

EVALUATION OF THE DAMAGE PROGRESSION AND STUDY OF WAVE OVERTOPPING IN RUBBLE-MOUND STRUCTURES

TIAGO AFONSO PALAS E SILVA GUIMARÃES

Dissertation submitted for partial satisfaction of the requirements of the degree of
Master in Civil Engineering - Specialization in Hydraulics

Advisor: Professor Paulo Jorge Rosa Santos

Co-advisor: Professor Francisco de Almeida Taveira Pinto

Co-advisor: Doctor Engineer Hugo Guedes Lopes

MESTRADO INTEGRADO EM ENGENHARIA CIVIL 2016/2017

DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ miec@fe.up.pt

Editado por

FACULDADE DE ENGENHARIA DA UNIVERSIDADE DO PORTO

Rua Dr. Roberto Frias

4200-465 PORTO

Portugal

Tel. +351-22-508 1400

Fax +351-22-508 1440

✉ feup@fe.up.pt

🌐 <http://www.fe.up.pt>

Reproduções parciais deste documento serão autorizadas na condição que seja mencionado o Autor e feita referência a *Mestrado Integrado em Engenharia Civil - 2016/2017 - Departamento de Engenharia Civil, Faculdade de Engenharia da Universidade do Porto, Porto, Portugal, 2017*.

As opiniões e informações incluídas neste documento representam unicamente o ponto de vista do respetivo Autor, não podendo o Editor aceitar qualquer responsabilidade legal ou outra em relação a erros ou omissões que possam existir.

Este documento foi produzido a partir de versão eletrónica fornecida pelo respetivo Autor.

To my Parents

*"We must let go of the life we have planned, so as to accept the one that is waiting for
us."*

Joseph Campbell.

HONORABLE MENTIONS

Being concluded this important stage of my life, here are some honorable mentions to the people that participated and supported me, during this adventure.

First, to my parents, for their support, financially and emotionally, for their love, education, orientation, and for being my pillars during this long and challenging period. Many obstacles came to my life during this academic time, and without them, I would not have been able to finish this college degree. Only they know what I mean to say. This new title of mine, is mainly dedicated to them, my Mother, Fernanda Guimarães, and my Father, Ricardo Guimarães.

To my girlfriend, mother of my child and future wife, Diana, and to my son, Tomás, for giving me the happiest day of my life, and for providing me joy, love and happiness, everyday.

To my sister, Bárbara Guimarães, for being the most special princess I have ever met, and for surprising me day after day.

To my aunt Manuela Palas, for being a second mother to me since I was born.

To my aunt Isabel Guimarães, for providing me happy moments in my childhood.

To my grandfather Licínio Guimarães. Before he unfortunately passed away, two promises were made to him by me. The first one, was to give him a great-grandson, which I eventually did earlier than I expected. And the second promise, was to achieve a college degree - check.

To my grandmother Lurdes, for all the prayers she made during all my college exams.

To my maternal grandfathers, Diamantina and José Palas, for providing me with several happy moments in my childhood, and also that for a few days, they will give me a very special gift, they know what I mean to say.

To José Pedro for being my best friend since I was born, and for becoming my son's godfather.

To my friend Carlos Catita, for being there for me, in the most difficult times of my academic course and life.

To my friends Hugo, Martim, João Marrana, Filipe Dalte and João Ferreira, for the fellowship and fun given during this academic adventure.

To Cristiano, Cristóvão and Fábio, for all the nights spent at FEUP, studying and eating McDonald's every night.

To my teacher and advisor of this thesis, professor Paulo Rosa Santos, for giving me everyday all the support, motivation and orientation in order to make this Master's thesis great and professional.

To my teacher and co-advisor, professor Francisco Taveira Pinto, for giving me the opportunity to do a dissertation in enterprise environment on APDL, and join this project.

To the hydraulic laboratory technician Miguel Guerra from FEUP, and to Ermano Sousa (student from Delf University, for the hard work, dedication, leadership, stories told and all the fun during summer season of 2016 and for becoming my friend. Without them, the construction of the model used in this thesis would not be possible.

To Hugo Lopes from APDL, for the guidance, knowledge and effort for my integration into APDL's business environment.

And finally, to Secil Britas SA – Penafiel, and the director Mr. Arménio Soares from the commercial department, for all the support and availability in the supply of all the gravel requested from FEUP.

RESUMO

As estruturas de proteção costeira são constantemente submetidas a vários tipos de solicitações do ambiente marítimo, que podem ter diferentes magnitudes, dependendo das condições externas.

Os efeitos das alterações climáticas, como por exemplo, a subida do nível médio da água do mar, estão a aumentar essas condições externas, influenciando, por exemplo, os eventos de galgamento.

O objetivo principal desta dissertação consiste no estudo do galgamento de um quebramar de taludes para diferentes condições de agitação marítima e na análise da progressão do dano da estrutura.

Tendo isso em vista, várias análises podem ser feitas com recurso a técnicas laboratoriais avançadas.

Os resultados experimentais foram comparados com os obtidos com recurso a ferramentas de previsão de galgamentos, para a gama de valores dada pelo Eurotop II (2016), sendo também algumas distribuições matemáticas aplicadas.

No final deste trabalho, concluiu-se qual a influência dos efeitos das alterações climáticas, bem como o efeito das ondas de crista-curta e variações da altura de onda significativa no galgamento e dano sobre a estrutura em estudo.

PALAVRAS-CHAVE: Alterações climáticas, subida do nível médio da água do mar, quebra-mares de taludes, galgamento, análise de dano, técnicas laboratoriais avançadas.

ABSTRACT

Coastal protection structures are constantly submitted to various types of loads from the maritime environment, that can have different magnitudes, depending on the external conditions.

Climate change effects, such as the sea level rise, are increasing those external conditions, influencing wave overtopping, for instance.

The main goal of this thesis, is based on the study of wave overtopping in rubble-mound structures for different conditions of the maritime environment, but also on the analysis of the damage progression in the structure.

Therefore, several analyses can be made using advanced laboratorial techniques.

The experimental results were compared with those obtained using the prediction tools for the range of values given by Eurotop II (2016), and some mathematical distributions are also applied.

At the end of this work, it was concluded the influence of the effects of climate change, as well as the effect of short-crested waves, and variations of the significant wave height, on wave overtopping and damage progression in the structure of this study.

KEY WORDS: Climate change effects, sea level rise, rubble-mound breakwaters, wave overtopping analysis, damage analysis, advanced laboratorial techniques.

INDEX

HONORABLE MENTIONS	i
RESUMO	iii
ABSTRACT	v
1. INTRODUCTION	1
1.1. Global Introduction	1
1.2. Research Objectives and structure.....	1
2. CLIMATE CHANGE EFFECTS ON COASTAL PROTECTION STRUCTURES	3
2.1. Climate Change: A First Approach	3
2.1.1. Greenhouse Gases and Global Warming	4
2.1.2. Decreasing Global Glacial Mass and Sea Level Rise	5
2.1.3. Coastline Retreat	7
2.2. Effects on the Function and Stability of Coastal Protection Structures	8
3. WAVE OVERTOPPING	11
3.1. INTRODUCTION	11
3.2. DEFINITION AND DESCRIPTION	12
3.2.1. Parameters related to wave overtopping	12
3.2.2. wave overtopping discharge and wave overtopping volume.....	16
3.3. OVERTOPPING ANALYSIS AND PREDICTION	18
3.3.1. INTRODUCTION.....	18
3.3.2. Empirical methods.....	18
3.3.3. Simplified formula for overtopping and comparison of structures	19
3.3.4. Numerical models – Wave overtopping	20
3.3.4.1. Non-linear Shallow Water Equations.....	20
3.3.4.2. Navier-Stokes Equations models	21
3.3.5. Smooth Particle Hydrodynamics (SPH) – wave overtopping	21
3.3.6. PC – Overtopping.....	22
3.3.7. Eurotop Neural Network prediction tool	23
3.3.8. Wave overtopping database – The Clash Project.....	26
3.3.9. Physical modelling	26

3.3.9.1. Introduction.....	26
3.3.9.2. Advantages and disadvantages	27
3.3.9.3. Similarity criteria	28
3.4. Overtopping analysis – probability and distribution functions	29
3.4.1. Weibull Probability Distribution.....	29
4. STABILITY AND DAMAGE ANALYSIS	35
4.1. Introduction	35
4.2. Failure modes of Rubble-mound breakwaters	36
4.3. Damage Evaluation Parameters	38
4.4. Techniques and Laboratorial Procedures for damage classification	40
4.5. Analysis of National and International Case Studies and Solutions.....	42
4.5.1. Portuguese Case Studies	43
4.4.1.1. The damaged breakwaters of the Port of Ericeira	43
4.4.1.2. The Port of Sines – report of the damages in the Sines breakwater	44
4.4.1.5. The north breakwater of Nazaré harbour	45
4.4.1.6. The Port of Leixões damaged breakwaters.....	46
4.5.2. International Case Studies: Rubble-mound Breakwaters Interventions	48
4.4.2.1. The Punta Lucero breakwater	48
4.4.2.2. Port of Long Beach – United States	49
4.5.3. Upgrade Solutions for Damaged Coastal Defence Structures	49
5. LABORATORY EXPERIMENTS WAVE OVERTOPPING.....	51
5.1. Introduction	51
5.2. Experimental Facility	52
5.2.1. Multidirectional wave basin	52
5.2.2. Wave Measurements	53
5.2.3. Overtopping measurement.....	54
5.2.4. Construction of the physical model	55
5.2.5. Test programme	60
5.3. Incident Wave Conditions.....	62
5.3.1. Test Series 1	62
5.3.2. Test Series 2	64
5.3.3. Test Series 3	65

5.3.4. Test Series 4	66
5.4. Analysis of the Mean Overtopping Discharges	67
5.5. Overtopping Rate and Freeboard.....	67
5.6. Normalized Overtopping Volumes	69
5.7. Wave-by-wave Overtopping Analysis.....	72
5.7.1. Measurements Procedure of Each wave Overtopping Event	72
5.7.2. Short-Crested Waves Tests	76
5.7.3. Wave-by-wave Overtopping Results	77
5.8. Distribution For Overtopping Wave Volumes.....	78
5.9. Sea Level Rise Effect on Wave Overtopping	80
5.10. Short-Crested Waves Influence on Wave Overtopping – Directional Spreading.....	85
5.11. Weibull Distribution and Probability Function – Data Analysis	91
5.11.1. Test Series 1	91
5.11.2. Test Series 2	95
5.11.3. Test Series 3	98
5.11.4. Test Series 4	102
5.12. NN-Overtopping – Data Modelling and Validation	107
6. LABORATORY EXPERIMENTS DAMAGE EVALUATION.....	113
6.1. Introduction	113
6.2. Damage Progression Analysis and Comparison Between Tests with the Same Initial Conditions	115
6.3. Analysis of the Effect of Sea Level Rise in the Damage Progression	117
7. CONCLUSIONS AND FUTURE DEVELOPMENTS ...	121
7.1. Conclusions	121
7.2. Future Developments	122
BIBLIOGRAPHIC REFERENCES	123
ANNEXE I.....	125
ANNEXE II.....	127

INDEX OF FIGURES

Figure 2.1 - Influence of different scenarios for chlorofluorocarbon volumes in the atmosphere, ...	4
Figure 2.2 - Glacial melt event, water flowing to the oceans (Noesises, 2009).....	5
Figure 2.3 - Global glacial mass balance through the years (Harwood, 2011).	5
Figure 2.4 - Projected future global sea level, present to 2100	6
Figure 2.5 - Sea level rise records of several regions of the world	6
Figure 2.6 - Short term coastline evolution.....	7
Figure 2.7 - Percentage of overtopping waves for rubble-mound breakwaters as a function of	9
Figure 3.1 - Wave overtopping at the marina of San Remo, Italy, ...	11
Figure 3.2 - Related parameters normally used in the characterization of wave overtopping of.....	12
Figure 3.3 - Wave propagation – longitudinal profile.	13
Figure 3.4 - Statistical distribution of ocean wave height. Significant wave height	14
Figure 3.5 - Typical cross section of a breaking wave and wave run-up.....	14
Figure 3.6 - Typical records of a wave overtopping measurement	16
Figure 3.7 - Wave probe measurement of an irregular wave overtopping, at the crest of the structure	17
Figure 3.8 - Comparison between Eurotop (2007) old formula	19
Figure 3.9 - Numerical modelling results using the hydrostatic NLSW WAF-TVD solver	20
Figure 3.10 - Time history related to wave overtopping load for various.....	22
Figure 3.11 - Typical cross-section of a dike-type structure for calculation.....	22
Figure 3.12 - Typical output of PC-Overtopping	23
Figure 3.13 - Configuration of a Neural Network (Gent et al., 2006).	24
Figure 3.14 - Setup of the parameters needed for NN-Overtopping	24
Figure 3.15 - Typical synthesis of the 15 selected dimensionless input parameters of the	25
Figure 3.16 - Typical output of the NN-Overtopping software calculator for a random	25
Figure 3.17 - Relation between overtopping rate and freeboard in rubble-mound	26
Figure 3.18 - Physical model of a rubble-mound breakwater, inside the wave tank of the	27
Figure 3.19 - Example of a wave tank (wave basin) of the Hydraulics Laboratory of the Faculty of the	28
Figure 3.20 - New Weibull shape factor, b, for rubble-mound structures, ...	30
Figure 3.21 - Curve of probability density function related to Weibull distribution with	31
Figure 3.22 - Mean overtopping discharge for 1:1.5 smooth and.....	32
Figure 3.23 - CLASH database for vertical walls.....	33
Figure 3.24 - Relative overtopping rate for a reshaping Icelandic type berm breakwater	34
Figure 4.1 - Breakwaters Dikkowita, Sri Lanka.	36

Figure 4.2 - Cross-section of a typical rubble mound breakwater	36
Figure 4.3 - Failure modes in a rubble-mound breakwater (Rock Manual, 2007).	37
Figure 4.4 - Hydraulic instabilities: a) rocking of unit during up and down-rush; b) rotation and subsequent down-slope displacement of unit during down-rush; c) rotation and subsequent up-slope displacement of unit during up-rush; d) sliding of several armour units (armour layer) during down-rush (CEM, 2011).	38
Figure 4.5 - The Port of Ericeira. Photograph from 1983, (Teixeira, 2012).	43
Figure 4.6 - Damage to the crown wall and dolos armour layer, taken immediately	44
Figure 4.7 - Dolos removed from the front of the seawall, broken dolos	44
Figure 4.8 - Evolution of the north breakwater's head: a) Before any significant damage;	45
Figure 4.9 - North breakwater head of Nazaré harbour in	45
Figure 4.10 - Layout of the north breakwater, and the different kind of armour	46
Figure 4.11 - North breakwater's head with submerged breakwater, ...	47
Figure 4.12 - Variation of the total number of replaced tetrapods	47
Figure 4.13 - Punta Lucero Breakwater: Port of Bilbao, ...	48
Figure 4.14 - Punta Lucero Breakwater: Port of Bilbao, original and reinforced	48
Figure 4.15 - Breakwater of the container Port of Long Beach, ...	49
Figure 4.16 - Situation before and after technical actions on damaged coastal.....	50
Figure 4.17 - Consideration of an extra armour layer on the front slope and a	50
Figure 4.18 - Protection of the revetment by a Detached breakwater on the	50
Figure 5.1 - Wave basin of SHRHA-FEUP. Physical model setup (plan view).	52
Figure 5.2 – SHRHA - Hydraulic Laboratory of Faculty of Engineering of the University of Porto.	52
Figure 5.3 - Wave probes layout in front of the rubble-mound breakwater.	53
Figure 5.4 - Water level variation during the tests, measured in one of the wave probes.....	53
Figure 5.5 - Six wave probes in a CERC6 arrangement.	54
Figure 5.6 - Water level variation inside the reservoir and the water level on the overtopping chute, measured	55
Figure 5.7 – Breakwater cross section: (a) current cross-section; (b) intermediate cross-section, ...	56
Figure 5.8 – Small reservoir, plan view	57
Figure 5.9 – Intermediate reservoir, plan view	57
Figure 5.10 – Large reservoir, plan view.	57
Figure 5.11 – (a) – Overtopping chute with 1 m width; (b) – Overtopping chute with 0.6 m width; (c) – Overtopping chute with 0.4 m width; (d) – Overtopping chute with 0.2 m width.....	58
Figure 5.12 – Physical model construction and preparation sequence in the wave basin of FEUP:	59
Figure 5.13 – Final model ready for the laboratory experiment.....	60

Figure 5.14 – Cumulative wave overtopping volumes of Series of tests 1.	63
Figure 5.15 – Cumulative wave overtopping volumes of the Series 2 of tests.	64
Figure 5.16 – Cumulative wave overtopping volumes of the serie 3 of tests.	65
Figure 5.17 – Wave overtopping volumes of the serie 4 of tests.	66
Figure 5.18 – Mean overtopping discharge for the 1:2 rubble-mound slope.	68
Figure 5.19 – Normalized wave overtopping volumes for the first three tests of test series S1.	69
Figure 5.20 – Normalized overtopping volume for different wave field conditions	70
Figure 5.21 – Normalized overtopping volume for different wave field conditions	70
Figure 5.22 – Normalized overtopping volume for different wave field conditions	71
Figure 5.23 – Wave-by-wave overtopping.	72
Figure 5.24 – Wave-by-wave overtopping.	73
Figure 5.25 – Zoom-In of Figure 5.15 from Test 3, Series 1.	74
Figure 5.26 – Wave-by-wave overtopping.	74
Figure 5.27 – Wave-by-wave overtopping.	75
Figure 5.28 – Wave-by-wave overtopping.	76
Figure 5.29 – Wave-by-wave overtopping.	77
Figure 5.30 – Distribution for overtopping wave volumes, ...	79
Figure 5.31 – Probability distribution function for wave overtopping volumes per wave, ...	79
Figure 5.32 – Cumulative overtopping volumes for two tests carried out for	80
Figure 5.33 – Flow thickness at the crest and water surface elevation with no sea level rise.	81
Figure 5.34 – Flow thickness at the crest and water surface elevation with sea level rise effects.	81
Figure 5.35 – Cumulative overtopping volumes for two tests carried out for	83
Figure 5.36 – Flow thickness at the crest and water surface elevation with no sea level rise.	83
Figure 5.37 – Flow thickness at the crest and water surface elevation with sea level rise consideration.	84
Figure 5.38 – Comparison of wave overtopping discharge between series, for test 1. Analysis of the 85	
Figure 5.39 – Comparison of wave overtopping discharge between series, for Test 2. Analysis of the	86
Figure 5.40 – Comparison of wave overtopping discharge between series, for Test 2. Analysis of the	87
Figure 5.41 – Comparison of wave overtopping discharge between series, for Test 3. Analysis of the	88
Figure 5.42 – Comparison of wave overtopping discharge between series, for Test 3. Analysis of the	88

Figure 5.43 – Comparison of wave overtopping discharge between series, for Test 4, and for the total test duration. Analysis of the short-crested waves influence in wave overtopping.....	89
Figure 5.44 – Comparison of wave overtopping discharge between series, for Test 4. Analysis of the	90
Figure 5.45 – Weibull distribution function, of overtopping values for Test 1.	91
Figure 5.46 – Weibull distribution function, of overtopping values for Test 2.	92
Figure 5.47 – Probability density function (pdf), for Test 2.	92
Figure 5.48 – Weibull distribution function of overtopping values, for Test 3.	93
Figure 5.49 – Probability density function (pdf), for Test 3.	93
Figure 5.50 – Weibull distribution function of overtopping values, for Test 4.	94
Figure 5.51 – Probability density function (pdf), for Test 4.	94
Figure 5.52 – Weibull distribution function of overtopping values, for Test 1.	95
Figure 5.53 – Probability density function (pdf), for Test 1.	95
Figure 5.54 – Weibull distribution function of overtopping values, for Test 2.	96
Figure 5.55 – Probability density function (pdf), for Test 2.	96
Figure 5.56 – Weibull distribution function of overtopping values, for Test 3.	97
Figure 5.57 – Probability density function (pdf), for Test 3.	97
Figure 5.58 – Weibull distribution function of overtopping values, for Test 1.	98
Figure 5.59 – Probability density function (pdf), for Test 1.	98
Figure 5.60 – Weibull distribution function of overtopping values, for Test 2.	99
Figure 5.61 – Probability density function (pdf), for Test 2.	99
Figure 5.62 – Weibull distribution function of overtopping values, for Test 3.	100
Figure 5.63 – Probability density function (pdf), for Test 3.	100
Figure 5.64 – Weibull distribution function of overtopping values, for Test 4.	101
Figure 5.65 – Probability density function (pdf), for Test 4.	101
Figure 5.66 – Weibull distribution function of overtopping values, for Test 1.	102
Figure 5.67 – Probability density function (pdf), for Test 1.	102
Figure 5.68 – Weibull distribution function of overtopping values, for Test 2.	103
Figure 5.69 – Probability density function (pdf), for Test 2.	103
Figure 5.70 – Weibull distribution function of overtopping values, for Test 3.	104
Figure 5.71 – Probability density function (pdf), for Test 3.	104
Figure 5.72 – Weibull distribution function of overtopping values, for Test 4.	105
Figure 5.73 – Probability density function (pdf), for Test 4.	105
Figure 5.74 – Weibull shape factor, b , for rubble-mound structures, spanning a large range.....	107

Figure 5.75 – Related parameters used in NN-Overtopping prediction tool (Deltares).	108
Figure 5.76 – Comparison between experimental results and the NN-overtopping prediction.	111
Figure 6.1 – Bathymetric survey obtained from stereophotogrametry	113
Figure 6.2 – Damage progression analysis, description and comparison between Test 1 of Series 1 and 3.	115
Figure 6.3 – Damage progression description and comparison, with consideration of the sea level rise.....	116
Figure 6.4 – Damage progression analysis and comparison between.....	117
Figure 6.5 – Damage progression analysis and comparison between.....	118
Figure 6.6 – Total damage description and comparison between Serie 1 and Serie 3.	119

INDEX OF TABLES

Table 3.1 – Limits for wave overtopping for structural design of breakwaters, seawalls, dikes and dams, (Eurotop II, 2016).	8
Table 3.2 – Range of values of wave heights, for different types of environmental scenarios.....	13
and structures (Eurotop II, 2016).....	13
Table 3.3 – Wave steepness: relation to wave's characteristics (Eurotop, 2016).	15
Table 3.4 – Wave overtopping discharge Classification (Eurotop, 2007).....	17
Table 3.5 – Regular scaling of factors, using the similarity of Froude.	29
Table 3.6 – Values for roughness factor γ_f for permeable rubble-mound structures.....	33
with slope of 1:1.5, (Eurotop II, 2016).	33
Table 4.1 – Damage Classification of the damage parameters D, Nod and S.	41
Table 4.2 – Damage level by D for two-layer armor.	42
Table 4.3 – Damage level by Nod for two-layer armor (Van der Meer 1988).....	42
Table 4.4 – Damage level by S for two-layer armor (van der Meer 1988).	42
Table 5.1 – Dimensions of the reservoirs used on the laboratorial experiments.....	57
Table 5.2 – Number of Dn50 associated to each overtopping chutes width.	59
Table 5.3 – Test program of the laboratorial experiments.....	61
Table 5.4 – Initial generation conditions. Range of values.	61
Table 5.5 – Description of each series purpose.	62
Table 5.6 – Incident wave conditions and other input parameters for Serie 1.	62
Table 5.7 – Dimension of the overtopping chute and type of reservoir used for each test of Serie 1.	63
Table 5.8 - Incident wave conditions and other input parameters for Serie 2.	64
Table 5.9 – Dimension of the overtopping chutes and type of reservoir used for each test of Serie 2.	64
Table 5.10 - Incident wave conditions and other input parameters for Serie 3.	65
Table 5.11 – Dimension of the overtopping chute and type of reservoir used for each test of Serie 3.	65
Table 5.12 - Incident wave conditions and other input parameters for Serie 4.	66
Table 5.13 – Dimension of the overtopping chute and type of reservoir used for each test of Serie 4.	66
Table 5.14 – Mean overtopping flow rate between tests and experiments.	67
Table 5.15 – Results of the wave-by-wave overtopping analysis, for S1-R1.	78
Table 5.16 – Characterization of the tests S2R2 and S3R3 used to assess the sea level rise effects.	80
Table 5.17 – Influence of the sea level rise in the number of waves and wave overtopping.	82
Table 5.18 – Generation conditions and results for two tests with a variation of water level.	82
Table 5.19 - Influence of the sea level rise in the number of waves and wave overtopping.	84
Table 5.20 – Experimental results from wave-by-wave overtopping analysis for Test 1.....	86

Influence of short-crested waves in wave overtopping.....	86
Table 5.21 – Experimental results from wave-by-wave overtopping analysis for Test 2.....	87
Influence of short-crested waves in wave overtopping.....	87
Table 5.22 – Experimental results from wave-by-wave overtopping analysis for Test 3.....	89
Influence of short-crested waves in wave overtopping.....	89
Table 5.23 – Experimental results from wave-by-wave overtopping analysis for Test 4.....	90
Influence of short-crested waves in wave overtopping.....	90
Table 5.24 – Weibull distribution parameters of all the tests.....	106
Table 5.25 – Input parameters values, for NN-Overtopping prediction.	109
Table 5.26 – NN-Overtopping prediction values for mean-overtopping discharge and different quantiles.	110
Table 6.1 – Initial conditions for each test, for damage analysis.	114
Table 6.2 – Comparisons definition for damage analysis.....	114

SYMBOLS, ACRONYMS AND ABBREVIATIONS

A_c	$[m^2]$	armour crest freeboard of the structure
a	$[-]$	scale factor for Weibull distribution and probability density function
A_e	$[m^2]$	eroded area of a slope
b	$[-]$	shape factor for Weibull distribution and probability density function
B_t	$[m]$	width of the berm
c	$[m/s]$	wave celerity at the structure toe
C_r	$[-]$	reduction factor for a wide rubble-mound crest
$\cotg(\alpha_d)$	$[-]$	angle of the down slope
$\cotg(\alpha_u)$	$[-]$	angle of the upper slope
d	$[m]$	water depth
D	$[m]$	weighted average of D_d and D_u
D_d	$[m]$	average element size at the down slope
D_u	$[m]$	average element size at the upper slope
D_n	$[m]$	nominal diameter of concrete armour unit
g	$[m/s^2]$	acceleration due to gravity ($= 9.81$)
G_c	$[m]$	width of structure crest
h	$[m]$	water depth in front of the toe of the structure
h_b	$[m]$	water depth on the berm (negative means berm is above SWL)
h_t	$[m]$	water depth on the toe of the structure
H	$[m]$	wave height
H_{m0}	$[m]$	estimate of significant wave height from spectral analysis
$H_{m0,t}$	$[m]$	H_{m0} determined at the toe of the structure
H_s	$[m]$	significant wave height
L	$[m]$	wave length measured in the direction of wave propagation
L_0	$[m]$	deep water wave length based on $T_{m-1,0}$
$L_{m-1,0}$	$[m]$	spectral wave length in deep water
L_{om}	$[m]$	mean wave length in deep water
m	$[-]$	slope of the foreshore
m_n	$[-]$	moment of spectral density
N_w	$[-]$	number of incident waves
N_{ow}	$[-]$	number of overtopping waves
P_{ow}	$[-]$	probability of overtopping per wave $= N_{ow}/N_w$

q	$[m^3/s/m]$	mean overtopping discharge per meter of structure width
R_c	$[m]$	crest freeboard of structure
$R_{u2\%}$	$[m]$	run-up level exceeded by 2% of the incident waves
s	$[-]$	wave steepness
$s_{m-1,0}$	$[-]$	wave steepness with L_o , based on $T_{m-1,0}$
s_{om}	$[-]$	wave steepness with L_o , based on T_m
s_p	$[-]$	wave steepness defined for the local water depth
SWL	$[m]$	still water level
T	$[s]$	wave period
T_m	$[s]$	mean wave period
$T_{m0,1}$	$[s]$	mean wave period
T_p	$[s]$	spectral peak wave period
V	$[m^3]$	volume of overtopping wave per unit crest width
V_{max}	$[m^3]$	maximum individual overtopping wave volume per unit crest width
α	$[^\circ]$	angle between overall structure slope and horizontal
β	$[^\circ]$	angle of wave attack
ρ_s	$[kg/m^3]$	mass density of armour units
γ_b	$[-]$	influence factor for a berm
γ_f	$[-]$	influence factor for the permeability and roughness of or on the slope
γ_v	$[-]$	influence factor for a vertical wall on the slope
γ_{BB}	$[-]$	influence factor for a berm breakwater
γ_β	$[-]$	influence factor for the oblique wave attack
ξ_o	$[-]$	breaker parameter based on s_o
ξ_{om}	$[-]$	breaker parameter based on s_{om}
$\xi_{m-1,0}$	$[-]$	breaker parameter based on $s_{m-1,0}$
Γ	$[-]$	mathematical gamma function

1. INTRODUCTION

1.1. GLOBAL INTRODUCTION

This work has the main goal of studying the damage progression and the study of wave overtopping phenomena on rubble-mound structures.

Also a part of this work includes these analyses with a consideration of climate change effects, short-crested waves, and variation of the wave heights, in order to analyse their influence in wave overtopping, and damage progression (in some cases).

In order to enable these studies, applications of advanced laboratory techniques are applied in this work, using physical models with proper scale. After the experimental work, the results are studied and it is made a data processing. This work also incorporates a component in work environment, performed in APDL – Administration of Ports of Douro, Leixões and Viana do Castelo, SA.

1.2. RESEARCH OBJECTIVES AND STRUCTURE

The thesis is subdivided in seven chapters. The first chapter addresses the objectives description and theme contextualization. Subsequently, there is the definition of the state of the art from chapter 2 to chapter 4, for the following objectives: laboratorial techniques and procedures for the damage and wave overtopping evaluation in coastal protection structures, climate change effects on the function and stability of coastal protection structures and adaptive measures to attend on the climate change effects.

Chapter 4 also addresses the analysis of national and international case studies.

Chapter 5 approaches the development of a laboratory experiment and data processing on wave overtopping, and the analysis of the climate change effects.

Chapter 6 addresses the laboratory experiment related to the analysis of damage progression on rubble-mound structures.

Chapter 7 address the conclusions and future developments related to this dissertation.

At the end of the dissertation, are the annexes with tables and graphics related to the laboratorial experiment. Also the work environment component in the Port of Leixões is addressed in the annexes.

2. CLIMATE CHANGE EFFECTS ON COASTAL PROTECTION STRUCTURES

2.1. CLIMATE CHANGE: A FIRST APPROACH

Coastal and port structures are constantly submitted to various types of loads from the maritime environment. Those loads can have different magnitudes depending on the external conditions and environment, such as wind, wave conditions, storm events, among others. Different places on earth have different conditions and actions on coastal structures. Furthermore, loads from the maritime environment can be increased by the climate change effects, for instance, the increase of the sea level rise, and the increase of the frequency and intensity of sea storms. Those effects influence the stability and wave overtopping of coastal protection structures (e.g. rubble-mound breakwaters).

In this context, it is necessary to carry out the analysis and characterization of the climate change effects on those structures, using physical modelling studies, advanced experimental techniques, and to promote the application of adaptive measures on existing structures (when needed).

Before entering the topic of overtopping and damage assessment on rubble-mound structures, it is important to understand how climate change issues were triggered over the years, around the world.

Mostly over the last two decades, climate change has become the most severe and addressed global issue, in the entire world (The Huffington post, 2014). Since the industrial age, until nowadays, that scientists from Met Office in Britain estimate that the average global temperature of the planet has increased 1°C. This temperature changes have negative effects already seen, such as variations in rainfall patterns, melting glaciers, sea level rise and population mass relocation.

NASA Global Climate Change, United States Environmental Protection Agency, IPCC (Intergovernmental Panel on Climate Change) and the Society of Environmental Journalists are other

examples of the vast number of organizations threatening and approaching this global environmental, social and political issue, nowadays.

The sea level rise and the changing weather in terms of more and stronger storm events may have a considerable influence on the damage and the wave overtopping magnitude of coastal protection structures, for instance, rubble-mound breakwaters.

2.1.1. GREENHOUSE GASES AND GLOBAL WARMING

It all began with a global increase of carbon dioxide concentration in the atmosphere. Nowadays, the climate change effects are considered on the entire world as a real and severe matter, being addressed on the scientific reports, political conferences, engineering summits, etc.

This increasing volumes of chlorofluorocarbons in the atmosphere, is causing Global Warming, this means an increase in average global temperatures. A correlation between carbon dioxide concentration in the atmosphere and the average global surface temperature, is presented by Figure 2.1 (PIK, 2012).

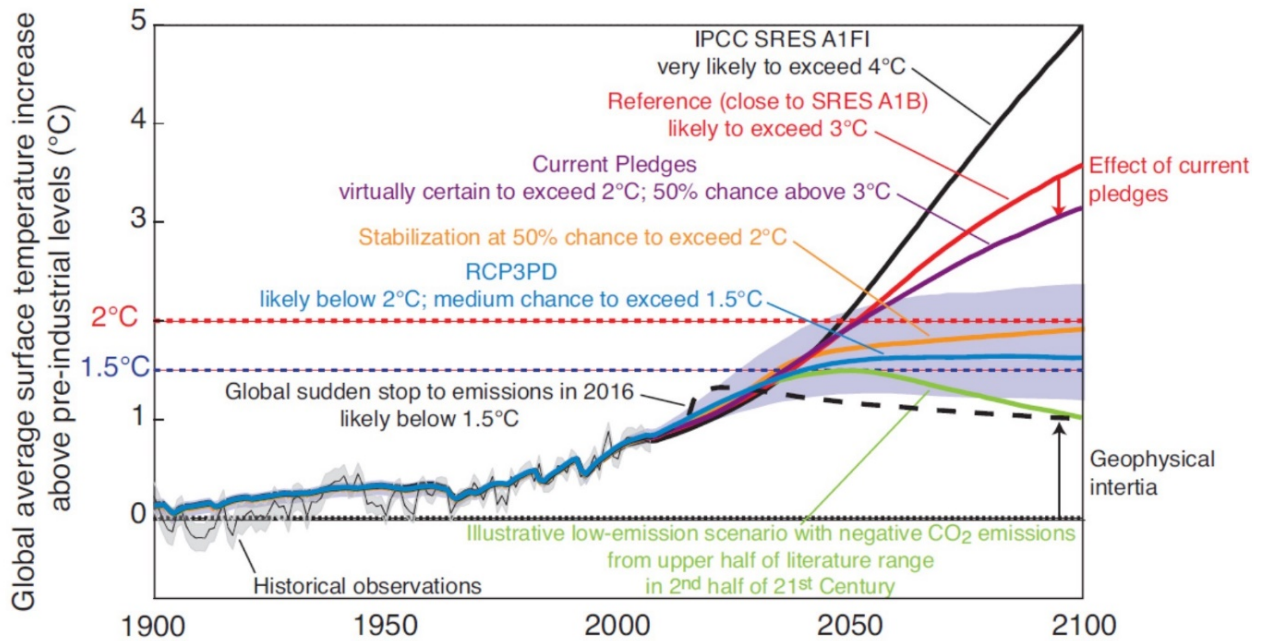


Figure 2.1 - Influence of different scenarios for chlorofluorocarbon volumes in the atmosphere, on the global average surface temperature (PIK, 2012).

These events generate global changes in the world, such as the decreasing global glacial mass, changes in rainfall patterns, sea level rise, and extreme temperature variations along the seasons, among other examples.

2.1.2. DECREASING GLOBAL GLACIAL MASS AND SEA LEVEL RISE

Other factor resulting from Global Warming is the decreasing global glacial mass. Obviously the stream of water resulting from the thaw of the Arctic, Antarctica, and other massive glacial areas, moves to the large oceans, Figure 2.2 (Noesises, 2009).



Figure 2.2 - Glacial melt event, water flowing to the oceans (Noesises, 2009).

In Figure 2.3, is presented the mass of glacial ice melted each year from 1960 to 2003. For the amount of ice values represented is used a scale given in units of 100 billion tons of ice per year (Richard Harwood, 2011).

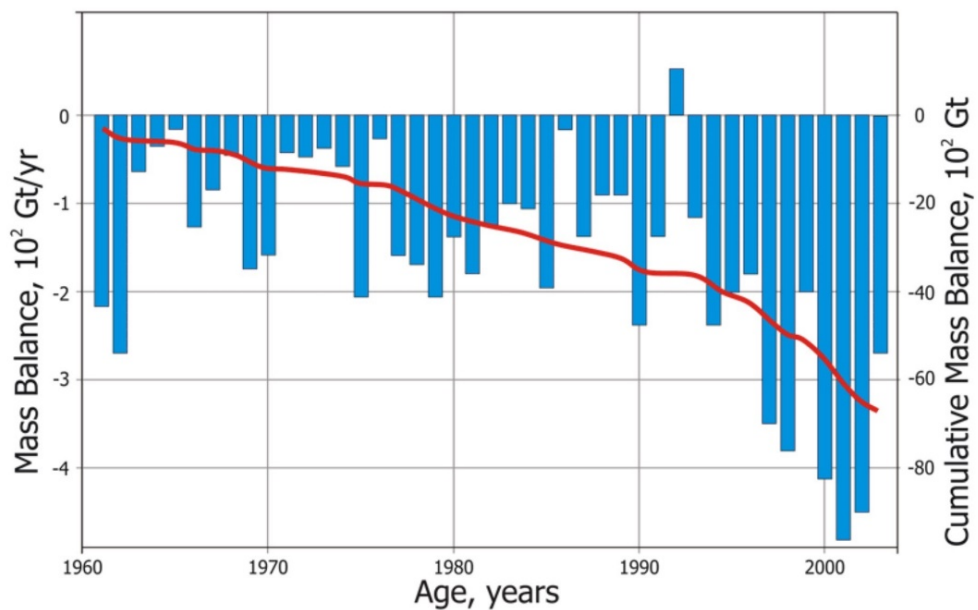


Figure 2.3 - Global glacial mass balance through the years (Harwood, 2011).

