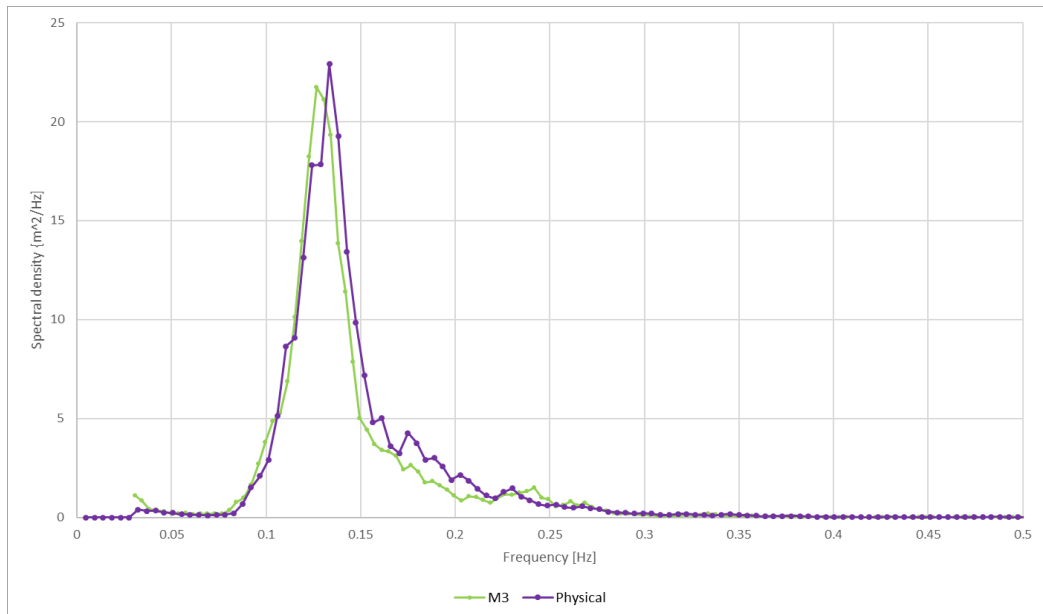


Fig. 4.101 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 317.

In the physical model occurs the same as in the previous cases. The wave gauges closest to the generator have higher wave heights, Fig. 4.102.

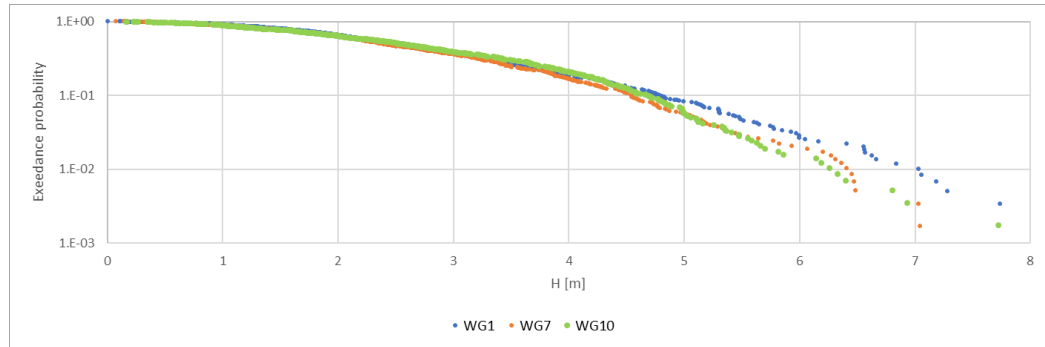


Fig. 4.102 - Wave height distribution of the physical model. Test no. 317.

The waves generated in M3 had some curious values. In the WG10 there is a higher probability of occurring waves bigger than 5 m than the in the WG1 and WG7, Fig. 4.103. That is an unexpected result once all wave gauges are in the flat part of the flume and the WG1 and WG7 are near to the wavemaker.

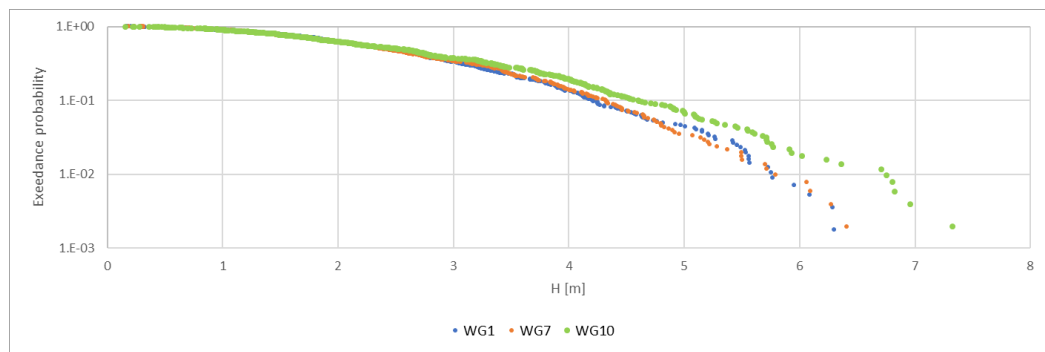


Fig. 4.103 - Wave height distribution of the M3 model. Test no. 317.

In the WG1, the probability of occurring waves smaller than 4 m is practically the same in both models, Fig. 4.104. However, in the physical model, there is a higher probability of occurring waves whose height is superior to 4 m.

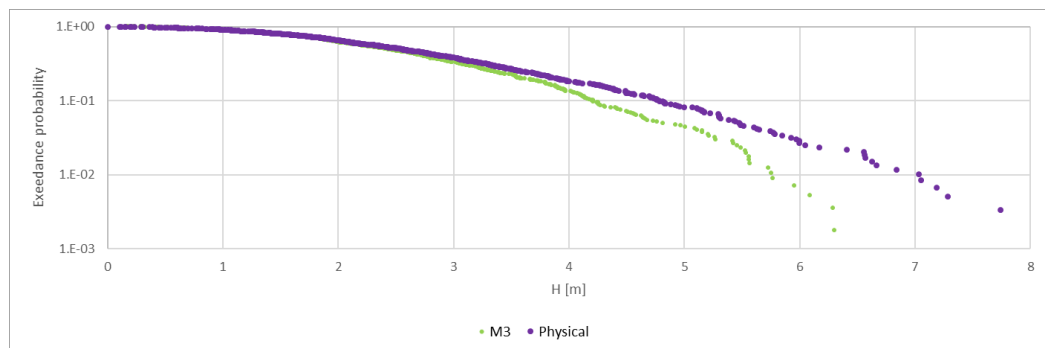


Fig. 4.104 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 317.

In the WG10, waves smaller than 5 m have similar probability to happen, either in the physical as in the numerical model. In its turn, in the numerical model it is found a higher probability of measuring waves superior than 5 m, Fig. 4.105.

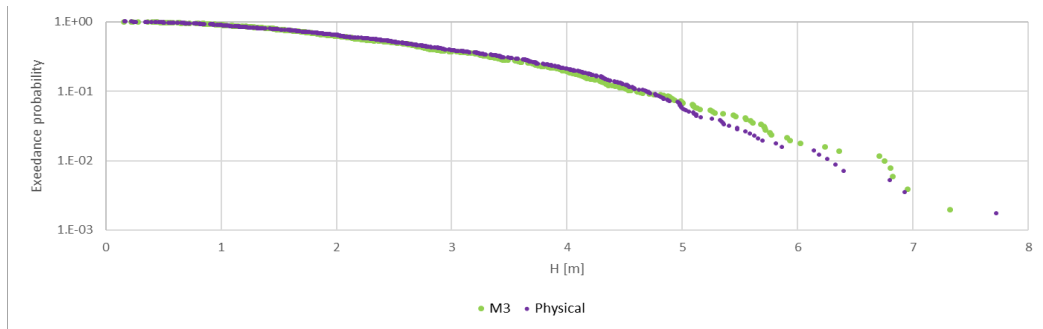


Fig. 4.105 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 317.

In Fig. 4.106, it is easily understood that each discharge event has a larger amount of water in the numerical model than in the physical. Other aspect to take into account is that the physical model had 133 overtopping events while the numerical model had 163.

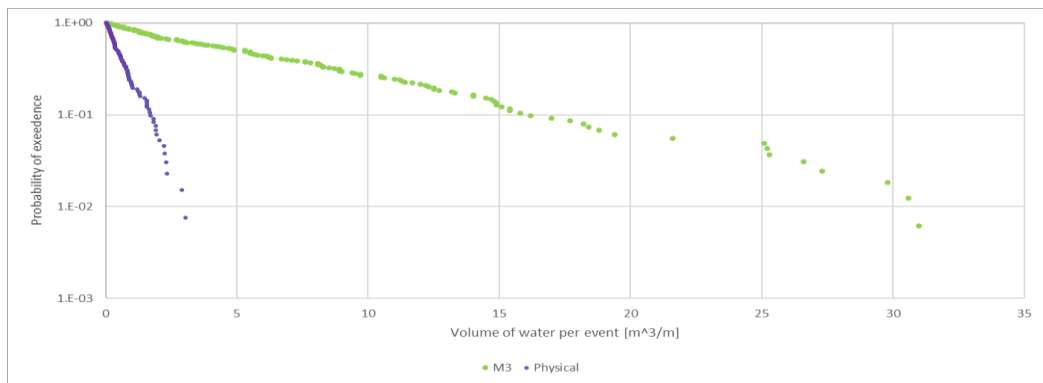


Fig. 4.106 - Event Volume distribution. Test no. 317.

Here it can be observed the accumulated discharge over the simulation time. The Fig. 4.107 shows an immense difference between both models in the final accumulated overtopping volume. This disparity is due to a major factor: the volume of water of each overtopping event.

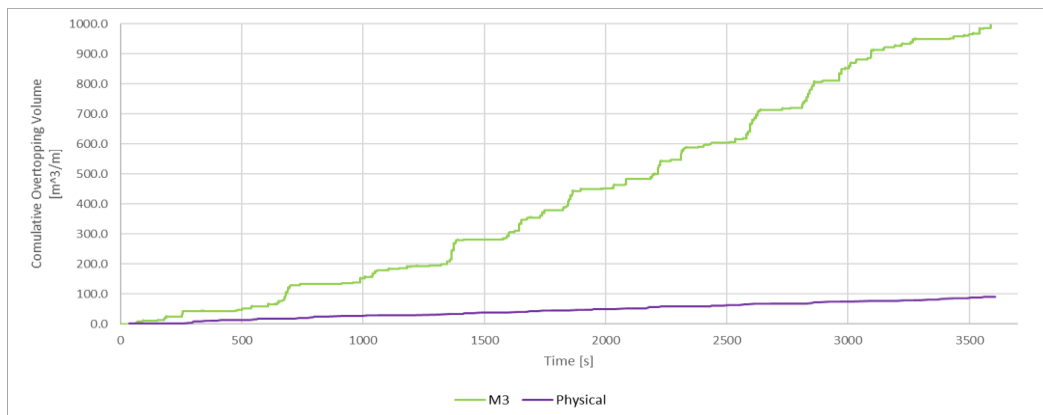


Fig. 4.107 - Time series of the accumulated overtopping volume. Test no. 317.

4.3.3.3 Test 318

The last test of all, test number 318, was set with design wave +10%, with an $H_{m0}=4.70$ m and a $T_p=8.07$ s. Although the spectrum of the numerical model has a higher peak, Fig. 4.108, the values of the H_{m0} are both 4.72 m.

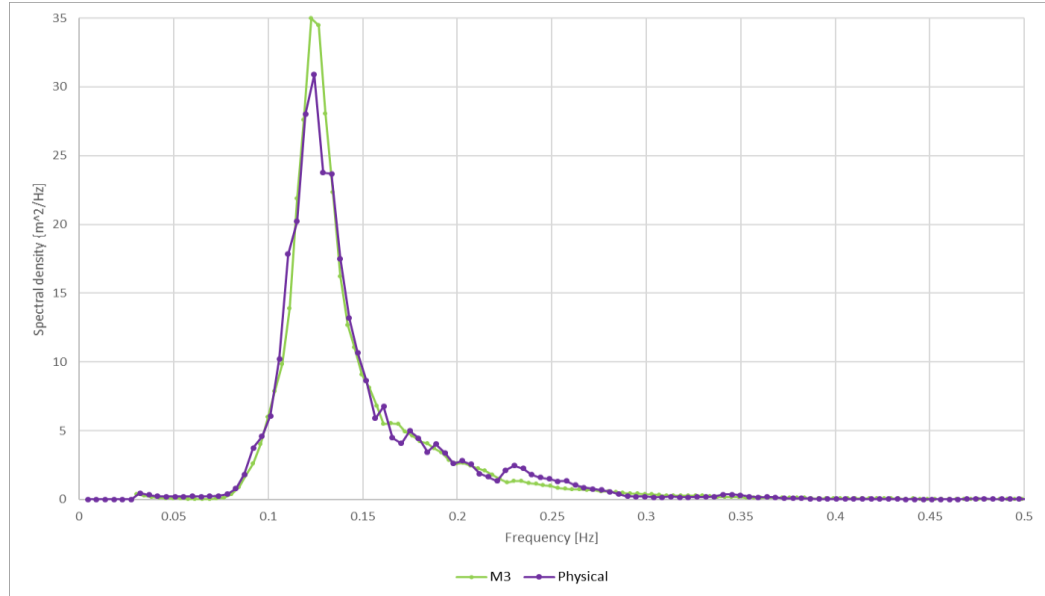


Fig. 4.108 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 318.

The second array of wave gauges show that the physical spectrum is slightly bigger than the spectrum of the M3 model, Fig. 4.109. The H_{m0} of the biggest spectrum is 4.55 m, and the H_{m0} of the M3 is 4.34 m.

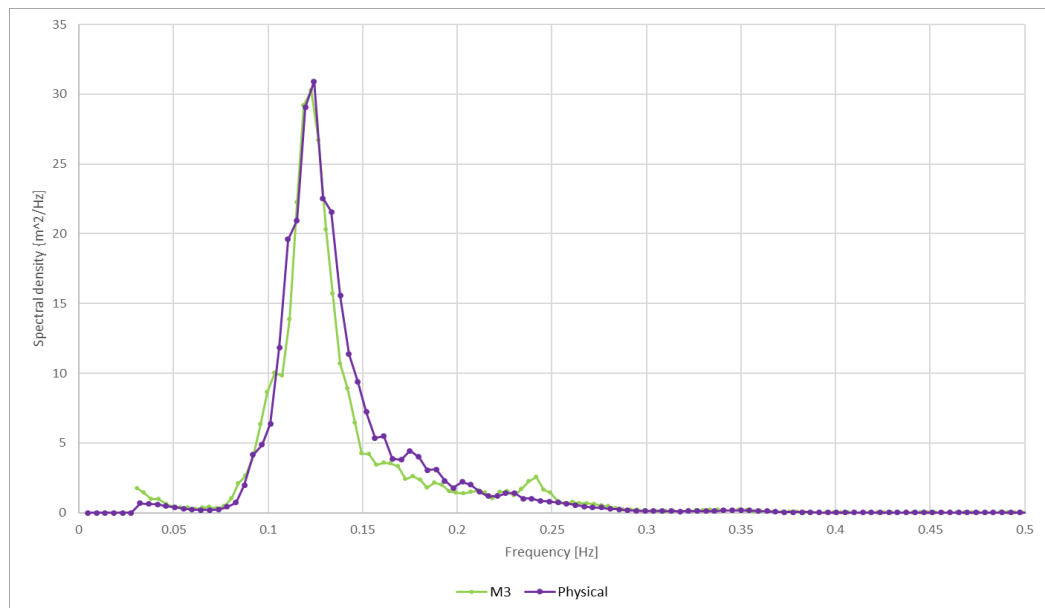


Fig. 4.109 - Incident spectrum of the M3 model (green) and the physical model (purple), from WG6 to WG10. Test no. 318.