The increasing storm events, precipitation volumes from the changes in the rainfall patterns, and the decreasing global glacial mass, leads to a critical scenario for coastlines and structures defending them, the sea level rise. Many studies and predictions for sea level rise have been made. Figure 2.4 presents a projected future line for the global sea level from year 1900 to 2100. Sea levels are given as deviation from the 1990 level (Richard Harwood, 2011).

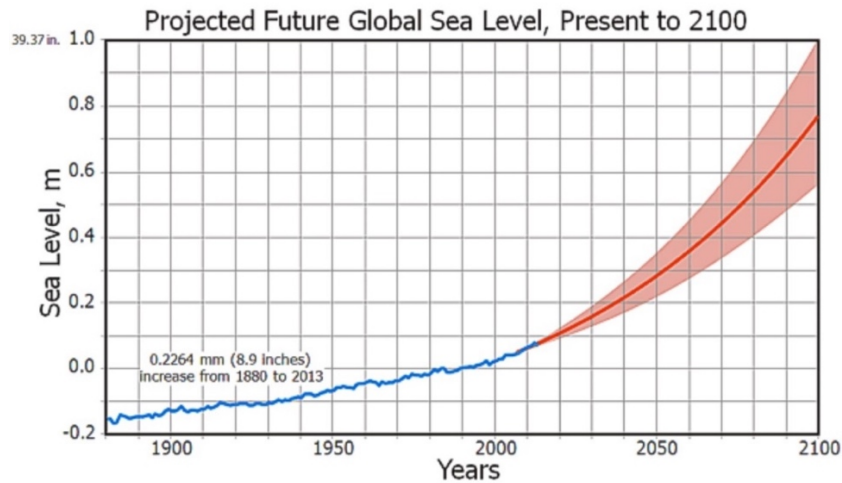


Figure 2.4 - Projected future global sea level, present to 2100
(Harwood, 2011).

Figure 2.5 presents sea level rise evolution from 1960 to the present, for several regions of the world (Santos, 2016).

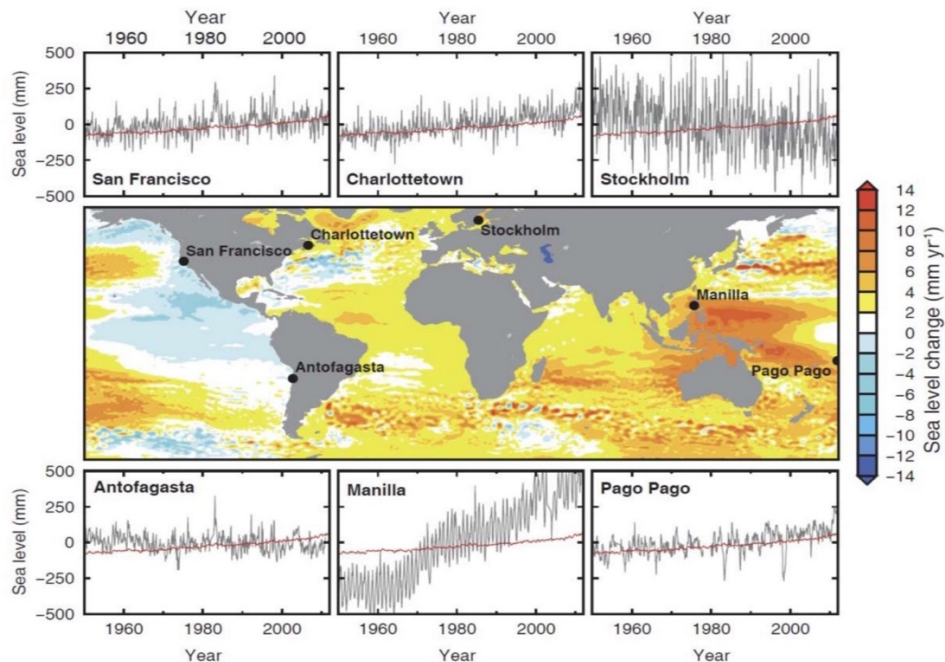


Figure 2.5 - Sea level rise records of several regions of the world
(Santos, 2016).

It has been estimated by a report from Intergovernmental Panel on Climate Change (IPCC, 2007), a mean value of 0.35 m for the sea level rise as a global average, in the range 0.18 – 0.59 m, by the end of the 21st century. And these estimated values are excluding future rapid dynamical changes in ice flow, which further increased the sea level rise (IPCC, 2007).

Another effect of climate change is an increase on the average wind velocities, which in turn causes the increase of the significant wave heights. About this parameter it is predicted for the North Sea an increase for significant wave height, of 2% and 5% for the years 2050 and 2100 respectively (STOWASUS, 2001).

2.1.3. COASTLINE RETREAT

In terms of coastal protection structures design and construction, it is necessary to consider new case scenarios with new environmental conditions and phenomena caused by the climate change effects, for instance, the sea level rise and the increase of the wave heights in general, and what consequences comes from this on breakwater's stability and wave overtopping events.

The most visible effect of climate change on coastal areas is the coastline retreat. This issue is related to the sea level rise phenomena.

Examples from all the world can be illustrated to sustain this fact, Figure 2.7.



Figure 2.6 - Short term coastline evolution.
Maracaipe. Brazil. (Macedo *et al.*, 2012).

Studies have been made in order to understand how these climate change effects influence coastal protection structures, and how those consequences can be evaluated for further solutions to attend on those effects.

2.2. EFFECTS ON THE FUNCTION AND STABILITY OF COASTAL PROTECTION STRUCTURES

For a proper comprehension of the influence of climate change effects on coastal protection structures, it is necessary to understand how those increasing external loadings interact with the structures. Burcharth *et al.*, 2014, established cause-effect relationships that connects climate change to the necessity of upgrading interventions on coastal protection structures. In the process of finding effective solutions and measures in the short and long term, it is necessary to understand how these structures are designed and built, and with what purpose. Concerning the design of coastal protection structures, due to the fact that they last a long time after their design and construction, and taking into account the reality of a continuing sea level rise, it is necessary to include a certain value of sea level rise in their structural design (and in a few areas of the world, a sea level decrease).

Normally, countries define a certain value of sea level rise that has to be considered in the design of port, coastal or flood protection structures. Concerning the return period with the consideration of Sea Level Rise, it may take different values, taking into account the sea level rise variation towards the future. Furthermore, each structure type has its complexity in terms of structural design, stability and function (Eurotop II, 2016).

For the different protection structures, fixed limits for wave overtopping were defined to be used in their structural design process. This means that for a climate change effect such as the already mentioned sea level rise, failure may occur if the wave heights and other parameters of the maritime environment become higher than the designed ones. Table 3.1 presents the limits given by Eurotop II (2016) for wave overtopping to be considered in the structural design of breakwaters, seawalls, dikes and dams.

Table 3.1 – Limits for wave overtopping for structural design of breakwaters, seawalls, dikes and dams, (Eurotop II, 2016).

Hazard type and reason	Mean discharge q (L/s per meter)	Maximum volume $V_{\max}$ (L/m)
Rubble-mound breakwaters; $H_{m0} > 5$ m; no damage.	1	2000 – 3000
Rubble-mound breakwaters; $H_{m0} > 5$ m; rear side designed for wave overtopping.	5 – 10	10 000 – 20 000
Grass covered crest and landward slope; maintained and closed grass cover; $H_{m0} = 1 – 3$ m	5	2000 – 3000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; $H_{m0} = 0.5 – 3$ m	0.1	500
Grass covered crest and landward slope; $H_{m0} < 1$ m	5 – 10	500
Grass covered crest and landward slope; $H_{m0} < 0.3$ m	No limit	No limit

More limits are presented in Eurotop II (2016) for wave overtopping, concerning areas with high concentration of people and vehicles. Furthermore, in situations where overtopping events cannot happen in any case, the zero overtopping is used (Eurotop II, 2016).

In respect to rubble-mound structures, many factors are taken into account in the structural design. One design variable that may be influenced by the climate change effects is the crest height of rubble-mound breakwaters. For a sea level rise scenario, an increase on wave overtopping may be expected, leading to a necessity of modifying the crest height of the structure.

Sea level rise causes an increase in wave overtopping volume and in the mean overtopping discharge. For a correct re-design of the structure in order to respond to those increased loadings, some modifications can be made in the design of structural elements, for instance, the already mentioned crest height of the structure. Figure 2.8 presents the relationship between the percentage of overtopping waves and the dimensionless crest height. For a higher design crest height, there is a smaller percentage of overtopping waves, and in the case of sea level rise, where each wave volume increases, the reduction of the number of overtopping waves is a goal to pursuit, and possible by increasing the crest height of the structure, compensating the increase of the crest freeboard.

This represents an example of an adaptive measure to attend on the climate change effects in coastal protection structures (Eurotop II, 2016).

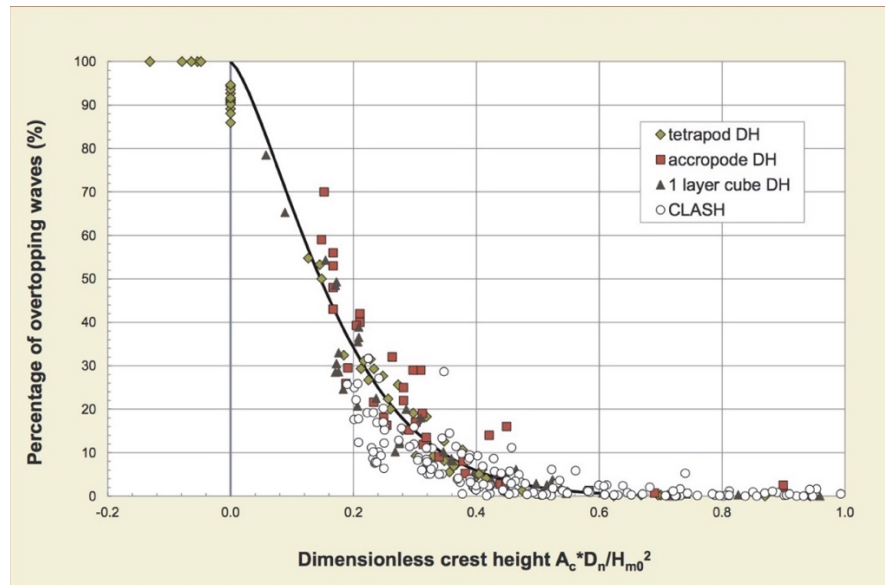


Figure 2.7 - Percentage of overtopping waves for rubble-mound breakwaters as a function of relative crest height and armour size (Eurotop II, 2016).

Concerning the climate change effects, and the new case scenarios with constant increasing loadings from the maritime environment, there is a need for re-designing and structural modification of the previous coastal protection structures, that once were sufficient to attend on the solicitations and conditions that were valid in the past, but are not sufficient, with the new sea state conditions influenced by the climate change effects.

There are many adaptive measures to attend on the Climate change effects on coastal protection structures. This matter is addressed in Chapter 4.

3. WAVE OVERTOPPING

3.1. INTRODUCTION

Over the years, in the study and design of coastal protection structures, the main subjects that have been evaluated in laboratory were the structure stability, the armour layer damage quantification and the evaluation of wave overtopping discharges and volumes.

The Eurotop (2007) and the Eurotop II (2016), present some examples that address those subjects.

Figure 3.1 presents an example of a wave-overtopping event, caught on camera, at the marina of San Remo, in Italy.



Figure 3.1 - Wave overtopping at the marina of San Remo, Italy, damaging small boats (Eurotop II, 2016).

Wave overtopping is an event that occurs when waves intersect a submerged reef or structure. Wave overtopping also occurs when waves meet an emerged reef or structure as long as the highest point of the structure is lower than the run-up. During wave overtopping events, two phenomena important to the coastal processes occur, namely, wave transmission and the crossing water over the structure. Wave transmission is defined as the waves generated on the lee side of a breakwater or other coastal structure, due to wave action on its immediate seaward side. After the impact in a structure, waves are reflected on the opposite direction of their propagation course.

3.2. DEFINITION AND DESCRIPTION

3.2.1. PARAMETERS RELATED TO WAVE OVERTOPPING

The parameters normally used in the characterization of wave overtopping of coastal structures, such as rubble-mound breakwaters (Figure 3.2) are.

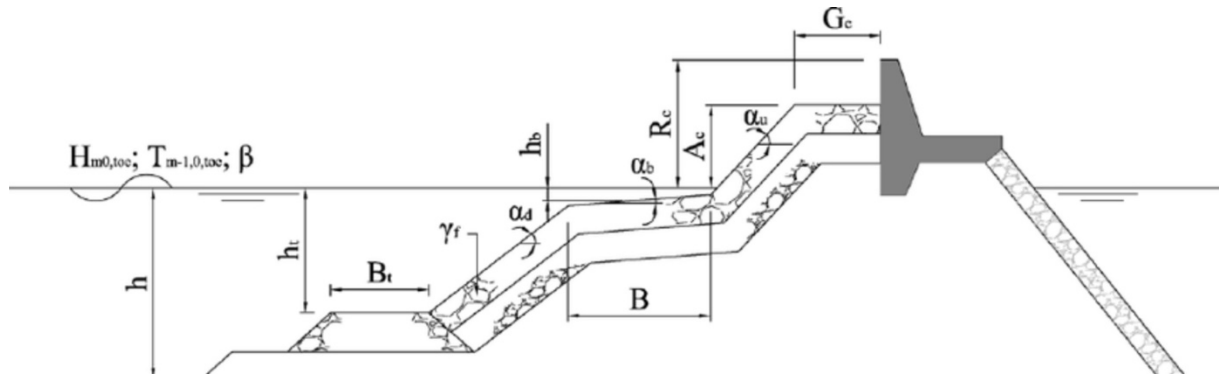


Figure 3.2 - Related parameters normally used in the characterization of wave overtopping of rubble-mound structures, (Karthika Pillai *et al.*, 2016).

- H_s : significant wave height;
- H_{m0} : spectral wave height;
- T_p : peak period;
- R_c : Crest freeboard;
- A_c : armour crest freeboard of the structure;
- G_c : width of structures crest;
- B_t : width of the berm;
- α : Slope angle of the armour layer;
- γ_f : roughness coefficient;
- h : water depth;
- $T_{m-1,0}$: spectral wave period.

3.2.1.1. Wave period

Various characteristic wave periods can be defined based on the wave spectra or time-domain zero-crossing techniques: namely the peak wave period, T_p , and the average wave period, T_m , and the significant wave period, $T_{1/3}$.

The conventional type is the peak period wherein the peak of the wave spectrum is defined. The second period type, T_m (average period), is calculated from the wave record preferably, but also can be determined from the spectrum of the wave. Finally, the significant period $T_{1/3}$, represents the average of the highest 1/3 of the waves (Eurotop, 2007).

3.2.1.2. Wave height

The wave height, is the main key parameter used in wave's studies and descriptions, and can be defined in different contexts. For instance, in respect to the wave run-up and overtopping formulae, the wave height represents the incident significant wave height, H_{m0} , at the toe of the structure, most known as the spectral wave height, which numerical expression is given by $H_{m0} = 4 \cdot (m_0)^{1/2}$. This equation can only be used after the determination of the wave spectrum.

Table 3.2 presents the different range of values of wave heights for different environmental scenarios and structures, Eurotop II (2016).

Table 3.2 – Range of values of wave heights, for different types of environmental scenarios and structures (Eurotop II, 2016).

Wave height – Range of values (m)	Type of environmental scenarios and structures
$H_{m0} \leq 1 \text{ m}$	Rivers, wide canals and small lakes. Embankments covered with grass.
$H_{m0} = 1 - 3 \text{ m}$	Sheltered seashores and large lakes. Embankments, seawalls with the wave attack zone protected by rock, concrete units or block revetments. Grass covered crest or protected promenades and boulevards.
$H_{m0} \geq 3 - 5 \text{ m}$	High waves and large water depths (above 10 meters) near the structure. Breakwaters, seawalls at land reclamations.

In another context, the wave height is defined also as the average of the highest third of the waves, $H_{1/3}$ (Eurotop, 2016). Figure 3.3 presents the longitudinal profile of wave propagation in a free surface of the sea.

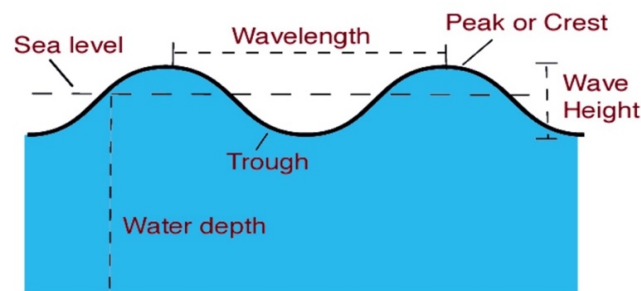


Figure 3.3 - Wave propagation – longitudinal profile.

The wave, itself, is not sufficient to predict the occurrence or non-occurrence of wave overtopping, because even when this parameter is smaller than the structure's crest level, the wave overtopping is still possible to occur, due to the wave run-up occurrence on the slope of the coastal protection structure. For that reason, it is necessary to study and relate more wave parameters and phenomena, such as the wave run-up.

3.2.1.3. Significant wave height

The significant wave height (H_s) is defined as the mean wave height of the highest third of the waves ($H_{1/3}$), and it is used in the design of coastal protection structures. Figure 3.4 presents the statistical distribution of ocean wave heights (Wikipedia, 2015).

This significant wave height is usually measured from a wave spectrum.

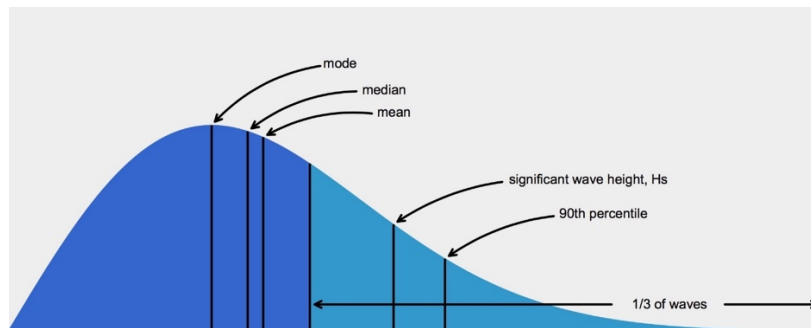


Figure 3.4 - Statistical distribution of ocean wave height. Significant wave height illustration (Wikiwand, 2015).

3.2.1.4. Wave run-up height

The wave run-up takes place when waves start to go up on the surface of a seawall for example, other type of structure or even in a beach surface slope. The wave run-up can reach different levels+ on the seawall, depending on the slope's angle, on the roughness of its surface and on the wave energy itself. In order to describe wave run-up, Figure 3.5 presents a typical cross section of a breaking wave and wave run-up event in a beach slope.

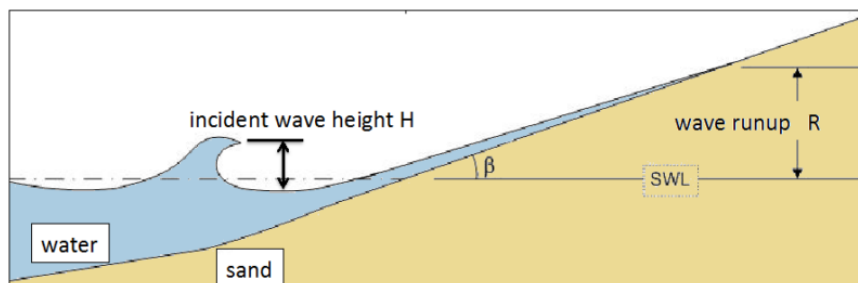


Figure 3.5 - Typical cross section of a breaking wave and wave run-up in a beach slope.

As it is illustrated in Figure 3.5, the wave run-up height is defined by the vertical distance R , known as $R_{u2\%}$, that represents the wave run-up level, which is measured vertically between the still water level and the end of the run-up on the slope of the structure exceeded by 2% of the number of incident waves. The amount of waves exceeding this value is considered as incoming waves and not as run-up waves, due to its size (Eurotop, 2007).

It is suggested on the Eurotop (2007), that on smooth slopes, the wave run-up level should be determined by the level where the water tongue has less than 2 cm of thickness, and that thinner layers thrown onto the surface of the slope by the wind strength, are not considered as wave run-up, since they contain a very small amount of water and cannot be properly reproduced in small scale model studies.

A thin layer in a run-up tongue is hard to measure accurately. The limit of water layer thickness in a smooth slope in model studies is about 2 mm. This means that prototype waves have a layer depth rounded value of 2 cm, depending on the scale. This thickness values for running-up water tongues incorporates very small quantities of water, and for that reason, it is just considered as run-up waves the ones with less than 2 cm of thickness (Eurotop, 2007).

Returning to the wave overtopping description, the water that passes over the crest of the structure is nominated “green water”. This is the conventional wave overtopping event.

Other form of overtopping takes place when the waves impact on the seaward surface, creating severe volumes of splash water, contributing as well, for overtopping volume quantifications.

Another form of overtopping, but not considered in overtopping volumes due to its small proportion in comparison with the total overtopping discharge volume, is the water in the form of spray caused by the wind strength that pushes waves into the seaward direction (EuroTop, 2007).

3.2.1.5. Wave steepness

The offshore wave steepness is given by the following relation (Eurotop, 2016).

$$s_0 = \frac{H_{m0}}{L_0} \quad (3.1.)$$

where H_{m0} represents the spectral significant wave height, and L_0 is the deep water wave length based on $T_{m-1,0} = gT_{m-1,0}^2/2\pi$.

This wave parameter represents the ratio between the significant wave height from spectral analysis and the wavelength, and it is directly related to the seabed topography, and it gives information on wave's history and its intrinsic characteristics. For instance, Table 3.3 presents typical wave steepness values for sea and swell waves (Eurotop, 2016).

Table 3.3 – Wave steepness: relation to wave's characteristics (Eurotop, 2016).

Wave steepness (s_0)	Sea type	Wave period type
$s_0 = 0.01$	typical swell sea	long period waves
$0.04 < s_0 < 0.06$	typical wind sea	short period waves

3.2.2. WAVE OVERTOPPING DISCHARGE AND WAVE OVERTOPPING VOLUME

3.2.2.1. Wave overtopping discharge – definition and description

One of the forms that wave overtopping discharge can be expressed, is by the mean discharge per linear meter of structure width (Parameter= q ; Units= $\text{m}^3/\text{s}/\text{m}$). This mean overtopping discharge can be calculated by dividing the total overtopped volume per meter width of the structure, for the total time extension of the measurement.

This variable is very irregular in time and magnitude, because during the period of time overtopping occurs, there is no constant discharge over the crest of the structure due to the irregularity and unpredictability of waves.

Wave overtopping is usually measured using wave probes placed in different spots of the wave basin, each one for different type of measurements.

For instance, Figure 3.6 presents a 200 s stretch of a typical time series, from overtopping discharge measurements, namely a cumulative overtopping curve on the upper graph (wave probe placed inside the reservoir), and flow depth curve in the lower graph from the wave probe placed in front of the structure (Eurotop, 2007).

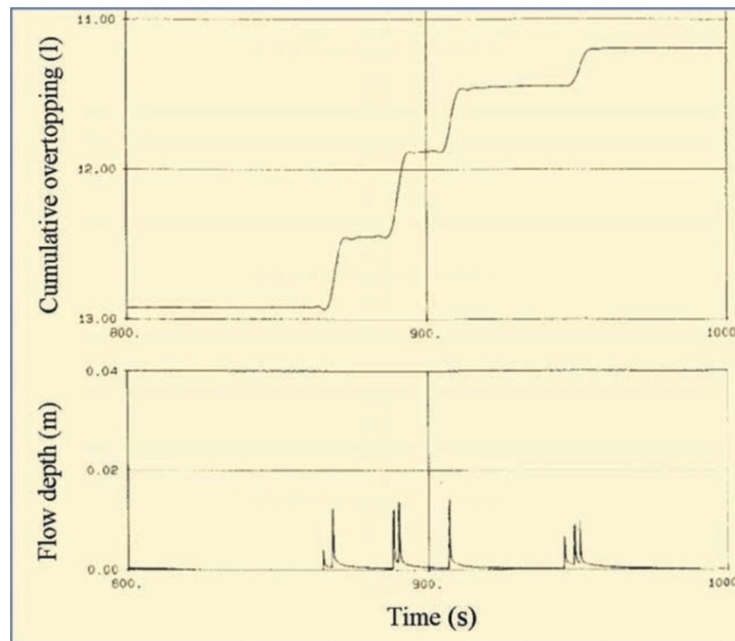


Figure 3.6 - Typical records of a wave overtopping measurement (Eurotop, 2007).

In Figure 3.6, in the lower graph, there is an evident illustration of wave overtopping irregularity in a short period of time, which explains the difficulty in the process of individual measurements of the volumes of each wave. On the upper graphic, it is illustrated the cumulative overtopping discharges measured in the overtopping tank used in this test, in the period of time of 200 s.

The graphic shows the overtopping occurrences as singular events, with individual waves and individual volumes, despite the cases when two different waves are in one wave group (Eurotop, 2007).

Another form of representation of wave overtopping events measured by wave probes, is presented in Figure 3.7 (Eurotop, 2016).

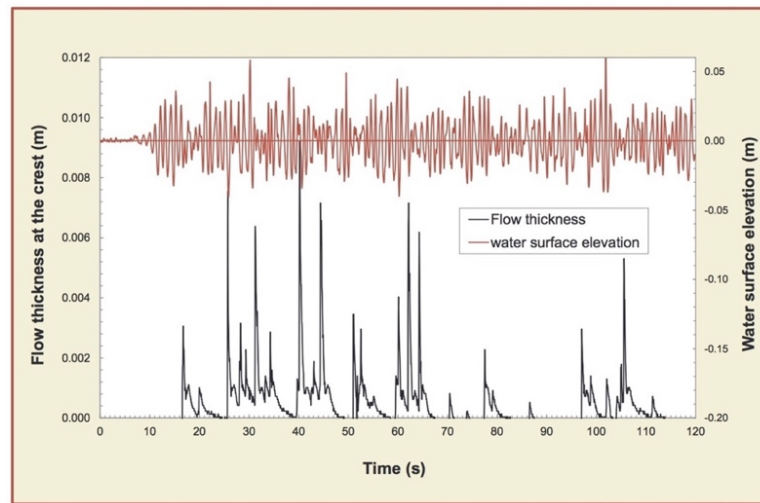


Figure 3.7 - Wave probe measurement of an irregular wave overtopping, at the crest of the structure (flow thickness) and the wave record in front of the structure (Eurotop, 2016).

Table 3.4 presents the mean overtopping discharge per meter of structure width classification (Eurotop, 2007).

Table 3.4 – Wave overtopping discharge Classification (Eurotop, 2007).

Mean overtopping discharge - q	Classification
$q < 0.1$ L/s per meter	Insignificant discharge in terms of crest and rear of structure's resistance.
$q = 1$ L/s per meter	Risk of small erosions on crest and inner slopes of the structure.
$q = 10$ L/s per meter	Significant overtopping discharge for dikes and embankments and some overtopping for rubble mound breakwaters.
$q = 100$ L/s per meter	Large quantities of overtopping and necessity for crest and inner slopes of dikes protection, using concrete/asphalt. With this amount of overtopping discharge, waves may be generated on rubble mound breakwaters.

3.2.2.1. Wave overtopping volume – definition and description

To describe precisely how many waves will overtop the crest of the structure and the volume of water overtopped in each wave, a mean overtopping discharge is not enough data, because the volume of water overtopped over the crest of the structure is irregular in time, and the mean overtopping discharge is measured in m^3 per wave per meter width.

Usually, most of the overtopping waves are small, but a few number of waves generate large volumes of water, which makes the process of getting individual wave overtopping volumes measurement a complex process (Eurotop, 2007).

With respect to storm duration, the longer it is the storm, the greater the number of overtopping waves and the maximum overtopped volume.

However, a high number of small overtopping waves may generate the same mean overtopping discharge as a small number of large waves for rough sea conditions.

In terms of maximum volume, this parameter will be much larger for rough sea conditions with large waves (Eurotop, 2007).

3.3. OVERTOPPING ANALYSIS AND PREDICTION

3.3.1. INTRODUCTION

Hazards caused by wave overtopping can be related to many parameters. The two principal parameters are (Eurotop II, 2016).

- Mean overtopping discharge – q ;
- Maximum individual volume – $V_{m\acute{a}x}$.

The main methods for wave overtopping prediction are:

- Empirical methods based on observation and comparison of structures;
- Numerical models that simulate wave overtopping as part of the numerical modelling of wave interaction with the structure, using complex flow equations;
- NN Overtopping – The Neural Network tool is another and very used method for overtopping prediction, based on artificial intelligence and supported by an extensive database of measured wave overtopping discharge and volumes;
- The PC-Overtopping (prediction software method) is used when empirical test data do not already exist, or when other methods don't give reliable results and clear conclusions, but presents some practical limitations;
- CFD (Computational Fluid Dynamics) numerical models, include RANS and SPH models, able to simulate coastal processes, such as run-up, wave breaking, and obviously overtopping;
- Physical modelling – allows the prediction of wave overtopping by laboratory experiments, and involves the reproduction of the real structure at an appropriate scale;

In addition, Eurotop also mentions a method/technique for overtopping simulation based on the use of hydraulic simulators. Since it is not the subject of this thesis, it will not be studied thoroughly.

In some cases, these methods are used in conjunction, in order to cross results of each method and achieve reliable results and conclusions.

3.3.2. EMPIRICAL METHODS

Empirical methods are one of the wave overtopping prediction methods, and one of the first to be used. These methods use simplified representation of the physics of the process with dimensionless equations to allow establishments of relationships between structures and parameters and therefore the estimation or prediction of wave overtopping volumes and discharges. These methods can sometimes be applied

directly, and other times, using iterative methods. In the past these empirical methods were presented graphically (Eurotop II, 2016).

3.3.3. SIMPLIFIED FORMULA FOR OVERTOPPING AND COMPARISON OF STRUCTURES

Empirical methods are frequently applied in the design of structures, and despite the use of simple analytical equations to calculate parameters (mean overtopping discharges, for instance), these models need additional empirical constants that are determined by fitting to experimental data from physical model tests.

The mean overtopping discharge is usually the main parameter in the process of wave overtopping determination. It is easy to measure in a laboratory wave flume or basin and it is easily related to the other parameters of wave overtopping.

The formulae used in wave overtopping analysis, relating the structure crest freeboard with the mean overtopping discharge and the spectral significant wave height, is presented in the following equation.

$$\frac{q}{\sqrt{gH_{m0}^3}} = a \exp \left[- \left(b \frac{R_c}{H_{m0}} \right)^c \right] \quad \text{for } R_c \geq 0 \quad (3.2.)$$

where q represents the mean overtopping discharge, R_c the crest freeboard of the structure, and H_{m0} represents the estimated spectral significant wave height.

Equation (3.2) represents a Weibull-shaped function. The overtopping discharge is given as dimensionless, $q/(gH_{m0}^3)^{1/2}$. The equation presented in Eurotop (2007) assumes $c=1$ for sloping structures, turning the equation into an exponential function.

It happens that this old formula is not valid anymore for certain specific conditions, as it is explained in the Figure 3.8 below, for breaking waves (Eurotop II, section 4.2.2, 2016).

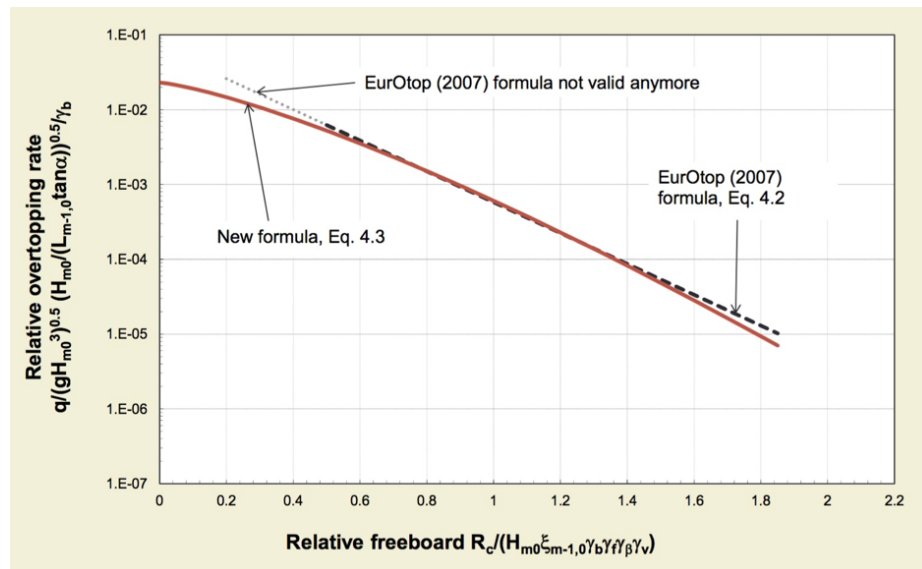


Figure 3.8 - Comparison between Eurotop (2007) old formula and Eurotop (2016) new formula.

The new formula for wave overtopping determination (Eurotop II, 2016) is presented by the following equation.

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.023}{\sqrt{\tan \alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp \left[- \left(2.7 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \gamma_b \gamma_f \gamma_\beta \gamma_v} \right)^{1.3} \right] \quad (3.3.)$$

where $\xi_{m-1,0}$ is the breaker parameter based on $s_{m-1,0}$ (wave steepness with L_0), γ_b is the influence factor for a berm, γ_f is the influence factor for the permeability and roughness of or on the slope, γ_β is the influence factor for oblique wave attack, γ_v is the influence factor for vertical wall on the slope. The main difference between the old formula from Eurotop (2007) and the new formula like this one from Eurotop (2016) is based on the fact that includes a zero freeboard (crest level equal to the water level), in the area for very low freeboards. This wave overtopping formula with dimensionless mean overtopping discharge, allows the comparison between different structures, by comparing each one's formulas/equations. In Section 3.4, are presented relations and graphics for relative overtopping rates with relative freeboards, for the case of rubble-mound structures, used in this work.

3.3.4. NUMERICAL MODELS – WAVE OVERTOPPING

The use of numerical models is another alternative for wave overtopping prediction. They simulate wave overtopping using numerical models and flow equations in the process. Several numerical models can be applied for the prediction of wave overtopping, from non-linear shallow water equation models, to *Reynolds Averaged Navier-Stokes* (RANS) equation models and Smooth Particle Hydrodynamics (SPH) models.

3.3.4.1. Non-linear Shallow Water Equations

Non-linear Shallow Water (NLSW) equations represent the Navier-Stokes equations in a simplified form, where the pressure is assumed as hydrostatic, the vertical forces are disdained and the study is processed in one dimension, mainly horizontal, with depth integrated free surface flow (Eurotop, 2016). The application field for these models is restricted to shallow waters case scenarios ($h/L < 0.05$) and require, from other tools, wave information from a wave propagation. NLSW models are applicable not only on permeable structures, but also on impermeable structures. However, this model has the benefit of its simplicity in terms of computational requirements, which provides the possibility of running long tests covering large physical domains, simulating an extensive numbers of incidents waves in 3D domains in a reasonable computational time (Figure 3.9).

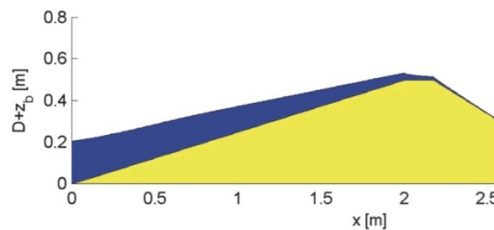


Figure 3.9 - Numerical modelling results using the hydrostatic NLSW WAF-TVD solver for wave overtopping, (Eurotop II, 2016).

A large number of NLSW models has been created and validated for wave overtopping prediction on coastal protection structures, namely the SWASH, RBREAK, AMAZON, ODIFLOCS, the ANEMONE, among others.

A few examples of numerical models applications on wave overtopping studies are presented here. For instance, Vanneste *et al.* (2014) apply three different numerical models and discuss their results in a validation study, where the main subject is wave overtopping estimation and the study of impacts on a sea wall. The model SWASH is based on nonlinear shallow water equations.

In respect to the SWASH model, it allows reliable estimates for the wave overtopping on a simple quay wall, and with lower computational cost than the Dual SPHysics and FLOW-3D, for instance.

It is a non-hydrostatic wave-flow model, mainly used for prediction of dispersive waves into the beach. This model has a large application field, from surf zones studies, to wave propagation and agitation analysis in ports and harbours, swash zone dynamics and can be applied also in coastal flooding studies (Vanneste *et al.*, 2014).

3.3.4.2. Navier-Stokes Equations models

The Navier-Stokes equations incorporate a complete flow description in three dimensions – pressure, the three flow velocity components, and turbulence in space and time.

Navier-Stokes equations can be solved in two different forms: using the Eulerian approach, or the Lagrangian approach. In the Eulerian approach, the fluid is treated as a continuum with the flow domain detached in control volumes. In the Lagrangian approach, the fluid domain is detached in particles, whose interactions are determined in space and time (Eurotop, 2016).

The Dual SPHysics and the FLOW-3D are based on the Navier-Stokes equations, and very used for sea walls design, because they provide reliable estimates of the wave impacts on the sea wall (Vanneste, 2014).

In respect to the DualSPHysics, the model is based on the SPH method (Smoothed Particle Hydrodynamics), that will be more precisely explained thereafter on this sub-chapter. It is also linked with the Navier-Stokes equations, and incorporates a method that detaches the entire fluid into a series of individual particles, thereafter transformed into computational units that represent nodal points in space and time (Vanneste, 2014).

3.3.5. SMOOTH PARTICLE HYDRODYNAMICS (SPH) – WAVE OVERTOPPING

The SPH (Smooth Particle Hydrodynamics) is a numerical model, able to simulate coastal processes, such as run-up, wave breaking, and obviously wave overtopping occurrences, and can be used in every type of coastal protection structure (Eurotop, 2016).

The Smooth Particle Hydrodynamics method is based on a mesh-free particle modelling approach, able to run and study free surface large deformations, and multi-interface by using a pure Lagrangian frame. Basically, in a SPH framework, by using an analytical kernel function, and through the interactions between particles, the movement of each one can be effectively calculated (Jaan Hui Pu, 2012).

In addition, the model incorporates a sub-particle scale turbulence model, named SPS, which allows the modelling of the generation of flow turbulence on wave breaking and overtopping occurrences (Jaan Hui Pu, 2012). However, SPH models are still very expensive in relation to computational costs, and still are not able to solve large domains and long duration wave trains that are required for wave-structure interaction analysis.

Figure 3.10 presents an output example of the SPH method.

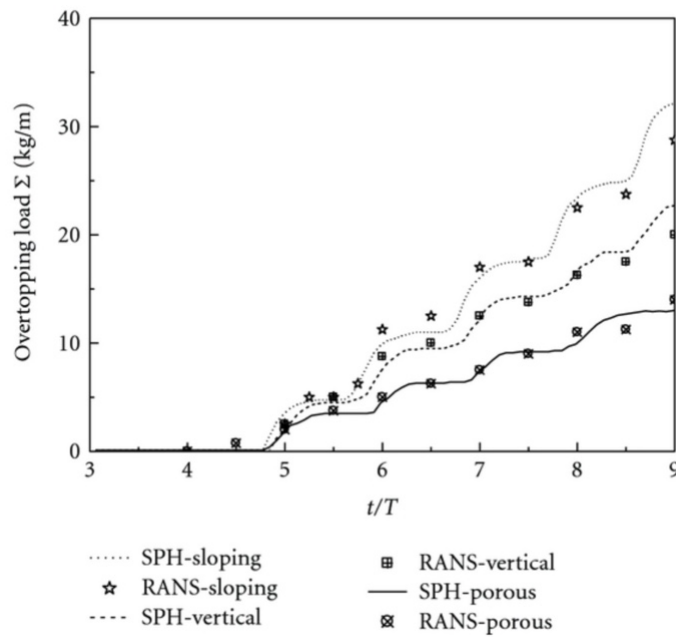


Figure 3.10 - Time history related to wave overtopping load for various breakwater designs (Hui Pu *et al.*, 2012).

3.3.6. PC – OVERTOPPING

Another important type of wave overtopping prediction method is based on the software named PC-Overtopping.

Created from the results of the Technical TAW report (dikes and embankments overtopping study), the PC-overtopping is a software applied in all water defences in the Netherlands, for the safety assessment (Eurotop, 2016). PC-overtopping is not applied to structures with crests built from permeable and rough rock or armour units. Despite those cases, permeability and roughness can be efficiently studied in this software. The typical application field rounds all kinds of dikes and embankments-type structures, as shown in Figure 3.11.

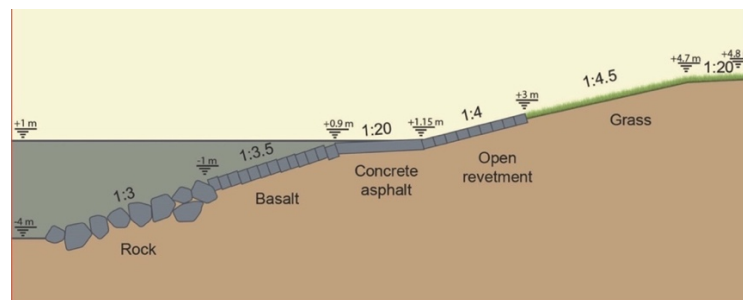


Figure 3.11 - Typical cross-section of a dike-type structure for calculation by PC-Overtopping (Eurotop, 2016).

The set-up made for PC-Overtopping allows the modelling process for practically every existing sloping structure, and with infinite number of cross-sections.

In respect to wave overtopping analysis, it calculates the main key parameters, e.g. mean overtopping discharge, the percentage of overtopping waves, the run-up level, etc. However, two variables are not calculated on this software, namely flow velocities and depths.

Another deprivation of the software's functionality, takes place on its inability in terms of overtopping discharges calculation for vertical structures (Eurotop, 2016). The typical output of PC-Overtopping is illustrated in Figure 3.12.

Calculated parameters	Average value
2% wave runup	4.486
2% wave runup	2.768
Percentage of overtopping	6.039

Overtopping[l/s/m]	Crest height[m]
0.100	2.728
1.000	1.274
10.000	0.999

Percentage[%]	Amount[l/m]
Number of waves	72
Vmax	1606.8
1.000%	0.000
10.000%	0.000
50.000%	0.000

Figure 3.12 - Typical output of PC-Overtopping
(HR Wallingford website calculation tool).

3.3.7. EUROTOP NEURAL NETWORK PREDICTION TOOL

The Eurotop Artificial Neural Network prediction tool were developed to estimate wave overtopping (as well as its newly upgraded version). This software represents a neural tool, since it simulates human intelligence by reproducing the structure of the human brains.

In respect to coastal engineering, the NN Overtopping is able to predict structure elements stability, forces acting on walls, wave transmission and reflection, and wave overtopping.

During and after the CLASH (described thereafter), the developments made in NN overtopping towards a new programme upgraded software, had the main goal to predict the average wave overtopping discharge, q , the wave transmission coefficient, K_t , and also the wave reflection coefficient, K_r .

In the chapter related to the laboratorial experiments, a comparison between laboratorial results and NN Overtopping results will be executed, and posteriorly validated by the experimental measures results.

The general configuration of a Neural Network is presented in Figure 3.13.

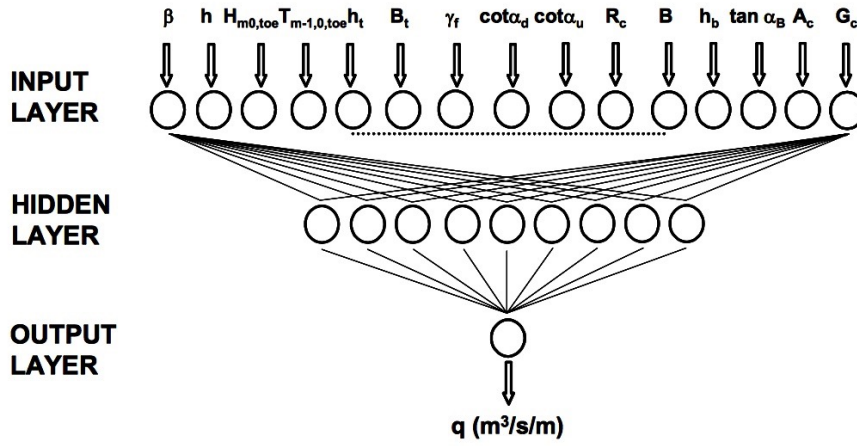


Figure 3.13 - Configuration of a Neural Network (Gent et al., 2006).

For the experimental results validation process addressed in chapter 5 of this thesis, it is used the NN-Overtopping software. This open source tool is available from the Deltares website.

The input parameters are represented in Figure 3.14.

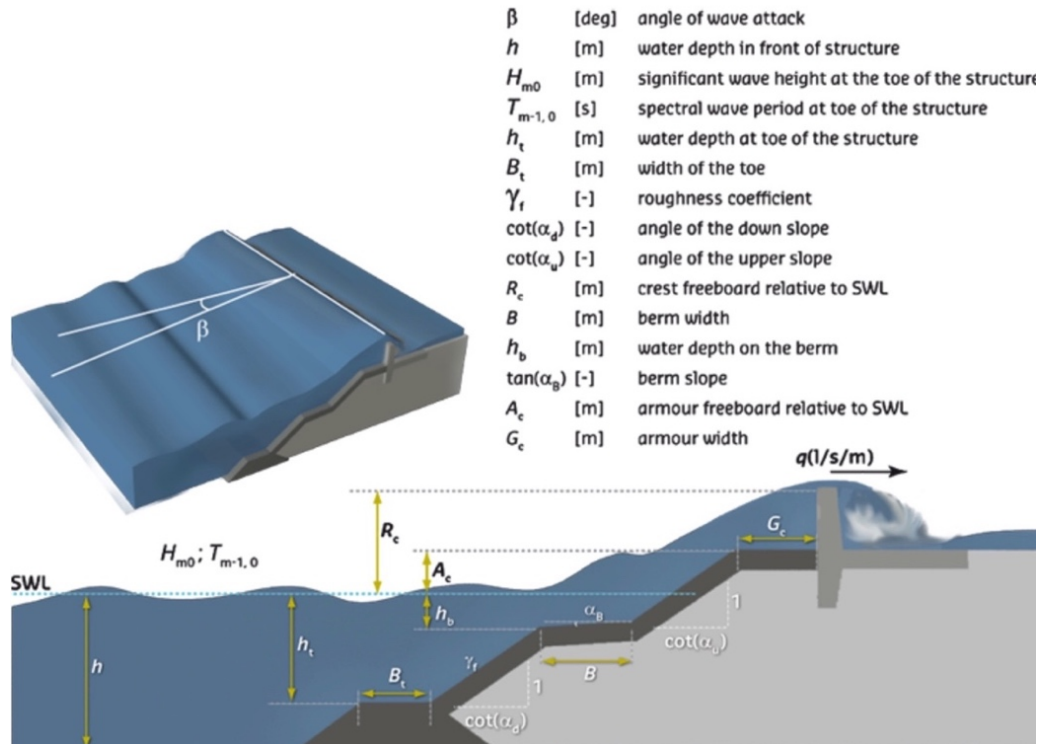


Figure 3.14 - Setup of the parameters needed for NN-Overtopping calculation (Eurotop, 2016).

A precisely description of the NN overtopping functionalities and software characteristics can be found in Eurotop II (2016) in section 4.5.3. The typical dimensionless input parameters of the software are illustrated in Figure 3.15.

#	Parameter	Type	Representation of
1	$H_{m0}/L_{m-1,0,t}$	Wave conditions	Wave steepness (breaking)
2	β [rad]	Wave conditions	Wave obliquity
3	$h/L_{m-1,0,t}$	Wave conditions	Shoaling parameter
4	$h/H_{m0,t}$	Geometry	Effect of the toe submergence
5	$B/L_{m-1,0,t}$	Geometry	Effect of the toe width
6	$d_b/H_{m0,t}$	Geometry	Effect of the berm level
7	$B/L_{m-1,0,t}$	Geometry	Effect of the berm width
8	$R_c/H_{m0,t}$	Geometry	Effect of the relative crest height (including the crown wall if present)
9	$A_c/H_{m0,t}$	Geometry	Effect of the relative crest height
10	$G/L_{m-1,0,t}$	Geometry	Effect of the crest width
11	m	Geometry	Effects of the foreshore slope
12	$\cot\alpha_d$	Geometry	Downstream slope
13	$\cot\alpha_{incl}$	Geometry	Average slope in the run-up/down area
14	$D/H_{m0,t}$	Structure characteristics	Indication of structure permeability and/or roughness
15	γ_t	Structure characteristics	Dissipation induced by structure roughness and permeability

Figure 3.15 - Typical synthesis of the 15 selected dimensionless input parameters of the new Artificial Neural Network (new NN Overtopping).

The main part of the output is the mean overtopping discharge, q , expressed in l/s/m. The other part corresponds to the quantiles $q_{5\%}$, $q_{25\%}$, $q_{50\%}$, $q_{75\%}$, $q_{95\%}$, and $q_{97.5\%}$. Figure 3.16 presents a typical output of the NN-Overtopping software (NN-Overtopping, Deltares).

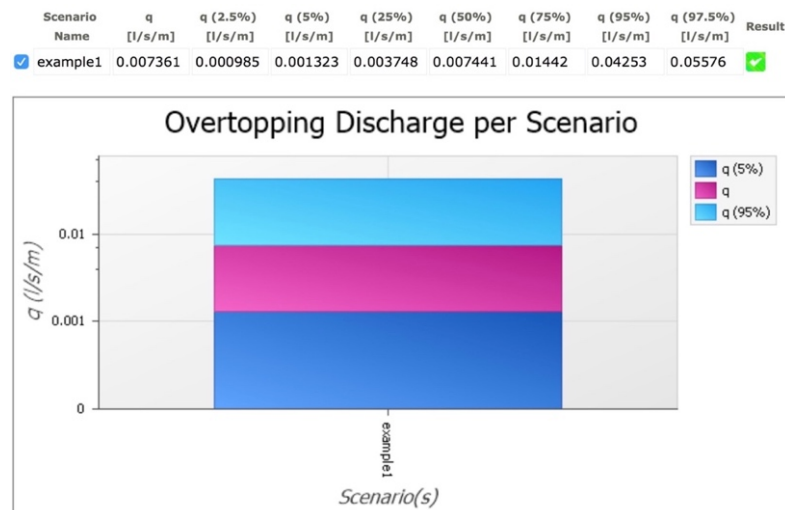


Figure 3.16 - Typical output of the NN-Overtopping software calculator for a random case example (NN-Overtopping, Deltares).

NN Overtopping has various field applications. For instance, after the prediction of the mean overtopping discharge, some relations and analysis can be made, such as the relation between the overtopping rate and the freeboard in rubble-mound structures, as it is presented in Figure 3.17 (van der Meer *et al.*, 2005).

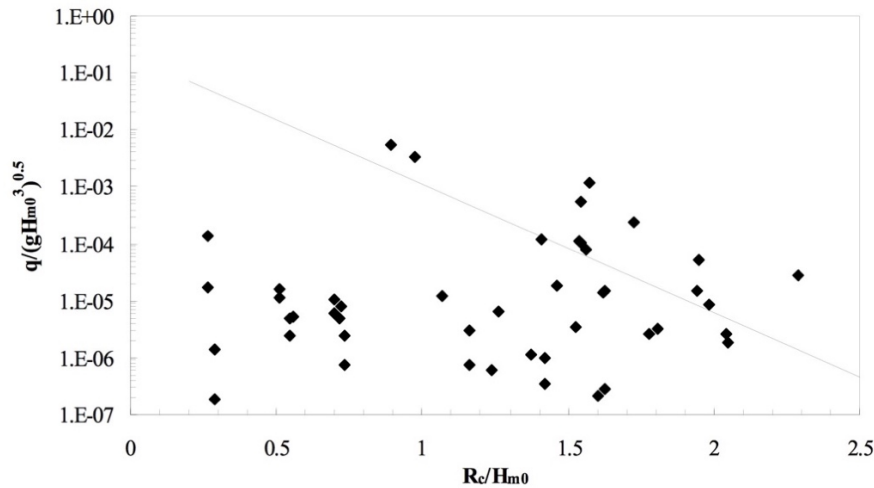


Figure 3.17 - Relation between overtopping rate and freeboard in rubble-mound structures (TAW, 2002).

3.3.8. WAVE OVERTOPPING DATABASE – THE CLASH PROJECT

The Clash Project was developed with the main goals of measuring wave overtopping in prototype scale, addressing the magnitude of scale effects in physical model tests, and providing an extensive and homogeneous database and toggle an artificial neural network for wave overtopping prediction, applicable to almost all types of structures, except the perforated ones.

This project database contains 10,000 organized wave overtopping discharge (q) tests, carried out all over the world, on different types of structures, such as dikes, rubble mound breakwaters, berm breakwaters, caissons, among others.

3.3.9. PHYSICAL MODELLING

3.3.9.1. Introduction

Through history, physical models have been used in hydraulic studies not just for coastal protection structures. Physical models may also be used for wave overtopping determination since they allow reliable reproductions of real cases at proper scales. They integrate a physical system reproduced at a reduced scale, where the major forces acting on that system are represented in an appropriate scale and in proportion to the actual physical system, named the prototype.

The irregularity of wave time-series and waves' behaviour, as well as the complexity and large size of some coastal protection structures, leads to the impossibility of obtaining reliable wave overtopping predictions using only empirical and numerical models. This stresses the relevance of using physical models, which are constructed and operated at reduced scale, and provide a new form of analysing coastal phenomena that cannot be properly studied analytically or numerically.

Figure 3.18 presents the physical model of a rubble-mound breakwater that was designed to assess wave overtopping and the damage progression on its armour layer.



Figure 3.18 - Physical model of a rubble-mound breakwater, inside the wave tank of the Faculty of Engineering of the University of Porto (FEUP, 2016).

3.3.9.2. Advantages and disadvantages

The use of physical models is very common in hydraulic studies due to their advantages, such as:

- The small size of the models allows an easier data collection and reliable results;
- The laboratory experiment techniques are one of the most reliable tools in coastal engineering due to the limits of deterministic fluid mechanics because of turbulence phenomena;
- Provide a high level of control and manipulation of the experiments, conceding the possibility of simulating various types of environmental conditions that interact with the physical model;
- Allow the reliable measurement of individual overtopping volumes;
- Visual feedback from the model is provided.

However, physical models also have limitations:

- Physical processes are represented with inherent problems related to scale effects, due to the size differences between the model and the prototype when it is not possible to correctly relate every variable with each other. An example of this is the scale effect of viscous forces which are smaller in the prototype than in the scale model (in coastal engineering models);
- Laboratory effects, caused from the difficulty in creating realistic forcing conditions in order to simulate an identical environment, are also a relevant issue. For example, in the model, unidirectional waves are sometimes generated to simulate the directional waves of the nature,

but sometimes this relation gives an unreliable approximation. Short crested waves were used in some laboratory experiment of this thesis;

- In most cases, physical models are more expensive to execute than numerical models. For that reason, numerical models are used instead of physical models when they are able to provide reliable and accurate results for wave overtopping and damage progression (for instance).

Figure 3.19 presents an example of a physical model test setup on a wave basin. This was the wave tank used for the laboratorial experiment of this thesis (Rosa-Santos *et al.*, 2008).

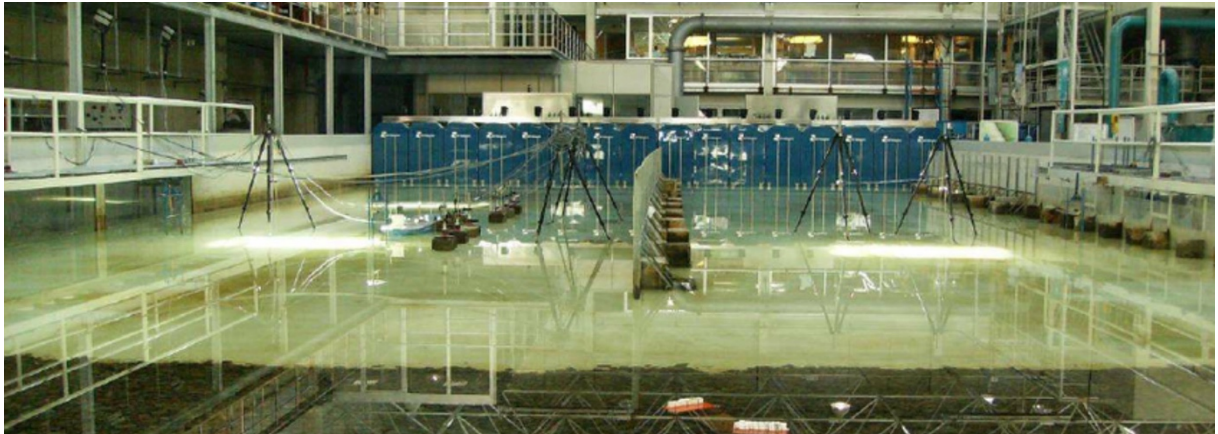


Figure 3.19 - Example of a wave tank (wave basin) of the Hydraulics Laboratory of the Faculty of the Engineering of the University of Porto (Rosa-Santos *et al.*, 2008).

3.3.9.3. Similarity criteria

The main aspect to be assured in physical modelling, is the similarity between the physical model and the prototype. For this matter, it is necessary to certify three types of similarity, namely, the geometric, kinematic and dynamic similarity.

As it was explained before, the model is designed and built in proportion with the prototype, so it can be analysed at a reduced size, but with equivalent results and conclusions. Some phenomena like wave refraction, breaking, or shoaling, are hard to reproduce accurately on a model's simulation due to their irregularity. However, these similarity of scales provide, in the best possible way, a reliable equality between the two environments (model and prototype) (Eurotop II, 2016).

Three principal methods for the similarity of scales are presented:

- Calibration;
- Differential equations;
- Dimensional analysis;

The similarity criteria used for this work was the similarity of Froude, in order to convert model values into prototype values.

For this type of analysis, the gravitational forces are the most relevant, and for that reason, the similarity criteria used is the similarity of Froude.

Table 3.5 presents the similarity of Froude with the regular scalling of factors used in this laboratorial experiment, from model to prototype values conversion (Yeo *et al.*, 2012).

Table 3.5 – Regular scalling of factors, using the similarity of Froude.

Froude (-)	Length (m)	Time (s)	Volume (m ³)	Flow (m ³ /s)
$Fr = \frac{U}{\sqrt{gL}}$	$\lambda_L = \lambda_L$	$\lambda_t = \lambda_L^{1/2}$	$\lambda_V = \lambda_L^3$	$\lambda_Q = \lambda_L^{5/2}$

Where U represents a characteristic flow velocity, g is the gravitational acceleration and L is a characteristic length.

Furthermore, the geometric scale used for this experimental work, was 1/35. This scale selection was based on two reasons. The first one is based on the need for conformity with the work performed in LNEC. The second reason is related to possible scale effects. The only chance to simulate a model completely equivalent to the real scale structure, is to design a model with the same size of the real one, scale 1/1. Due to practical and economic reasons, the models have to be designed in a smaller scale than the prototype.

However, if a very large scale is used, for instance 1/60 or 1/100, the model will be too small and the external forces acting on the structure suffer large scale effects. The smaller is the model, the bigger are the scale effects consequences on the physical modelling analysis. So it is advised, in order to reasonably avoid scale effects, a scale lower than 1/40.

Taking that into account, the geometric scale used for this laboratorial experiment model was 1/35.

Laboratory effects have also to be taken into account in these type of analysis. On this case, two examples are given for laboratorial effects:

- Wave reflection from the shovels of the wave tank at FEUP (the absorption system is not perfect);
- Barrier to wave propagation caused by the wall supporting the breakwater's model, causing reflection and other phenomena that influence damage and possibly the wave overtopping as well.

3.4. OVERTOPPING ANALYSIS – PROBABILITY AND DISTRIBUTION FUNCTIONS

3.4.1. WEIBULL PROBABILITY DISTRIBUTION

Wave overtopping represents a dynamic and irregular process that cannot be described only using the mean overtopping discharge q . It is necessary to consider also the storm duration, t , and the number of overtopping waves in that period, N_{ow} , to characterize this irregular and dynamic phenomena, among other parameters. There are many forms of overtopping analysis and results representation. One of the analysis done in this dissertation is based on fitting distribution and probability functions to individual overtopping wave volumes.

In Eurotop II (2016), the distribution of individual wave overtopping volumes is represented by the Weibull probability distribution function, that gives information on exceedance probabilities in terms of wave overtopping in this case (Equation 3.4). This probability distribution function is used later in the overtopping analysis (Chapter 5).

$$P_{V\%} = P(V_i \geq V) = \exp \left[- \left(\frac{V}{a} \right)^b \right] \quad (3.4.)$$

where P_V represents the probability between 0 and 1, that an individual wave volume, V_i , will be larger than a specific volume V , and $P_{V\%}$ the percentage of overtopping waves exceeding the specified volume V .

The two parameters of the Weibull distribution are the parameter b , that represents the non-dimensional shape factor, that defines the extreme tail of the distribution, and the parameter a , that represents the dimensional scale factor, used to normalize the distribution (Eurotop II, 2016).

The two Weibull distribution parameters a and b , are defined in Equations (3.5) and (3.6), respectively (Eurotop II, 2016).

$$a = \left(\frac{1}{\Gamma(1+\frac{1}{b})} \right) \left(\frac{qT_m}{P_{ov}} \right) \quad (3.5.)$$

$$b = 0.85 + 1500 \cdot \left(\frac{q}{gH_{m0}T_{m-1,0}} \right)^{1.3} \quad (3.6.)$$

where Γ represents the mathematical gamma function expressed by $\frac{1}{\Gamma(1+\frac{1}{b})}$, T_m is the average wave period from time domain analysis, q is the mean overtopping discharge per meter structure width, a and b , the scale factor and the shape factor, respectively, P_{ov} represents the probability of overtopping presented in Equation (3.7), and H_{m0} represents the spectral significant wave height. Figure 3.20 taken from Eurotop II (2016), presents the new Weibull shape factor, b , for rubble-mound structures, for a large range of relative freeboards.

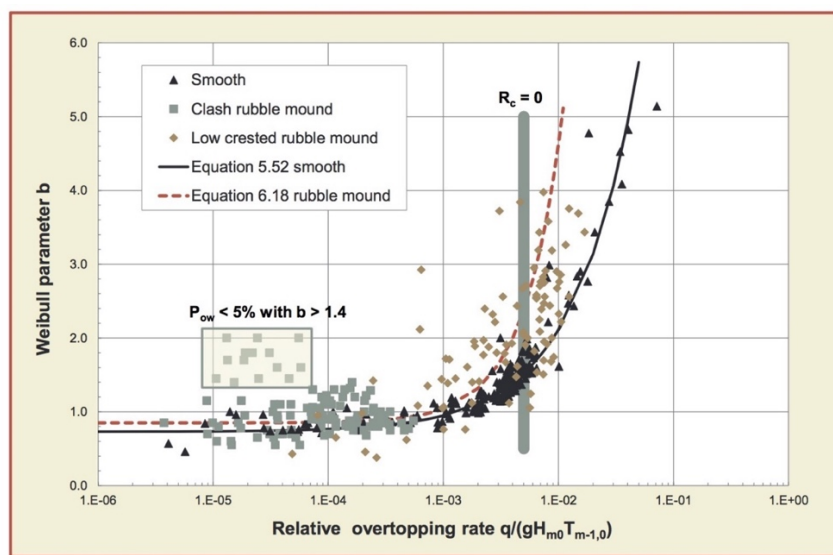


Figure 3.20 - New Weibull shape factor, b , for rubble-mound structures, spanning a large range of relative freeboards (Eurotop II, 2016).

In order to be able to use Equation (3.5), it is necessary to calculate the probability of overtopping using the following equation (Eurotop II, 2016).

$$P_{ov} = \frac{N_{ow}}{N_w} \quad (3.7.)$$

where P_{ov} is the probability of overtopping, N_{ow} is the number of overtopping wave volumes, and N_w is the total number of incident waves of one storm event (or test duration).

Another important parameter for wave overtopping volumes analysis addressed in Eurotop II (2016) is the maximum overtopping wave volume, $V_{m\acute{a}x}$, that depends on the storm duration and can be calculated from the number of overtopping waves, by the Equation (3.8), given in Eurotop II (2016).

$$V_{m\acute{a}x} = a \cdot [\ln(N_{ow})]^{1/b} \quad (3.8.)$$

Where “a” and “b” represent the scale and shape factors, respectively, and N_{ow} represent the number of overtopping waves.

Equation (3.9) presents the distribution function of the wave-by-wave overtopping masses that are expected to follow a two-parameter Weibull distribution, with probability density function (Eurotop, 2007).

$$f(V) = \frac{b}{a} \left(\frac{V}{a}\right)^{b-1} \cdot \exp \left[- \left(\frac{V}{a}\right)^b \right] \quad (3.9.)$$

Where V is the wave-by-wave overtopping volume, expressed in m^3/m , and the parameters “a” and “b” are given by the same Equations 3.5 and 3.6, respectively, presented before. A typical graphic from this expression is presented in Figure 3.21 (Eurotop, 2007), with the influence of the shape factor b in the probability density function.

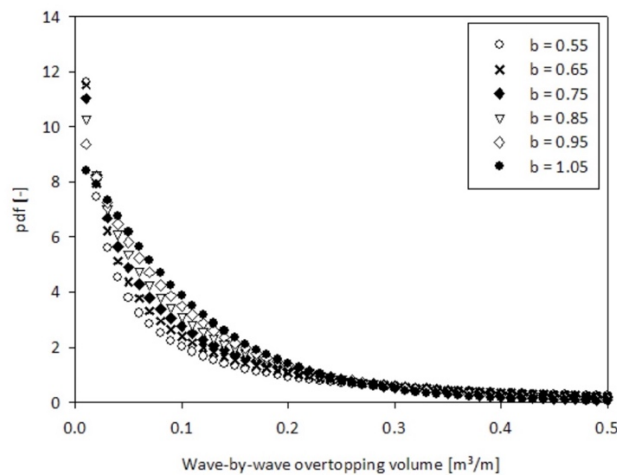


Figure 3.21 - Curve of probability density function related to Weibull distribution with scale factor $a=0.102$, for different values of the shape factor, b (Eurotop, 2007).

For rubble-mound structures, the Eurotop II (2016) presents the relationship between the relative freeboard and the relative overtopping rate for smooth and rubble-mound slopes, Figure 3.22.

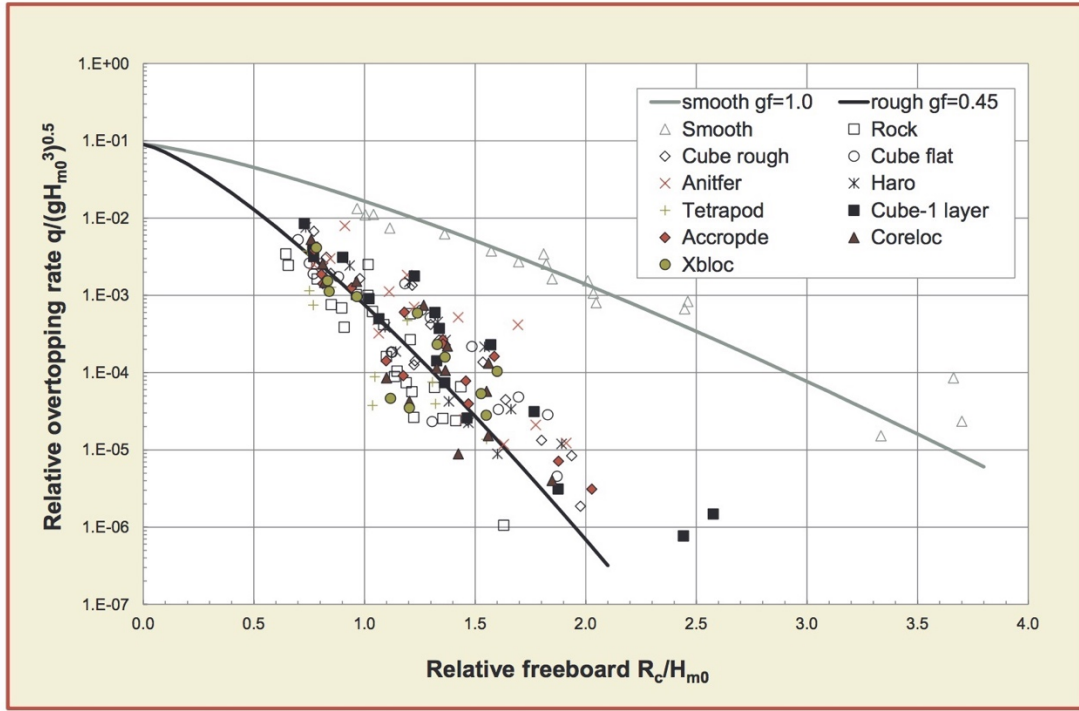


Figure 3.22 - Mean overtopping discharge for 1:1.5 smooth and rubble-mound slopes (Eurotop II, 2016).

This analysis is made for the laboratory experiment of this thesis in Chapter 5. One of the conclusions that can be taken from Figure 3.22, is the fact that there is more overtopping rate for smooth slopes, than for rubble-mound slopes. For the average of the measured data, predictions and comparisons with measurements (mean value approach), giving the graphic illustrated in Figure 3.22 above, the Equation 3.10 is presented.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp \left[- \left(\frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right)^{1.3} \right] \quad (3.10.)$$

where γ_f represents the influence factor for the permeability and roughness of the slope, γ_β is the influence factor for oblique wave attack, q is the mean overtopping discharge per meter of structure width, R_c is the crest freeboard of the structure, and H_{m0} is the spectral significant wave height.

For the selection of the parameter γ_f , Table 3.6 from Eurotop II (2016) presents the different values of γ_f for each type of structure's material and number of structural elements. The value used for this work case was 0.4, for a rubble-mound breakwater with two layers covering a permeable core.

Table 3.6 – Values for roughness factor γ_f for permeable rubble-mound structures with slope of 1:1.5, (Eurotop II, 2016).

Type of armour layer	γ_f
Smooth impermeable surface	1.00
Rocks (1 layer, impermeable core)	0.60
Rocks (1 layer, permeable core)	0.45
Rocks (2 layers, impermeable core)	0.55
Rocks (2 layers, permeable core)	0.40
Antifers	0.50
HARO's	0.47
Tetrapods	0.38
Dolosse	0.43
Accropode	0.46
Cubipods (one layer)	0.49
Cubipods (two layers)	0.47

The Eurotop II (2016) presents an equation for the parameter γ_β estimation. But taking into account the fact that this parameter represents the influence factor for oblique wave attack, and the laboratorial experiment model has no oblique waves, $\gamma_\beta = 1$. In order to understand how the type of structure influence the variation of the overtopping rate compared with the freeboard, more examples are presented, for other types of structures. For instance, for vertical walls, the relation between the relative overtopping rate and the relative freeboard is different, Figure 3.23.

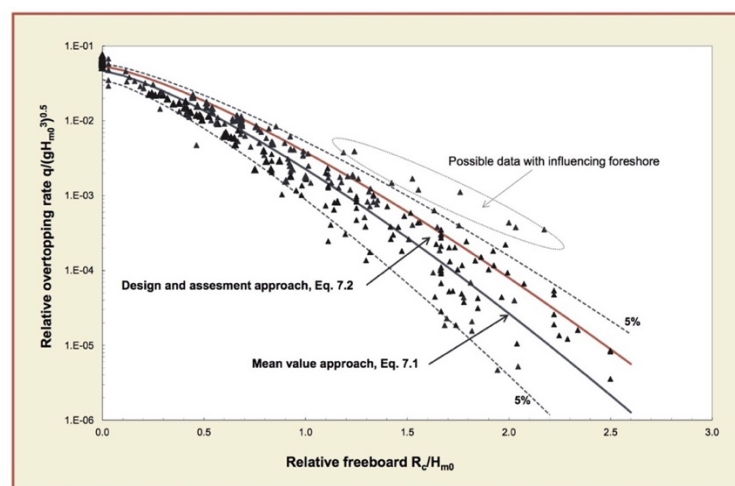


Figure 3.23 - CLASH database for vertical walls (Eurotop II, 2016).

One final example for this work, is the type berm breakwater structure. The relation between the relative overtopping rate and the relative freeboards for this case is presented in Figure 3.24 (Eurotop II, 2016).

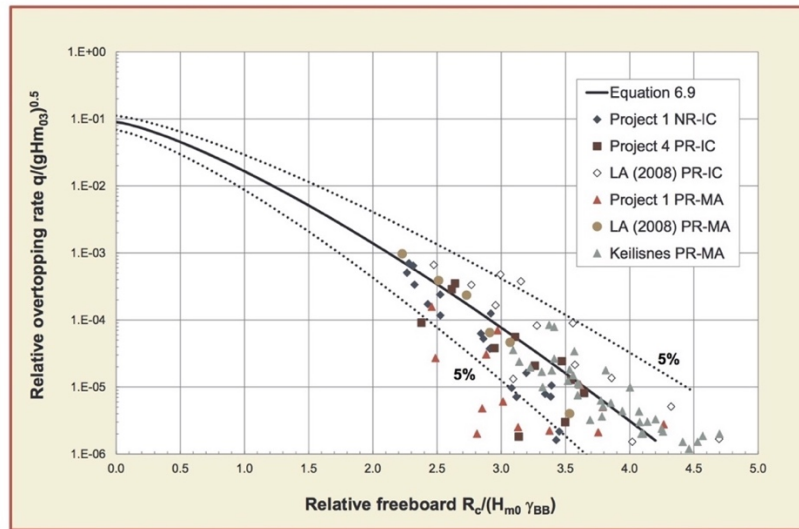


Figure 3.24 - Relative overtopping rate for a reshaping Icelandic type berm breakwater (Eurotop II, 2016).

4. STABILITY AND DAMAGE ANALYSIS

4.1. INTRODUCTION

Coastal engineering is the branch of engineering that studies the general structures granted at or near the coast and the development of the coast itself. The interaction between those structures with the hydrodynamic impact of waves, storm surges, tides and tsunamis are the common concerns and case studies of this engineering field.

Breakwaters are one of the most common solutions to protect ports against wave action. Breakwaters are wave energy barriers, with the purpose of protecting any landform or water zone behind them from the direct impact of waves.

Despite their relevance, breakwaters have been mainly used on harbour protection or for navigation purposes, due to their higher cost in comparison to onshore structures, such as seawalls, for instance. The wave conditions, as well as the wind and topography restrictions and erosion, create different solicitations for coastal protection structures, during their life cycle.

In the past decades the climate change effects have become the main cause for those changing field conditions. In order to attend those increasing loadings, it is necessary to understand how breakwaters and other coastal protection structures can overcome this environmental conditions.

The study of the function of each structural element, as long as the observation of existing structures through time are important for a proper comprehension of the stability issues in a breakwater interaction with the external solicitations.

A description of upgrade solutions for coastal defence structures, damage quantification parameters, and laboratorial procedures for damage quantification is made in this chapter.

Figure 4.1 presents an example of a rubble-mound breakwater with an implemented superstructure in a section of its extension.