In the physical model, the wave gauges that are close to the wave generator are bigger than the most distant ones, Fig. 4.110. Since the WG7 and WG10 gauges are close to each other and both are in the flat zone of the flume, the results are identical. Although those of WG10 are smaller because in the distance between wave gauges the waves loose energy.

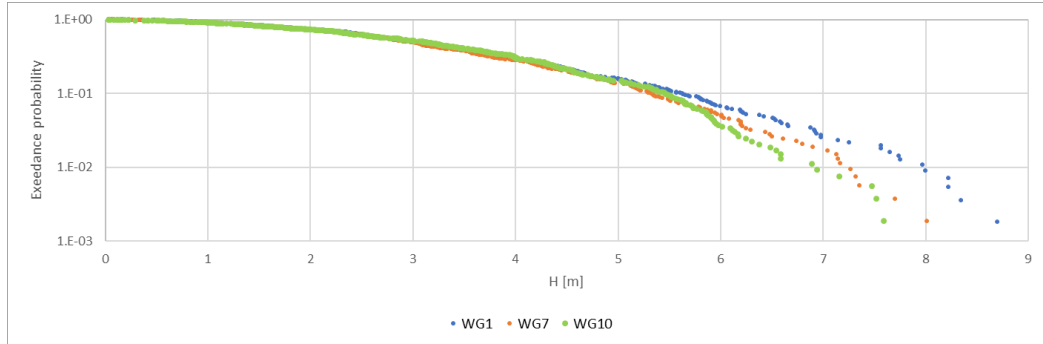


Fig. 4.110 - Wave height distribution of the physical model. Test no. 318.

In its turn, the wave heights values in the numerical simulations were identical to other numerical tests: the biggest waves were measured in the last wave gauge, Fig. 4.111.

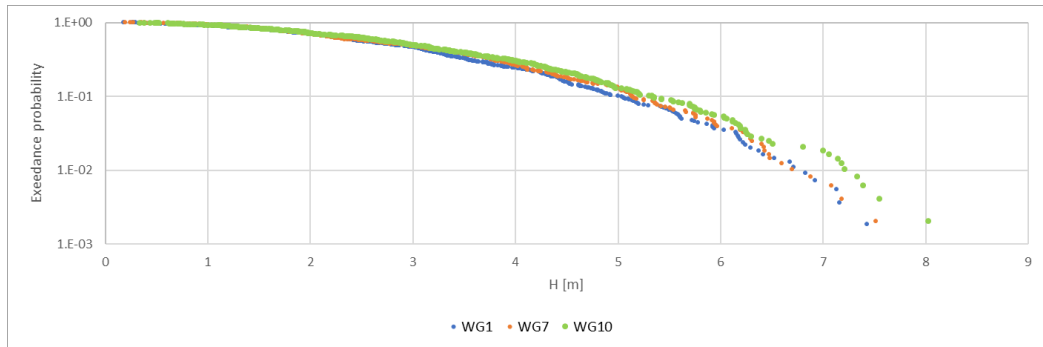


Fig. 4.111 - Wave height distribution of the M3 model. Test no. 318.

In the WG1, the probability of occurring waves smaller than 4 m is practically the same in both models, Fig. 4.112. However, in the physical model, there is a higher probability of occurring waves whose height is superior to 4 m.

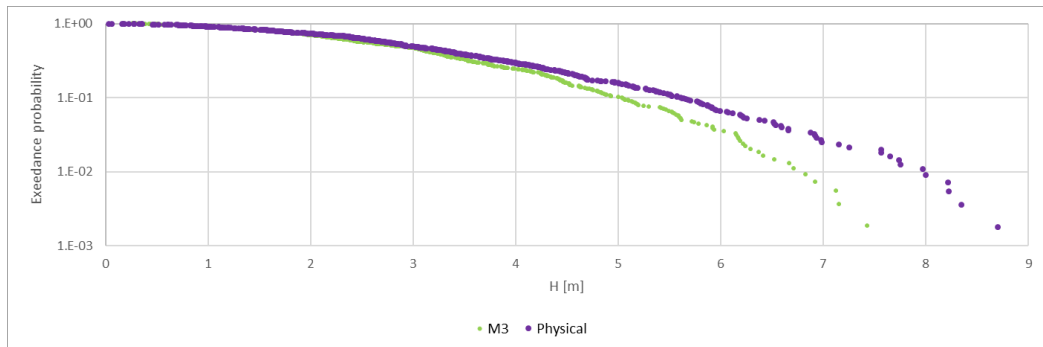


Fig. 4.112 - Wave height distribution at WG1 ($x=2.00$ m). Test no. 318.

The situation reverses when the WG10 is analysed. After a set of wave heights where the probability of occurring is identical, it is found that the wave heights in the numerical model are bigger and more likely to occur, Fig. 4.113.

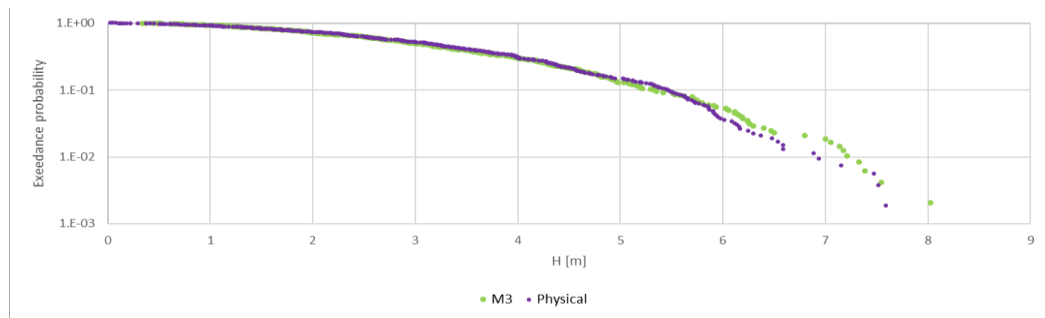


Fig. 4.113 - Wave height distribution at WG10 ($x=16.60$ m). Test no. 318.

Observing the graphic, it is easily understood that each discharge event has a larger amount of water in the numerical model than in the physical, Fig. 4.114. In its turn, the number of events is similar, the physical model had 129 overtopping events while the numerical model had 124.

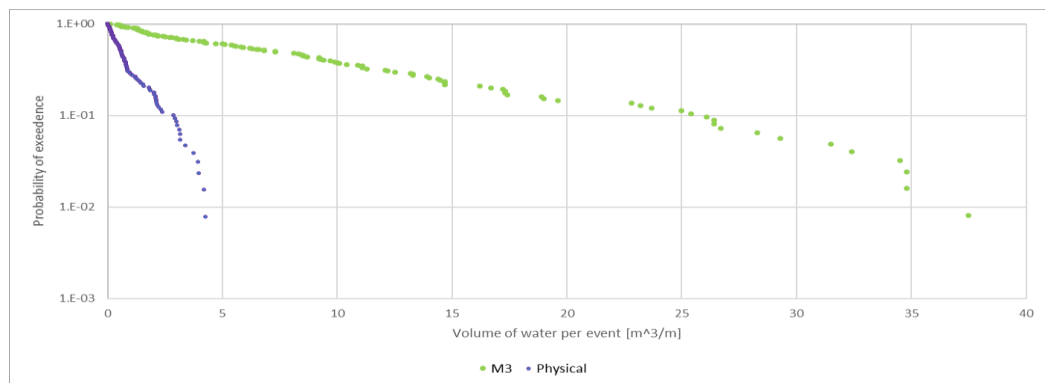


Fig. 4.114 - Event Volume distribution. Test no. 318.

In the time series of the accumulated discharge, the numerical model had a larger volume of water passing over than the physical model, Fig. 4.115. This is due to three factors: bigger waves near the dike, more overtopping events and the bigger volume of water of each event.

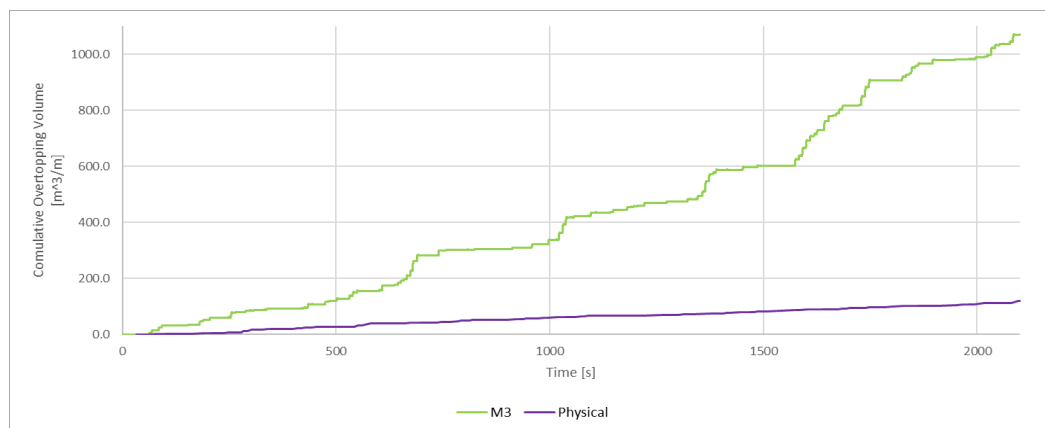


Fig. 4.115 - Time series of the accumulated overtopping volume. Test no. 318.

4.3.3.4 H4d-5 Synthesis

In general, the correlation of the numerical model with the physical model is good. The relative freeboard is almost the same and the difference in the relative overtopping rate is acceptable, Fig. 4.116.

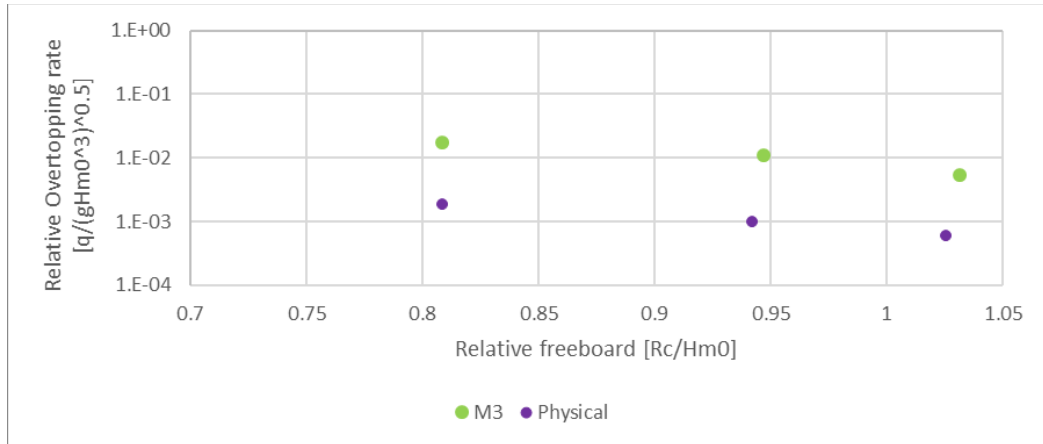


Fig. 4.116 - Comparison of the overtopping in the physical model and the M3 model, profile H4d-5.

Summing up, the results show that the numerical model lost more energy than the physical between the two arrays of wave gauges, and that the number of overtopping events is higher in the M3 model, except in the test 318, Table 4.5.

Table 4.5 - Synthesis of H4d-5 tests.

Profile	Test	H _{m0} [m]				no. of overtopping events	
		WG1-WG5		WG6-WG10			
		M3	Physical	M3	Physical	M3	Physical
H4d-5	316	3.7	3.72	3.39	3.61	121	100
	317	4.03	4.05	3.75	3.93	163	133
	318	4.72	4.72	4.34	4.55	124	129

5

CONCLUSIONS AND FUTURE DEVELOPMENTS

5.1 CONCLUSIONS

The objective initially defined in this work was to carry out a software sensitivity analysis, comparing the data collected in the wave flume for the *Afsluitdijk* case, with the results obtained with the MIKE 3 Wave FM. Although the relative overtopping rate had values that are in the same range of the results of other authors' tests, the results obtained with the software indicate that the user do not have fully control over the numerical model. The increasing wave heights along the domain and the volume of each overtopping event were aspects that were worse than expected.

The results presented above show that the relationship between the results of the M3 model and the physical model ones is good. However, there are some values that deserve some attention. For example, the study of wave heights: in the numerical model wave height grows when moving away from the wave generator, which is the opposite trend (and expected) observed in the physical model, in which the wave heights decrease. This could be an effect of reflections and/or standing waves in the physical model. Nevertheless, with the data provided in the physical model, it was not possible to confirm whether this was the case or not. Regardless to the wave heights in the second array of wave gauges and the incident spectrum, the comparison is not correct. The incident spectrum only contains the incident energy, and the analyses made with the WS Reflection Analyses removed the reflected energy. The wave height distribution measured in the simulations, is the sum of both energy spectra, the incident and the reflected. So, that is why is not unrealistic when exists a fairly good match between the incident spectra of M3 and the one of the physical model, while the individual waves are bigger in M3 compared to the physical model tests.

Another aspect is the volume of water of each overtopping event. In all tests, the amount of water that passes over the crest of the dike, in the numerical model, is far superior than in the physical model. The reason for that is the incorrect definition of some parameters in the numerical model, specially the Flood and Dry. This conclusion is due to the fact that some simulations were made with the goal of achieving a very close result of the accumulated discharge (Annex B), disregarding the practical application of the model. In Annex C, is possible to see a comparison between the numerical models, the adopted one and the one with a Flood and Dry that grants an excellent overtopping ratio. But the behavior of the waves is physically not possible. The discarded model was set with the exact characteristics of the adopted model, except for the Flood and Dry parameters. The physical problem in that model was that the waves ended in a vertical wall, something that should not happen.

Still considering the volume of water of each event, it is necessary to refer to cases where the overtopping was different from the theoretical value, in the profiles H1-4 and H4d-5. In the profile H1-4 the ribs were set, as explained in the porosity chapter, as a continuous rib, but in the physical model the ribs had

gaps that allow the water to run back. Another feature that would also alter the volume of water is the angle of the ribs in relation to the slope. It is not possible to state in what configuration would pass a larger volume of water, but the question is that it would not be the same. In the profile H4d-5, the angle between the porosity zone and the slope is also responsible for some loss of characteristics. In both cases, the porosity of the ribs, in H1-4, and the bullnose, in H4d-5, was a solution that is not the same as an impermeable object, although this has been attempted with the unrealistic values of turbulence resistance.

In the course of this work, during the simulations of the models, some bugs were identified in the M3 software. The first bug is related with the Message Passing Interface (MPI) domain decomposition when the boundary between two subdomains is in a dry area. This bug was discovered because the division between two subdomains coincided with the output line of the discharge and the results were different, when the input was exactly the same. The other bug is related with the porosity zones in 3D map. The new version of the software is already available, but the present work was performed with the previous one.

Concluding, although some results demonstrate that the models presented in this work were not accurate, the software is ready for use and the results are reliable.

5.2 FUTURE DEVELOPMENTS

Based on the work accomplished, the observations made and the results obtained, some recommendations are presented for future developments of similar numerical models. These recommendations are related with aspects that were not considered or optimized during the study and options that were not adequate during the numerical modelling process. For futures studies, it is considered necessary to examine in more detail the following aspects:

- Calibrate the Flood and Dry parameter with values that represent better the overtopping phenomenon considering the volume of water of each event;
- Increase the resolution of the models in both axis, x and y. This could represent better the wave propagation and the overtopping events;
- Replicate in the numerical model, physical models tests where were gather data of the reflection and standing waves, to be possible to understand and quantify the wave height propagation;
- Define the ribs and the bullnose was part of the bathymetry instead of setting it as a porous zone. If the objects have an angle, as in the cases of H1-4 and H4d-5, impossible to describe, is better to smooth the angle to values that are possible in the discretization of the bathymetry;
- Create a tool in the MIKE software to calculate individual overtopping events. This can prevent errors when determining results;
- Use the software tool Neural Network (NN) Overtopping 2 to create a similar profile and compare the results with the numerical model. Only the profile P-1 could be analysed in NN Overtopping 2 since the profile H4d-5 and H1-4 have a complex bathymetry that is impossible to reproduce (ribs and the bullnose).