Figure 4.1 - Breakwaters Dikkowita, Sri Lanka.

4.2. FAILURE MODES OF RUBBLE-MOUND BREAKWATERS

Rubble-mound breakwaters are the main subject of this dissertation. These structures are built with quarried rock. Armourstone or artificial concrete units are frequently used in the outer armour layer, exposed to direct wave attack. A typical cross-section of a rubble-mound breakwater is presented in Figure 4.2 (Rock Manual, 2007).

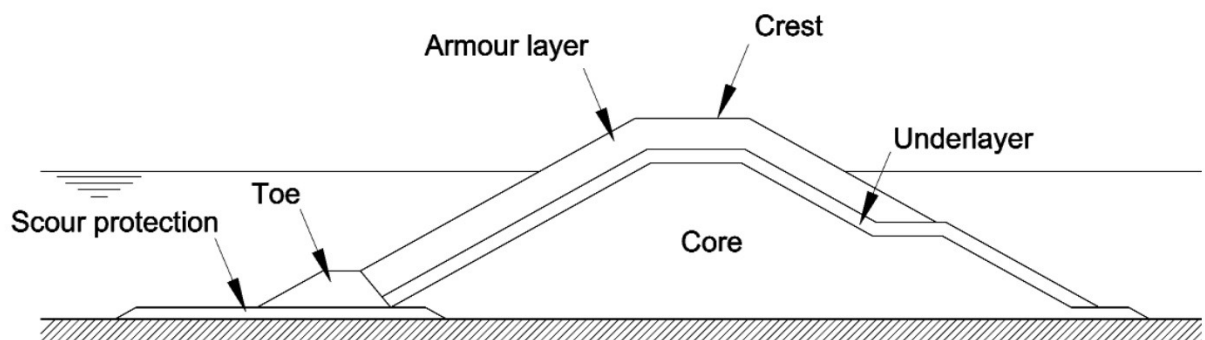


Figure 4.2 - Cross-section of a typical rubble mound breakwater
(Rock Manual, 2007).

In order to design and build a rubble-mound breakwater, it is primarily necessary to understand its failure modes. For rubble-mound breakwaters, failures are generally caused by wave action, or by geotechnical or structural factors.

Failure in rubble-mound breakwaters means an exceedance of the design damage criterion defined in the project. For instance, it can be expressed in terms of displacement of armour units or magnitude of wave overtopping. The typical failure modes considered in the design a rubble-mound breakwaters are (Gravesen, 2008):

- Geotechnical failure modes:
 - Core settlement;
 - Slip failure of various components.
- Hydraulic failure modes:
 - Armour layer, rear slope layer and toe berm erosion;
 - Large overtopping events;
 - Sliding and tilting of the superstructure.
- Structural failure modes:
 - Breakage of the armour units;
 - Breakage of the superstructure.

These failure modes are connected to each other, that is, the occurrence of one failure mode might trigger another (different) failure mode in a different section of the structure (Gravesen, 2008). Figure 4.3 presents the possible failure modes of a rubble-mound breakwater (Rock Manual, 2007).

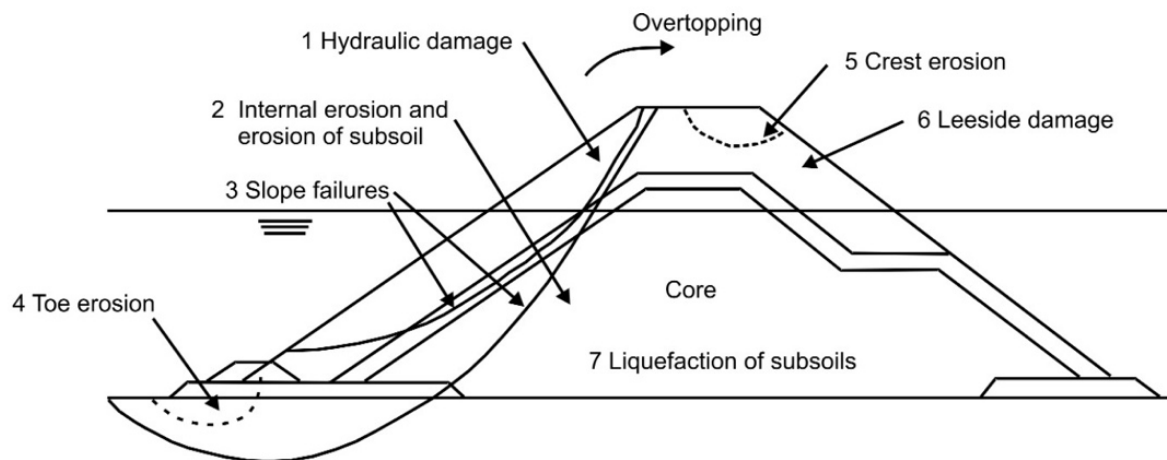


Figure 4.3 - Failure modes in a rubble-mound breakwater (Rock Manual, 2007).

Rubble-mound structures may suffer different kinds of damage and in different areas of the structure.

For that reason, for a proper stability study and damage characterization, it is necessary to study the entire structure and understand how the damage is produced and analyse the interaction with the external loads. The armour layer stability is of paramount importance.

The hydraulic instability of the armour layer can be caused by the action of wave on the rubble-mound slope, creating the movement of armour units.

Four different types of movements can be identified, namely: rocking, displacement of units out of the armour layer, sliding of a blanket of armour units, and finally, settlement, due to compaction of the armour layer (CEM, 2011), Figure 4.4.

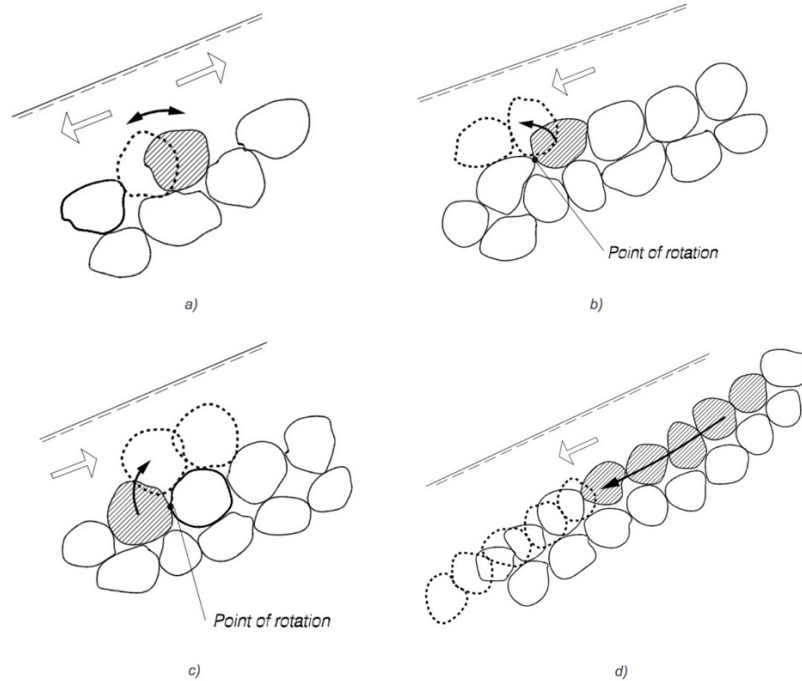


Figure 4.4 - Hydraulic instabilities: a) rocking of unit during up and down-rush; b) rotation and subsequent down-slope displacement of unit during down-rush; c) rotation and subsequent up-slope displacement of unit during up-rush; d) sliding of several armour units (armour layer) during down-rush (CEM, 2011).

The illustrated displacements of armour units can be caused by the water impact forces that remove the units out of the layer or into a different gap of the armour layer by sliding events. The other origin for the displacement of units is due to existing steep slopes, creating loss of support between units of the armour layer, and subsequently, a re-arrangement is generated (CEM, 2011).

The displacement of units, as it is represented on section 4.1, is a form of damage generally given as the relative displacement, D , expressed as the quotient between the number of displaced units and the total number of units within reference area.

4.3. DAMAGE EVALUATION PARAMETERS

The different failure modes caused by different loadings from different environmental conditions through the structures lifetime, have to be quantified. That quantification and evaluation is made by related parameters.

The stability analysis of rubble-mound structures is complex for several reasons. One of them is due to the irregularity of the propagation of waves and their impact forces on the surface of armour layers, where the calculation of those flow forces acting on the armour units are almost impossible. Another issue is related to the complexity of the shape of the armour units placed together and in a random way, which does not allow the calculation of the reaction forces between adjacent armour units (CEM, 2011).

Taking into account that the instantaneous armour unit stability conditions cannot be estimated by deterministic calculations, the stability formulae are then based on hydraulic model tests instead. Subsequently, the response of the armour units in terms of movement are then directly associated to parameters of incident waves, and the actual forces are analysed as a “black box” transfer function (CEM, 2011).

The number of parameters used on the stability analysis of these coastal structures is large. In terms of armour layer hydraulic stability, the parameters are organised in three groups: sea state parameters, structural parameters and combined parameters.

The sea state parameters are:

- Characteristic wave heights – H_s , $H_{1/3}$, H_{mo} , $H_{1/10}$, etc;
- Wave asymmetry;
- Characteristic wave steepness – s_m , s_{om} , s_p , etc.
- Characteristic wave length – L_m , L_{om} , L_p , etc;
- Wave grouping;
- Wave incident angle, β ;
- Shape of wave spectrum and double peak spectra;
- Water depth, h ;
- Mass density of water, ρ_w ;
- Number of waves, N_z .

The structural parameters used in coastal structures stability analysis are as follows:

- Mass density of armour units, ρ_s ;
- Mass M and shape of armour units;
- Grading of rock armour;
- Porosity and permeability of under layers, filter layers and core;
- Seaward profile of the structure, including armour layer slope angle α , freeboard, etc;
- Packing density, placement pattern and layer thickness of the main armour.

Some dimensionless parameters establish important relationships between the sea state parameters and the structural parameters, giving additional information about the hydraulic stability (Comola, 2012):

- $\Delta = \frac{\gamma_s}{\gamma_w} - 1$ Relative density;
- $N_s = \frac{H_s}{\Delta D_n}$ Stability number;
- $\frac{W}{\gamma_r \cdot H^3}$ Dimensionless weight;
- $H_0 T_0 = N_s T_m \sqrt{\frac{g}{D_n}}$ van der Meer (1988);
- $\xi_m = \frac{\tan \alpha}{\sqrt{s_{om}}}$ Iribarren number (Battjes, 1974);

- $\frac{H_s^2}{T v^2}$ Reynolds number;

In addition to these numerical expressions, s_p is the local wave steepness, T_m represents the mean wave period and the parameter $s_{om} = \frac{2\pi H_s}{g T_m^2}$, where H_s is the significant wave height defined as the highest one-third of wave heights = $H_{1/3}$.

CEM (2011) presents the definition and relation between the three main damage parameters: D , N_{od} and S .

- a. Relative displacement within an area:

$$D = \frac{\text{number of displaced units}}{\text{total number of units within reference area}}; \quad (4.1)$$

- b. Number of displaced units within a strip with width D_n (Van der Meer 1988):

$$N_{od} = \frac{\text{number of units displaced out of armor layer}}{\text{width of tested section}/D_n}; \quad (4.2)$$

- c. Relative number of displaced units within total height of armour layer (van der Meer, 1988):

$$\frac{N_{od}}{N_a}, \text{ where } N_a \text{ is the total number of units within a strip of horizontal width } D_n.$$

- d. Percent erosion of original volume (Hudson, 1958):

$$D = \frac{\text{average eroded area from profile}}{\text{area of average origina profile}} * 100\% \quad (4.3)$$

- e. Related eroded area (Broderick, 1983):

$$S = \frac{A_e}{D_{n50}^2} \quad (4.4)$$

4.4. TECHNIQUES AND LABORATORIAL PROCEDURES FOR DAMAGE CLASSIFICATION

Some previous damage characterization procedures, presented by Iribarren (1938) and Hudson (1958), were based on the eroded cross-section area parameter, A_e , around the SWL (Still Water Level).

The difference between them is based on the fact that Iribarren defined the limit of severe damage to occur when erosion depth reached the level of D_n in the armour layer and, on the other hand, Hudson defined D as the percent erosion of the original volume (CEM, 2011).

The parameter D (relative displacement) can be related to all types of movement (section 4.2) and also to the total number of units within a vertical strip of width D_n that is stretched from the bottom to the top of the armour layer.

For this fillet displacement, the van der Meer (1988) formula used the parameter N_{od} defined in section 4.3, for displaced units out of the layer, and for rocking units, the N_{or} parameter was used. However, this damage characterization procedure has the disadvantage of the dependency that these two parameters carry of the slope length (CEM, 2011).

As it is represented in Section 4.2, the expression $S = \frac{A_e}{D_{n50}^2}$, Broderick (1983), defined a damage parameter as dimensionless, and for a riprap and rock armour. This parameter is defined in the CEM (2011), as the number of squares having a D_{n50} length that could be placed in the eroded area. The parameter S, unlike the previous one, is independent of the slope's length.

Despite allowing the treat of vertical settlements, it does not consider settlements and sliding that are parallel to the slope. Another limitation of this formula is that for complex types and shapes of armour like tetrapods or dolos, the parameter S is unreliably suitable, due to complications in defining a proper surface profile (CEM, 2011).

In order to explain the damage classification, the Coastal Engineering Manual gives examples of damage classification and related values of the damage parameters D, N_{od} and S, for a two-layer armour, Table 4.1.

Table 4.1 – Damage Classification parameters D, N_{od} and S.

No damage	Non-occurrence of unit displacement. Possibility of the parameter S not being equal to zero as a result of settlements.
Initial damage	A reduced number of units is displaced. This damage level is related to the no-damage level addressed in the Shore Protection Manual (1977) and to the Hudson (1984) formula stability coefficient, in which the no-damage level is considered as 0-5% displaced units inside the area extending from the intermediate value of the crest height down to the seaward face to a deepness below the Still Water Level equal to a H_s value that causes the damage 0-5%.
Intermediate damage ranging from moderate to severe damage	Occurrence of unit displacements, although with non-exposure of the filter or under layer to direct wave attack.
Failure	Under layer or filter layer directly exposed to wave attack.

The Coastal Engineering Manual (2011) also presents values for the damage parameters D, N_{od} and S, for different types of armour layer units and slopes.

Table 4.2 presents the damage level by D for two-layer armour.

Table 4.2 – Damage level expressed by D for two-layer armor.

Unit	Slope	Initial damage	Intermediate damage	Failure	Reference
Rock	1:2 - 1:3	0 – 5%	5 – 10%	$\geq 20\%$	Jackson (1968)
Cube	1:1.5 – 1:2	-	4%	-	Brorsen et al. (1974)
Dolos	1:1.5	0 – 2%	-	$\geq 15\%$	Burcharth and Liu (1992)
Accropode	1:1.33	0 %	1 – 5%	$\geq 10\%$	Burcharth et al. (1998)

It is important to mention that for rock units, the parameter D is defined as a percentage of the eroded volume. For cube units, it is considered as the percentage of units moved more than D_n within some limited areas. In the case of Accropode units, a one-layer armour layer is considered.

Table 4.3 presents the damage level by N_{od} for two-layer armour.

Table 4.3 – Damage level N_{od} for two-layer armor (van der Meer 1988).

Unit	Slope	Initial damage	Intermediate damage	Failure
Cube	1:1.5	0	-	2
Tetrapods	1:1.5	0	-	1.5
Accropode	1:1.33	0	-	0.5

Table 4.4 presents the damage level by S for two-layer armour.

Table 4.4 – Damage level S for two-layer armor (van der Meer 1988).

Unit	Slope	Initial damage	Intermediate damage	Failure
Rock	1:1.5	2	3-5	8
Rock	1:2	2	4-6	8
Rock	1:3	2	6-9	12
Rock	1:4 – 1:6	3	8-12	17

4.5. ANALYSIS OF NATIONAL AND INTERNATIONAL CASE STUDIES AND SOLUTIONS

Coastal structures are commonly designed to resist severe storms with no significant damage, while undergoing tolerable damage during more extreme events. Due to the climate change effects, storm events are expected to become more intense in the future. In addition, and as a result of the expected sea level rise, wave overtopping and damage in coastal protection structures should increase through the

years. A few examples of national and international case-studies of rubble-mound structures are given, for which damage and overtopping records were found in the literature.

4.5.1. PORTUGUESE CASE STUDIES

In the case of Portugal, with a continental Atlantic coast of about 800 km, the highly and permanent occurrence of erosion and flooding phenomena in several different parts of its Coast, makes this country subject to many coastal interventions. These severe phenomena on the Portuguese Coast, creates hazards on man's assets as well as on natural systems near the coast.

In order to prevent material losses and life threatening caused by these events, different kinds of coastal protection interventions have been built to secure and to preserve the shoreline, (Veloso-Gomes *et al.*, 2000).

Currently, the main defence works that contribute to the protection of the Portuguese coastline are the following:

- Groins,
- Revetments;
- Artificial sand nourishment of beaches;
- Dune restoration interventions;
- Rubble-mound breakwaters.

The significant wave height is one of the variables that influence the design of the crest height of breakwaters, in order to mitigate wave overtopping events and consequences.

4.4.1.1. The damaged breakwaters of the Port of Ericeira

The initial Port of Ericeira is an example of a port with breakwaters damaged by large wave overtopping events that occurred near the 70s, Figure 4.5. The armour layers, made of tetrapods, suffered severe damages, characterized by the displacement of several armour units and the breaking of individual elements (Teixeira, 2012).



Figure 4.5 - The Port of Ericeira. Photograph from 1983, (Teixeira, 2012).

4.4.1.2. The Port of Sines – report of the damages in the Sines breakwater

The Port of Sines also suffered damages in their breakwaters in the past. In a report of the damages on the Sines breakwater (Baird *et al.*, 1980), a full description of the damage events occurred during the 1978 storm is made. During this storm, measurements at Cabo da Roca indicated the presence of waves with significant wave heights of 8 to 9 m. Figure 4.6 and 4.7 present the damages that occurred on the Port of Sines.



Figure 4.6 - Damage to the crown wall and dolos armour layer, taken immediately after the February 1978 storm, (Baird, *et al.*, 1980).



Figure 4.7 - Dolos removed from the front of the seawall, broken dolos and superstructure, (Baird, *et al.*, 1980).

The official model studies were done by LNEC. However, those studies were not for the designer, but for CAS and for that reason, the irregular wave tests results were not available for the designer before the design was complete.

The effect of wave grouping that occurs in the wave data recorded at Sines, was not taken into account in those model tests.

4.4.1.5. The north breakwater of Nazaré harbour

The north breakwater (the smaller of the two breakwaters) protecting the Nazaré harbour is another example of a Portuguese breakwater damaged. Some damages occurred through time since the initial construction of the breakwater and therefore it was subjected to some rehabilitation works in 2004. However, after that, the structure was then severely damaged after the severe storms that occurred between 2012 to 2013, and 2013 to 2014 winter seasons, resulting in a complete destruction of the breakwater's head, and more or less 30 m of its trunk (Ferreira *et al.*, 2015). Figure 4.8 presents the conditions of the breakwater's head of Nazaré harbour, before and after those storms.



Figure 4.8 - Evolution of the north breakwater's head: a) Before any significant damage; b) Photograph taken on 7th January 2014, after the two storm events, (Ferreira *et al.*, 2015).

Figure 4.9 presents more clearly the damage at the breakwater's head, with deterioration and large displacements of armour units and broken structural elements (Ferreira *et al.*, 2015).



Figure 4.9 - North breakwater head of Nazaré harbour in December 2013, (Ferreira *et al.*, 2015).

This structure was constructed in a sandy foundation, close to Nazaré canyon, where long and high waves are generated. The head of the breakwater and the trunk section connected directly to it were protected with a double layer of Tetrapods, with a total weight of about 40 ton, placed over one layer of 5 to 12 ton of rock blocks (Ferreira *et al.*, 2015).

For the destruction of the head of Nazaré north breakwater, probably many mechanisms contributed. In order to find possible causes for this damage level, many *in situ* observations and overall analysis were made (Ferreira, 2015).

The main conclusion was that probably the rupture mechanism of the breakwater was due to partial liquefaction of the foundations (sandy soil). The direct wave attacks of high waves in conjunction with a relative low permeability of the sands caused that liquefaction.

The rehabilitation works begun in August 2014, and took 1 year to be completed. At first, a recover of the tetrapod and rock blocks, and also a preceding dredging works for the construction of the new section and breakwater's head (Ferreira, 2015).

Takin into account that the damage on the breakwater was due to wave action above than the estimated in the design, a new solution was considered (Santos-Ferreira, 2015), Figure 4.10.

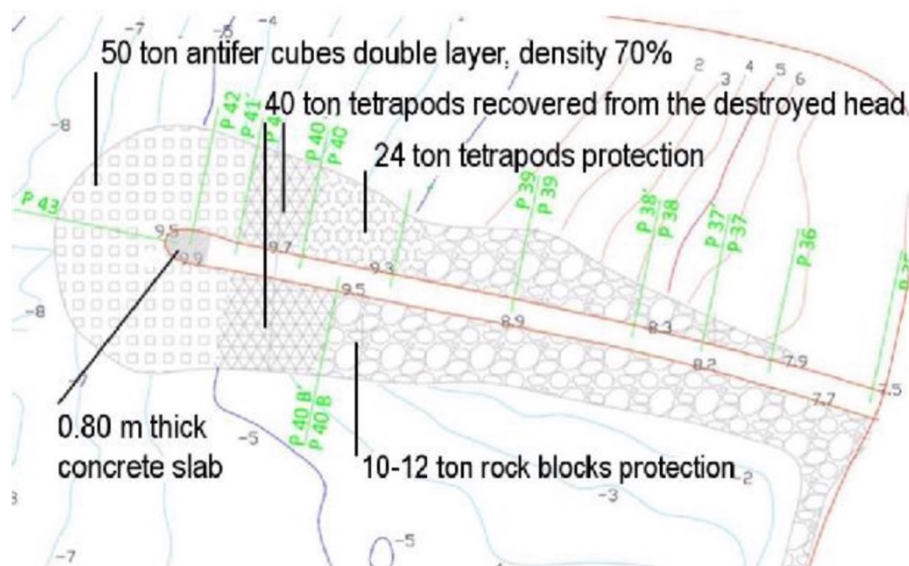


Figure 4.10 - Layout of the north breakwater, and the different kind of armour selected for the rehabilitation works (Ferreira, 2015).

4.4.1.6. The Port of Leixões damaged breakwaters

One final example of damaged breakwaters in Portugal is the case study in the Port of Leixões. The construction of this Port was completed in 1895. Through the years, it was concluded that the recreational harbor conditions were not satisfactory for the increasing commercial traffic, mainly during storm events.

Therefore, a submerged breakwater with 1000 m extension, was constructed and completed in 1942. With the increasing commercial traffic of the port, in the late sixties of last century, the submerged breakwater was raised (crest of the structure increased), in order to protect the new oil terminal (Lopes, 2005).

As usual, the critic zone in this structures is the roundhead, as it is for the Port of Leixões case. After many storm events, for instance, the storm in 1979, the damages led to the Port administration taking the decision to study new solutions for the roundhead reinforcement.

This construction of the protection solution took place in November 1979 and was completed in 1982. It consisted of a structure similar to the old submerged breakwater, designed to mitigate wave overtopping events, and increase the stability of the structure (Lopes, 2005).



Figure 4.11 - North breakwater's head with submerged breakwater, Port of Leixões, (Veloso – Gomes *et al.*, 2013).

Figure 4.12 presents the variation of the total number of tetrapods on the northern breakwater in Port of Leixões, replaced through the years (Lopes, 2005).

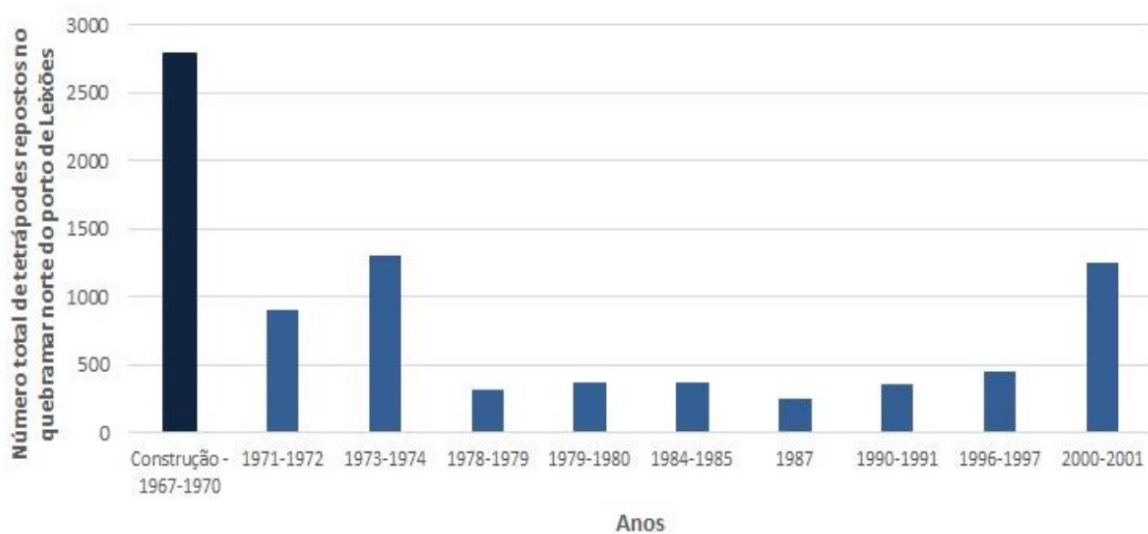


Figure 4.12 - Variation of the total number of replaced tetrapods through the years (Lopes, 2005).

4.5.2. INTERNATIONAL CASE STUDIES: RUBBLE-MOUND BREAKWATERS INTERVENTIONS

4.4.2.1. The Punta Lucero breakwater

Another example is the case of “Punta Lucero” breakwater placed on the Port of Bilbao, Figure 4.13.

This rubble-mound breakwater, built between 1971 and 1976, was damaged in several places (about 140 m of the breakwater were destroyed), so it was necessary to reinforce and restore the structure to recover its original function.



Figure 4.13 - Punta Lucero Breakwater: Port of Bilbao, damage view (Serret *et al.*, 2010).

After many studies, considering all the design parameters of the breakwater and the storm that caused the damage, the causes of the failures were established. The original cross-section as well as the reinforced cross section are presented in Figure 4.14.

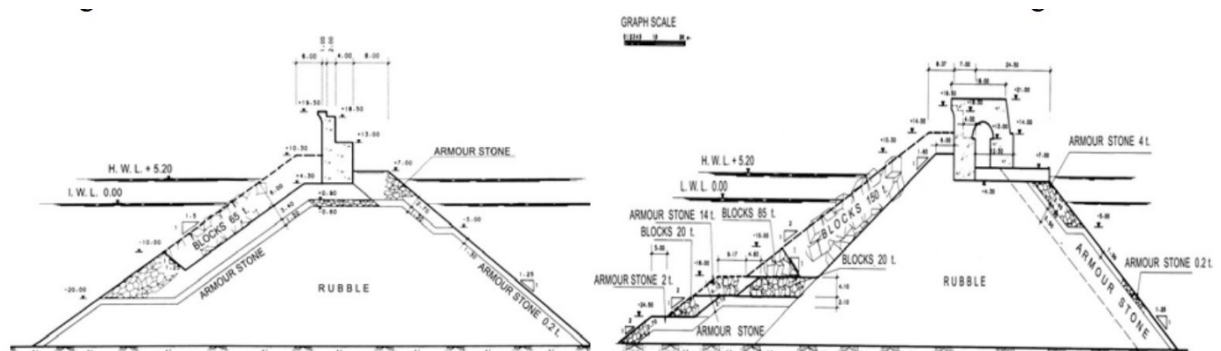


Figure 4.14 - Punta Lucero Breakwater: Port of Bilbao, original and reinforced cross sections (Serret *et al.*, 2010).

4.4.2.2. Port of Long Beach – United States

Another international example is the container port of Long Beach, in the United States of America. Between August and September of 2014, the Hurricane Marie caused heavy surf and overtopping in one of the breakwaters, causing severe damage in the middle section, including dozens of breaches, as it is presented in Figure 4.15.



Figure 4.15 - Breakwater of the container Port of Long Beach, United States.

Concluding, several examples of structures designed and built in the past worldwide have experienced significant damage and their main function, due mainly to large loads of wave overtopping.

In a context of climate change, with the increasing SLR (Sea Level Rise) and more frequent and intense storms, it is crucial to take into account these factors on the future design and reconstruction of coastal protection structures and study what can be expected in the future years, in terms of maritime environment conditions.

4.5.3. UPGRADE SOLUTIONS FOR DAMAGED COASTAL DEFENCE STRUCTURES:

There are various solutions and technical actions that can be applied on damaged coastal protection structures, which can also be used to upgrade coastal structures to withstand the effects of climate change.

Depending on the characteristics of each case, as well as on economic, political or geographic restrictions, the possible solutions are presented in Figure 4.16 (Veloso-Gomes *et al.*, 2007, Burcharth *et al.*, 2014).

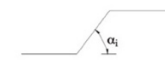
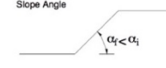
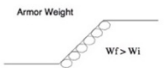
Lowering of bottom foundation level and toe protection		Foundation 
Decrease armour slope angle		Slope Angle 
Increase weight of armour units		Armor Weight 
Beach sand nourishment		Sand Nourishment 
Consideration of filter layers		Filter Layer 
Increase of crest height		Crest Height 
Consideration of toe berms and submerged berms		Toe berm Geotextile filter 

Figure 4.16 - Situation before and after technical actions on damaged coastal defence structures (Veloso-Gomes *et al.*, 2007).

Other examples of technical actions performed on damaged coastal defence structures are presented in Figures 4.17 and 4.18 (Burcharth *et al.*, 2014).

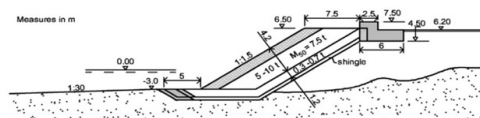


Figure 4.17 - Consideration of an extra armour layer on the front slope and a wave wall extended over the crest level (Burcharth *et al.*, 2014).

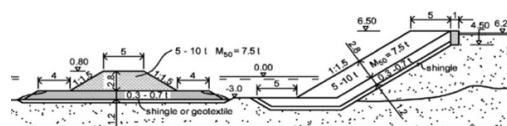


Figure 4.18 - Protection of the revetment by a Detached breakwater on the foreshore (Burcharth *et al.*, 2014).

5. LABORATORY EXPERIMENTS WAVE OVERTOPPING

5.1. INTRODUCTION

The multidirectional wave basin of the Hydraulic Laboratory of the Hydraulics, Water Resources and Environment Division (SHRHA), of the Faculty of Engineering of the University of Porto (FEUP), Portugal, was used, between July and September of 2016, to perform a tridimensional physical model study of a reference rubble-mound breakwater with a rocky armour layer, reproduced on a geometric scale of 1/35.

This study was prepared in cooperation with Deltares, The Netherlands, and LNEC, Portugal, but also received the advice and input from HR Wallingford, UK.

The experimental study was designed and carried out in order to allow the analysis of the damage progression in the trunk (front and rear slopes) and roundhead of the breakwater, for two water levels, and considering the effect of sea level rise.

The tests also included overtopping measurements, which are the main subject of the present dissertation.

The work was done by SHRHA staff and a M.Sc. researcher from TUDelft and Deltares. The damage on the structure was measured using a stereophotogrammetry equipment, developed and borrowed by Deltares.

Although the physical model considered was not intended to be based on a real case study, the results and conclusions obtained provide new insights and information valuable for the analysis, design and damage quantification of common rubble-mound breakwaters including roundheads, as well as to the quantification of wave overtopping.

5.2. EXPERIMENTAL FACILITY

5.2.1. MULTIDIRECTIONAL WAVE BASIN

The multidirectional wave basin of SHRHA-FEUP is 28.0 m long, 12.0 m wide and 1.2 m deep. It has a multidirectional, piston-type wavemaker, composed of 16 independent paddles that is equipped with a dynamic wave absorption module (model HR Wallingford, UK), Figure 5.1. The wavemaker is able to generate regular and irregular waves, either long crested or short crested.

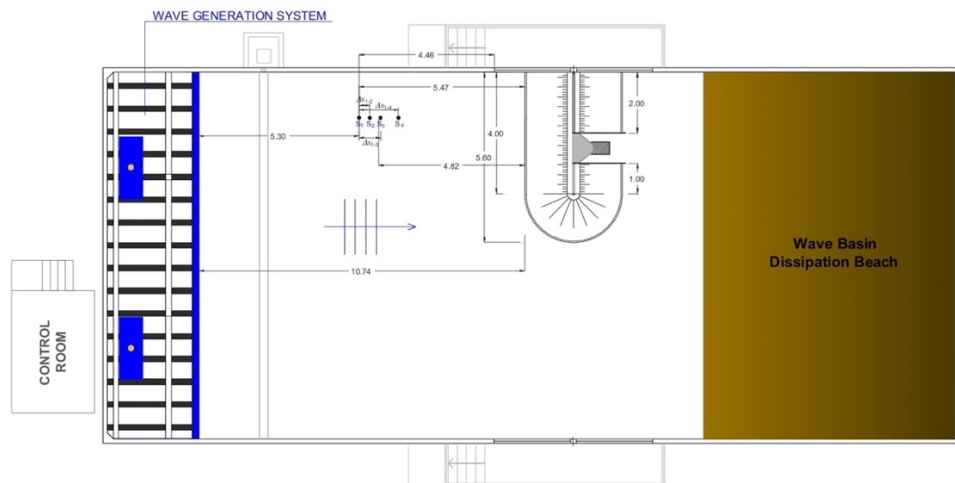


Figure 5.1 - Wave basin of SHRHA-FEUP. Physical model setup (plan view).

A dissipation beach, installed at the end of the wave tank, ensures that the amount of wave energy reflected back in the direction of the wave generation system is minimized, Figure 5.1. A general overview of the wave basin is presented in Figure 5.2.



Figure 5.2 – SHRHA - Hydraulic Laboratory of Faculty of Engineering of the University of Porto.

5.2.2. WAVE MEASUREMENTS

Resistive wave probes were used to record the water free surface elevation during the tests in order to estimate the incident wave conditions in front of the model, and also to monitor the water level inside the overtopping reservoir.

In the tests with irregular long crested waves, the incident wave conditions were estimated based on the measurements of four aligned wave probes placed in front of the rubble-mound breakwater, Figure 5.3. The spacing between the probes was determined based on the characteristics of the tested sea states.

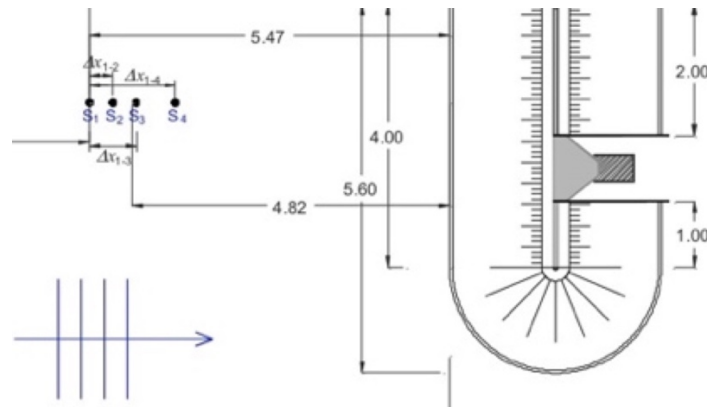


Figure 5.3 - Wave probes layout in front of the rubble-mound breakwater.

The water free surface elevation measured in front of the model during one of the experimental tests is presented in Figure 5.4.

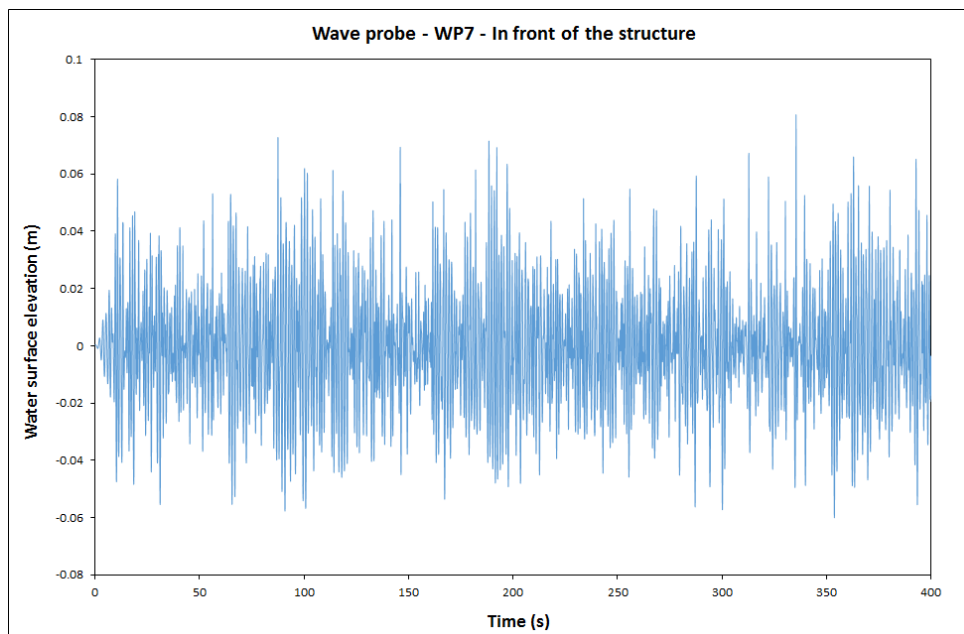


Figure 5.4 - Water level variation during the tests, measured in one of the wave probes in front of the breakwater.

The wave-probe set-up used in the tests with short crested waves was composed of six wave probes following a CERC6 arrangement, Figure 5.5.

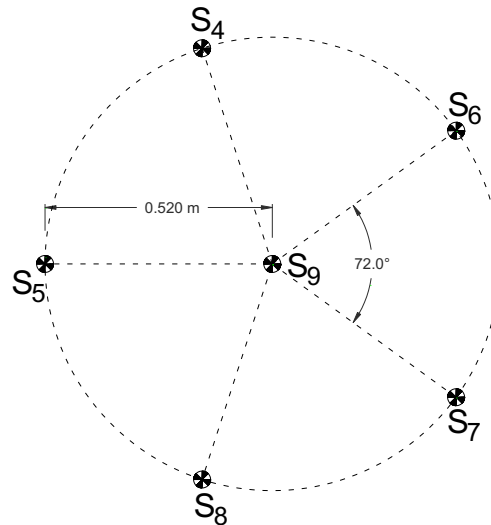


Figure 5.5 - Six wave probes in a CERC6 arrangement.

A wave probe calibration was also made on the preparation phase of the experimental work, allowing reliable readings of the wave probes.

5.2.3. OVERTOPPING MEASUREMENT

The characterization of wave overtopping during the experimental tests is based on the measurements of two additional wave probes: one placed at the entrance of the overtopping chute that conducts the water to the overtopping reservoir and the other one placed inside the reservoir to monitor the water level variation during the test.

The overtopped volumes were then calculated by multiplying the water level variations inside the reservoir by the reservoir plant area.

The wave gauge installed at the entrance to the overtopping chute is not as important as the previous one; however, it has a key role in the identification of individual wave overtopping events during the tests, since it registers when water reaches the crest of the breakwater and enter the overtopping chute.

In fact, when an overtopping mass of water crosses the chute's wave gauge, this one registers a peak in the "water level variation" signal, which corresponds to the occurrence of a wave overtopping event, which is subsequently registered in the reservoir's wave gauge as an abrupt increase of the water level.

It is important to mention that the signal from the probe installed inside the reservoir is already converted to volume of overtopped water per meter width of the breakwater crest.

The records of the two probes mentioned are represented together in Figure 5.6, where it is possible to observe the correlation between both signals.

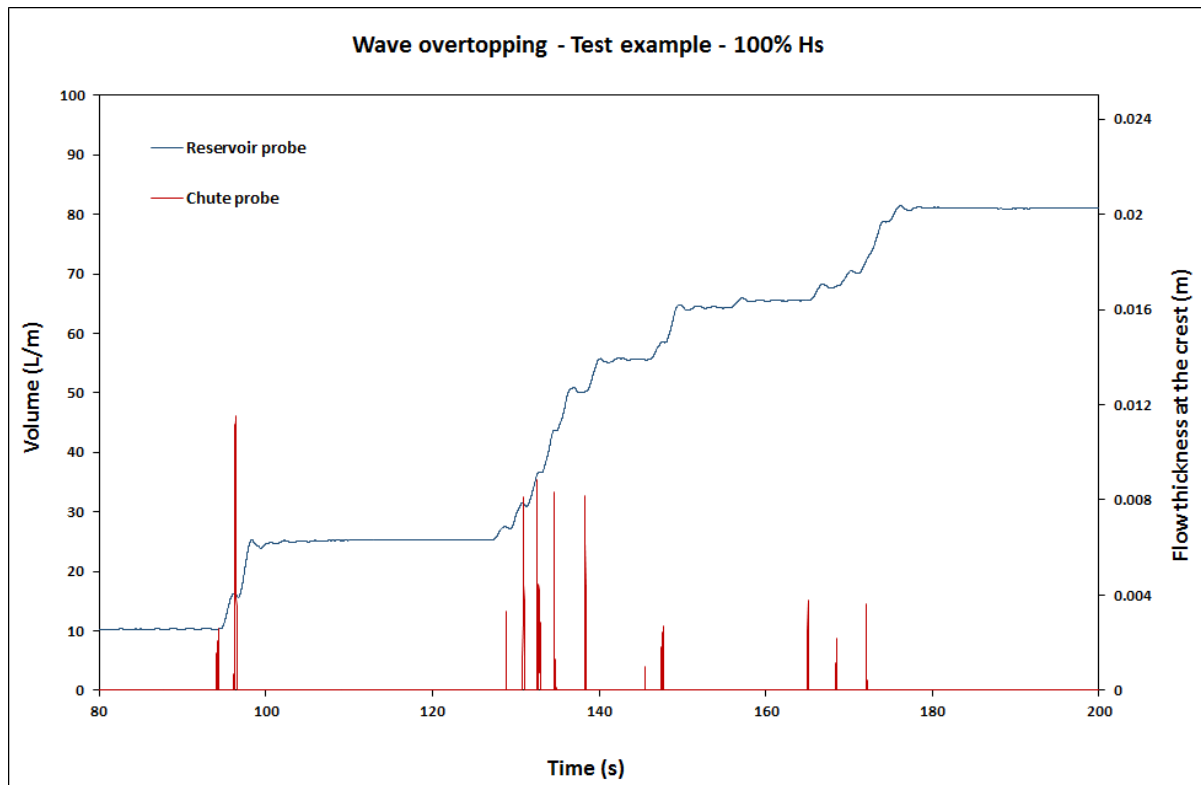


Figure 5.6 - Water level variation inside the reservoir and the water level on the overtopping chute, measured by the two wave probes placed in the reservoir and on the chute.

Based on the information presented in Figure 5.6 it is possible to make a wave-by-wave overtopping analysis, i.e., besides the mean overtopping discharge and the total overtopped volume, the number of overtopped waves and the individual overtopped volumes can be identified and analysed.

Later, it will be verified if these experimental results are correctly described by existing formulae (*e.g.*, Eurotop, 2007 and 2016) and the NN Overtopping tool.

The Weibull distribution function will also be fitted to the experimental data and the agreement with it discussed.

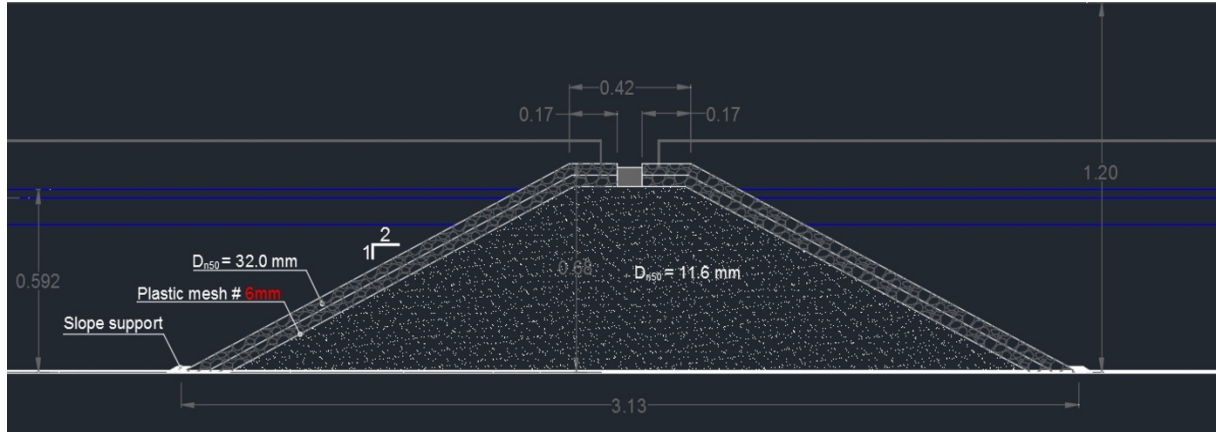
5.2.4. CONSTRUCTION OF THE PHYSICAL MODEL

The model of the breakwater was built, as mentioned, on a geometric scale of 1:35. The core of the breakwater was made of crushed stone ($D_{n50} = 11.6$ mm) and separated from the armour layer by a plastic mesh (6 mm).

The armour layer has a 1:2 slope and is made of two layers of rocks $D_{n50} = 32$ mm (each one 3.80 cm thick), supported by a toe materialized by geosynthetic sand containers, in order to increase the stability of the armour layer, preventing the crushed stones to slip, and decreasing the water depth at the beginning of the slope, therefore, anticipating the waves breaking occurrence.

Figure 5.7 (a) presents the typical cross-section of the breakwater and Figure 5.7 (b) an intermediate cross section that includes the overtopping reservoir in the inner side. A global overview of the rubble-mound breakwater is presented in the photograph of Figure 5.13.

a)



b)

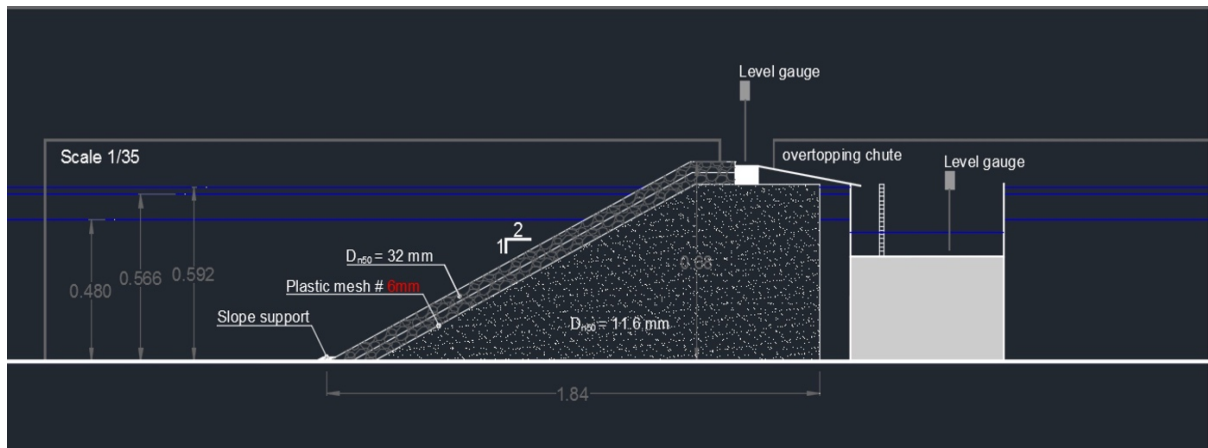


Figure 5.7 – Breakwater cross section: (a) current cross-section; (b) intermediate cross-section, with overtopping reservoir on the inner side.

The wave overtopping measurement system is illustrated in Figure 5.7 (b). The reservoir is placed on the inner slope, leaning against the axis of the breakwater, in its intermediate zone.

The overtopping reservoirs have two parts, being the front part an impact zone where the overtopping water enters the reservoir. The rear part is more sheltered and the water's fluctuation is much lower, allowing reliable water level readings.

Therefore, the level gauge was placed inside the rear part of the reservoir to measure continually in time the water level, which is then used to determine the overtopped volume of water that crossed over the crest of the structure.

On this laboratory experiment, different types of reservoirs and overtopping chutes were used, depending on the wave characteristics (significant wave height and wave peak period) and water level.

Figures 5.8, 5.9 and 5.10 present the three different types of reservoir used in this laboratory experiment, including its main dimensions.

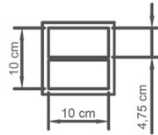


Figure 5.8 – Small reservoir, plan view.

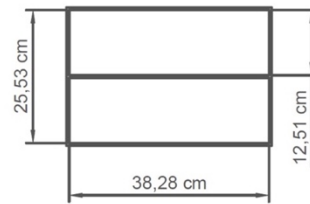


Figure 5.9 – Intermediate reservoir, plan view.

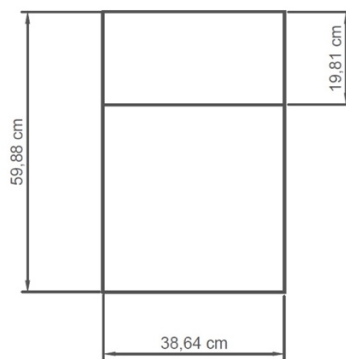


Figure 5.10 – Large reservoir, plan view.

Table 5.1 presents the dimensions of the reservoirs used on the laboratorial experiments. For the overtopping analysis, these values were later converted to prototype dimensions (geometric scale of 1:35).

Table 5.1 – Dimensions of the reservoirs used on the laboratorial experiments.

Reservoir internal measures	Large reservoir	Intermediate reservoir	Small reservoir
Effective height (cm)	40.15	25.1	36.35
Left side and right side (cm)	59.88	25.53	10
Front and rear side (cm)	38.64	38.28	10
Interior walls (cm)	0.18	0.51	0.5
Plan Area (m ²)	0.23283	0.0958	0.0095

The large reservoir, with 40.15 cm height and a plan area of 0.2325 m^2 , was used on the tests with the larger wave overtopping events, otherwise the reservoir would be filled very quickly and only a few wave-overtopping events would be recorded.

On the other hand, on tests with small wave heights and overtopping occurrences, a small reservoir was used, in order to allow the wave gauge installed inside to record accurately in time, every water level variation, during the test.

This small reservoir was built with 36.35cm height and an area of 0.01m^2 , having a slender shape, so that the water level variations could be accurately measured, otherwise, with only a few and small magnitude wave overtopping events, it would be very difficult to separate the volumes corresponding to each overtopping event.

The intermediate reservoir with 25.1cm in height and an area of 0.0957 m^2 was used in some tests, where the sea conditions and parameters generated medium wave overtopping occurrences.

Four overtopping chutes were used in this laboratorial experiment. The combination of the different reservoirs and overtopping chutes allowed to measure properly wave overtopping for a larger range of wave conditions.

On the cases where small wave overtopping events occurred, chutes with a larger width were used, and for the large wave overtopping cases, narrow chutes were applied. This was made so that in the tests with large overtopping, the reservoir would not be full on the first minutes, and logically, in tests with small wave overtopping events, so that the reservoir could be filled before the test was concluded.

Figure 5.11 presents the overtopping chutes that were used in the laboratorial experiments.

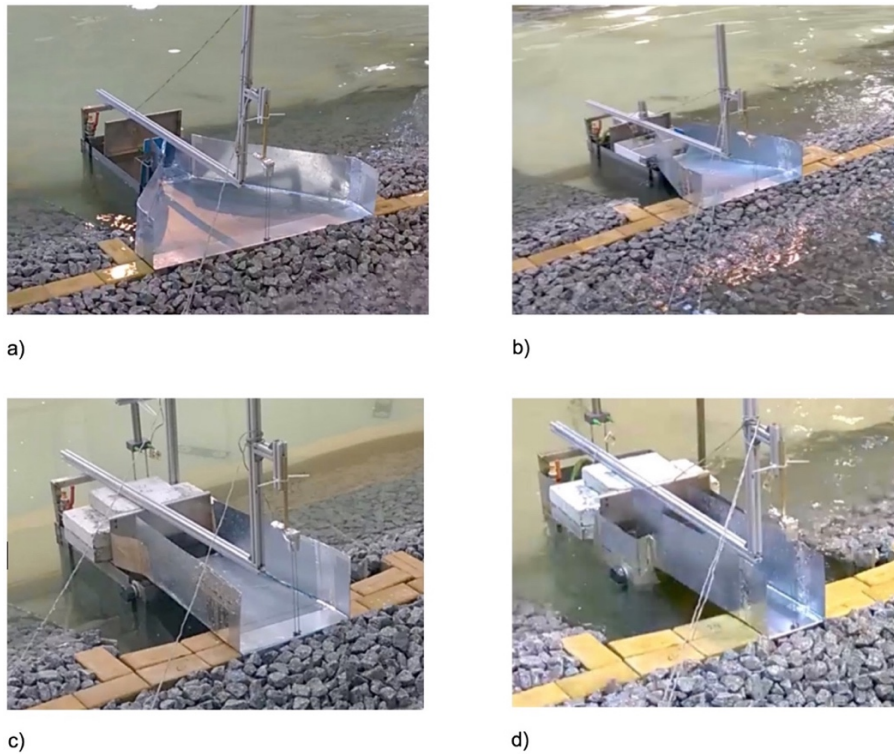


Figure 5.11 – (a) – Overtopping chute with 1 m width; (b) – Overtopping chute with 0.6 m width; (c) – Overtopping chute with 0.4 m width; (d) – Overtopping chute with 0.2 m width.

For each overtopping chute width, Table 5.2 presents the corresponding number of armour units (D_{n50}).

Table 5.2 – Number of D_{n50} associated to each overtopping chutes width.

Overtopping chute	Chute width (m)	number of units
a)	1.0	31.25
b)	0.6	18.75
c)	0.4	12.5
d)	0.2	6.25

The number of armour units for each overtopping chutes width were calculated using Equation 5.1.

$$D_{n50}(Chute) = \frac{\text{Width of the chute}}{D_{n50}(\text{Armour layer})} \quad (5.1.)$$

Figure 5.12 presents the model construction in the wave basin of the Hydraulics Laboratory, of the Hydraulics, Water Resources and Environment Division of FEUP, during the summer season of 2016.

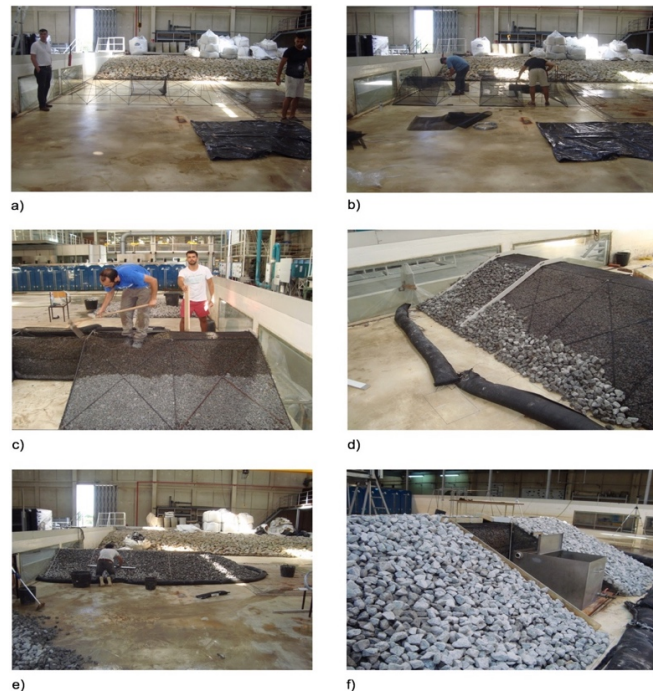


Figure 5.12 – Physical model construction and preparation sequence in the wave basin of FEUP:
 (a) - placement of the metal structure of the breakwater inside the basin; (b) - placement of a plastic mesh (6mm) tied up to the structure; (c) - placement of crushed stone ($D_{n50}=11.6$ mm) inside to materialize the core of the breakwater; (d) - placement of two armor layers of 3.80 cm thickness made of gravel ($D_{n50}=32$ mm); (e) - manual compaction of the armor layers; (f) - placement of reservoir behind the model for wave overtopping measurement.

Figure 5.13 presents the physical model after its construction.



Figure 5.13 – Final model ready for the laboratory experiment.

5.2.5. TEST PROGRAMME

Four different series with four tests each were executed in this laboratory experiment, Table 5.3. From Test 1 to Test 4 of each serie there is an increase on the percentage of the significant wave height ($H_s=60\%$, 80% , 100% and 120% of the design wave height). The water depth used was 0.566 m, and in the Test 3 and 4 of Serie 3, a Sea Level Rise was considered for a water depth of 0.589 m.

A wave steepness of 0.03 is used (given as $s_p = H_s \cdot 2 \cdot \pi / g \cdot T_p^2$).

The wave peak period, T_p is also larger from Test 1 to Test 4 of each serie. Wave conditions are increased from Test 1 to Test 4 of each Serie, in order to evaluate their increase influence in wave overtopping events.

The time duration, as long as the overtopping chutes and reservoirs of each test, were selected, accordingly to the wave conditions of each test, in a way to allow a reliable comparison between the different tests results readings. This issue is better explained in Section 5.25.

In all the tests, long-crested waves were considered, except for Serie 4, where a directional spreading of waves, this is, short-crested waves consideration, was used, in order to evaluate their influence in wave overtopping.

The complete test programme defined for the laboratory experiments is presented in Table 5.3.

Table 5.3 – Test program of the laboratorial experiments.

Series	Date	Test	Water depth (m)	L/S crested	% Hs	Tp (s)	sp	Time (s)	Width of gutter (m)	Reservoir
S1	05-Aug	Run1	0.566	Long	60%	1.29	0.03	1097	1	Small
	05-Aug	Run2	0.566	Long	80%	1.58	0.03	1339	1	Intermediate
	07-Aug	Run3	0.566	Long	100%	1.87	0.03	1591	0.4	Large
	08-Aug	Run4	0.566	Long	120%	2.18	0.03	1850	0.2	Large
S2	10-Aug	Run1	0.566	Long	80%	1.58	0.03	1339	0.6	Intermediate
	12-Aug	Run2	0.566	Long	100%	1.87	0.03	1591	0.2	Large
	29-Aug	Run3	0.566	Long	120%	2.18	0.03	1850	0.2	Large
S3	31-Aug	Run1	0.566	Long	60%	1.29	0.03	1097	1	Small
	02-Sep	Run2	0.566	Long	80%	1.58	0.03	1339	0.6	Intermediate
	05-Sep	Run3	0.589	Long	100%	1.85	0.03	1572	0.2	Large
	06-Sep	Run4	0.589	Long	120%	2.15	0.03	1825	0.2	Large
S4	10-Sep	Run1	0.566	Short - cos2	60%	1.29	0.03	1097	1	Small
	12-Sep	Run2	0.566	Short - cos2	80%	1.58	0.03	1339	0.6	Intermediate
	13-Sep	Run3	0.566	Short - cos2	100%	1.87	0.03	1591	0.2	Large
	14-Sep	Run4	0.566	Short - cos2	120%	2.18	0.03	1850	0.2	Large
S2	16-Sep	Run2r	0.566	Long	100%	1.87	0.03	1591	0.2	Large

The wave breaking conditions were the most limiting ones for the definition of wave conditions to be considered in the tests. A JONSWAP spectrum, with a peak enhancement factor of 3.3, was selected to define the irregular sea states.

The range of values considered for some relevant variables in the study is presented in Table 5.4.

Table 5.4 – Initial generation conditions. Range of values.

Water depth	s _p (Local water depth)	Wave periods - T _p (s)	Wave heights - H _s (m)	Test duration (average)
0.566 – 0.589 m (19.8 m – 20.6 m at prototype)	0.030	1.28 – 2.18 s (7.6 – 12.9 s at prototype)	0.071 – 0.142 m (2.5 – 5.0 m at prototype)	1000 waves

Table 5.5 presents a brief description and the main goals of each test Serie.

Table 5.5 – Description of each series purpose.

Series	Brief description
S1	▪ Long crested waves; ▪ Cumulative damage; ▪ Damage and wave overtopping analysis;
S2	▪ Long-crested waves; ▪ Rebuilt before each test; ▪ Wave overtopping analysis;
S3	▪ Long-crested waves; ▪ Cumulative damage; ▪ Sea level rise effect; ▪ Damage and wave overtopping analysis;
S4	▪ Directional spreading, short-crested waves; ▪ Wave overtopping analysis; ▪ Comparison with other tests with the same initial conditions.

In order to allow to performing a large range of analysis, different conditions were defined for the four series of tests. For instance, in series S3, the climate change effects on armour layer damage and wave overtopping of rubble-mound structures was also considered.

Some cross analysis can also be made between test series. For instance, the test 1 of Series S1, S3 and S4 was carried out for the same initial wave conditions (*i.e.*, significant wave height and peak wave period) and water level.

This also applies to other experimental tests. The similarities were planned in order to allow evaluating the influence of other variables (directional spreading, armour layer rebuilt) on the wave overtopping and damage, but always with the same base conditions.

5.3. INCIDENT WAVE CONDITIONS

5.3.1. TEST SERIES 1

Table 5.6 presents the incident wave conditions for test series 1, S1.

Table 5.6 – Incident wave conditions and other input parameters for Series 1.

Serie	Test	Water depth	L/S crested	Gain	%Hs	H _s (m)	T _p (s)	Breaking	H _{m0}	CR
S1	R1	0.566	Long	0.92	60%	0.071	1.302	No	0.076	0.245
	R2	0.566	Long	0.92	80%	0.094	1.666	No	0.102	0.284
	R3	0.566	Long	0.88	100%	0.118	1.811	No	0.126	0.255
	R4	0.566	Long	0.88	120%	0.142	2.193	No	0.151	0.284

Table 5.7 presents the dimensions of the overtopping chutes and the type of reservoir used in each test of test series S1.

Table 5.7 – Dimension of the overtopping chute and type of reservoir used for each test of Series 1.

Serie	Width of the overtopping chute (m)	Type of reservoir
S1	1	Small
	1	Intermediate
	0.4	Large
	0.2	Large

The records of the wave probes installed inside the reservoirs used in the different tests allow the identification of the number of wave overtopping events. Figure 5.14 presents the variation of the accumulated overtopping volumes for the four tests of test series S1. The larger the significant wave height, the bigger is the total volume water collected during the test.

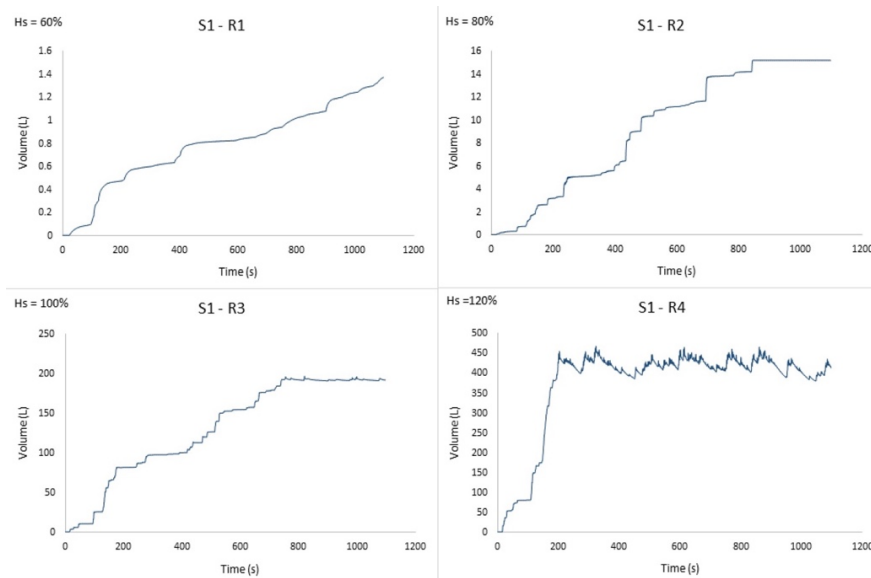


Figure 5.14 – Cumulative wave overtopping volumes of Series of tests 1.

The different increasing levels in the graphics represent the wave overtopping events recorded by the wave probe placed inside the reservoir, as it was explained before. The tests have different measurement and test total duration. The duration of the test is defined as the time of the generation of waves (pre-defined in the test-programme). The measurement time is defined between the beginning of the test and the end of the last overtopping wave that filled the reservoir. Although, in order to quantify all the wave overtopping events, the last overtopping event that fill the reservoir it is not considered, for the level variation inside the reservoir stops in the moment it is filled. In some tests with severe wave conditions such as in the Test 4 of these series, the reservoir is full after 200 s of wave generation. More conclusions such as the number of overtopping waves, or individual volume of each event, are presented later.

5.3.2. TEST SERIES 2

Table 5.8 presents the incident wave conditions and other input parameters for test series 2, S2.

Table 5.8 - Incident wave conditions and other input parameters for Serie 2.

Serie	Test	Water depth	L/S crested	Gain	%Hs	H _s (m)	T _p (s)	Breaking	H _{m0}	CR (reflection coefficient)
S2	R2	0.566	Long	0.92	80%	0.094	1.666	No	0.104	0.270
	R3	0.566	Long	0.88	100%	0.118	1.811	No	0.124	0.268
	R4	0.566	Long	0.88	120%	0.142	2.193	No	0.153	0.302

Table 5.9 presents the dimensions of the overtopping chutes and the type of reservoir used in each test of test series S2.

Table 5.9 – Dimension of the overtopping chutes and type of reservoir used for each test of Serie 2.

Serie	Width of the overtopping chute (m)	Type of reservoir
S2	0.6	Intermediate
	0.2	Large
	0.2	Large

Figure 5.15 presents the variation of the accumulated overtopping volumes for the four tests of S2.

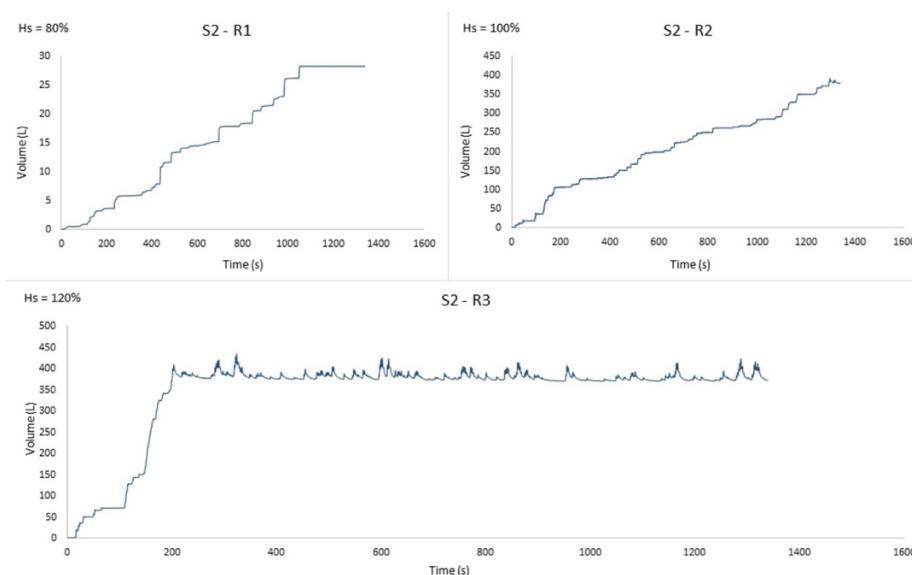


Figure 5.15 – Cumulative wave overtopping volumes of the Series 2 of tests.

5.3.3. TEST SERIES 3

Table 5.10 presents the incident wave conditions and other input parameters for test series 3, S3.

Table 5.10 - Incident wave conditions and other input parameters for Series 3.

Serie	Test	Water depth	L/S crested	Gain	%Hs	H _s (m)	T _p (s)	Breaking	H _{m0}	CR
S3	R1	0.566	Long	0.92	60%	0.071	1.302	No	0.076	0.243
	R2	0.566	Long	0.92	80%	0.094	1.666	No	0.102	0.270
	R3	0.589	Long	0.87	100%	0.118	1.736	No	0.124	0.265
	R4	0.589	Long	0.88	120%	0.142	2.193	No	0.154	0.276

Table 5.11 presents the dimensions of the overtopping chutes and the type of reservoir used in each test of test series S3.

Table 5.11 – Dimension of the overtopping chute and type of reservoir used for each test of Series 3.

Serie	Width of the overtopping chute (m)	Type of reservoir
S3	1	Small
	0.6	Intermediate
	0.2	Large
	0.2	Large

Figure 5.16 presents the variation of the accumulated overtopping volumes for the four tests of S3.

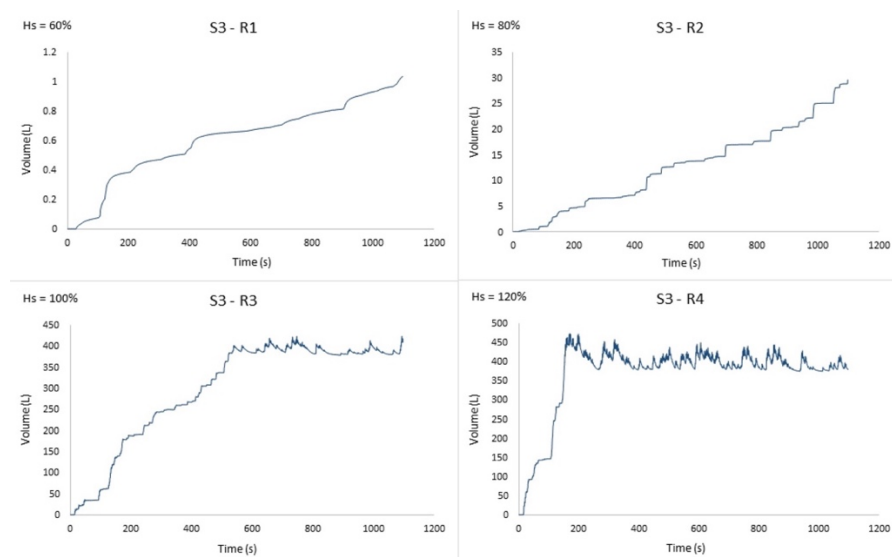


Figure 5.16 – Cumulative wave overtopping volumes of the serie 3 of tests.

5.3.4. TEST SERIES 4

Table 5.12 presents the incident wave conditions and other input parameters for test series 4, S4.

Table 5.12 - Incident wave conditions and other input parameters for Serie 4.

Serie	Test	Water depth	L/S crested	Gain	%Hs	H _s (m)	T _p (s)	Breaking	H _{m0}
S4	R1	0.566	Long	0.93	60%	0.071	1.225	No	0.071
	R2	0.566	Long	0.88	80%	0.094	1.666	No	0.094
	R3	0.566	Long	0.85	100%	0.118	1.894	No	0.118
	R4	0.566	Long	0.82	120%	0.142	2.08	No	0.142

Table 5.13 presents the dimensions of the overtopping chutes and the type of reservoir used in each test of test series S4.

Table 5.13 – Dimension of the overtopping chute and type of reservoir used for each test of Serie 4.

Serie	Width of the overtopping chute (m)	Type of reservoir
S4	1	Small
	0.6	Intermediate
	0.2	Large
	0.2	Large

Figure 5.17 presents the variation of the accumulated overtopping volumes for the four tests of S4.

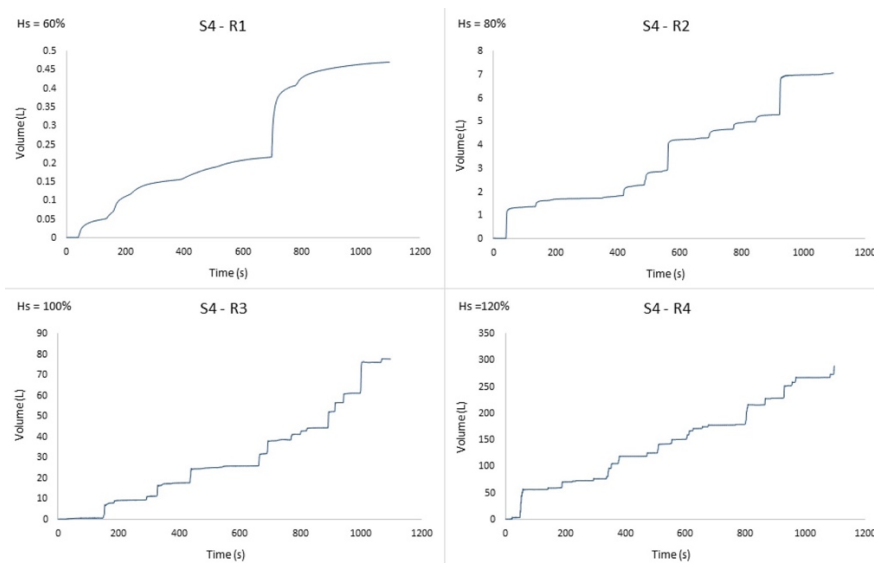


Figure 5.17 – Wave overtopping volumes of the serie 4 of tests.

5.4. ANALYSIS OF THE MEAN OVERTOPPING DISCHARGES

Table 5.14 presents the mean overtopping discharges obtained for the different tests carried out during the experimental work.

Table 5.14 – Mean overtopping flow rate between tests and experiments.

Hs %	Test	q (m ³ /s/m)	Notes
60 %	S1 – R1	0.0002553	Accumulated damages
	S3 – R1	0.0001871	
	S4 – R1	0.0001083	
80 %	S1 – R2	0.003483	Rebuild
	S2 – R1	0.005162	
	S3 – R2	0.005484	
	S4 – R2	0.001227	
100 %	S1 – R3	0.05304	Accumulated damages
	S2 – R2	0.05958	
	S3 – R3	0.1487	
	S4 – R3	0.01264	
120 %	S1 – R4	0.4282	Accumulated damages
	S2 – R3	0.3680	
	S3 – R4	0.4857	
	S4 – R4	0.05121	

Where Hs is the significant wave height, and q is the mean overtopping discharge of each test, S represents the series of tests, and R is the test.

From the mean overtopping discharge of the different tests presented in Table 5.11, it can be concluded that there is a trend to the fact that the larger the significant wave height (given as a percentage of the design wave height), higher the values for mean overtopping discharge, q.

5.5. OVERTOPPING RATE AND FREEBOARD

The roughness and permeability characteristics of the armour layer also have influence on wave overtopping.

The relationship between the overtopping rate and the freeboard can be expressed by the graphic presented in Figure 5.18.

It is used an expression for the calculation of the average of the measured data and it is also used for prediction and comparisons between different measurements (Eurotop II, 2016).

Equation 5.2 was used to achieve the theoretical curve presented in Figure 5.18.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp \left[- \left(1.5 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right)^{1.3} \right] \text{ for steep slopes } 1:2 \text{ to } 1:4/3 \quad (5.2.)$$

For permeable slopes with one armour layer, the Eurotop (2016), gives a roughness factor of $\gamma_f = 0.45$.

In order to validate that relationship expressed by equation 5.2., the experimental results obtained in this work were presented in Figure 5.18 that presents the relative overtopping rate as a function of the relative freeboard of the structure.