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ANNEXES

ANNEX A

Summary of the input information in the software

// Created : 2018-12-24 14:57:23

// Executable : C:\Program Files (x86)\DHI\2019\bin\x64\femenginehd.exe

// Version : 17.1.0.12331

=====Computing Environment=====

Local machine name : NUSM-PC1.dhi.dk

Number of subdomains : 4

Local subdomain id : 0

Specified threads per subdomain : 4

=====

Creator:

File : C:\Users\nusm\Desktop\Afsluitdijk CASE\P-1\P-1.dfsu

Title : porosity

Start date Start time Sets Interval Unit

2018-01-01 00:00:00 1 1 second

Test 5

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	58842	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-0.845399	0.1536	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	28000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	58842	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-0.845399	0.1436	meter
Porosity zones	Grid Codes	0	1	Integer

Section: ZONE_1 -----

porosity : 0.4

grain_diameter : 0.05

alpha : 1000

beta : 2.8

oscillating_period : 8

gamma : 0.34

Section: SPONGE_LAYER -----
 type : 1
Section: SPONGE_ZONES_BOUNDARY -----
 number_of_zones : 1
Section: ZONE_1 -----
 include : 1
 boundary_code : 4
 width : 5
 approach : 3 (default)
 characteristic_resolution : 0.2
 a : 7.5
 r : 0.725
 s_reference : 0
 u_reference : 0 (default)
 v_reference : 0 (default)
Section: BODY_FORCE -----
Section: INTERNAL_CONDITIONS -----
 number_of_zones : 1
Section: ZONE_1 -----
 include : 1
Section: RELAXATION_DATA -----
 coordinate_type : NON-UTM
 number_of_points : 2
Section: POINT_1 -----
 x : 4.6
 y : 1
Section: POINT_2 -----
 x : 4.6
 y : 0
 width_ramp_up_zone : 4.5
 width_pressure_zone : 4.5 (default)
 type_of_ramp_up_space : 1
 factor_ramp_up : 3.5
 type_of_ramp_up_time : 0
 type_of_ramp_down_time : 0
Section: WAVE_DATA -----
 type_of_waves : 2
 water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.253128

peak_wave_period : 1.81

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 1261

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.253128

Peak wave period Tp (s) : 1.809009

Mean wave period T01 (s) : 1.517339

Mean wave period T02 (s) : 1.430642

Mean wave direction MWD (deg): 0

=====

Test 16

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	58842	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter

Z-coord	Item geometry 3-dime	-0.845399	0.1536 meter
Code	Grid Codes	2	5 Integer
Element id	Grid Codes	1	28000 Integer
Element type	Grid Codes	33	33 Integer
No of nodes	Grid Codes	8	8 Integer
Connectivity	Grid Codes	1	58842 Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-0.845399	0.1436	meter
Porosity zones	Grid Codes	0	1	Integer

 Section: ZONE_1 -----

porosity : 0.4
 grain_diameter : 0.05
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1
 boundary_code : 4
 width : 5
 approach : 3 (default)
 characteristic_resolution : 0.2
 a : 7.5
 r : 0.725
 s_reference : 0
 u_reference : 0 (default)
 v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.286589

peak_wave_period : 1.9

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 848

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.286589

Peak wave period Tp (s) : 1.893376

Mean wave period T01 (s) : 1.591816

Mean wave period T02 (s) : 1.49949

Mean wave direction MWD (deg): 0

=====

Test 36

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	58842	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-0.845399	0.1536	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	28000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	58842	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-0.845399	0.1436	meter
Porosity zones	Grid Codes	0	1	Integer

Section: ZONE_1 -----

porosity : 0.4

grain_diameter : 0.05

alpha : 1000

beta : 2.8

oscillating_period : 8

gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

boundary_code : 4

width : 5

approach : 3 (default)

characteristic_resolution : 0.2

a : 7.5

r : 0.725

s_reference : 0

u_reference : 0 (default)

v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.235445

peak_wave_period : 1.72

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 2541

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.235445

Peak wave period Tp (s) : 1.719674

Mean wave period T01 (s) : 1.442952

Mean wave period T02 (s) : 1.361909

Mean wave direction MWD (deg): 0

=====

Test 85

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-0.845399	0.170599	meter
Code	Grid Codes	2	5	Integer

Element id	Grid Codes	1	56000 Integer
Element type	Grid Codes	33	33 Integer
No of nodes	Grid Codes	8	8 Integer
Connectivity	Grid Codes	1	114882 Integer
Dynamic items:			
Item name	Item type	Minimum	Maximum Unit
Z coordinate	Item geometry 3-dime	-0.845399	0.1606 meter
Porosity zones	Grid Codes	0	3 Integer

Section: ZONE_1 -----

porosity : 0.4
 grain_diameter : 0.026
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_2 -----

porosity : 0.4
 grain_diameter : 0.05
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_3 -----

porosity : 0.2
 grain_diameter : 0.05
 alpha : 1000
 beta : 580000000
 oscillating_period : 8
 gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1
 boundary_code : 4
 width : 5

approach : 3 (default)
characteristic_resolution : 0.2
a : 7.5
r : 0.725
s_reference : 0
u_reference : 0 (default)
v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.21919

peak_wave_period : 1.72

gamma : 3.3
sigma_A : 0.07
sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0
type_of_angle : 2
main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)
minimum_frequency : 0.0005
maximum_frequency : 3
number_of_components : 2541
minimum_amplitude : 1.E-08 (default)
seed : 100
rescale : 1
wave_theory : 1 (default)
second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.21919
Peak wave period Tp (s) : 1.719674
Mean wave period T01 (s) : 1.442952
Mean wave period T02 (s) : 1.361909
Mean wave direction MWD (deg): 0

=====

Test 86

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-0.845399	0.170599	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	56000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	114882	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
-----------	-----------	---------	---------	------

Z coordinate Item geometry 3-dime -0.845399 0.1606 meter
 Porosity zones Grid Codes 0 3 Integer

 Section: ZONE_1 -----

porosity : 0.4
 grain_diameter : 0.026
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_2 -----

porosity : 0.4
 grain_diameter : 0.05
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_3 -----

porosity : 0.2
 grain_diameter : 0.05
 alpha : 1000
 beta : 580000000
 oscillating_period : 8
 gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1
 boundary_code : 4
 width : 5
 approach : 3 (default)
 characteristic_resolution : 0.2
 a : 7.5
 r : 0.725
 s_reference : 0
 u_reference : 0 (default)

```
v_reference : 0 (default)
Section: BODY_FORCE -----
Section: INTERNAL_CONDITIONS -----
    number_of_zones : 1
Section: ZONE_1 -----
    include : 1
Section: RELAXATION_DATA -----
    coordinate_type : NON-UTM
    number_of_points : 2
Section: POINT_1 -----
    x : 4.6
    y : 1
Section: POINT_2 -----
    x : 4.6
    y : 0
    width_ramp_up_zone : 4.5
    width_pressure_zone : 4.5 (default)
    type_of_ramp_up_space : 1
    factor_ramp_up : 3.5
    type_of_ramp_up_time : 0
    type_of_ramp_down_time : 0
Section: WAVE_DATA -----
    type_of_waves : 2
    water_depth : 0.845
    s_reference : 0
Section: IRREGULAR_WAVES -----
Section: FREQUENCY_SPECTRUM -----
    type_of_frequency_spectrum : 2
Section: JONSWAP -----
    type_of_JONSWAP : 1
    significant_wave_height : 0.24642
    peak_wave_period : 1.81
    gamma : 3.3
    sigma_A : 0.07
    sigma_B : 0.09
Section: DIRECTIONAL_DISTRIBUTION -----
    type_of_directional_distribution : 0
    type_of_angle : 2
```

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 2541

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.246419

Peak wave period Tp (s) : 1.81165

Mean wave period T01 (s) : 1.517343

Mean wave period T02 (s) : 1.430651

Mean wave direction MWD (deg): 0

=====

Test 87

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-0.845399	0.170599	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	56000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	114882	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-0.845399	0.1606	meter
Porosity zones	Grid Codes	0	3	Integer

Section: ZONE_1 -----

porosity : 0.4

grain_diameter : 0.026

```
alpha : 1000
beta : 2.8
oscillating_period : 8
gamma : 0.34
Section: ZONE_2 -----
porosity : 0.4
grain_diameter : 0.05
alpha : 1000
beta : 2.8
oscillating_period : 8
gamma : 0.34
Section: ZONE_3 -----
porosity : 0.2
grain_diameter : 0.05
alpha : 1000
beta : 580000000
oscillating_period : 8
gamma : 0.34
Section: SPONGE_LAYER -----
type : 1
Section: SPONGE_ZONES_BOUNDARY -----
number_of_zones : 1
Section: ZONE_1 -----
include : 1
boundary_code : 4
width : 5
approach : 3 (default)
characteristic_resolution : 0.2
a : 7.5
r : 0.725
s_reference : 0
u_reference : 0 (default)
v_reference : 0 (default)
Section: BODY_FORCE -----
Section: INTERNAL_CONDITIONS -----
number_of_zones : 1
Section: ZONE_1 -----
include : 1
```

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.26833

peak_wave_period : 1.9

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 2541

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.268329

Peak wave period Tp (s) : 1.90113

Mean wave period T01 (s) : 1.591823

Mean wave period T02 (s) : 1.499508

Mean wave direction MWD (deg): 0

=====

Test 316

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-1.0954	0	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	56000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	114882	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-1.0954	0	meter
Porosity zones	Grid Codes	0	2	Integer

Section: ZONE_1 -----

porosity : 0.4

grain_diameter : 0.05

alpha : 1000

beta : 2.8

oscillating_period : 8

gamma : 0.34

Section: ZONE_2 -----

porosity : 0.1

grain_diameter : 0.005

alpha : 1000

beta : 5.8E+11

oscillating_period : 8

gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

boundary_code : 4

width : 5

approach : 3 (default)

characteristic_resolution : 0.2

a : 7.5

r : 0.725

s_reference : 0

u_reference : 0 (default)

v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

```

type_of_ramp_up_time : 0
type_of_ramp_down_time : 0
Section: WAVE_DATA -----
type_of_waves : 2
water_depth : 0.845
s_reference : 0
Section: IRREGULAR_WAVES -----
Section: FREQUENCY_SPECTRUM -----
type_of_frequency_spectrum : 2
Section: JONSWAP -----
type_of_JONSWAP : 1
significant_wave_height : 0.21005
peak_wave_period : 1.72
gamma : 3.3
sigma_A : 0.07
sigma_B : 0.09
Section: DIRECTIONAL_DISTRIBUTION -----
type_of_directional_distribution : 0
type_of_angle : 2
main_direction : 0
Section: AUXILIARY_PARAMETERS -----
type_of_frequency_discretization : 1 (default)
minimum_frequency : 0.0005
maximum_frequency : 3
number_of_components : 2541
minimum_amplitude : 1.E-08 (default)
seed : 100
rescale : 1
wave_theory : 1 (default)
second_order_correction : 0
=====Wave parameters - generated spectrum=====
Significant wave height Hm0 (m) : 0.210049
Peak wave period Tp (s) : 1.719674
Mean wave period T01 (s) : 1.442952
Mean wave period T02 (s) : 1.361909
Mean wave direction MWD (deg): 0
=====

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Test 317

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-1.0954	0	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	56000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	114882	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-1.0954	0	meter
Porosity zones	Grid Codes	0	2	Integer

 Section: ZONE_1 -----

porosity : 0.4
 grain_diameter : 0.05
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_2 -----

porosity : 0.1
 grain_diameter : 0.005
 alpha : 1000
 beta : 5.8E+11
 oscillating_period : 8
 gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1
 boundary_code : 4

width : 5
approach : 3 (default)
characteristic_resolution : 0.2
a : 7.5
r : 0.725
s_reference : 0
u_reference : 0 (default)
v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.22826

peak_wave_period : 1.81

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 2541

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.22826

Peak wave period Tp (s) : 1.81165

Mean wave period T01 (s) : 1.517343

Mean wave period T02 (s) : 1.430651

Mean wave direction MWD (deg): 0

Test 318

Static items:

Item name	Item type	Minimum	Maximum	Unit