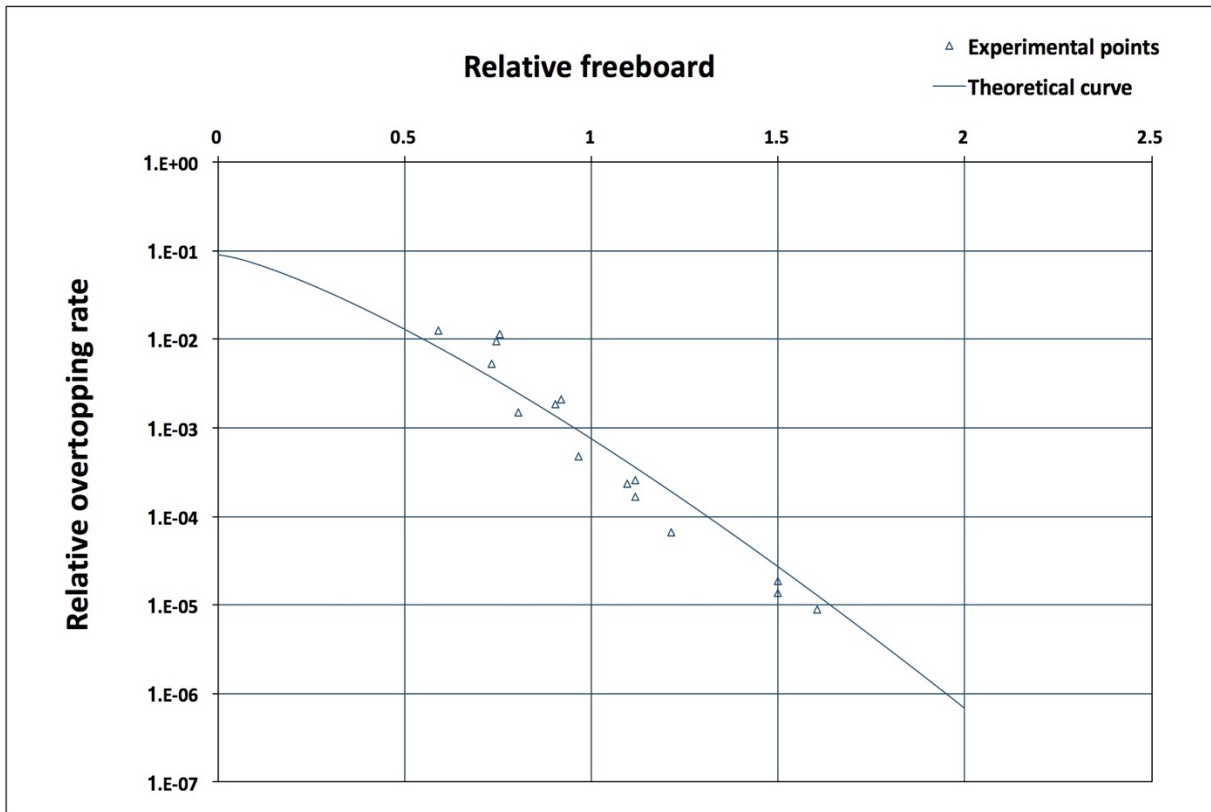


Figure 5.18 – Mean overtopping discharge for the 1:2 rubble-mound slope.

The theoretical curve given in Figure 5.18, represents an average of range of values.

The required crest height for each mean overtopping discharge (for this steep rubble-mound structure in this case) is also given in Figure 5.18.

Summing up, by observing the graphic, it can be seen that for larger variations of the relative freeboard (caused either by the variation of water level and either by modifications in the crest height), smaller variations are given for the overtopping rate.

5.6. NORMALIZED OVERTOPPING VOLUMES

In order to analyse the influence of the different tests conditions in the accumulated wave overtopping volumes along the test, the normalized overtopping volumes of each test series are presented with the results of each test superimposed on a chart.

In respect to test series 1, the overtopping reservoir was filled first in test 4 (S1-R1). Due to the rapid filling of the reservoir on this test, it was impossible to normalize those volumes together with the other runs. The normalized overtopping volume as a function of time for test series 1 is presented in Figure 5.19. The accumulated overtopping volumes were normalized by the total volume collect at the end of each test. It should be mentioned that the structure was not changed between tests and the damage to the armour layer was cumulative (*i.e.*, no rebuilt between tests).

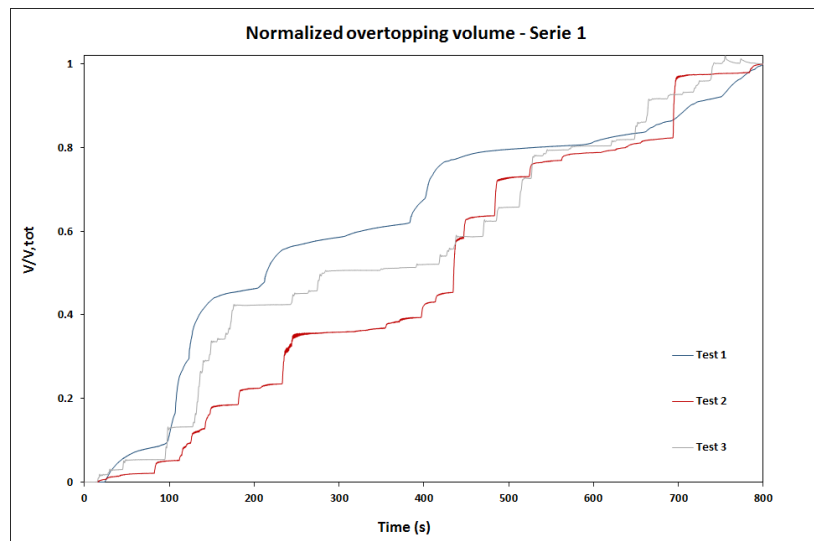


Figure 5.19 – Normalized wave overtopping volumes for the first three tests of test series S1.

The number of waves in the 800 s of test duration presented in Figure 5.19 was 766, 621 and 514 for run 1 (60% of $H_{s,d}$), run 2 (80% of $H_{s,d}$) and run 3 (100% $H_{s,d}$), respectively.

Figure 5.19 is obtained by dividing the accumulated overtopping volume by the total overtopping volume measured in the same test (up to 800 s), and overlapping the results for the relevant tests of that series. Test 1 was carried out for the lowest significant wave height and is the test that needed less overtopping events to produce the total measured overtopping volume.

On the other hand, the R3 was the test of series S1 where more wave overtopping events were observed until the total overtopping volume was reached. In some parts of Figure 5.19 the curves associated to two different tests intersection each other, namely where large overtopping events occur.

However, the exceedance probability of the extreme events in the extreme value distribution is not the same. With that said, the wave that creates the maximum overtopping event in one test can lead to, for instance, the fifth largest wave overtopping in another test.

Furthermore, in Test 3, the number of incident waves in 800 s was 514; the lowest number of waves in the tests, but the volume of each overtopping wave is larger. This aspect is analyzed later in the wave-by-wave overtopping analysis.

Figure 5.20 presents the normalized overtopping volumes for test series 2.

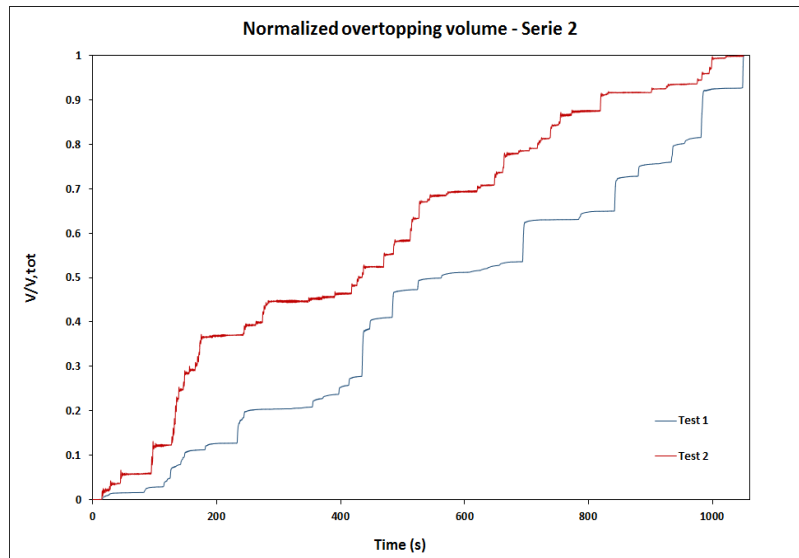


Figure 5.20 – Normalized overtopping volume for different wave field conditions and the same structure for Series 2.

The number of waves in the 1050 s of test duration presented in Figure 5.20 was 830 and 672 for run 1 (80% of $H_{s,d}$) and run 2 (100% $H_{s,d}$), respectively. In Serie 2 of tests, a comparison between normalized volumes of Test 1 and two, is made. Test 3 is not considered on this analysis, due to the rapid filling of the reservoir, resulting in a very short duration of test, not allowing a reliable comparison. However, in Test 2, a significant wave height of 100 %, resulted in a recorded number of incident waves of about 672. On the other hand, in Test 2, for a lesser number of incident waves, the number of overtopping waves were higher than in Test 1. Figure 5.21, presents the normalized overtopping volume for S3.

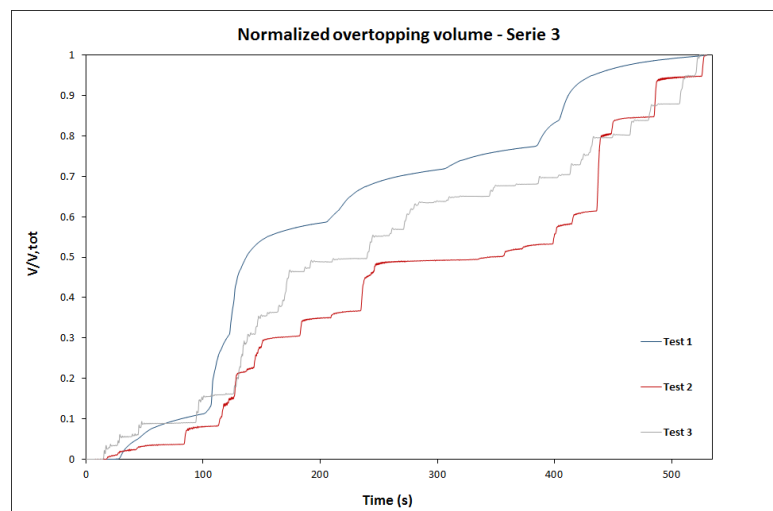


Figure 5.21 – Normalized overtopping volume for different wave field conditions and the same structure for Series 3.

The number of waves in the 550 s of test duration presented in Figure 5.21 was 511, 414 and 347 for run 1 (60% of $H_{s,d}$), run 2 (80% of $H_{s,d}$) and run 3 (100% $H_{s,d}$), respectively. On Figure 5.21, a comparison of normalized overtopping volumes between Tests 1, 2 and 3 of Serie 3, is made.

Test 1, with a significant wave height of 60 %, registered 511 incident waves. However, by observing Figure 5.21, Test 1 was the test with lesser overtopping waves. In Test 2, for an increase of the significant wave height to 80 %, a decrease on the number of incident waves, occurred, and an increase on the number of overtopping waves, was registered.

Finally, Test 3, the test with the higher value for significant wave height of 100 %, registered 347 number of incident waves (the lesser number of incident waves for this three tests), and the higher number of overtopping waves. Figure 5.22 show the normalized overtopping volumes for test series 4.

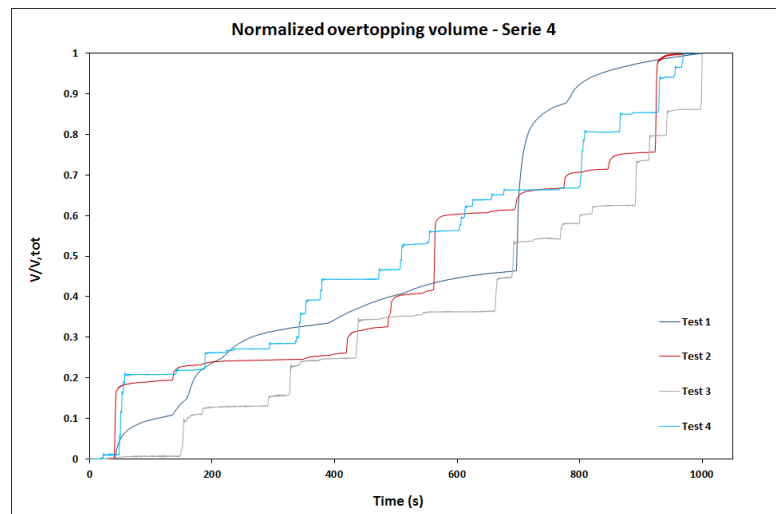


Figure 5.22 – Normalized overtopping volume for different wave field conditions and the same structure for Serie 4.

The number of waves in the 1050 s of test duration presented in Figure 5.22 was 975, 764, 628 and 545 for run 1 (60% of $H_{s,d}$), run 2 (80% of $H_{s,d}$), run 3 (100% $H_{s,d}$) and run 4 (120% $H_{s,d}$), respectively.

In this case, it is possible to analyse the four tests with reliable readings, due to the fact that all the tests were neither very short and neither very long in duration, filling the respective reservoir.

The same relation between the significant wave height and the number of incident waves and overtopping waves occurred in this series of tests. From Test 1 to Test 4, the significant wave height increases, with a range of values between 60 % to 120 %.

As the value is increased, the number of registered incident waves decreases, and in detriment, the number of overtopping waves increases. For this case, the Test with more overtopping waves (by observing Figure 5.22) is the Test 4. The subject matter related to the number of overtopping waves is more approached in the next section.

Summing up, an increase on the significant wave height results in a larger number of overtopping waves producing the total overtopping volume, i.e., the overtopping probability increases, that is given by the ratio between the number of overtopping waves and the number of incident waves.

5.7. WAVE-BY-WAVE OVERTOPPING ANALYSIS

This dissertation also included a wave-by-wave overtopping analysis. In order to define the individual wave overtopping events, a visual analysis of the curve that presents the cumulative overtopping volumes for each test was made.

5.7.1. MEASUREMENTS PROCEDURE OF EACH WAVE OVERTOPPING EVENT

For each experimental test, all individual wave-overtopping events were identified. Both the time series of the cumulative overtopping volumes and the record of the probe located at the entrance of the overtopping chute were taken into account in the analysis. All the graphics from each test are presented in the annexes. Some cases from this wave-by-wave overtopping analysis are presented in this section in order to explain all the process and measurements made for the different conditions of test and different levels of information available for each test. Figure 5.23 presents an example of the wave-by-wave overtopping analysis for Test 1 of Serie 1.

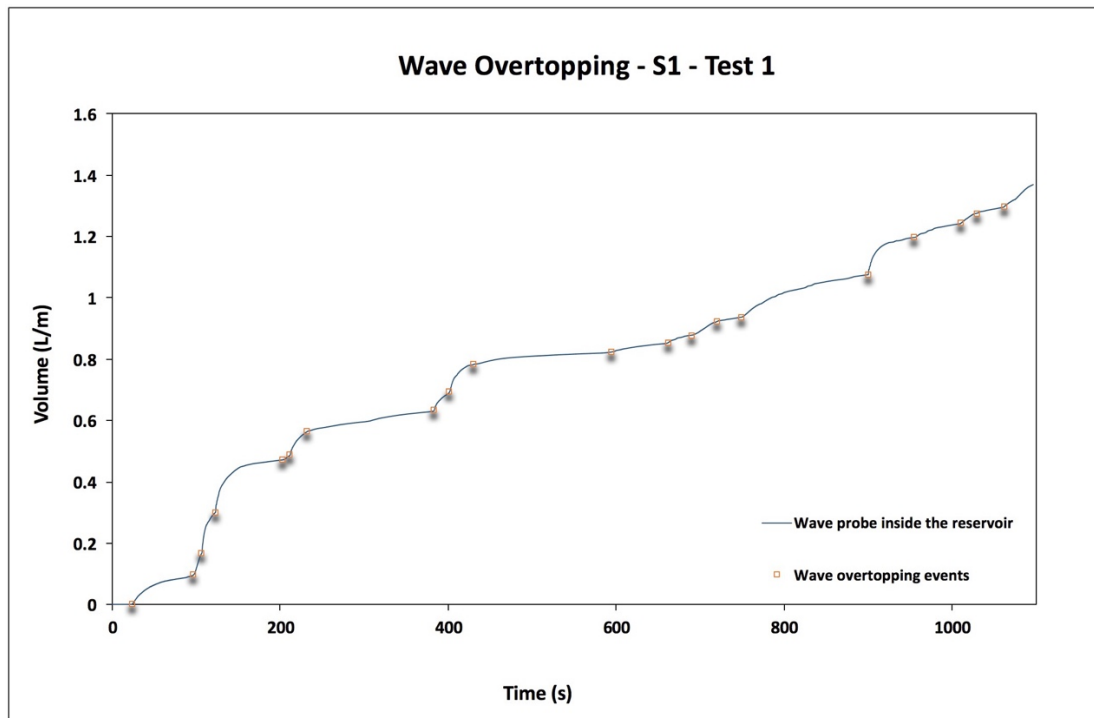


Figure 5.23 – Wave-by-wave overtopping.
Individual identification of the events for Test 1, Series 1.

The wave probe placed inside the reservoir for each test, recorded various water levels (inside the reservoir) during the generation of waves.

Those levels multiplied by the height of the used reservoir give the volume variation in the time domain of the test. Thus, it is possible to measure every volume variation inside the reservoir, i.e., every volume of each wave overtopping event.

Every “step” of the graphic is considered as a wave overtopping event. However, in some cases, the fluctuation of the water surface inside the reservoir when an overtopping wave enters the reservoir, might cause some unreliable readings.

For that matter, as it was referred before, some wave probes placed on top of the gutter gives the flow thickness at the crest of the structure, i.e., the water level that passes the overtopping chute, which allows to identify and support the readings of wave overtopping events.

When a wave overtops the crest of the structure, two wave probes record the event. In this test, there was no records from the wave probe placed on top of the overtopping chute recording each passage of overtopped water, due to the small wave overtopping volumes with very low water.

The wave is so small, that there is no contact between the wave probe and the water passing the overtopping chute, which for that reason, the overtopping identification in this test was made only with the information of the wave probe placed inside the reservoir.

In tests with 60% $H_{s,d}$, the wave overtopping events are very few and have small volumes per wave overtopping. For Test 3, the wave overtopping events were identified, Figure 5.24.

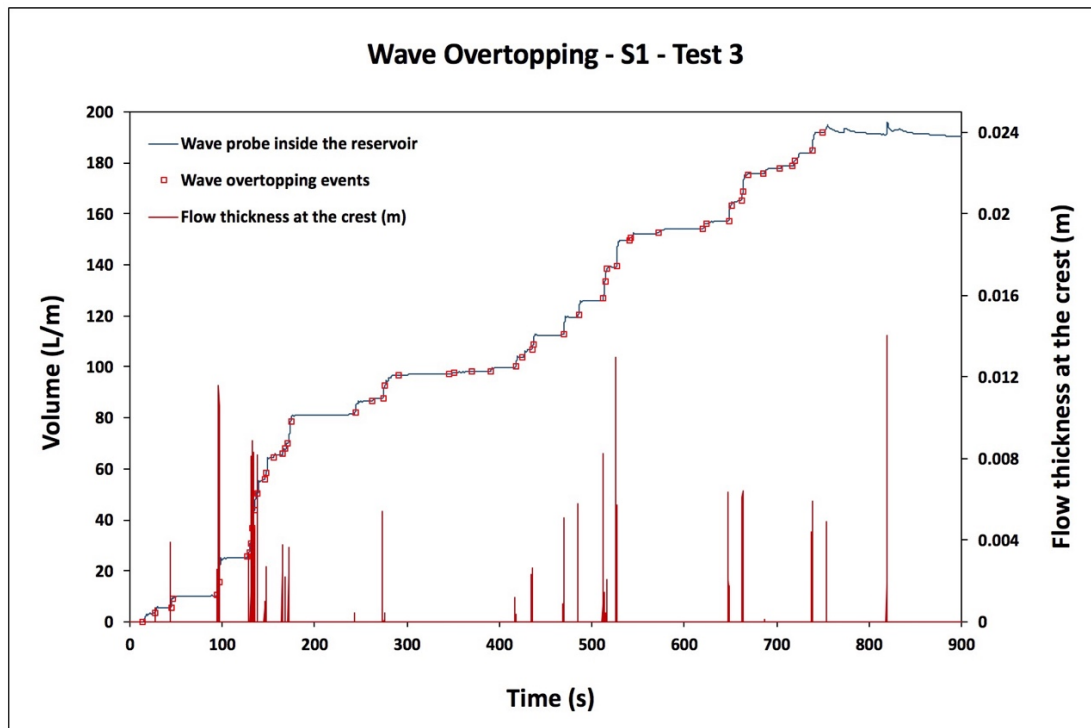


Figure 5.24 – Wave-by-wave overtopping.
Individual identification of the events for Test 3, Series 1.

In some cases such as the Test 3 of Serie 1 (e.g., Figure 5.24), the number of overtopping waves was very high, and in some parts of the record it is possible to identify the occurrence of consecutive wave overtopping events, by the observation of the wave probe placed at the crest of the structure on the overtopping chute.

For that reason, the analysis was made with additional detail and precautions, in order to identify each different event and the volume associated with it.

Furthermore, in test 3 of test series S1, there is many information given by the wave probe placed on top of the overtopping chute, which allows even more reliable readings on the wave overtopping events.

Figure 5.25 presents a zoom-in of test 3 of test series S1.

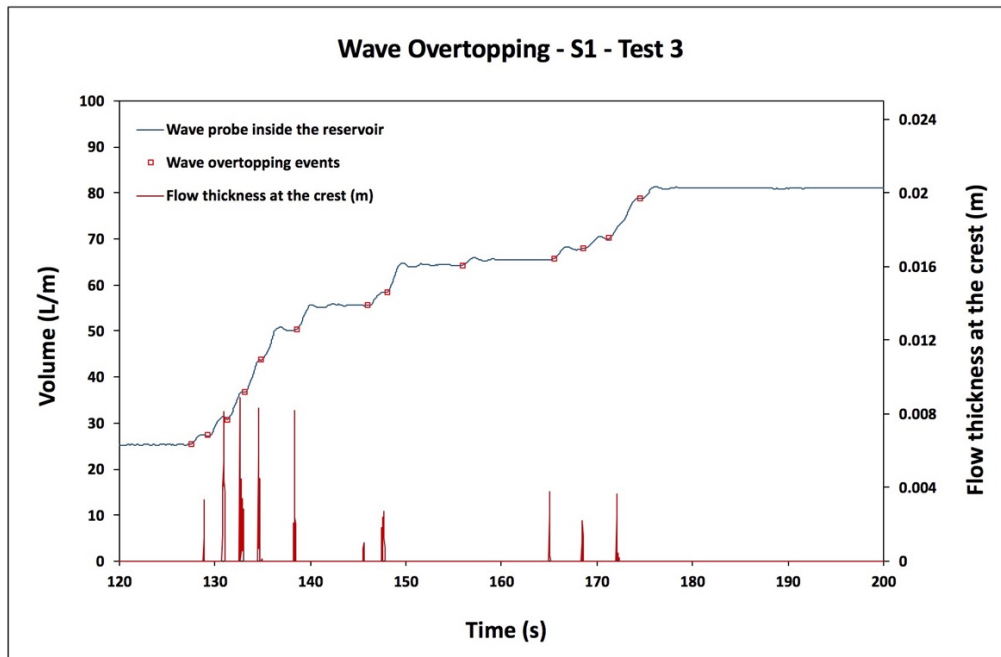


Figure 5.25 – Zoom-In of Figure 5.15 from Test 3, Series 1.

For Test 4, the identified wave overtopping events are presented in Figure 5.26.

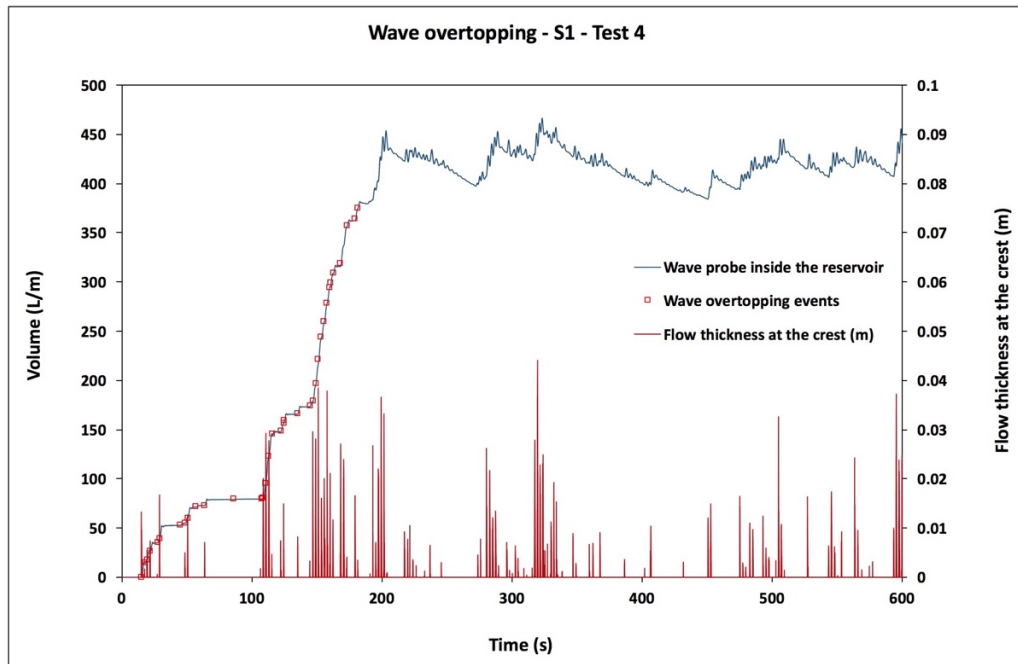


Figure 5.26 – Wave-by-wave overtopping.
Individual identification of the events for Test 4, Series 1.

The overtopping measurement in this test was very short, due to the rapid filling of the reservoir for a $H_s=120\%$ $H_{s,d}$. This behaviour occurred for all the four tests with 120% of $H_{s,d}$, due to the severe wave conditions used for these tests.

Another important aspect of this analysis is presented in Figure 5.27.

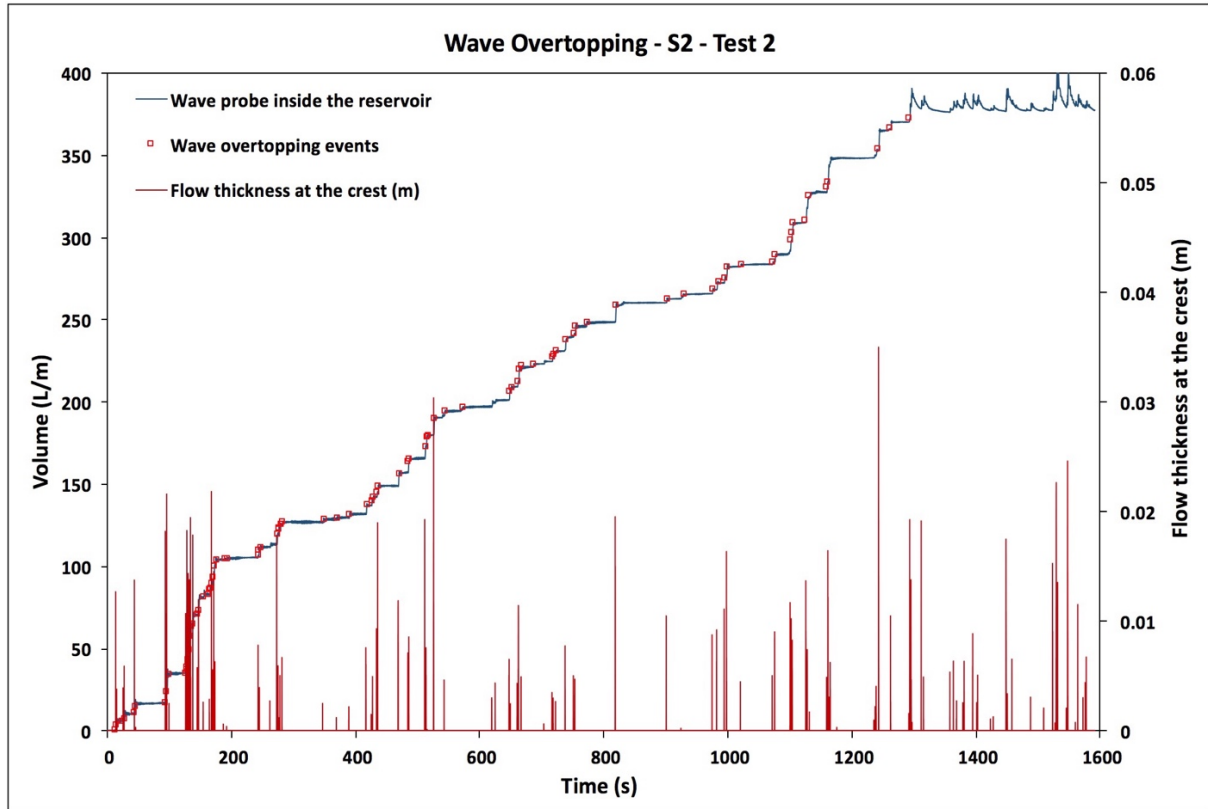


Figure 5.27 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Series 2.

In some tests, such as Test 2 of Serie 2, the wave probe placed on top of the overtopping chute recorded many overtopping events, given important information for this measurement.

This graphic presents an interesting test result, with many overtopping events, measured either by the wave probe inside the reservoir, either by the one placed on top of the overtopping chute, allowing very reliable measurements.

For Test 2 of Serie 3, the wave-by-wave overtopping analysis was made. In some cases, the wave probe placed at the crest of the structure (overtopping chute entrance) could not register all overtopping events.

This could be due to the fact that in tests with small overtopping waves, the flow thickness was smaller than the vertical gap between the chute and the wave probe, which could not be in contact since the chute was metallic.

So for this matter, the wave probe that gives more reliable records is the wave probe placed inside the reservoir, due to the fact that gives all the variations that occur inside the reservoir.

On the other hand, the wave probe placed on top of the overtopping chute, does not record all of the overtopping events as it was explained before.

In Figure 5.28, many wave overtopping events can be identified based on the record of the wave probe installed inside the reservoir, however, the wave probe at the crest of the structure only registered a few.

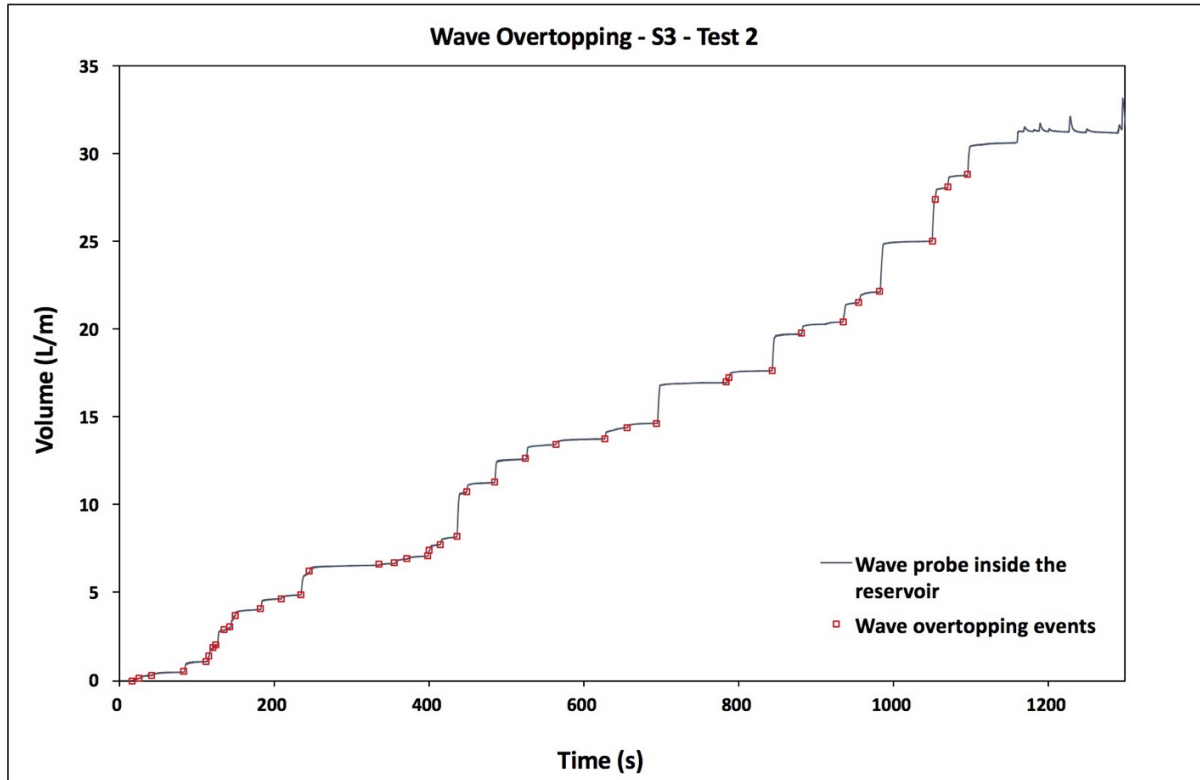


Figure 5.28 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Serie 3.

On this test, among others, the reservoir was only filled near the end of the test. Some tests end with the reservoir not completely filled, and in other cases the reservoir is full already at a few minutes of test duration.

On the case of this test, the last measured overtopping event is considered to be the penultimate wave overtopping event of the test, due to the fact that the last level variation or “step” of this graphic does not represent the entire wave overtopping volume of that last overtopping wave that filled the reservoir.

5.7.2. SHORT-CRESTED WAVES TESTS

All tests of test series 4 were carried out with short-crested waves. For Test 4, the wave-by-wave overtopping analysis was made. For Serie 4, with a consideration of short-crested waves, the wave-by-wave overtopping tended to have many wave overtopping events with small volumes per wave in average. This subject matter is further more elaborated in Section 5.10.

The disposition of wave overtopping events in the test 4 of Serie 4 is presented in Figure 5.29.

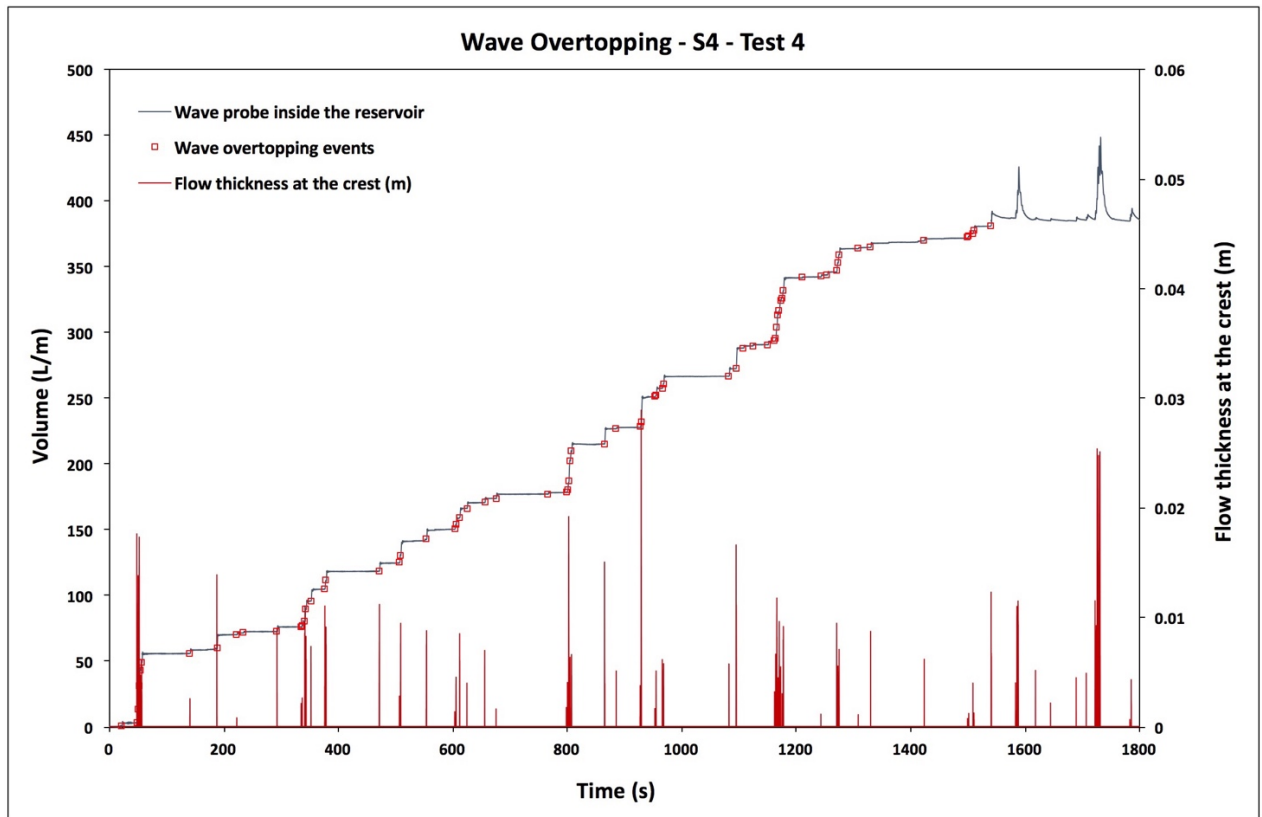


Figure 5.29 – Wave-by-wave overtopping.
Individual identification of the events for Test 4, Serie 4.

5.7.3. WAVE-BY-WAVE OVERTOPPING RESULTS

On Table 5.15 all the results from the wave-by-wave overtopping analysis for this laboratory experiment are presented.