Node id	Grid Codes	1	114882	Integer
X-coord	Geographical coordin	0	28	meter
Y-coord	Geographical coordin	0	1	meter
Z-coord	Item geometry 3-dime	-1.0954	0	meter
Code	Grid Codes	2	5	Integer
Element id	Grid Codes	1	56000	Integer
Element type	Grid Codes	33	33	Integer
No of nodes	Grid Codes	8	8	Integer
Connectivity	Grid Codes	1	114882	Integer

Dynamic items:

Item name	Item type	Minimum	Maximum	Unit
Z coordinate	Item geometry 3-dime	-1.0954	0	meter
Porosity zones	Grid Codes	0	2	Integer

Section: ZONE_1 -----

porosity : 0.4
 grain_diameter : 0.05
 alpha : 1000
 beta : 2.8
 oscillating_period : 8
 gamma : 0.34

Section: ZONE_2 -----

porosity : 0.1
 grain_diameter : 0.005
 alpha : 1000
 beta : 5.8E+11
 oscillating_period : 8
 gamma : 0.34

Section: SPONGE_LAYER -----

type : 1

Section: SPONGE_ZONES_BOUNDARY -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1
 boundary_code : 4
 width : 5
 approach : 3 (default)
 characteristic_resolution : 0.2
 a : 7.5
 r : 0.725
 s_reference : 0
 u_reference : 0 (default)
 v_reference : 0 (default)

Section: BODY_FORCE -----

Section: INTERNAL_CONDITIONS -----

number_of_zones : 1

Section: ZONE_1 -----

include : 1

Section: RELAXATION_DATA -----

coordinate_type : NON-UTM

number_of_points : 2

Section: POINT_1 -----

x : 4.6

y : 1

Section: POINT_2 -----

x : 4.6

y : 0

width_ramp_up_zone : 4.5

width_pressure_zone : 4.5 (default)

type_of_ramp_up_space : 1

factor_ramp_up : 3.5

type_of_ramp_up_time : 0

type_of_ramp_down_time : 0

Section: WAVE_DATA -----

type_of_waves : 2

water_depth : 0.845

s_reference : 0

Section: IRREGULAR_WAVES -----

Section: FREQUENCY_SPECTRUM -----

type_of_frequency_spectrum : 2

Section: JONSWAP -----

type_of_JONSWAP : 1

significant_wave_height : 0.26679

peak_wave_period : 1.9

gamma : 3.3

sigma_A : 0.07

sigma_B : 0.09

Section: DIRECTIONAL_DISTRIBUTION -----

type_of_directional_distribution : 0

type_of_angle : 2

main_direction : 0

Section: AUXILIARY_PARAMETERS -----

type_of_frequency_discretization : 1 (default)

minimum_frequency : 0.0005

maximum_frequency : 3

number_of_components : 2541

minimum_amplitude : 1.E-08 (default)

seed : 100

rescale : 1

wave_theory : 1 (default)

second_order_correction : 0