It is not very accurate or correct to compare the total overtopped volume and mean overtopping discharge between all the tests, taking into account the fact that all the tests didn't end up at the same time, nor the reservoir of each test was completely full at the same time.

However, a few observations and conclusions can be made from this table, one of which, is the fact that for an increase of the significant wave height (Test 1 to Test 4 of all series), there is an increase on the total overtopped volume and mean overtopping discharge.

Despite that, these conclusions are just superficial, because in some cases, a test with an $H_{s,d}$ of 100% might filled the reservoir in less time than the other tests (for instance, 5 minutes), and in that case, the mean overtopping discharge and the total overtopped volume would not be comparable with other tests where the reservoir was filled in 20 minutes. These comparisons are correctly made in the following sections of this chapter.

Being the wave-by-wave overtopping analysis concluded, the next step is to represent and analyse the data in different forms, using the available formulae, mathematical distributions, softwares (e.g. NN-Overtopping), among other, in order to achieve the established objectives of this dissertation.

The results of all the tests series from this wave-by-wave overtopping analysis, are presented in Table 5.15.

Table 5.15 – Results of the wave-by-wave overtopping analysis, for S1-R1.

Test	Number of overtopping waves (Now)	Time of the first overtopping event (s)	Final time of the last overtopping event (s)	Total overtopped volume in model (L/m)	Total overtopped volume in prototype (m ³ /m)	q in prototype (m ³ /s/m)
S1 – R1	20	24.03	1080	1.330	1.630	0.0002553
S1 – R2	39	16.07	841.4	14.16	17.34	0.003483
S1 – R3	53	14.34	749.1	191.9	235.1	0.05304
S1 – R4	34	15.24	181.5	375.34	459.79	0.4282
S2 – R1	43	15.34	1048.28	26.13	32.01	0.00516
S2 – R2	87	13.8	1292.58	371.95	455.64	0.05958
S2 – R3	30	16.4	198.06	351.98	431.18	0.36798
S3 – R1	9	29	1066	0.963	1.180	0.000187
S3 – R2	40	17.54	1160	30.72	37.64	0.00548
S3 – R3	64	14.29	533.98	365.76	469.72	0.1487
S3 – R4	27	15.31	150.01	351.86	431.03	0.4857
S4 – R1	6	40	778	0.4068	0.4983	0.0001083
S4 – R2	18	40.57	1308	7.752	9.496	0.00123
S4 – R3	39	30	1490.41	90.959	111.424	0.0126
S4 – R4	69	20.86	1540.34	380.94	466.65	0.05121

5.8. DISTRIBUTION FOR OVERTOPPING WAVE VOLUMES

In this section is made a relation between the overtopping wave volume of each event (divided by the average of all the overtopping events of each test, in order to allow a reliable analysis and comparison between tests with different wave conditions), and the number of overtopping waves of the same test, in ascending order.

Figure 5.30 presents another representation for wave overtopping volumes per wave, for all the tests of the experiment overlapped in one graphic.

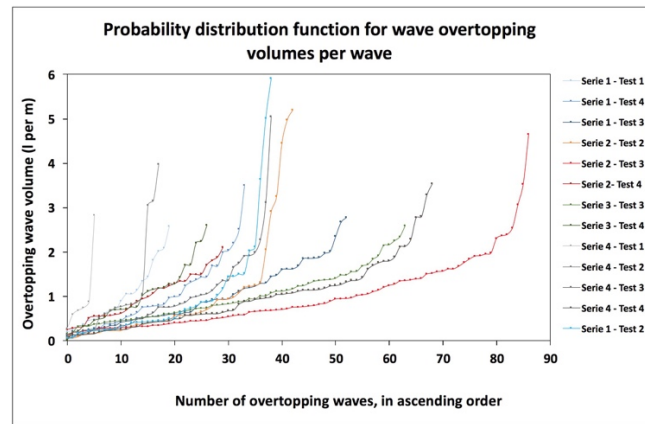


Figure 5.30 – Distribution for overtopping wave volumes, of all the tests.

The results obtained for each test series are presented separately in Figure 3.31.

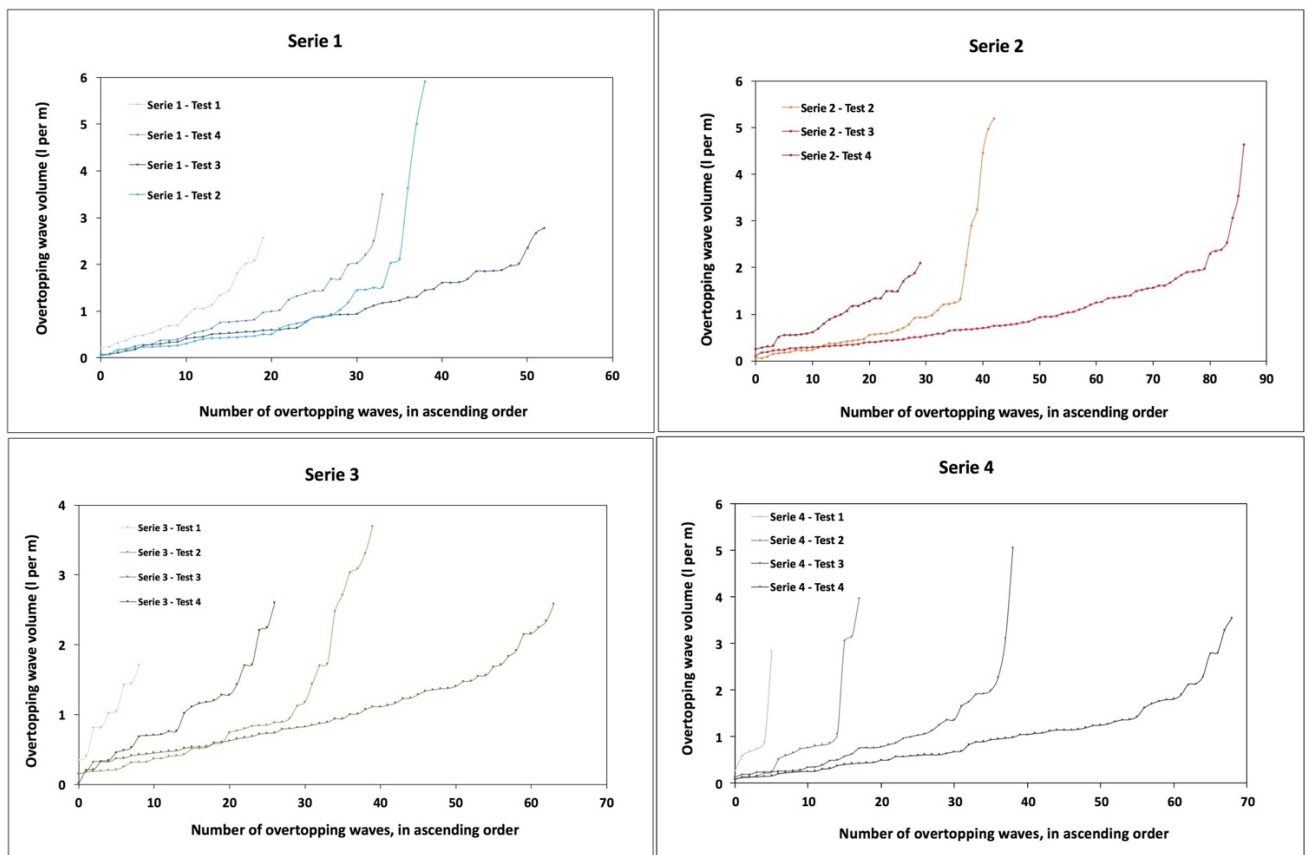


Figure 5.31 – Probability distribution function for wave overtopping volumes per wave, of each series of tests, separated.

It can be concluded that in some cases, with a smaller number of overtopping waves, a larger relative overtopped volume is reached.

5.9. SEA LEVEL RISE EFFECT ON WAVE OVERTOPPING

In order to analyze the sea level rise effects on wave overtopping, two tests (S3 – Test 3 and Test 4) were carried out with a water level increase of 0.023 m, from 0.566 to 0.589 m (model values). For Test 3 of S3, a comparison to the Test 2 of S2, with identical conditions (Table 5.16) is made.

Table 5.16 – Characterization of the tests S2R2 and S3R3 used to assess the sea level rise effects.

Test	Water Level (m)	H _s %	H _s (m)	T _p (s)	Type of Reservoir	Overtopping Chute Width (m)
S2 – R2	0.566	100 %	0.118	1.811	Large	0.2
S3 – R3	0.589	100 %	0.118	1.736	Large	0.2

Figure 5.32 presents the wave-by-wave overtopping curves for the two tests. The effect of the sea level rise introduced in the test S3-R3 is clearly visible on the measured overtopping volumes.

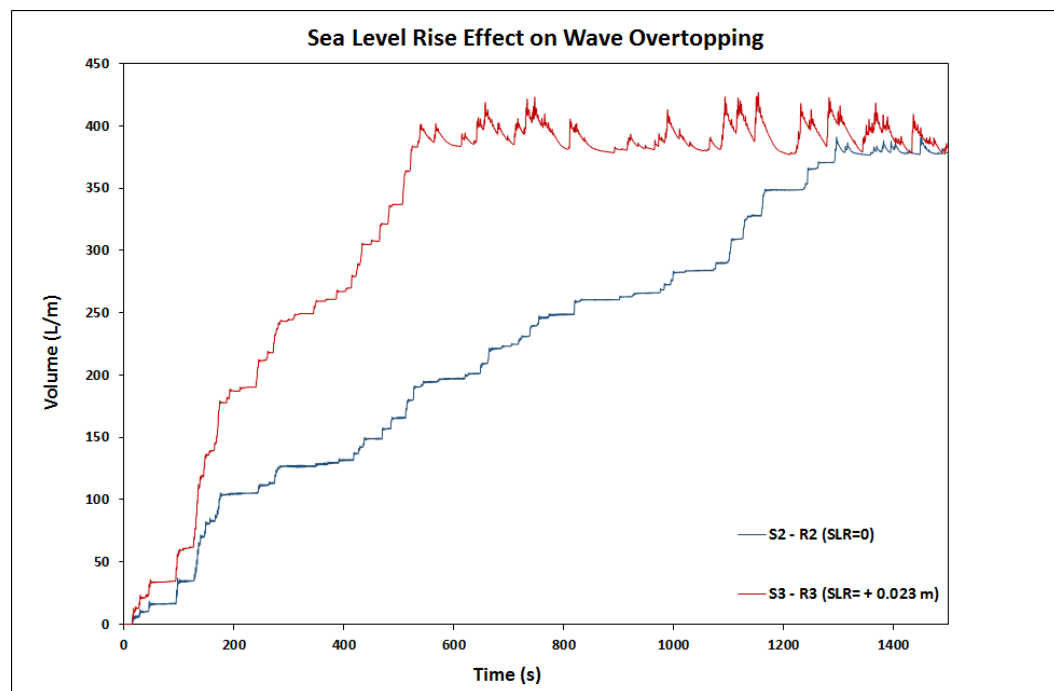


Figure 5.32 – Cumulative overtopping volumes for two tests carried out for the same wave conditions, but for different mean sea levels.

In Figure 5.32, at the time of 600 seconds more or less, the two graphics stop increasing the volume, and remain with many variations but with the same mean water level inside the reservoir, which means the reservoirs are filled in the two tests, while overtopping waves continued to enter.

The sea level rise effects are also visible in the curve that presents the flow thickness at the crest, as it can be seen in Figures 5.33 and 5.34.

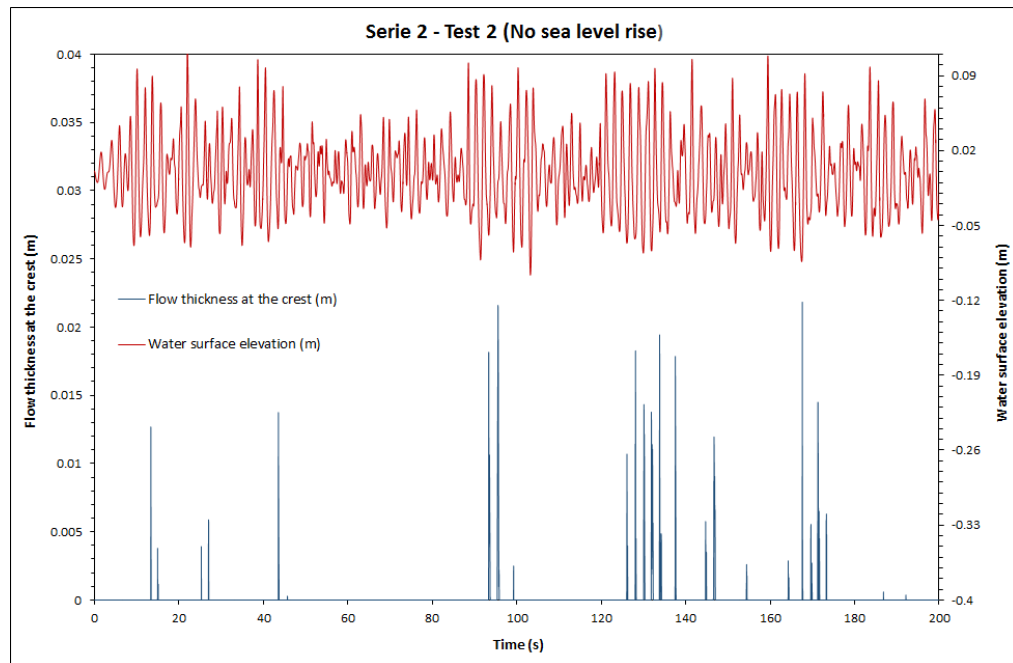


Figure 5.33 – Flow thickness at the crest and water surface elevation with no sea level rise.

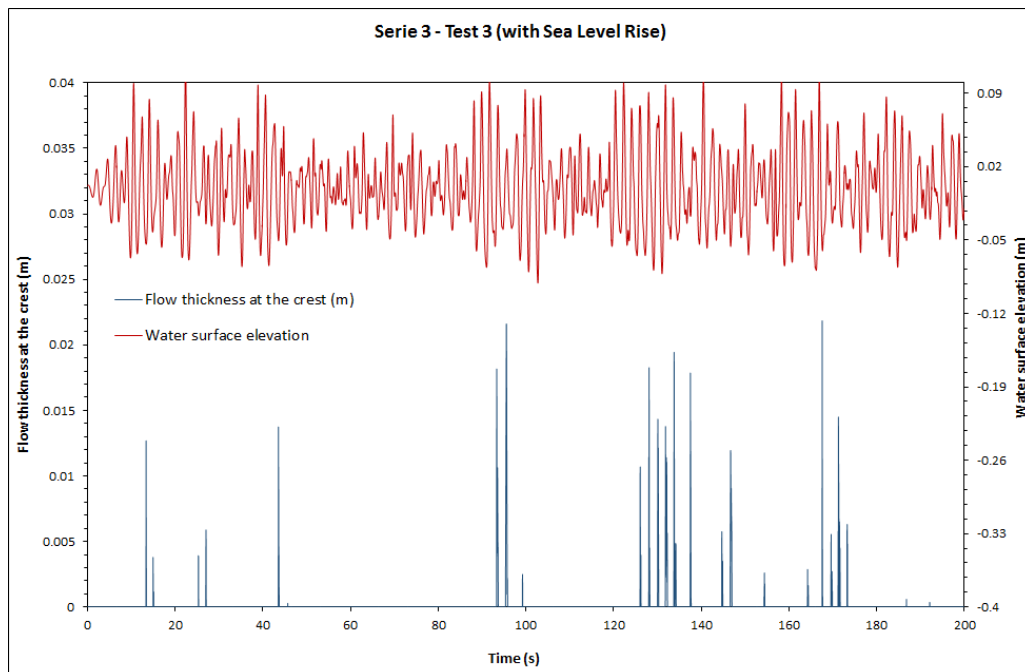


Figure 5.34 – Flow thickness at the crest and water surface elevation with sea level rise effects.

In a stretch of 520 s of the two tests, the results of the experimental analysis are presented in Table 5.17.

Table 5.17 – Influence of the sea level rise in the number of waves and wave overtopping.

Series	Number of generated waves in 520 s	Number of overtopping waves in 520 s	Total overtopped volume (m^3/m) in 520 s in prototype	Mean overtopping discharge q ($\text{m}^3/\text{s}/\text{m}$) in 520 s in prototype
S2 – R2	333	52	232.3	0.07582
S3 – R3	340	62	450.4	0.1463

The total overtopped volume and mean overtopping discharge are presented in prototype values. The mean overtopping discharge is calculated for the test duration of 520 s, a fixed value used for this assess.

Table 5.17 shows that for an identical number of incident waves and overtopping waves, the total overtopped volume and the mean overtopping discharge are larger in the case of a sea level rise consideration.

This means that on average, each overtopping wave in Test 3 have a larger volume than the overtopping waves from test 2.

The mean overtopping discharge is larger for the Test 3, which also means that the overtopping reservoir was filled in less time than in Test 2 (with no sea level rise).

For Test 4 of S3, a comparison to the Test 3 of S2, with identical conditions (Table 5.18) is made.

Table 5.18 – Generation conditions and results for two tests with a variation of water level.

Test	Water Level (m)	$H_s\%$	H_s (m)	T_p (s)	Type of Reservoir	Overtopping Chute Width (m)	Number of waves
S2 – R3	0.566	120 %	0.142	2.193	Large	0.2	1022
S3 – R4	0.589	120%	0.142	2.193	Large	0.2	1025

Figure 5.35 presents the wave-by-wave overtopping graphic of the two tests of Table 5.18 overlapped, also recorded by the wave probe placed inside the reservoir.

On Test 4 of Serie 3, the effects of the sea level rise on overtopping is clearly visible.

Considering the sea level rise (Test 4), each overtopping wave has larger volume than with no sea level rise (Test 3).

In Figure 5.35, it is visible that the overtopping events of the two different tests are mainly happening at the same time, but with different volumes for each overtopping wave.

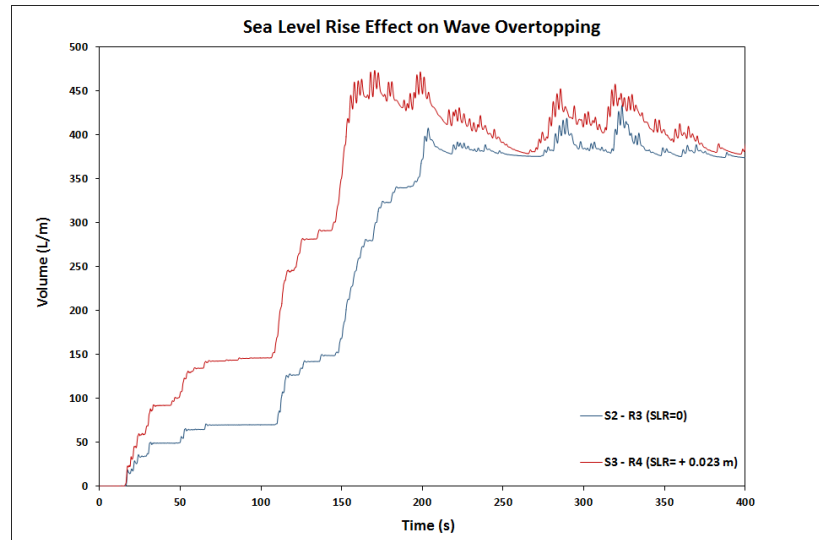


Figure 5.35 – Cumulative overtopping volumes for two tests carried out for the same wave conditions, but for different mean sea levels.

In Figure 5.35, it is visible that the overtopping events of the two different tests are mainly happening at the same time, but with different volumes for each overtopping wave. Considering the sea level rise (Test 4), each overtopping wave has larger volume than with no sea level rise (Test 3). The sea level rise effects are also visible in a graphic of the flow thickness at the crest, and in water surface variation, as it can be seen in Figure 5.36 and Figure 5.37. In these cases, it is used a stretch of 150 s, for a T_p of 2.193 s.

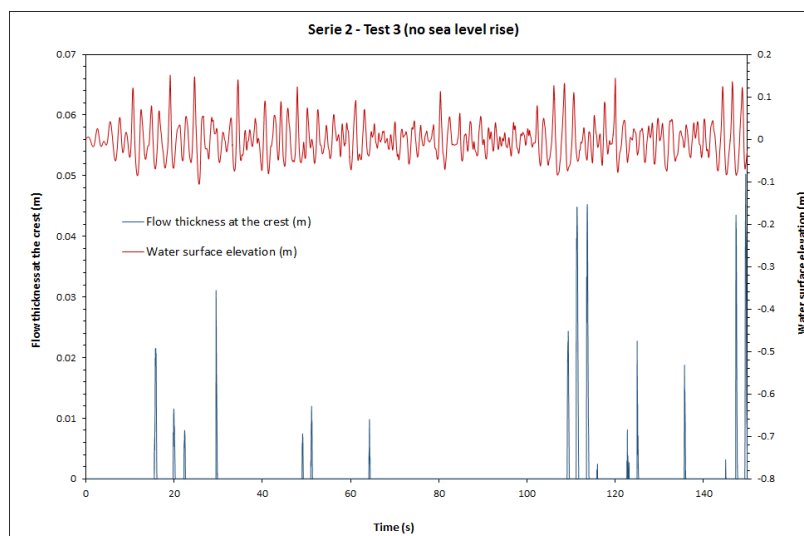


Figure 5.36 – Flow thickness at the crest and water surface elevation with no sea level rise.

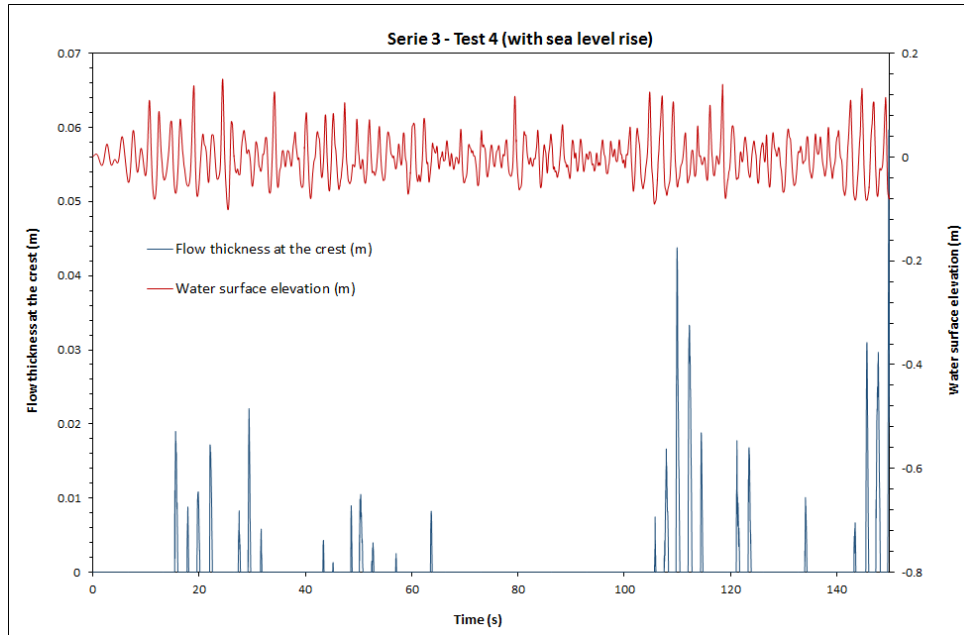


Figure 5.37 – Flow thickness at the crest and water surface elevation with sea level rise consideration.

Table 5.19 presents the results comparison between the Test 3 without sea level rise consideration, and Test 4 with that value taken into account.

Table 5.19 - Influence of the sea level rise in the number of waves and wave overtopping.

Series	Number of generated waves in 150 s	Number of overtopping waves in 150 s	Total overtopped volume of the test (m^3/m) in 150 s in prototype	q ($\text{m}^3/\text{s}/\text{m}$) in 150 s in prototype
S2 – R3	83	17	230.5	0.2598
S3 – R4	84	27	431.0	0.4857

On Table 5.19 is presented the number of generated waves in 150 s (the time of the measurement), such as the wave overtopping events that occurred for each test. The total overtopped volume and the mean overtopping discharge are from the entire tests measurement and expressed in prototype values as well.

Again, considering a sea level rise of 0.023 m, the number of generated waves and overtopping waves are approximately the same. The total overtopped volume of the two tests are equal for a 150 s duration,

However, the mean overtopping discharge is higher in the Test 4 with sea level rise consideration, which means that in Test 4, the reservoir was filled faster than in Test 3.

In this case, the total overtopped volume is the same because the reservoir used in these two tests was the same of the large reservoir. Taking into account that in 150 s of test, the two reservoirs were filled, it is logical the identical values for total overtopped volume.

Concluding, the consideration of a sea level rise scenario results in higher wave overtopping volumes. However, for the conditions tested in this work, the number of overtopping waves is not significantly affected.

For the same value of significant wave height, it appears that the wave overtopping events for the two groups of two tests, even with different water levels (with or without SLR), occurred at the same time, but with different magnitudes, causing higher mean overtopping discharge in the tests with sea level rise.

Another aspect observed in Table 5.17 for the first comparison, is that the total overtopped volume was double in the case of a sea level rise consideration.

This can be explained by the fact that the two reservoirs were not filled at 200 s duration of test, unlike the second comparison.

5.10. SHORT-CRESTED WAVES INFLUENCE ON WAVE OVERTOPPING – DIRECTIONAL SPREADING

In this experimental work, short-crested waves were used in teste series S4, in order to evaluate their influence in wave overtopping. Comparisons between the results of S4 tests with the ones from the other series, for the same wave conditions, is made.

Figure 5.38 presents a comparison between wave overtopping discharge for test 1 of test series S1, S3 and S4, for the total duration of tests.

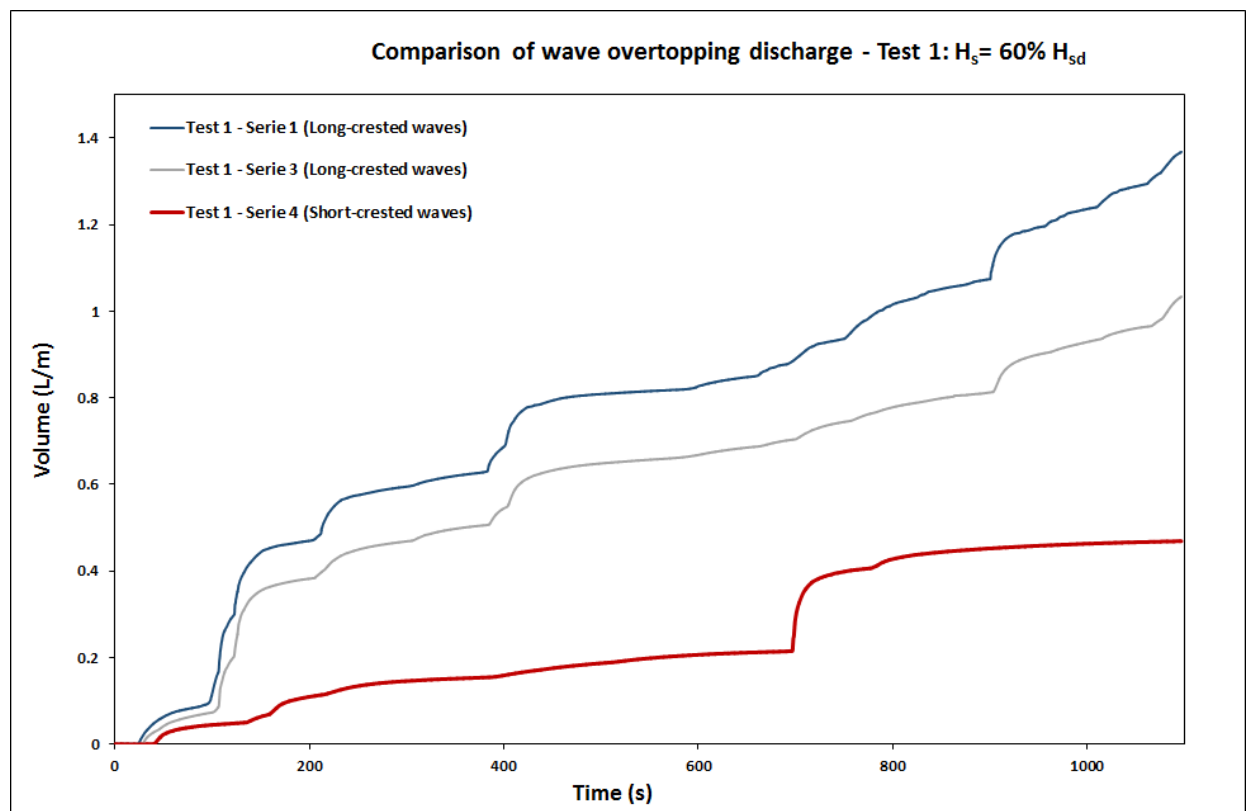


Figure 5.38 – Comparison of wave overtopping discharge between series, for test 1. Analysis of the short-crested waves influence in wave overtopping.

For this analysis, in order to allow a reliable comparison between the series, a fixed value of time is used. The lowest time of the group of tests is fixed and used for the other tests. Then it is possible to do a comparison for the same time, and analyse the influence of short-crested waves in overtopping with reliability. The experimental results obtained from the wave-by-wave analyzes in Section 5.6, are displayed in Table 5.20.

Table 5.20 – Experimental results from wave-by-wave overtopping analyzes for Test 1.
Influence of short-crested waves in wave overtopping.

Test 1	L/S crested	Time of the total test (s)	Number of incident waves (N_w)	Number of overtopping waves (N_{ow})	P_{ow} (Probability of overtopping per wave)	Volume in prototype (m^3/m)	q (m^3/s) in prototype
Serie 1	Long	1080	1051	20	0.01903	1.631	0.0002553
Serie 3	Long	1066	1057	9	0.008515	1.180	0.0001871
Serie 4	Short	778	1070	6	0.007905	0.4983	0.0001083

On the first analysis, a comparison between test 1 of Serie 1, 3 and 4 is made. With a significant wave height of 60% of the design wave height, these three tests are compared for the total duration of each test, due to the fact that none of the reservoirs were completely filled. On Table 5.20 it is shown how short-crested waves influence the wave overtopping, as well as the mean overtopping discharge and probability of overtopping per wave. There is less number of overtopping waves in the case of a short-crested waves test. The total overtopped volume and the mean overtopping discharge are lower in the short-crested waves test in Serie 4. So, for this comparison, it can be concluded that for a short-crested waves scenario, the number of incident waves is larger, the number of overtopping waves are lower, and the total wave overtopping such as the mean overtopping discharge decreases to at least half of the value for the case scenario with long-crested waves. Figure 5.39 presents a comparison between wave overtopping discharge of the four series, for Test 2, and for the total time of the tests.

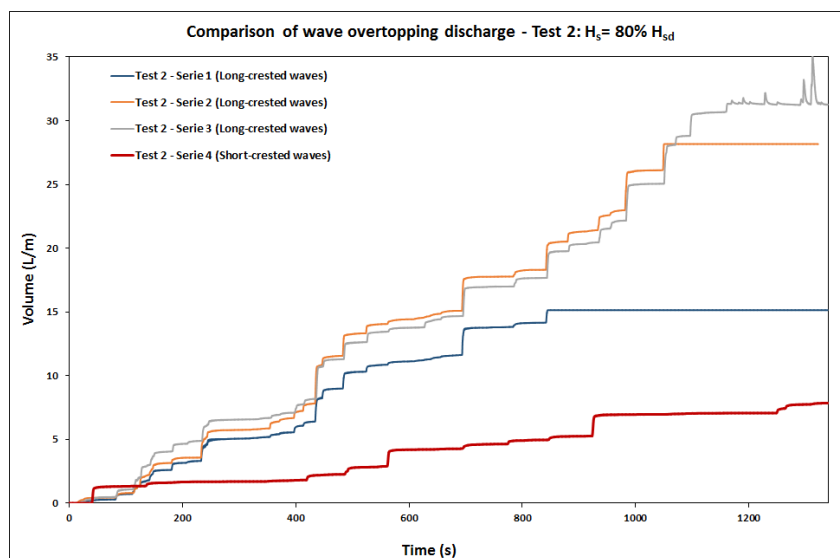


Figure 5.39 – Comparison of wave overtopping discharge between series, for Test 2. Analysis of the short-crested waves influence in wave overtopping.

Figure 5.40 presents a comparison between wave overtopping discharge of the four series, for Test 2, and for 841.42 s of test duration.

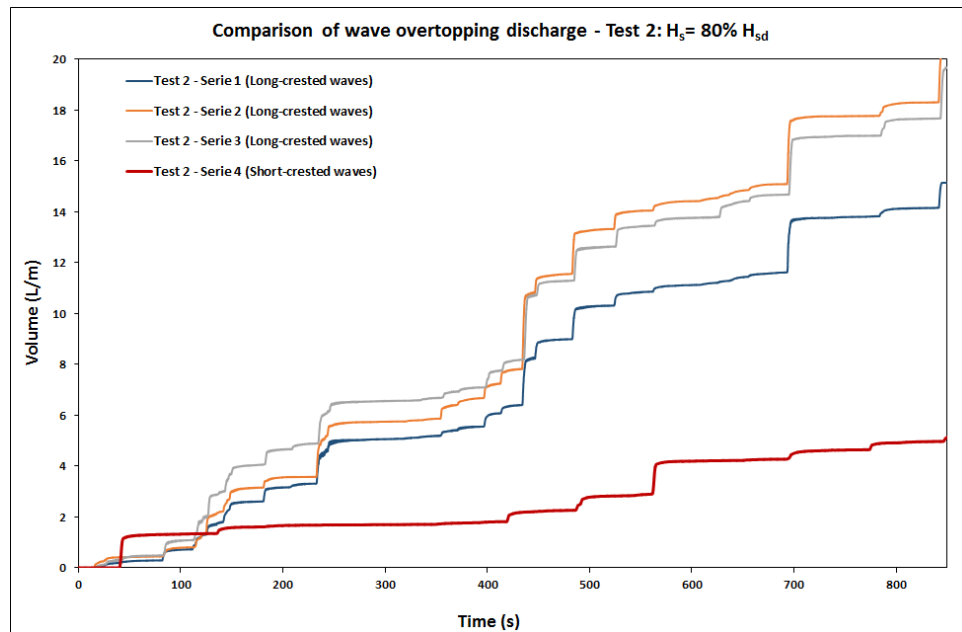


Figure 5.40 – Comparison of wave overtopping discharge between series, for Test 2. Analysis of the short-crested waves influence in wave overtopping.

For an analysis in the same time domain, the comparisons are made for values relative to a 841.42 s duration of test for Test 2, Table 5.21.

Table 5.21 – Experimental results from wave-by-wave overtopping analysis for Test 2.

Influence of short-crested waves in wave overtopping.

Test 2 (In 841.42 s of test duration)	L/S crested	Time of the total measurement (s) in model	Number of incident waves during measurement (N_w)	Number of overtopping waves during measurement (N_{ow})	P_{ow} (Probability of overtopping per wave)	Total overtopped volume in prototype (m^3/m)	q (m^3/s) in prototype
Serie 1	Long	841.42	653	39	0.05972	17.34	0.003483
Serie 2	Long	1048.3	665	36	0.05414	24.84	0.004990
Serie 3	Long	1160	657	32	0.04871	24.22	0.004866
Serie 4	Short	1308	643	13	0.02022	6.452	0.001296

For the second analysis, the Test 2 of all the series are compared and overlapped in one graphic.

The same significant wave height of 80 % H_{sd} is used. Series 1, 2 and 3 used long-crested waves, and serie 4 used short-crested waves. In order to make a reliable comparison, the same time domain of 841.42 s is used for the overlapped graphic presented in Figure 5.41.

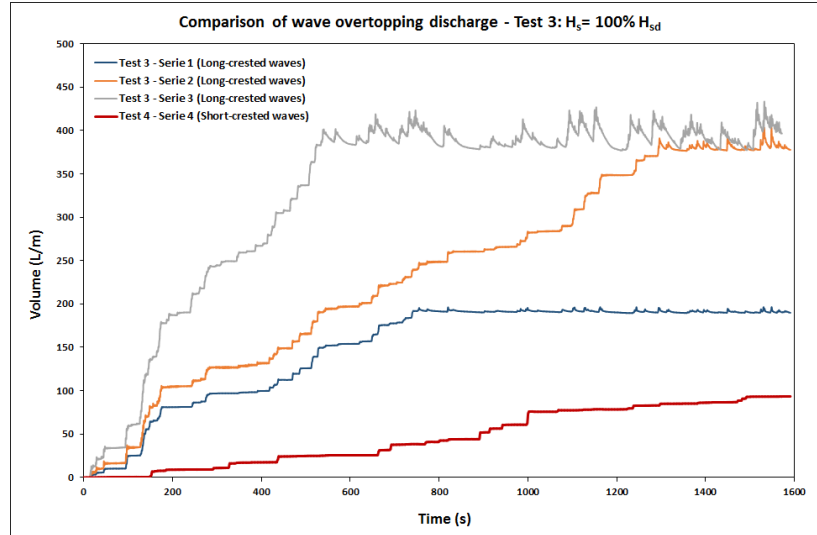


Figure 5.41 – Comparison of wave overtopping discharge between series, for Test 3. Analysis of the short-crested waves influence in wave overtopping.

It is presented a comparison between wave overtopping discharge during the total test of the four Series, for Test 3. For that duration, it appears that the number of incident waves, such as the overtopping waves are lower in Serie 4 with short-crested waves. The volume of each wave is also lower on these cases, resulting in the lowest total overtopped volume and mean overtopping discharge of the tests. Figure 5.42 presents a comparison between wave overtopping discharge of the four series, for Test 3, and for 533.98 s of test duration.