=====Wave parameters - generated spectrum=====

Significant wave height Hm0 (m) : 0.26679

Peak wave period Tp (s) : 1.90113

Mean wave period T01 (s) : 1.591823

Mean wave period T02 (s) : 1.499508

Mean wave direction MWD (deg): 0

=====

ANNEX B

Results of the numerical model discarded

Test 5

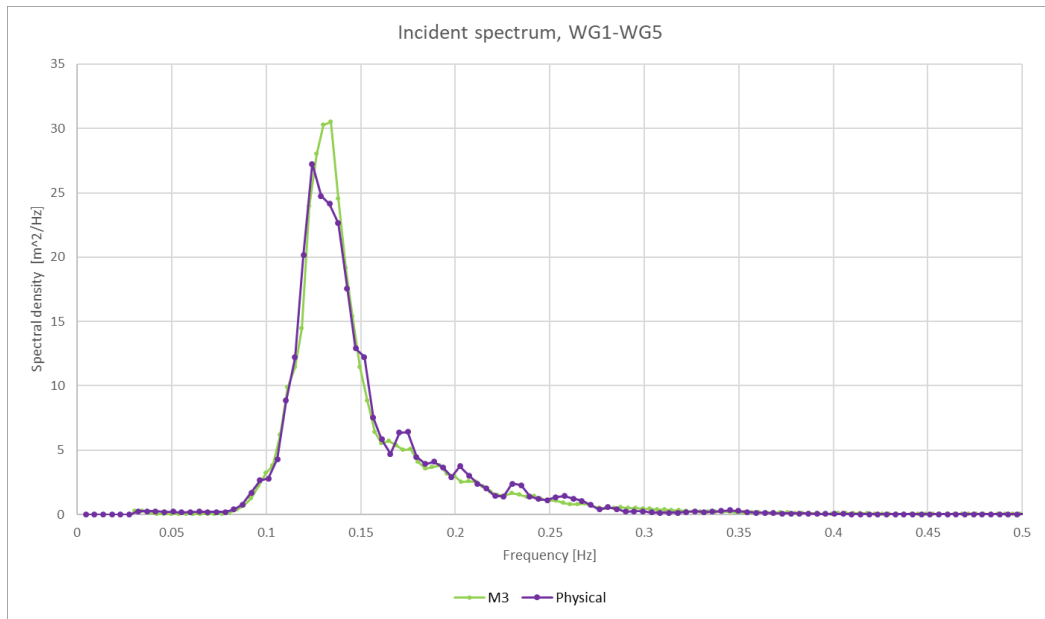


Fig. B.0.1 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 5.

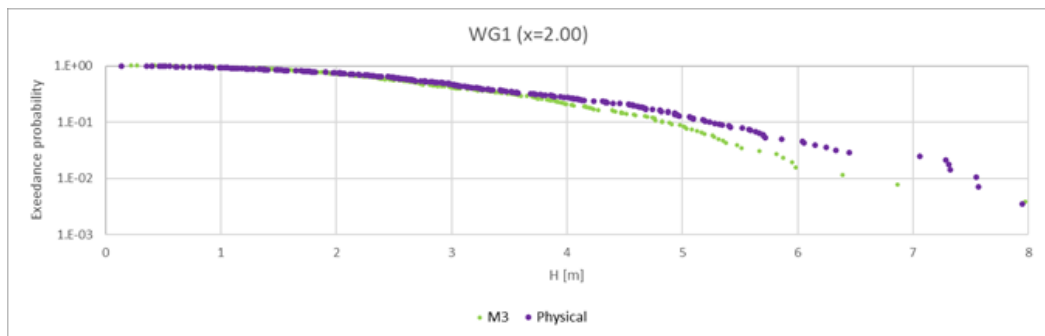


Fig. B.0.2 - Wave height distribution of the discarded model at WG1 (x=2.00 m). Test no. 5.

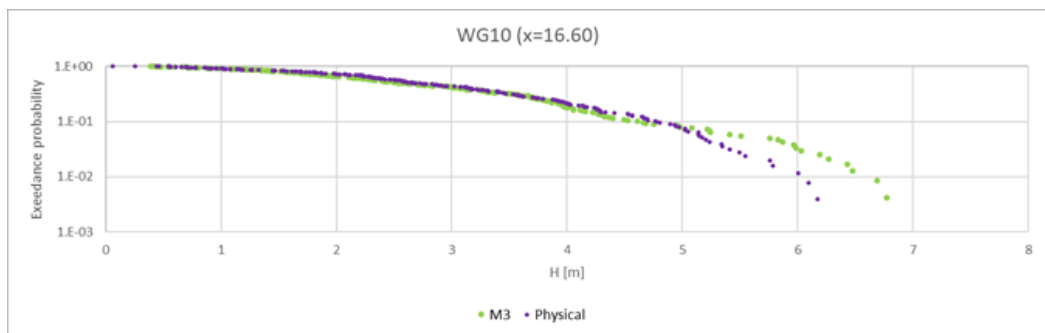


Fig. B.0.3 - Wave height distribution of the discarded model at WG10 (x=16.60 m). Test no. 5.

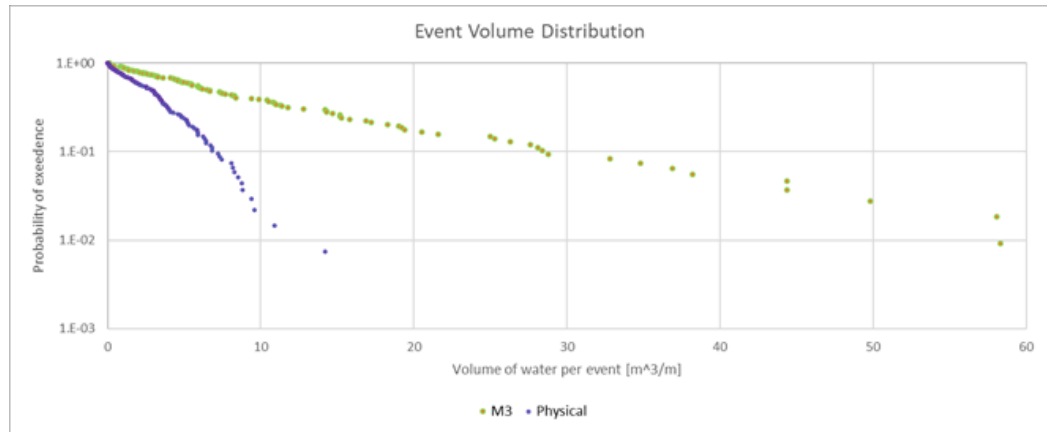


Fig. B.0.4 - Event Volume distribution of the discarded model. Test no. 5.

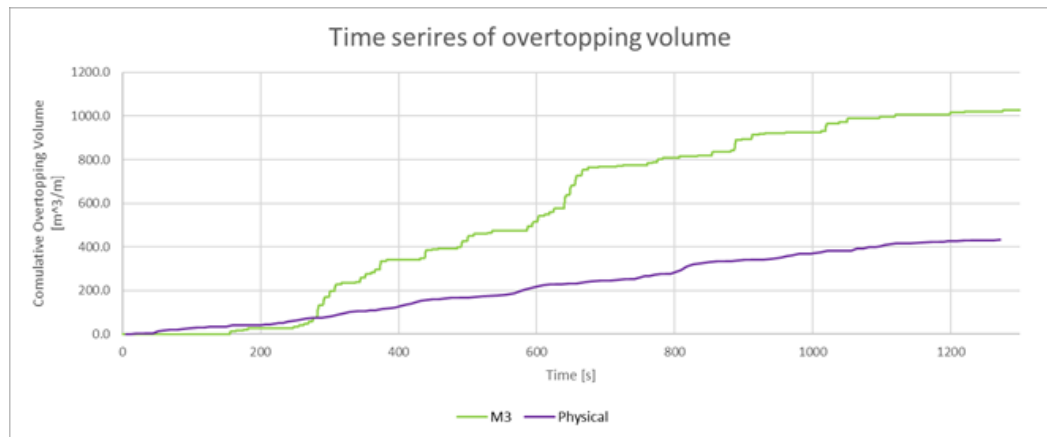


Fig. B.0.5 - Time series of the accumulated overtopping volume of the discarded model. Test no. 5.

Test 16

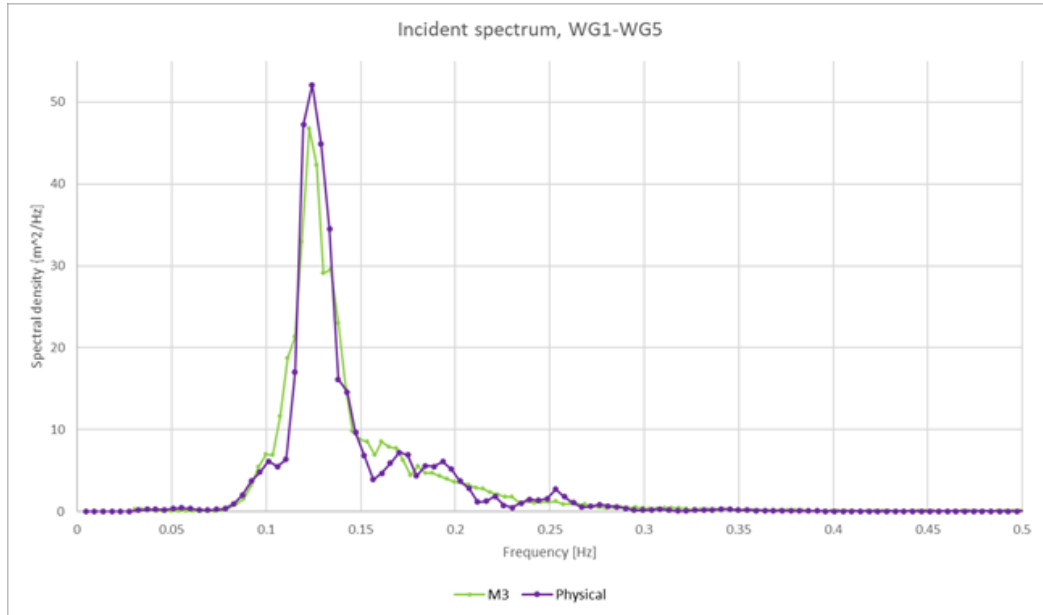


Fig. B.0.6 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 16.

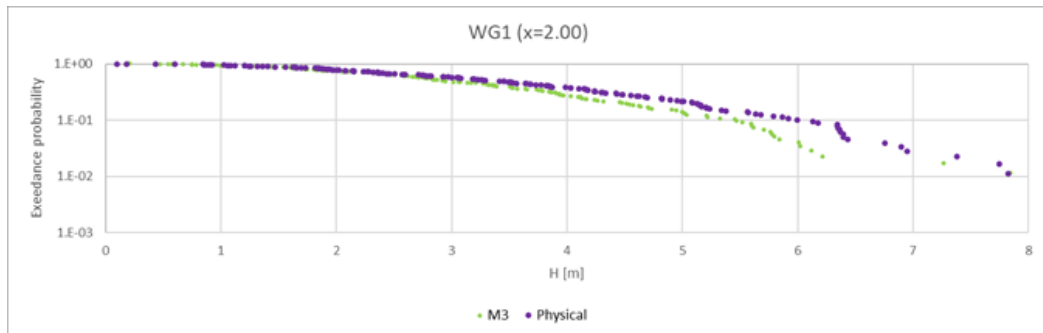


Fig. B.0.7 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 16.

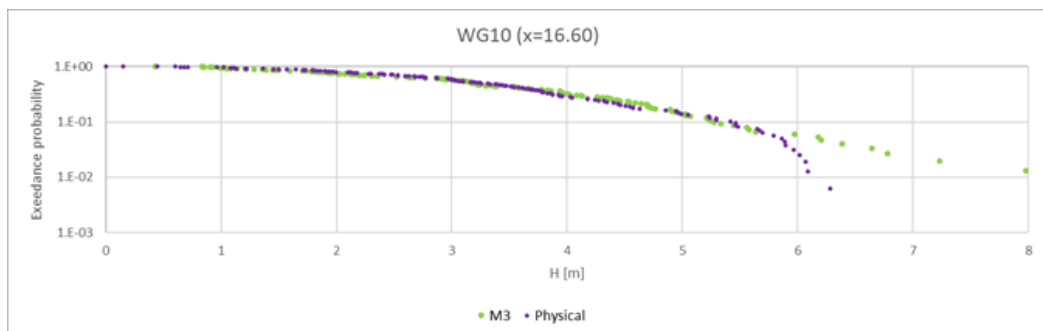


Fig. B.0.8 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 16.

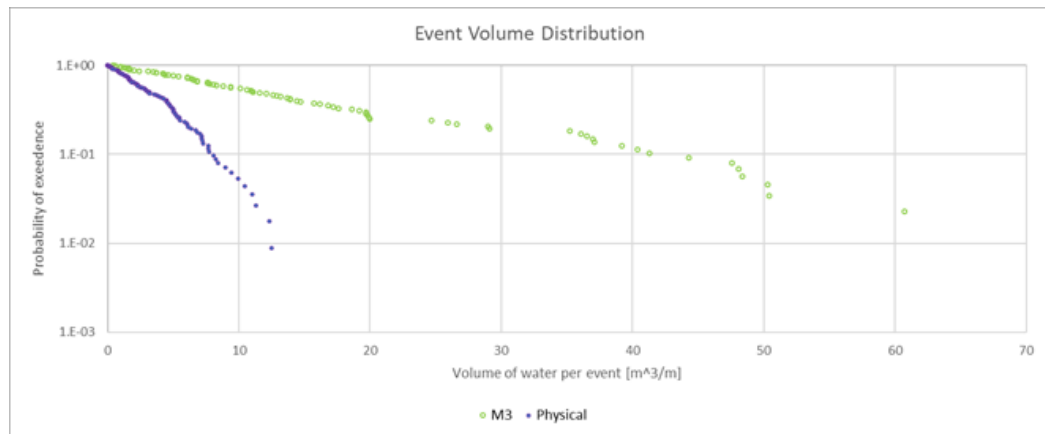


Fig. B.0.9 - Event Volume distribution of the discarded model. Test no. 16.

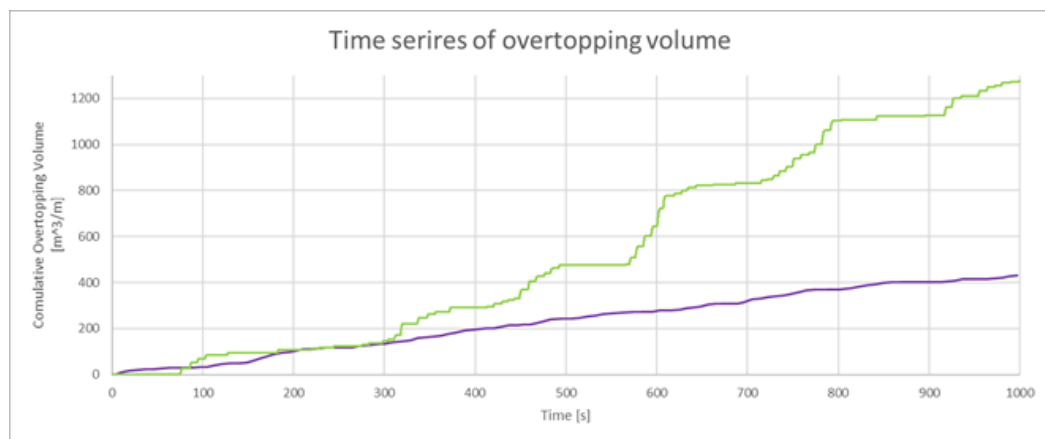


Fig. B.0.10 - Time series of the accumulated overtopping volume of the discarded model. Test no. 16.

Test 36

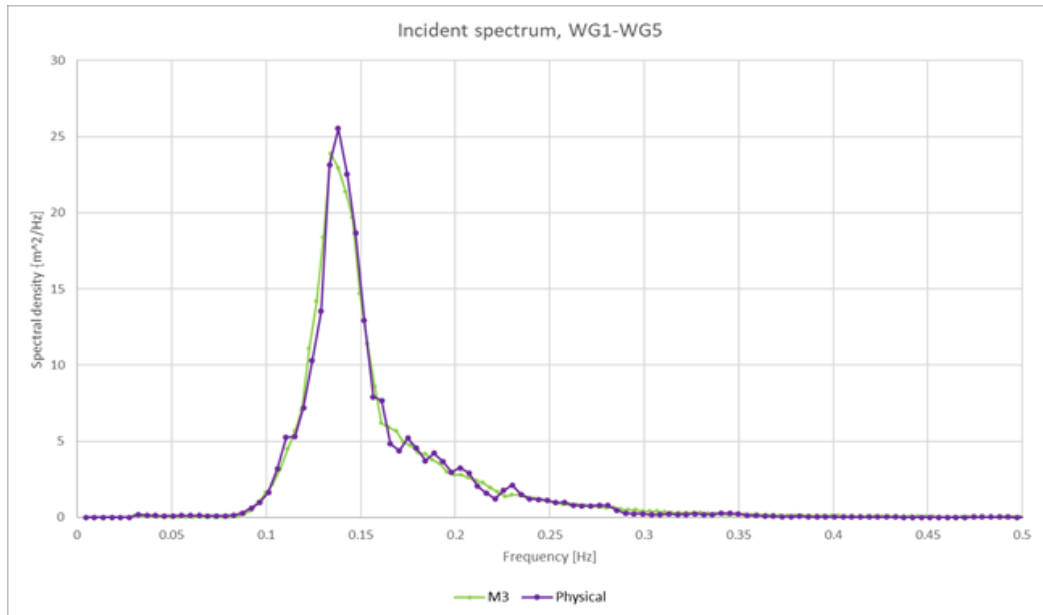


Fig. B.0.11 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 36.

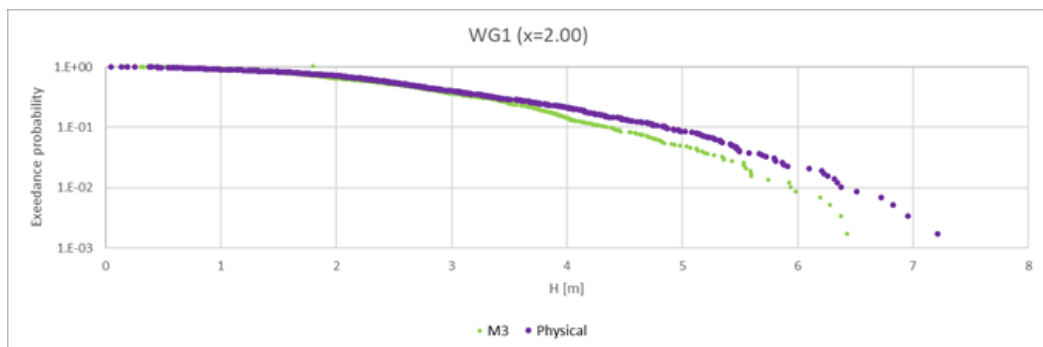


Fig. B.0.12 - Wave height distribution of the discarded model at WG1 (x=2.00 m). Test no. 36.

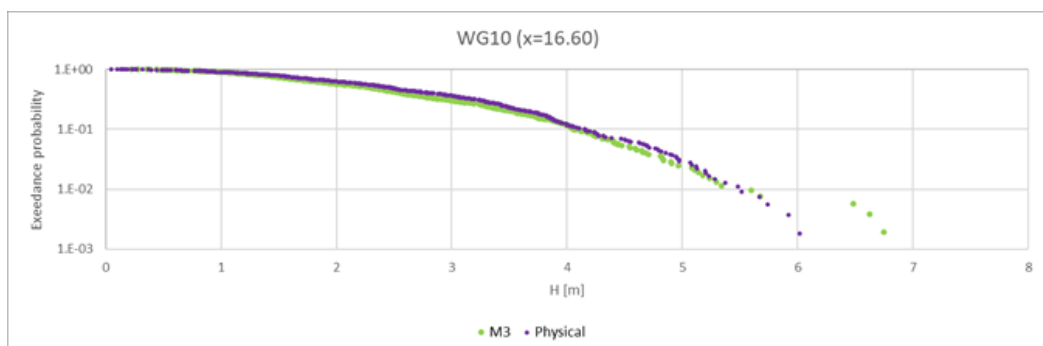


Fig. B.0.13 - Wave height distribution of the discarded model at WG10 (x=16.60 m). Test no. 36.

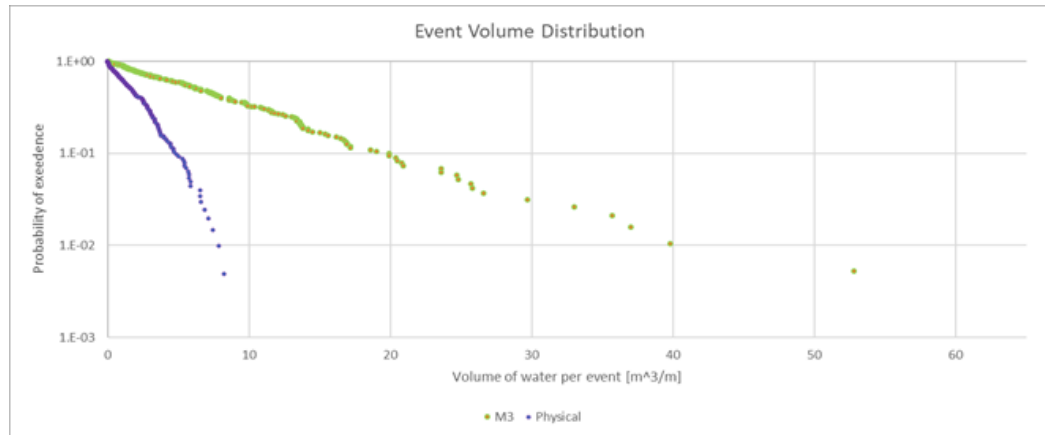


Fig. B.0.14 - Event Volume distribution of the discarded model. Test no. 36.

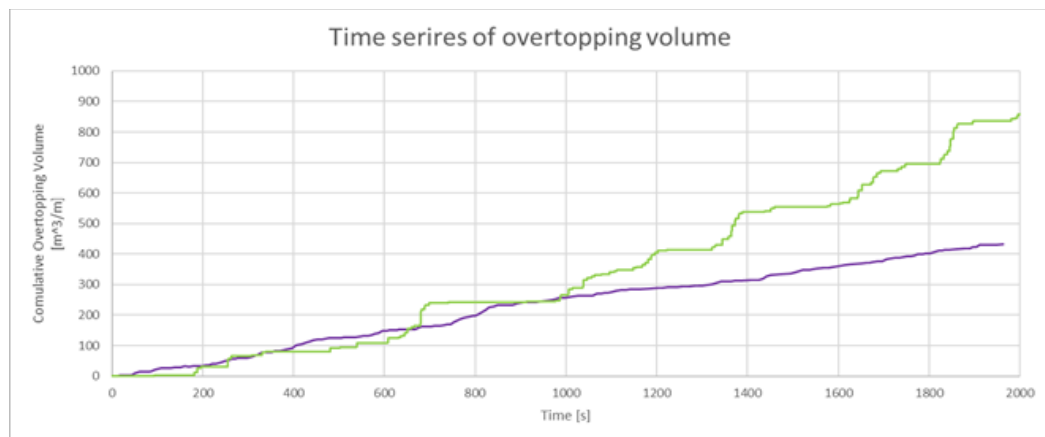


Fig. B.0.15 - Time series of the accumulated overtopping volume of the discarded model. Test no. 36.

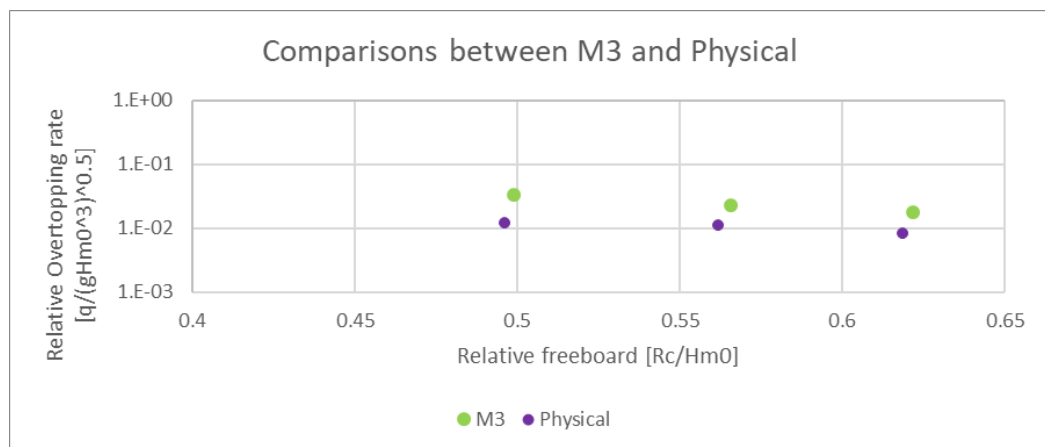


Fig. B.0.16 - Comparison of the overtopping in the physical model and the discarded M3 model, profile P-1.

Test 85

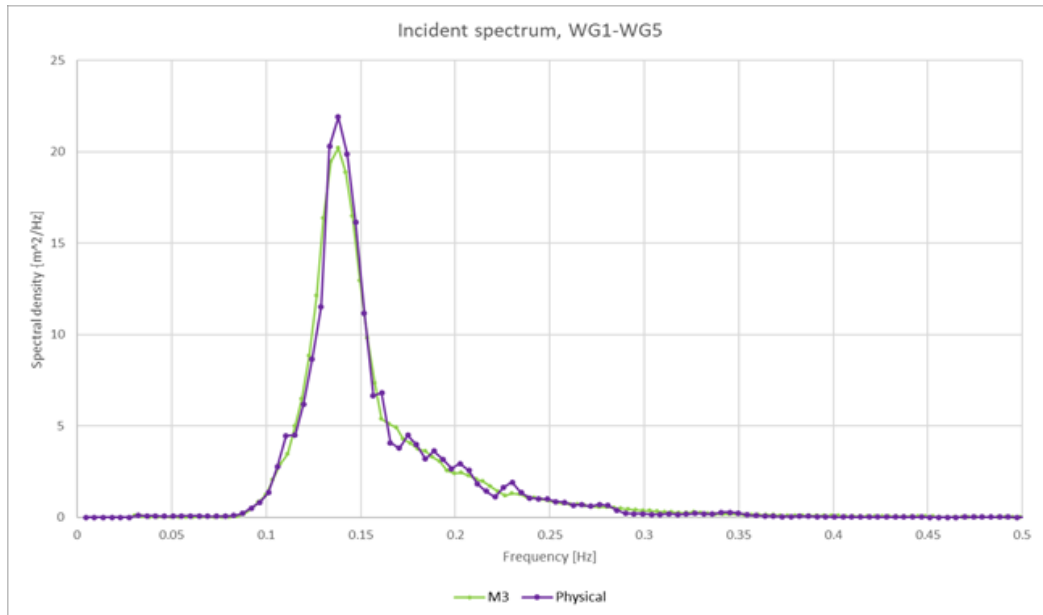


Fig. B.0.17 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 85.

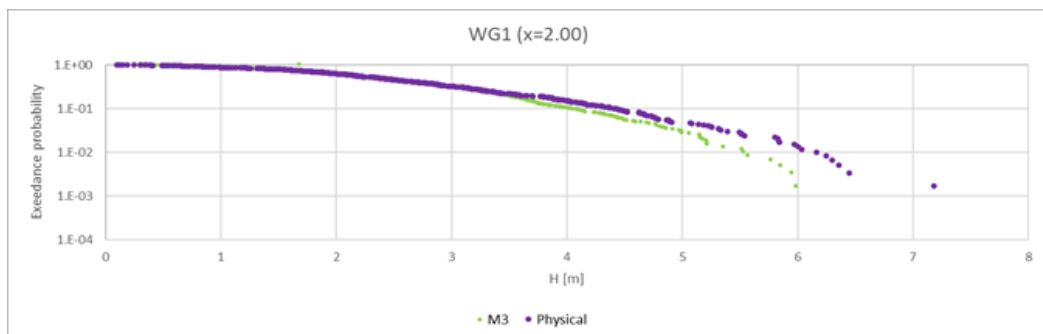


Fig. B.0.18 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 85.

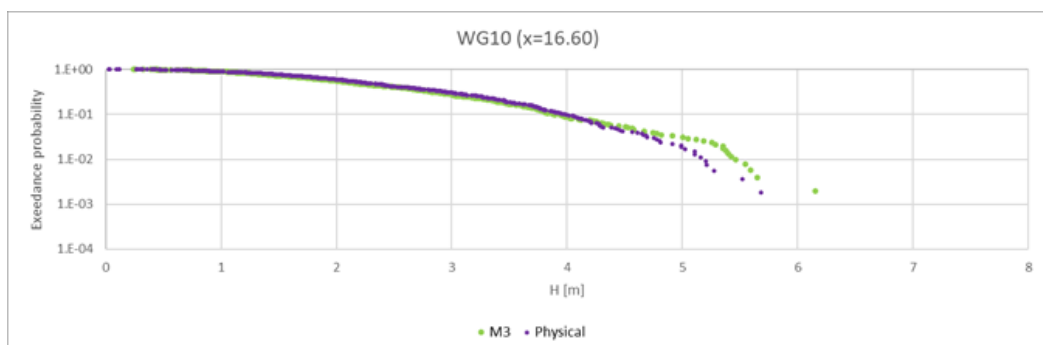


Fig. B.0.19 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 85.

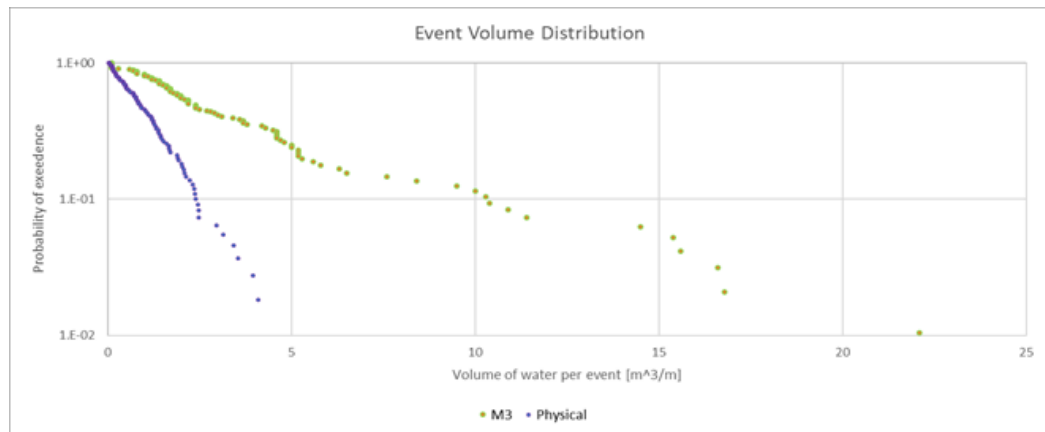


Fig. B.0.20 - Event Volume distribution of the discarded model. Test no. 85.

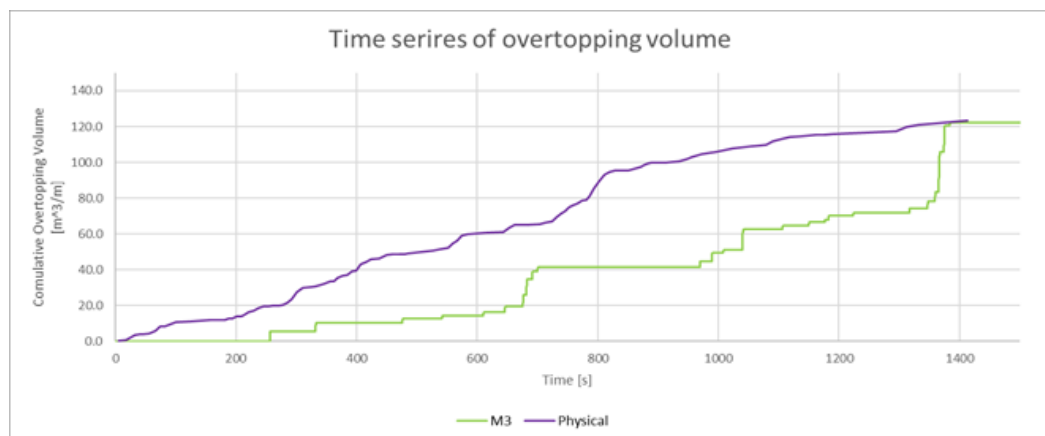


Fig. B.0.21 - Time series of the accumulated overtopping volume of the discarded model. Test no. 85.

Test 86

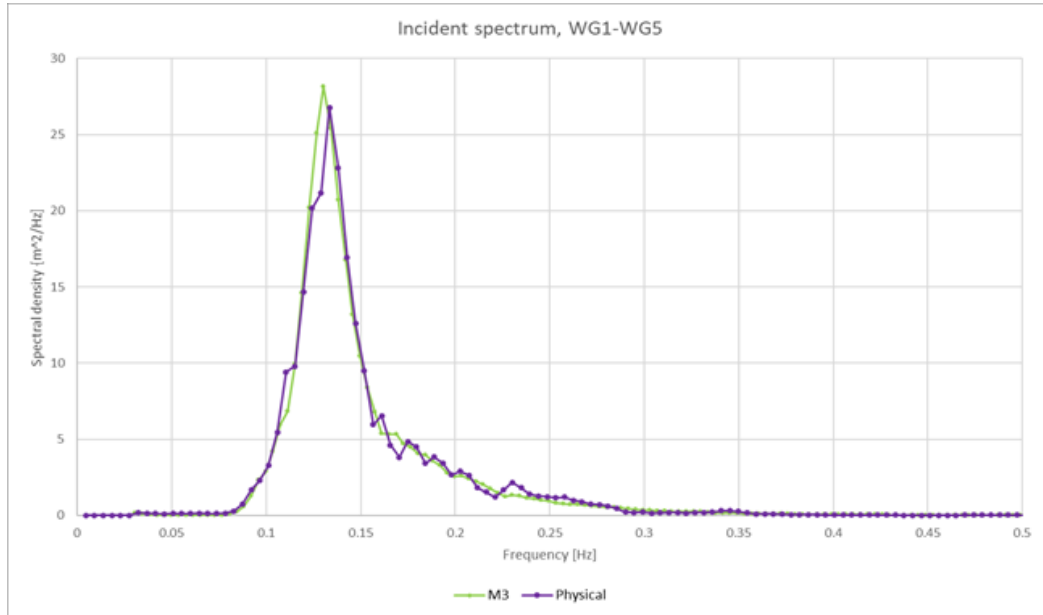


Fig. B.0.22 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 86.

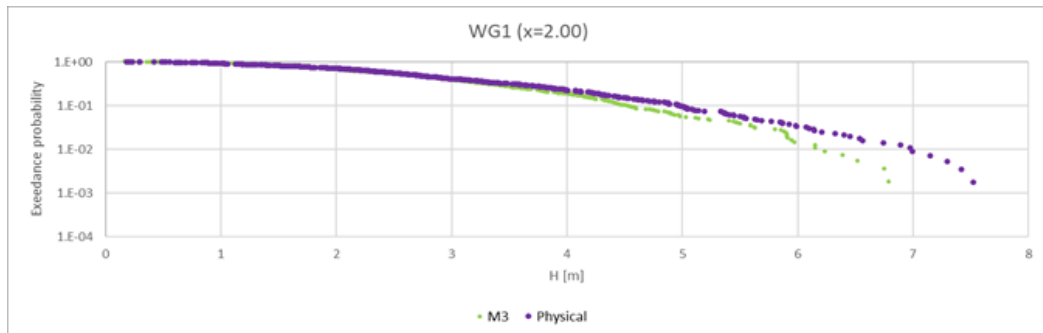


Fig. B.0.23 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 86.

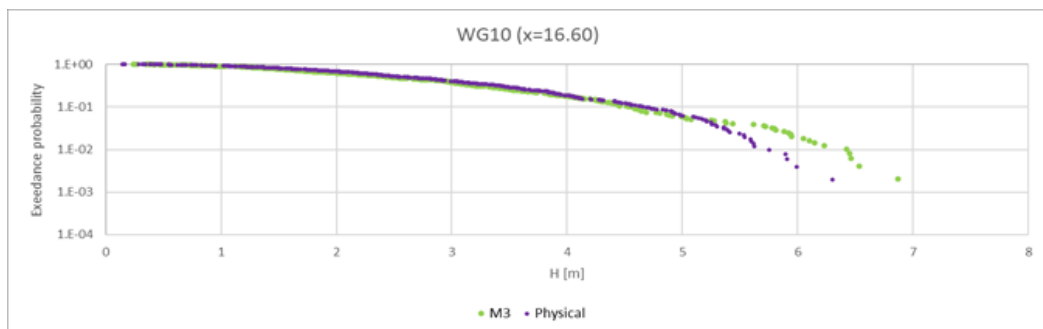


Fig. B.0.24 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 86.

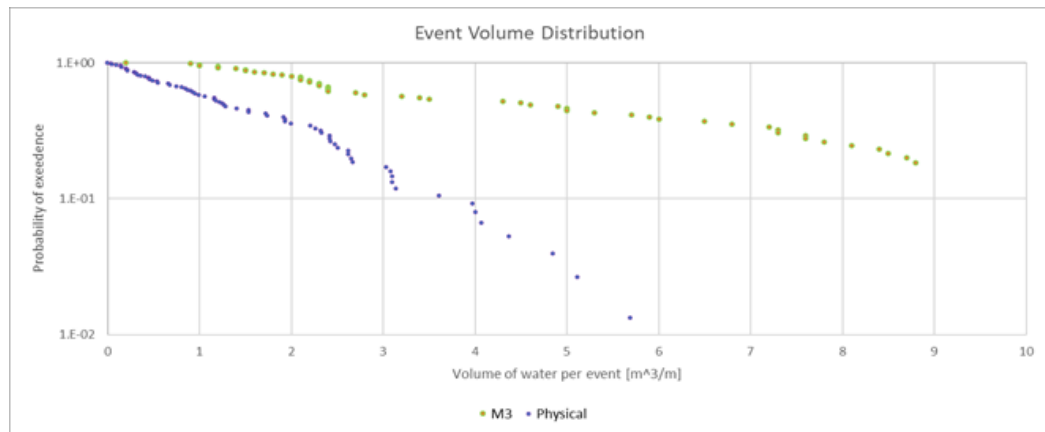


Fig. B.0.25 - Event Volume distribution of the discarded model. Test no. 86.

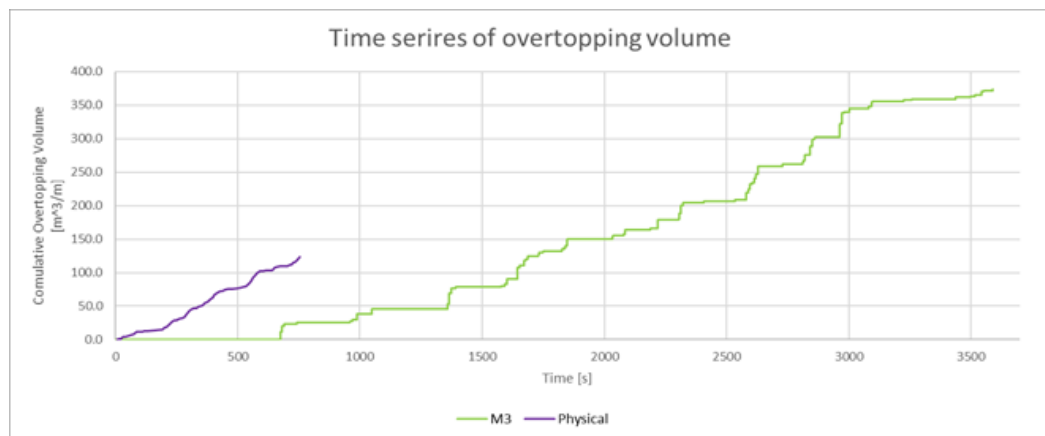


Fig. B.0.26 - Time series of the accumulated overtopping volume of the discarded model. Test no. 86.

Test 87

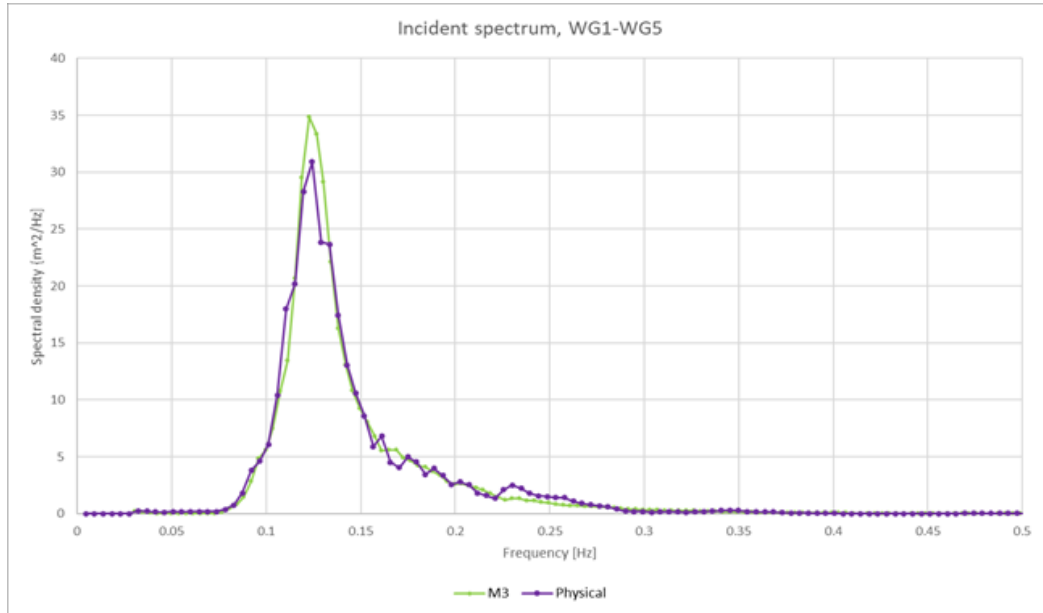


Fig. B.0.27 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no. 87.

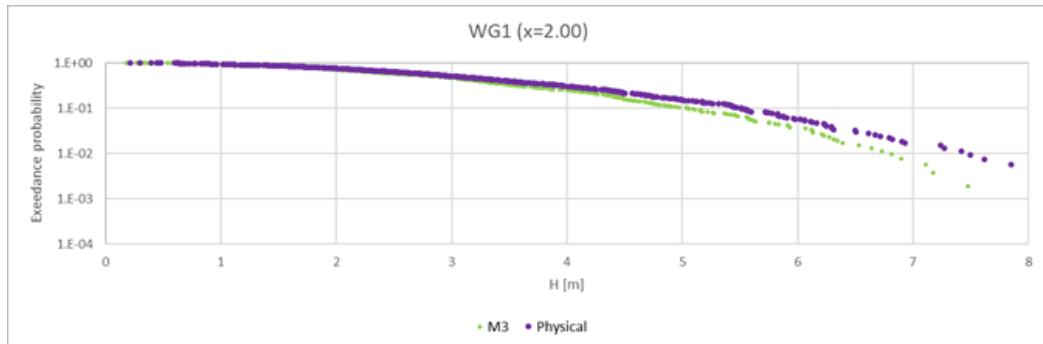


Fig. B.0.28 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 87.

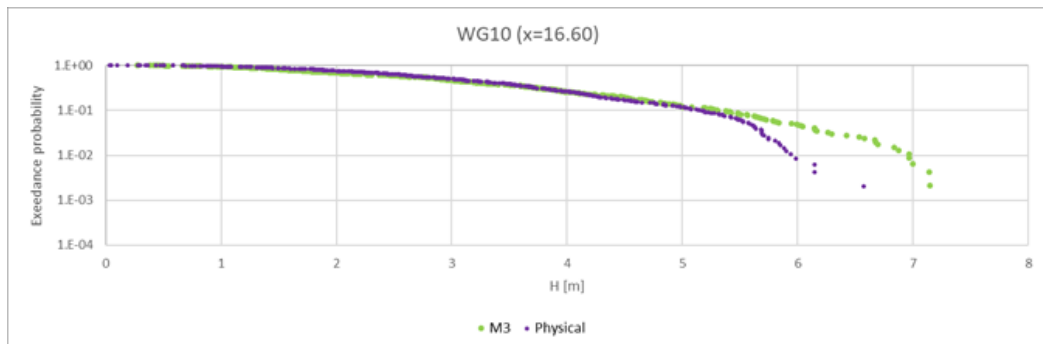


Fig. B.0.29 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 87.

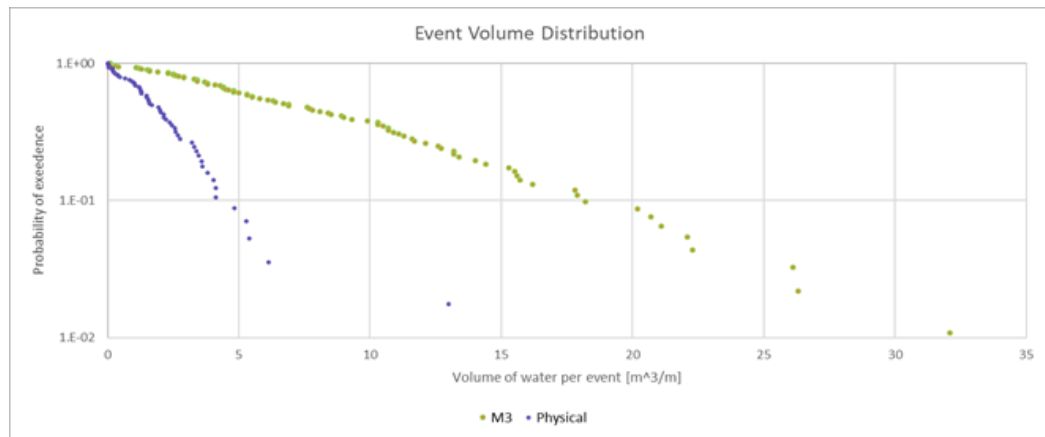


Fig. B.0.30 - Event Volume distribution of the discarded model. Test no. 87.

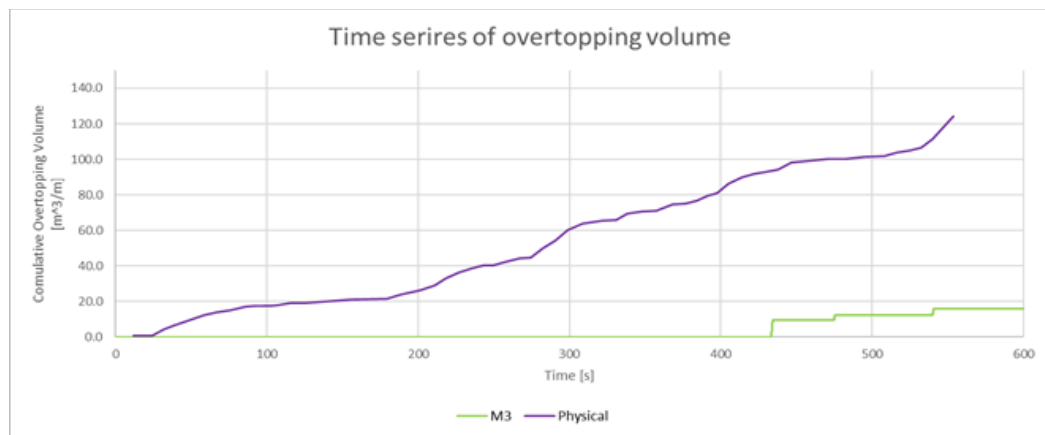


Fig. B.0.31 - Time series of the accumulated overtopping volume of the discarded model. Test no. 87.

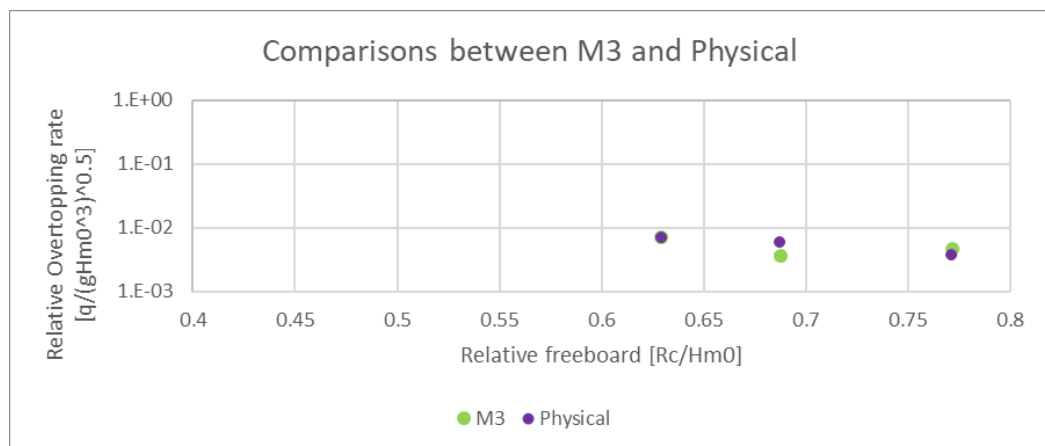


Fig. B.0.32 - Comparison of the overtopping in the physical model and the discarded M3 model, profile H1-4.

Test 316

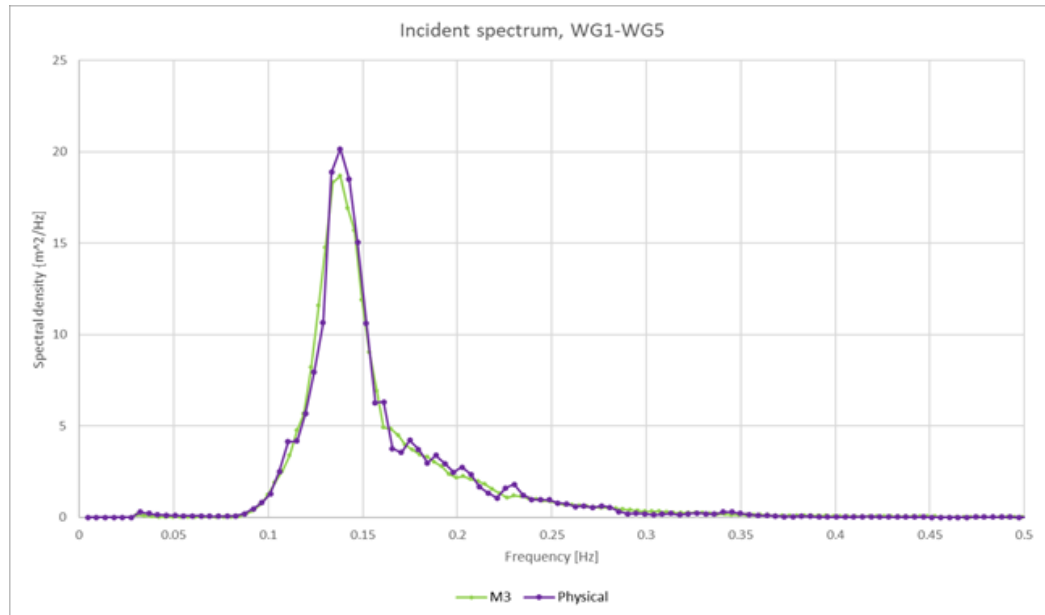


Fig. B.0.33 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no.316.

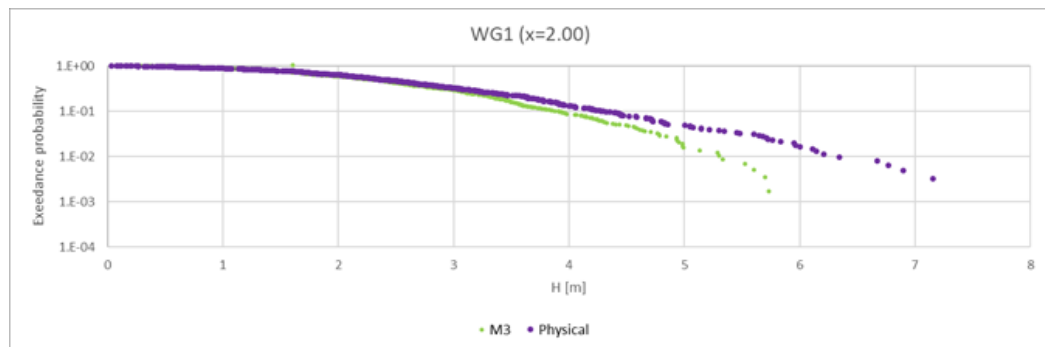


Fig. B.0.34 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 316.

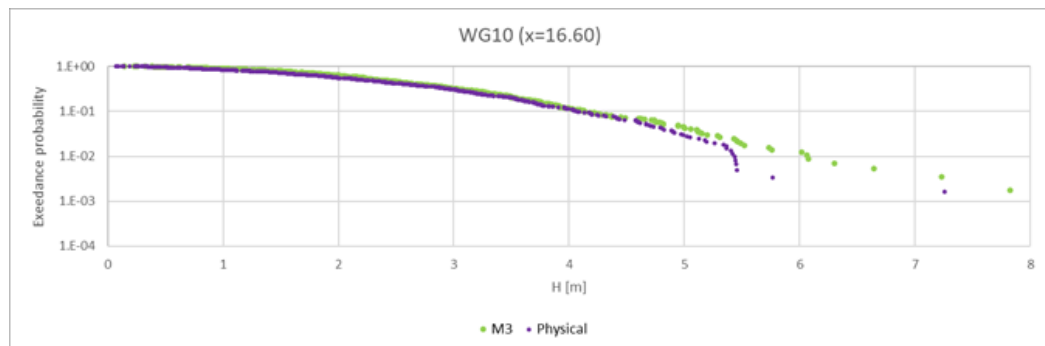


Fig. B.0.35 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 316.

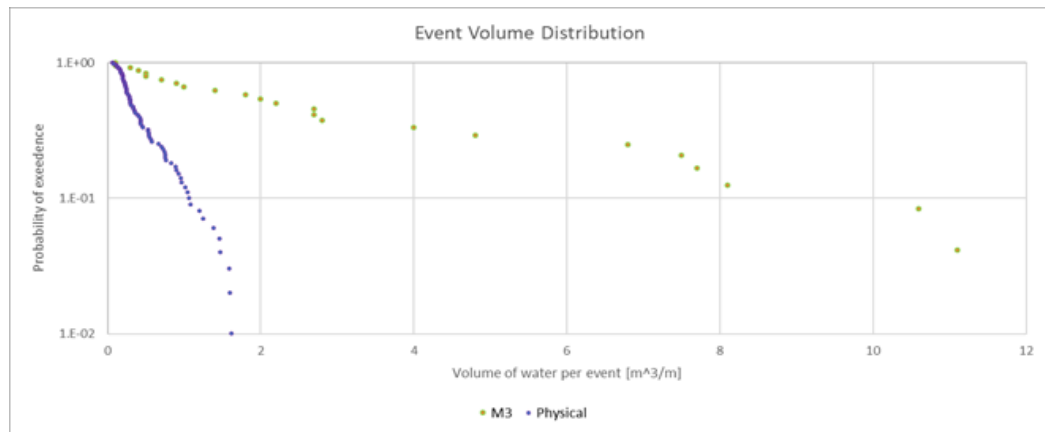


Fig. B.0.36 - Event Volume distribution of the discarded model. Test no. 316.

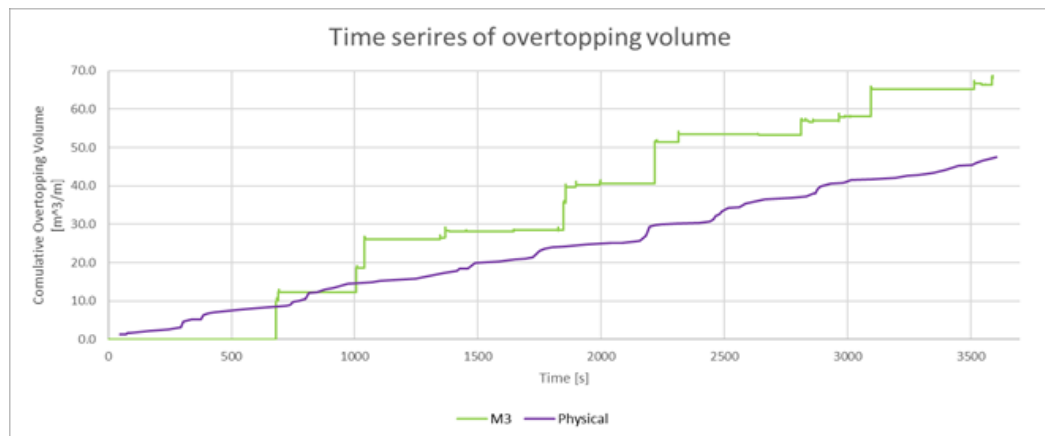


Fig. B.0.37 - Time series of the accumulated overtopping volume of the discarded model. Test no. 316.

Test 317

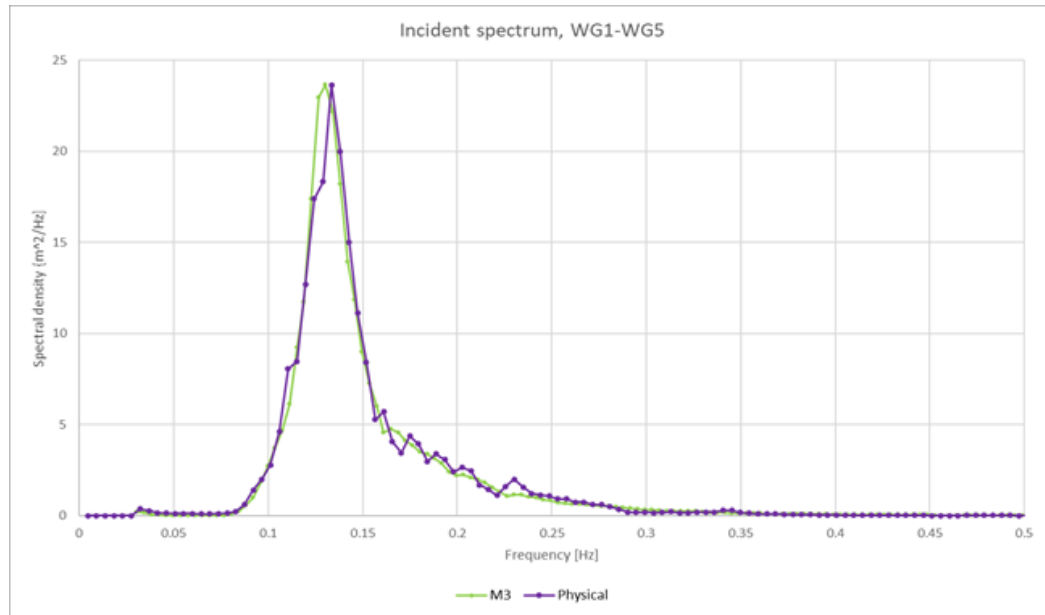


Fig. B.0.38 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no.317.

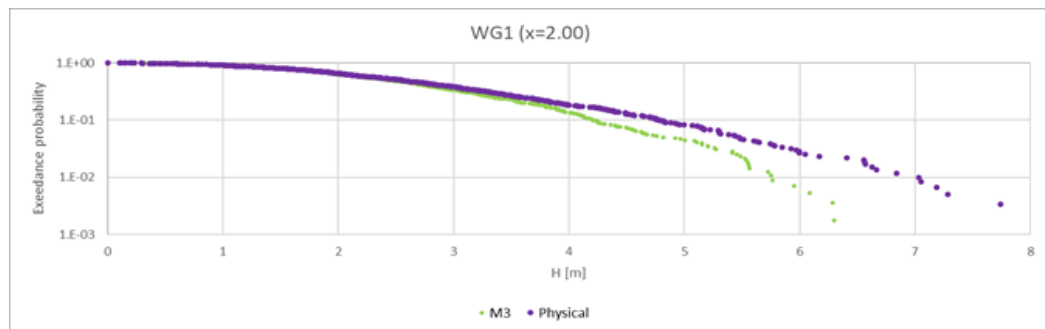


Fig. B.0.39 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 317.

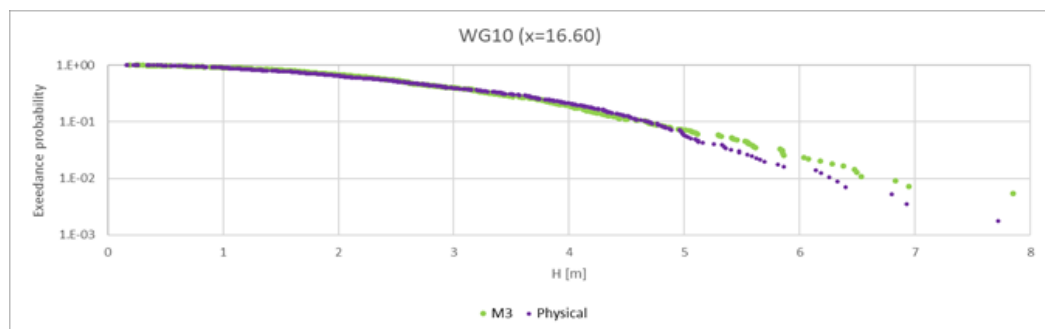


Fig. B.0.40 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 317.

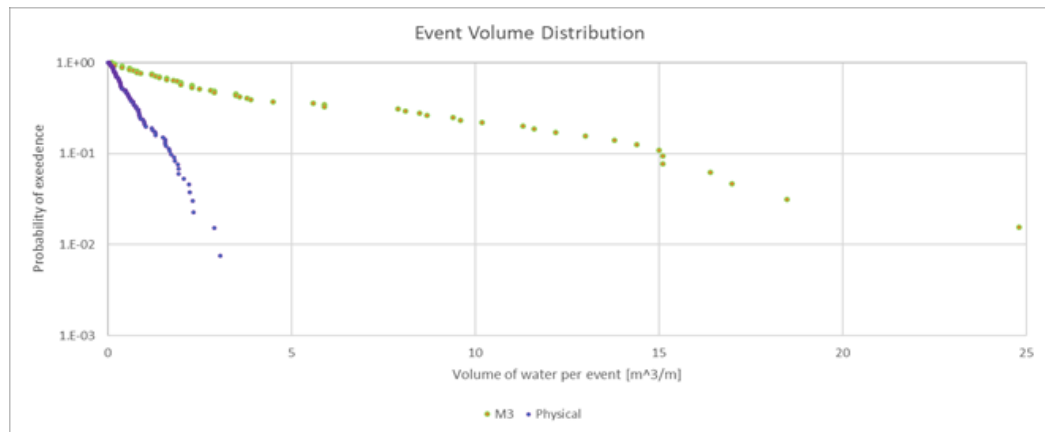


Fig. B.0.41 - Event Volume distribution of the discarded model. Test no. 317.

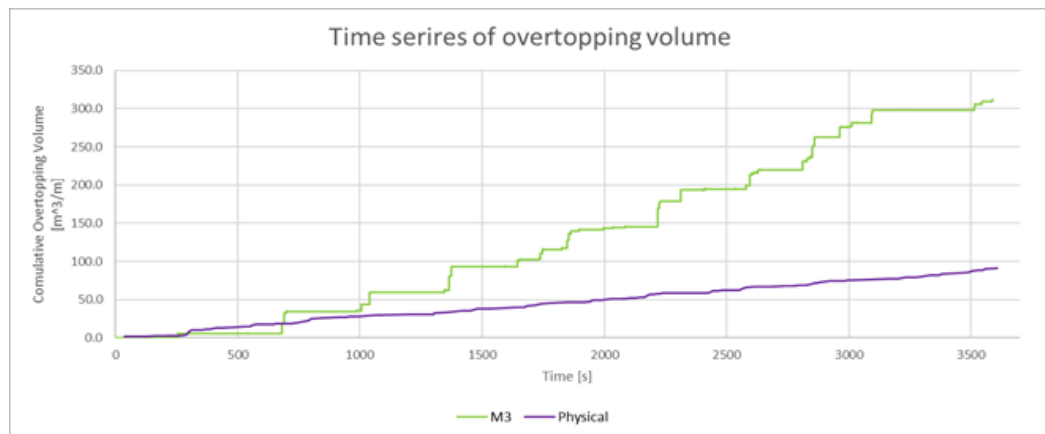


Fig. B.0.42 - Time series of the accumulated overtopping volume of the discarded model. Test no. 317.

Test 318

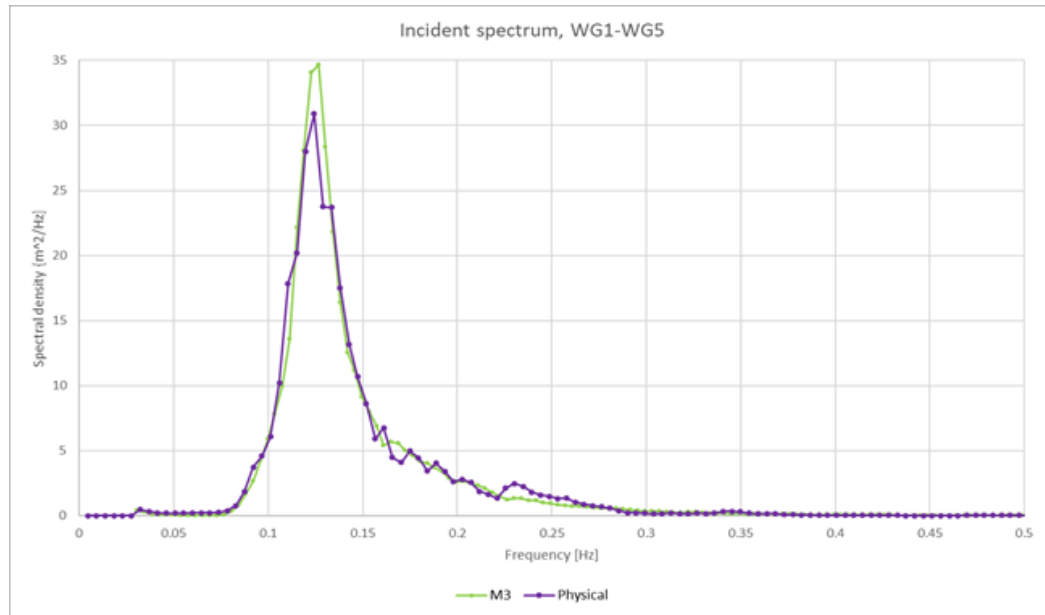


Fig. B.0.43 - Incident spectrum of the discarded M3 model (green) and the physical model (purple), from WG1 to WG5. Test no.318.

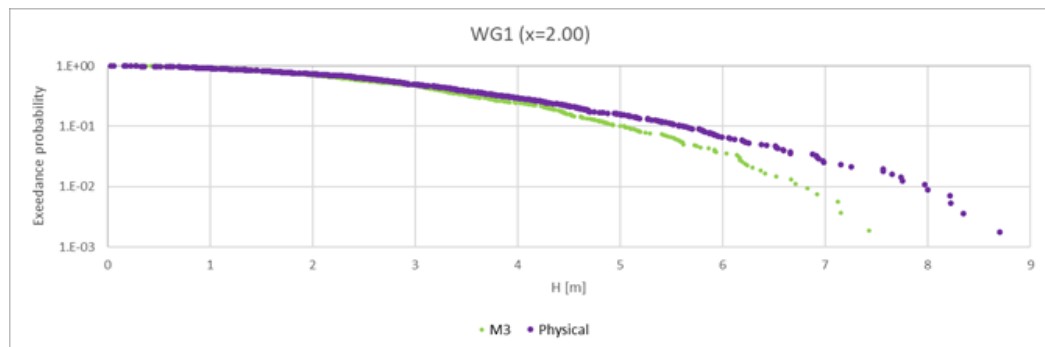


Fig. B.0.44 - Wave height distribution of the discarded model at WG1 ($x=2.00$ m). Test no. 318.

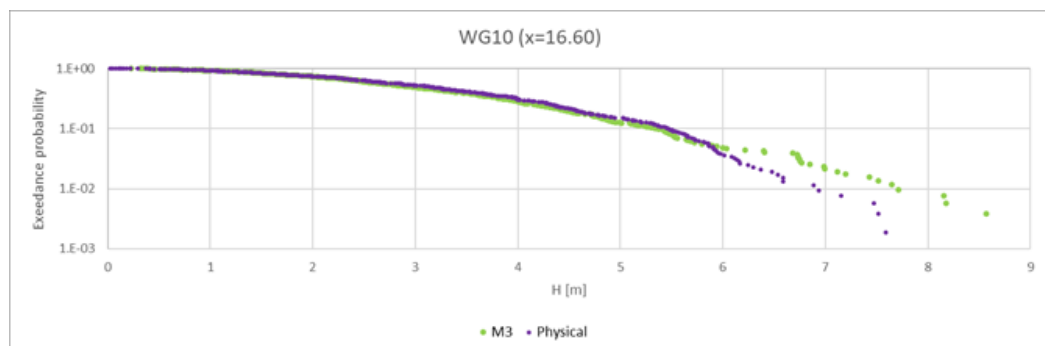


Fig. B.0.45 - Wave height distribution of the discarded model at WG10 ($x=16.60$ m). Test no. 318.

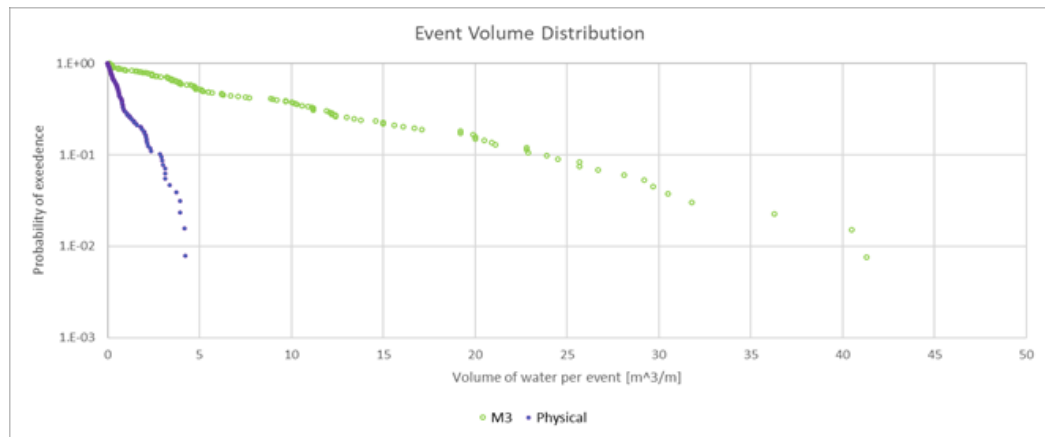


Fig. B.0.46 - Event Volume distribution of the discarded model. Test no. 318.

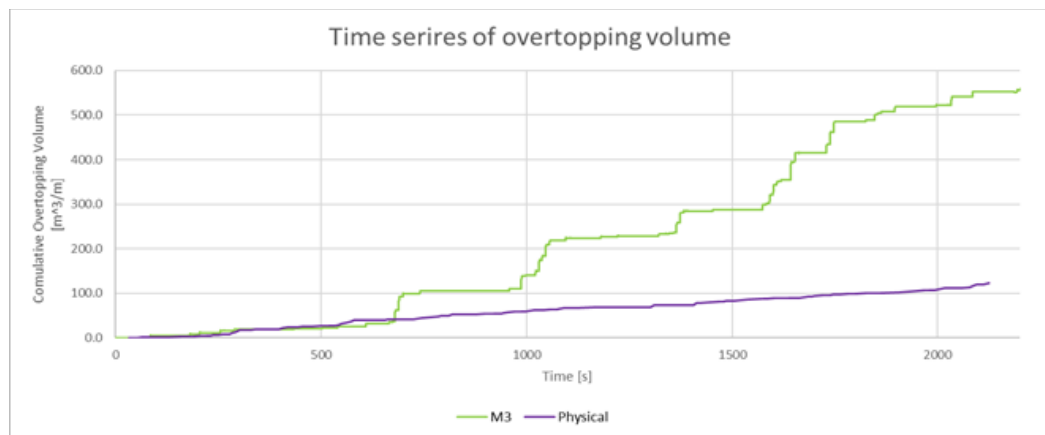


Fig. B.0.47 - Time series of the accumulated overtopping volume of the discarded model. Test no. 318.

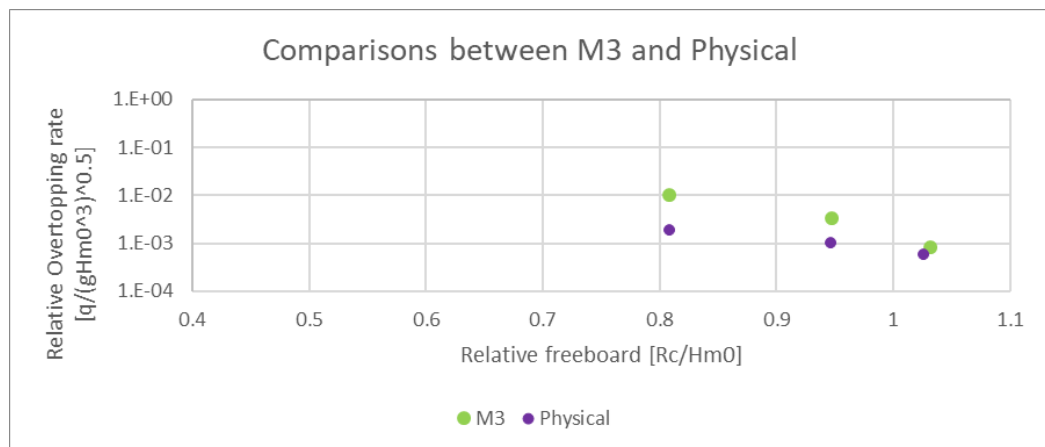


Fig. B.0.48 - Comparison of the overtopping in the physical model and the discarded M3 model, profile H4d-5.

ANNEX C

Transect of the numerical model adopted vs model discarded

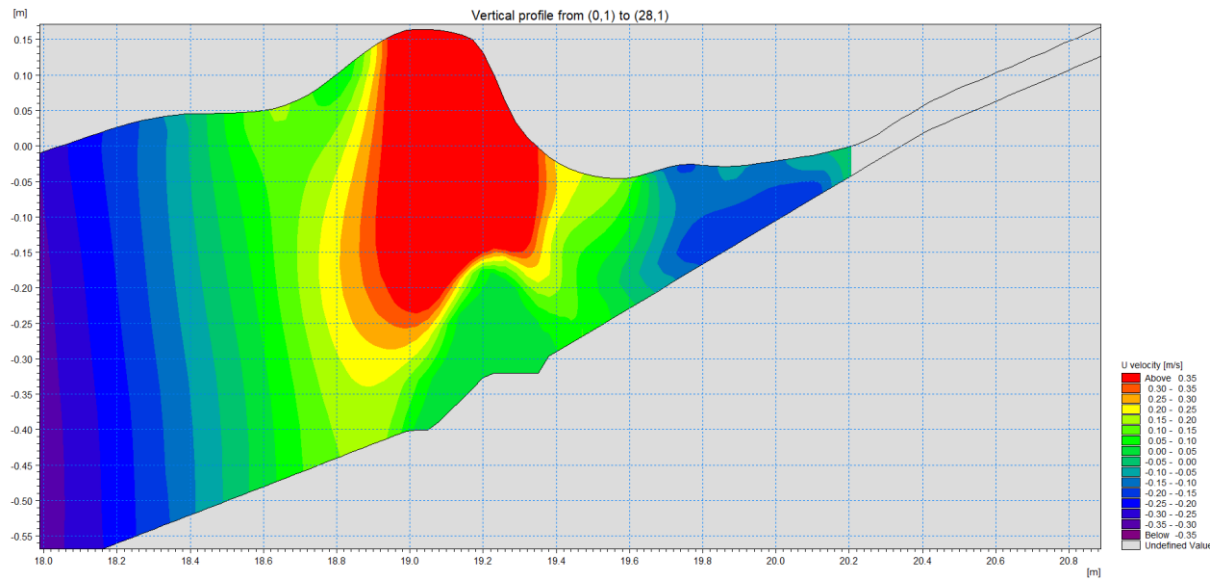


Fig. C.1 - Discarded model Transect.

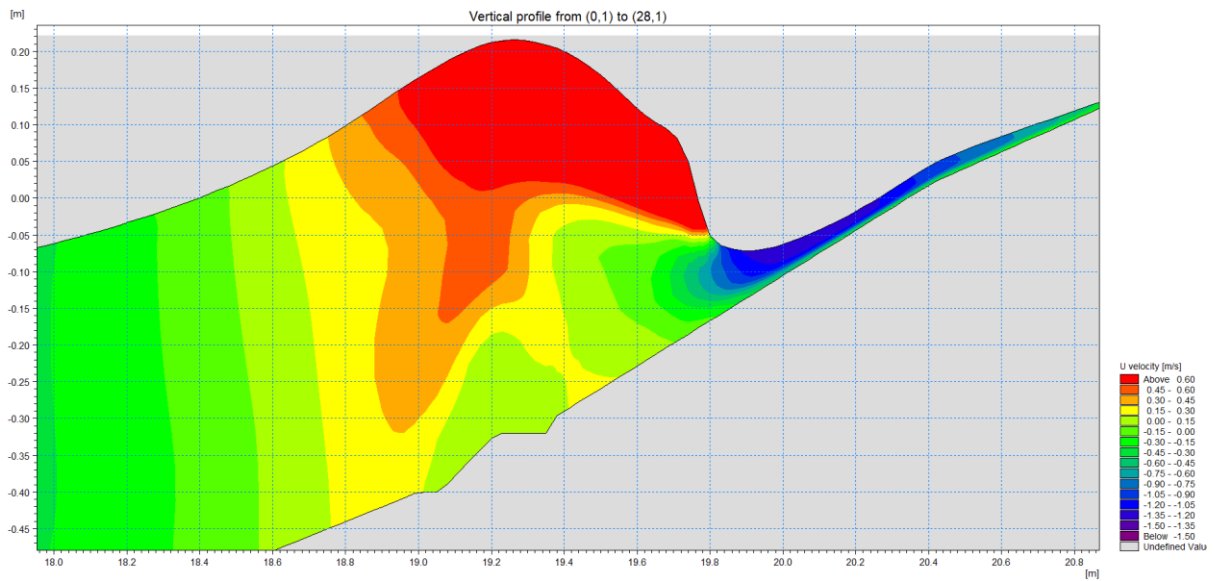


Fig. C.2 - Adopted model Transect.