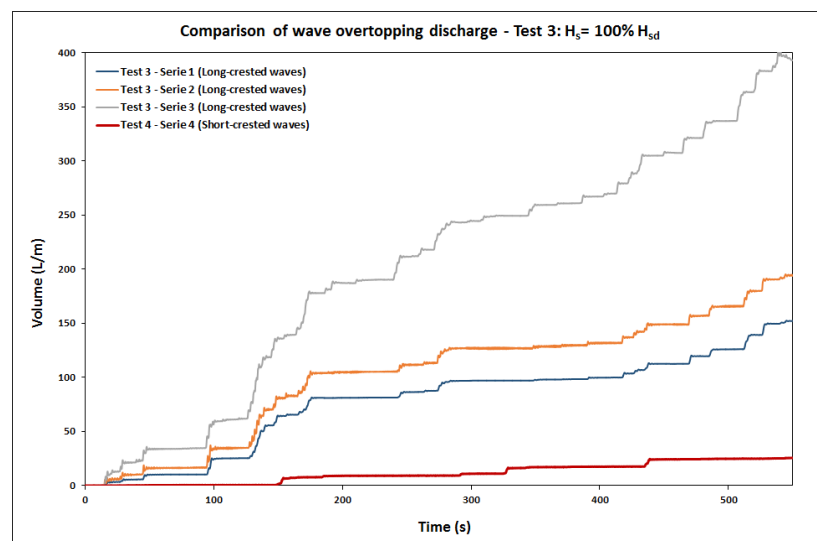


Figure 5.42 – Comparison of wave overtopping discharge between series, for Test 3. Analysis of the short-crested waves influence in wave overtopping.

For an analysis in the same time domain, the comparisons are made for values respective to a 533.98 s duration of test for Test 3, Table 5.22.

Table 5.22 – Experimental results from wave-by-wave overtopping analysis for Test 3.
Influence of short-crested waves in wave overtopping.

Test 3 (In 533.98 s of test duration)	L/S crested	Time of the total measurement (s) in model	Number of incident waves during measurement (N_w) in 533.98 s	Number of overtopping waves during measurement (N_{ow}) in 533.98 s	P_{ow} (Probability of overtopping per wave)	Total overtopped volume in prototype (m^3/m) in 533.98 s	q (m^3/s) in prototype in 533.9 s
Serie 1	Long	749.1	343	38	0.1108	183.2	0.05799
Serie 2	Long	1292.6	342	53	0.1550	237.7	0.07525
Serie 3	Long	533.9	350	64	0.1829	469.7	0.1487
Serie 4	Short	1490.4	335	14	0.04179	31.55	0.009988

For a significant wave height of 100 % $H_{s,d}$, the differences between tests with short-crested waves and long-crested waves increases. Through the observation of Figure 5.42, for a duration of 533.98 s, the number of incident waves and overtopping waves are lowest in Serie 4 (case scenario with short-crested waves). For long-crested waves, the total overtopped volumes registered a range of values of 183 to 470 m^3/m (in prototype). However in short-crested waves scenario, the total overtopped volume was 32 m^3/m . The mean overtopping discharge also decreases severely for a short-crested waves consideration. The same conclusions about the influence of short-crested waves in wave overtopping can be made for this comparison. Figure 5.43 presents a comparison between wave overtopping discharge of the four series, for Test 4, in the total duration of the tests.

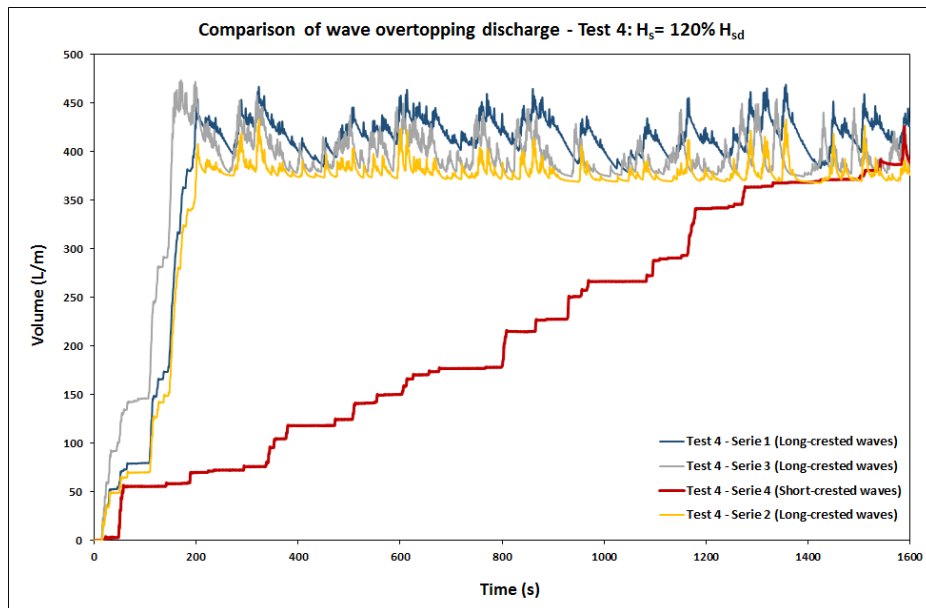


Figure 5.43 – Comparison of wave overtopping discharge between series, for Test 4, and for the total test duration. Analysis of the short-crested waves influence in wave overtopping.

Figure 5.44 presents a comparison between wave overtopping discharge of the four series, for Test 4, in 150.01 s of test duration.

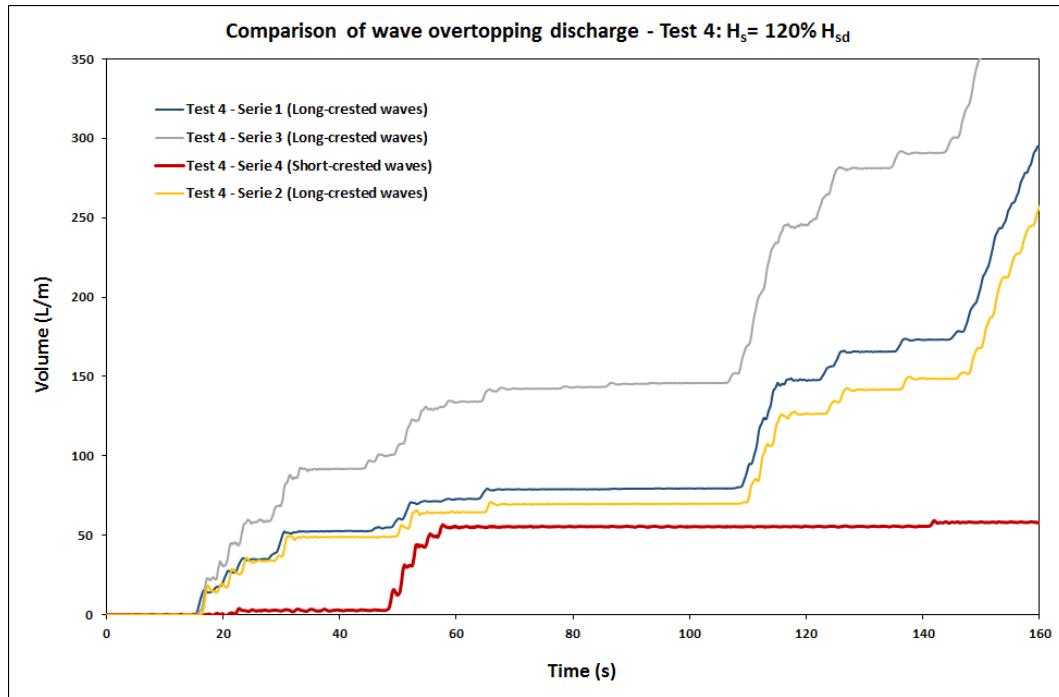


Figure 5.44 – Comparison of wave overtopping discharge between series, for Test 4. Analysis of the short-crested waves influence in wave overtopping.

For an analysis in the same time domain, the comparisons are made for values relative to 150.01 s of duration of test for Test 4, Table 5.23.

Table 5.23 – Experimental results from wave-by-wave overtopping analysis for Test 4.
Influence of short-crested waves in wave overtopping.

Test 4 (In 150 s of test duration)	L/S crested	Time of the total measurement (s) in model	Number of incident waves during measurement (N_w) in 150 s	Number of overtopping waves during measurement (N_{ow}) in 150 s	P_{ov} (Probability of overtopping per wave = N_{ow}/N_w)	Total overtopped volume in prototype (m^3/m) in 150 s	q (m^3/s) in prototype in 150.01 s
Serie 1	Long	181.5	83	24	0.2892	271.5	0.3059
Serie 2	Long	198.1	83	17	0.2048	230.5	0.2597
Serie 3	Long	150	84	27	0.3214	431	0.4857
Serie 4	Short	1540.3	82	8	0.09756	72.87	0.08211

It is visible that in the tests with long-crested waves, the effective overtopping measurement duration was very short, due to the fast filling of the reservoirs.

After 200 s of test, the overtopping reservoir was already filled in the three tests with long-crested waves. On the contrary, in the test with short-crested waves, the reservoir was only filled after 1500 s of test duration.

Although the number of incident waves in the tests are almost the same, for the tests with short-crested waves, a lesser number of overtopping waves was registered.

For time duration of 150 s, the lowest value of total overtopped volume and mean overtopping discharge was obtained in the test of series 4, carried out with short-crested waves.

5.11. WEIBULL DISTRIBUTION AND PROBABILITY FUNCTION – DATA ANALYSIS

5.11.1. TEST SERIES 1

On this section, a presentation of the data analysis using the Weibull distribution and probability density function (see Chapter 3) is made. Figure 5.45 presents the exceedance probability function for Test 1 of Serie 1.

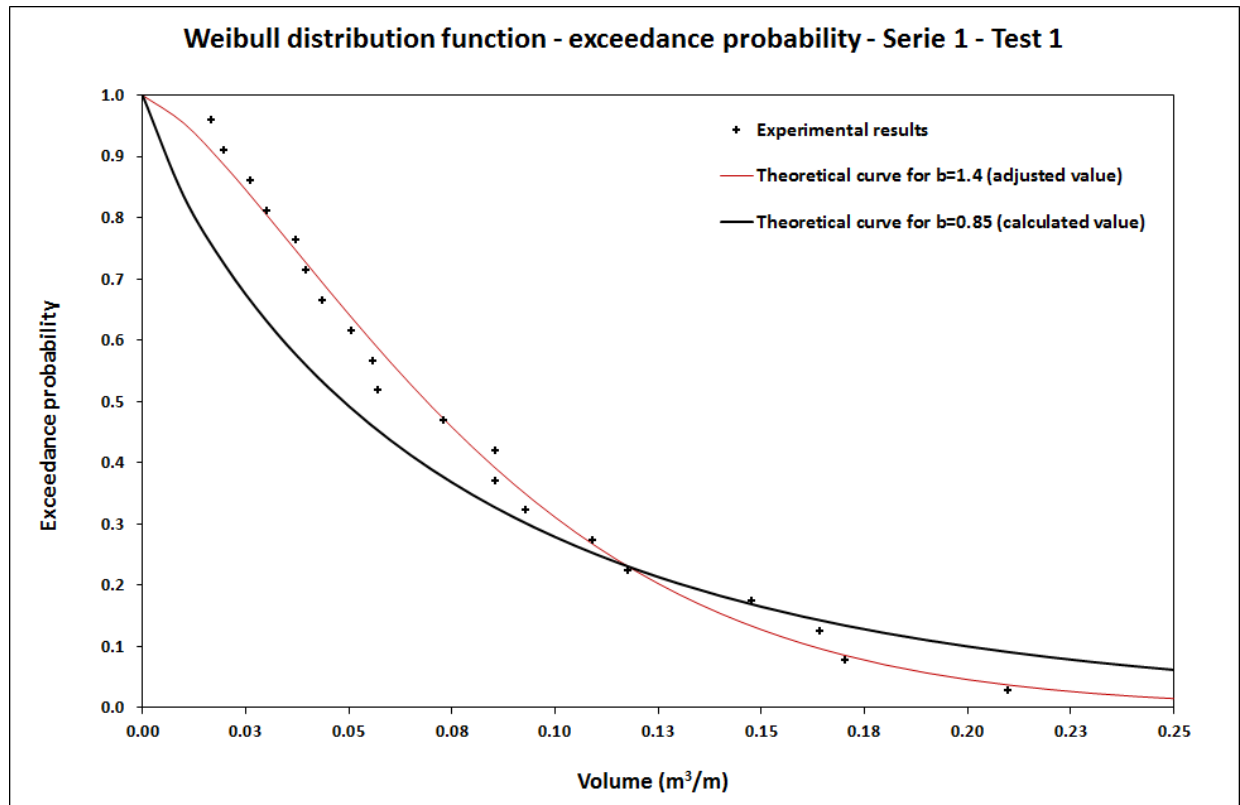


Figure 5.45 – Weibull distribution function, of overtopping values for Test 1.

Figure 5.46 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 1 of Serie 1.

Figure 5.47 presents the exceedance probability function for Test 2 of Serie 1.

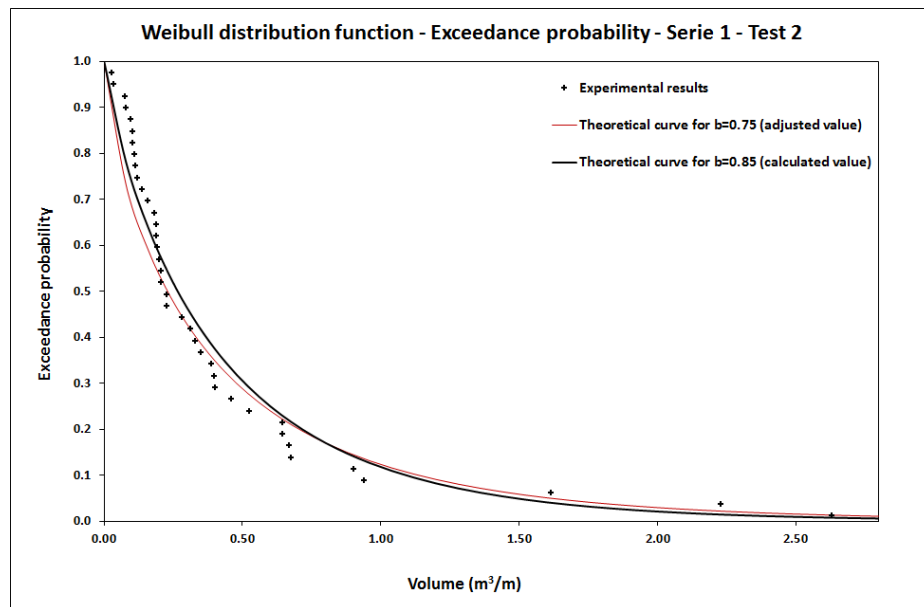


Figure 5.46 – Weibull distribution function, of overtopping values for Test 2.

Figure 5.48 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 2 of Serie 1.

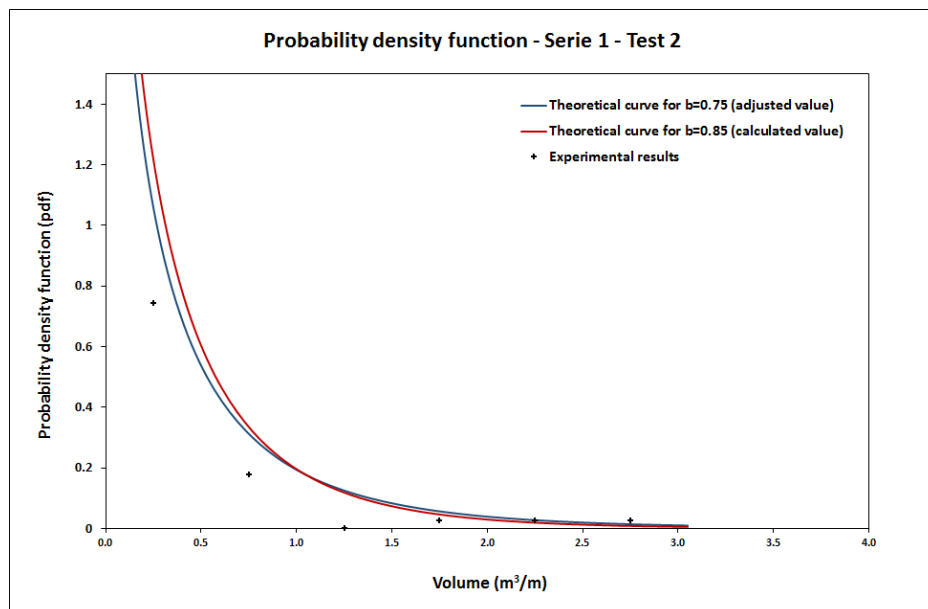


Figure 5.47 – Probability density function (pdf), for Test 2.

Figure 5.49 presents the exceedance probability function for Test 3 of Serie 1.

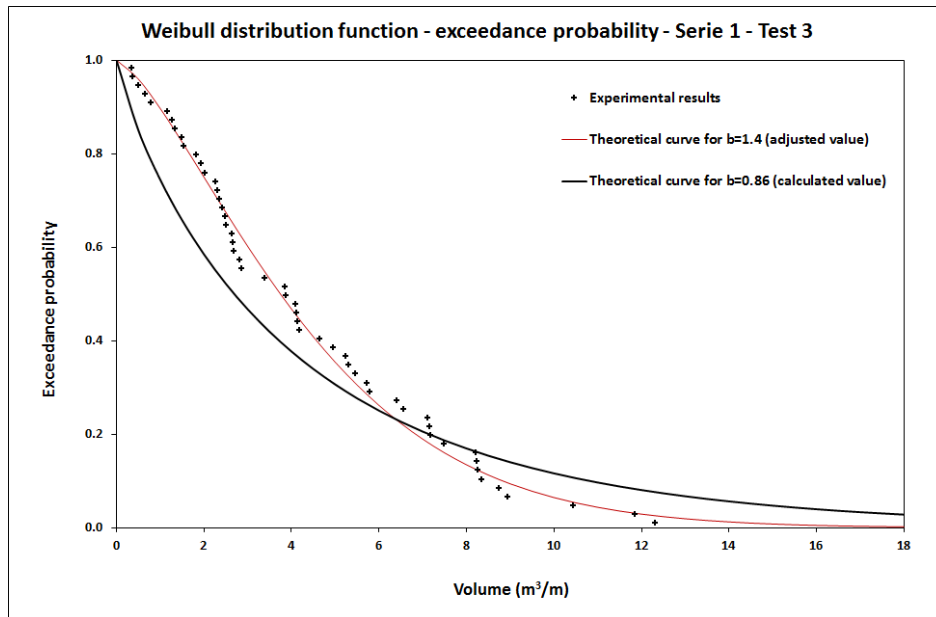


Figure 5.48 – Weibull distribution function of overtopping values, for Test 3.

Figure 5.50 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 3 of Serie 1.

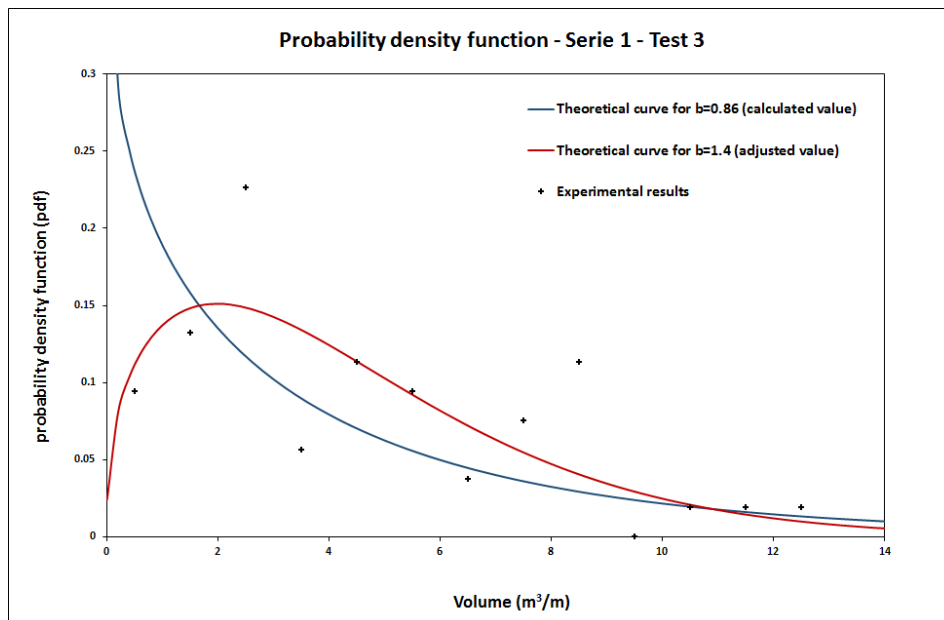


Figure 5.49 – Probability density function (pdf), for Test 3.

Figure 5.51 presents the exceedance probability function for Test 4 of Serie 1.

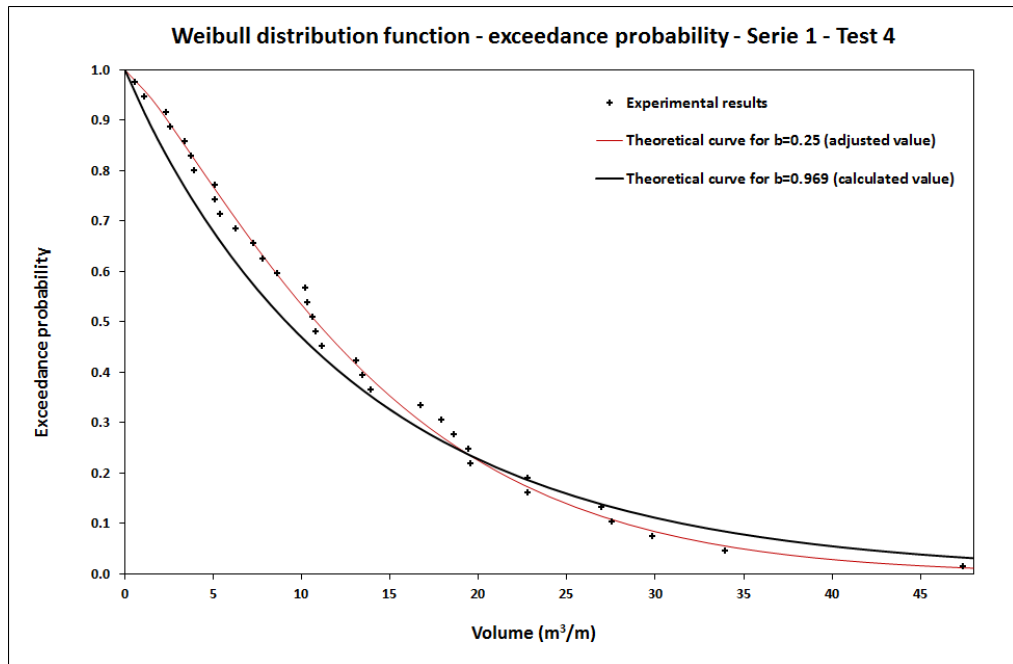


Figure 5.50 – Weibull distribution function of overtopping values, for Test 4.

Figure 5.52 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 4 of Serie 1.

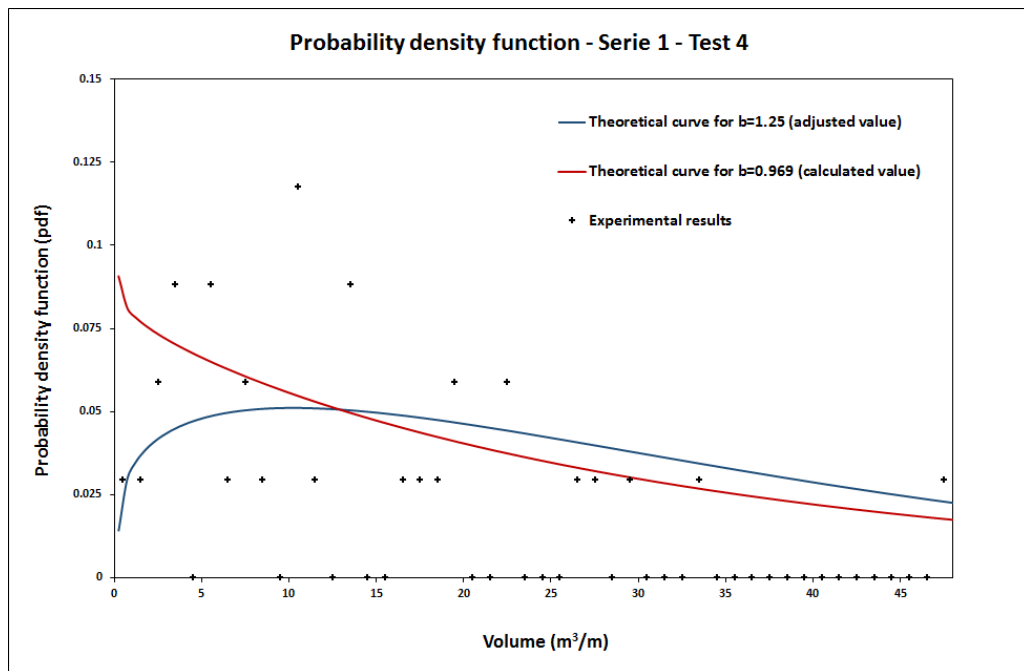


Figure 5.51 – Probability density function (pdf), for Test 4.

5.11.2. TEST SERIES 2

Figure 5.53 presents the exceedance probability function for Test 1 of Serie 2.

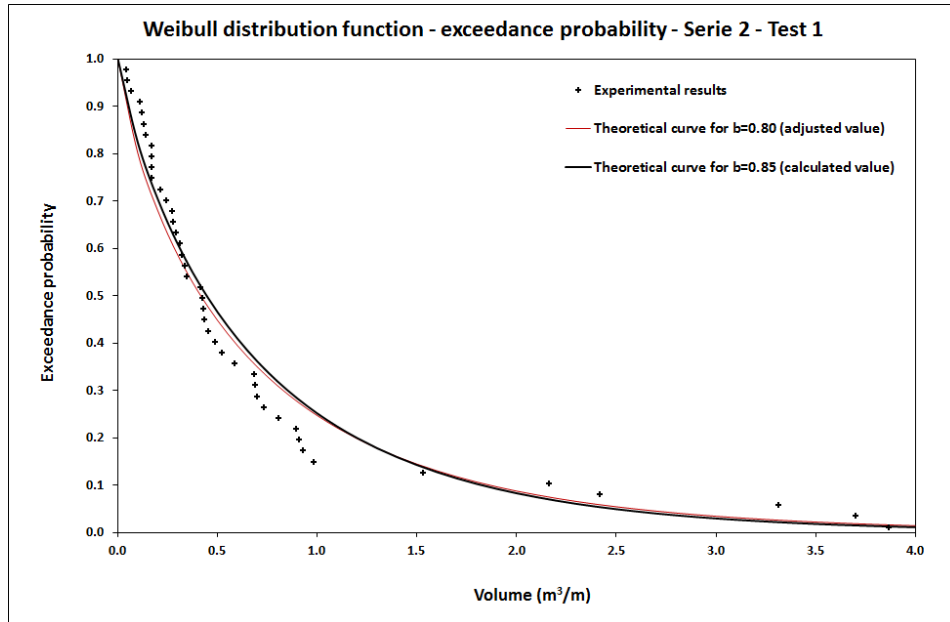


Figure 5.52 – Weibull distribution function of overtopping values, for Test 1.

Figure 5.54 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 1 of Serie 2.

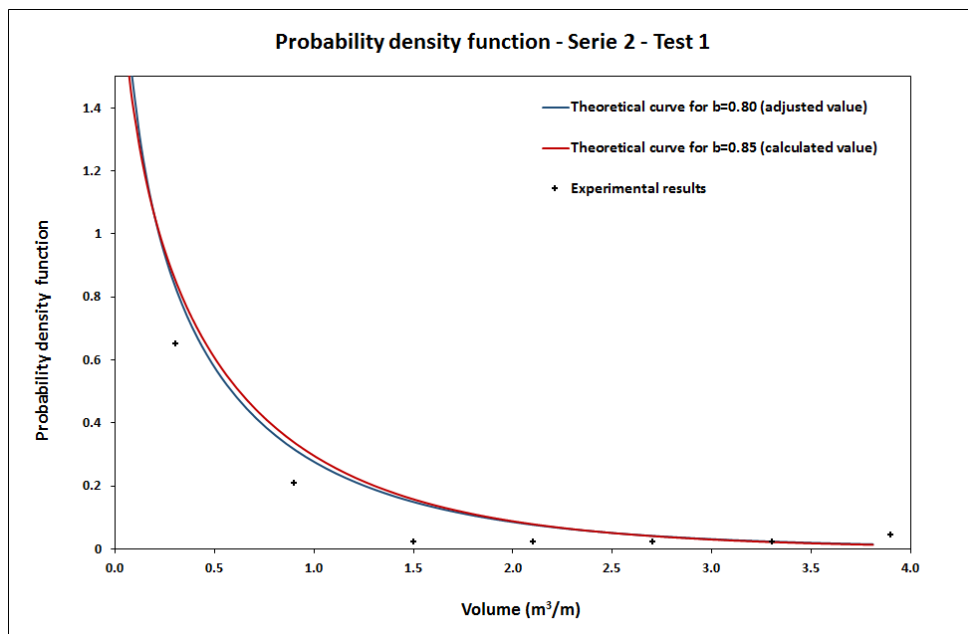


Figure 5.53 – Probability density function (pdf), for Test 1.

Figure 5.55 presents the exceedance probability function for Test 2 of Serie 2.

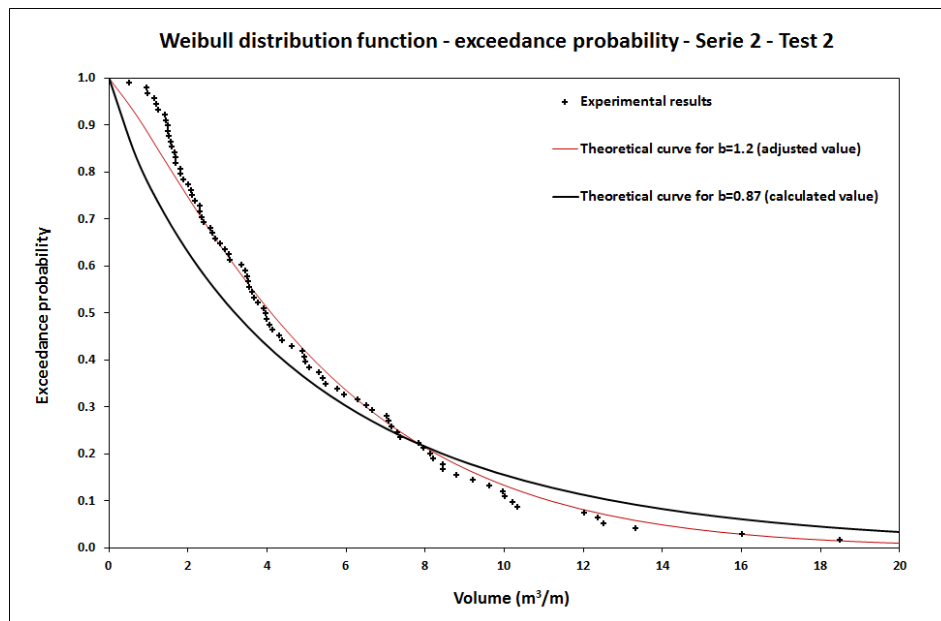


Figure 5.54 – Weibull distribution function of overtopping values, for Test 2.

Figure 5.56 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 2 of Serie 2.

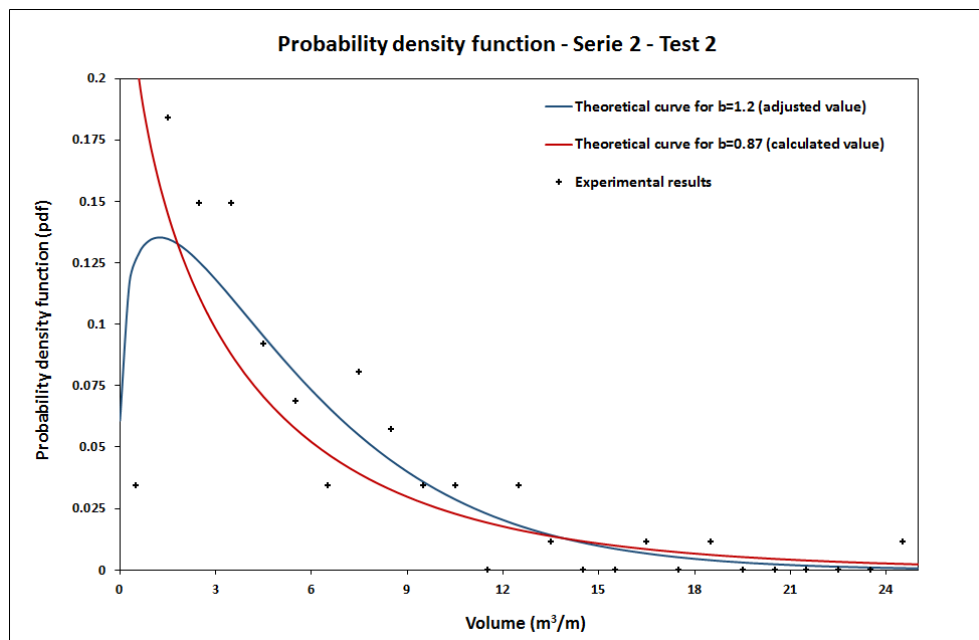


Figure 5.55 – Probability density function (pdf), for Test 2.

Figure 5.57 presents the exceedance probability function for Test 3 of Serie 2.

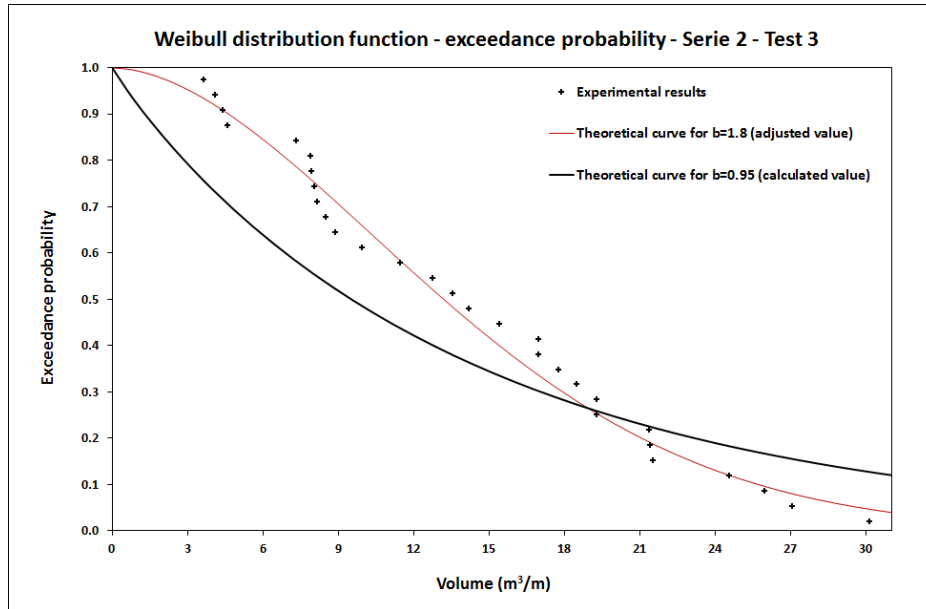


Figure 5.56 – Weibull distribution function of overtopping values, for Test 3.

Figure 5.58 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 3 of Serie 2.

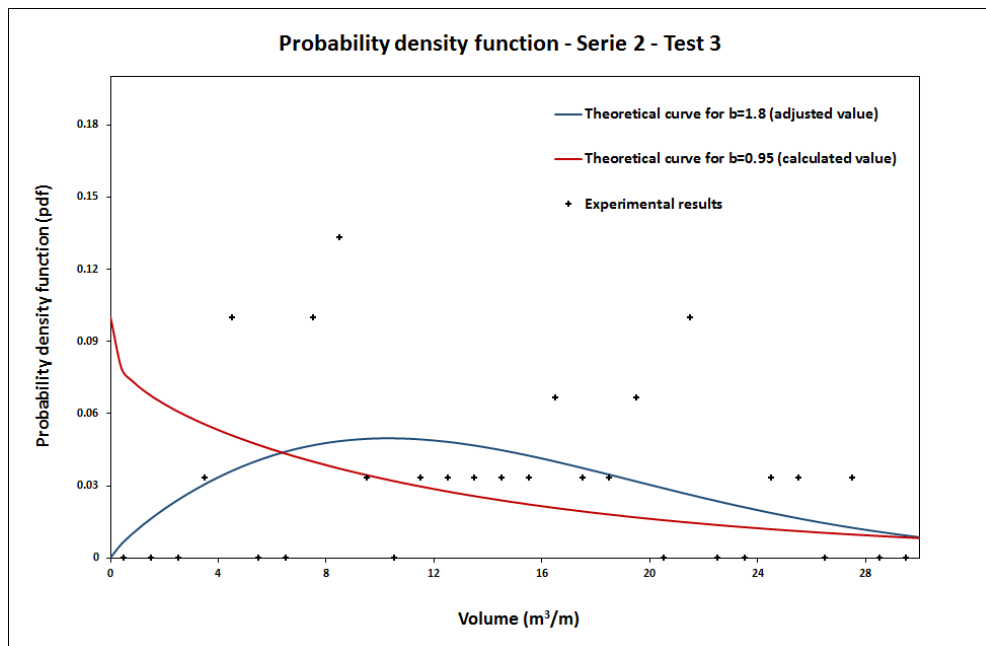


Figure 5.57 – Probability density function (pdf), for Test 3.

5.11.3. TEST SERIES 3

Figure 5.59 presents the exceedance probability function for Test 1 of Serie 3.

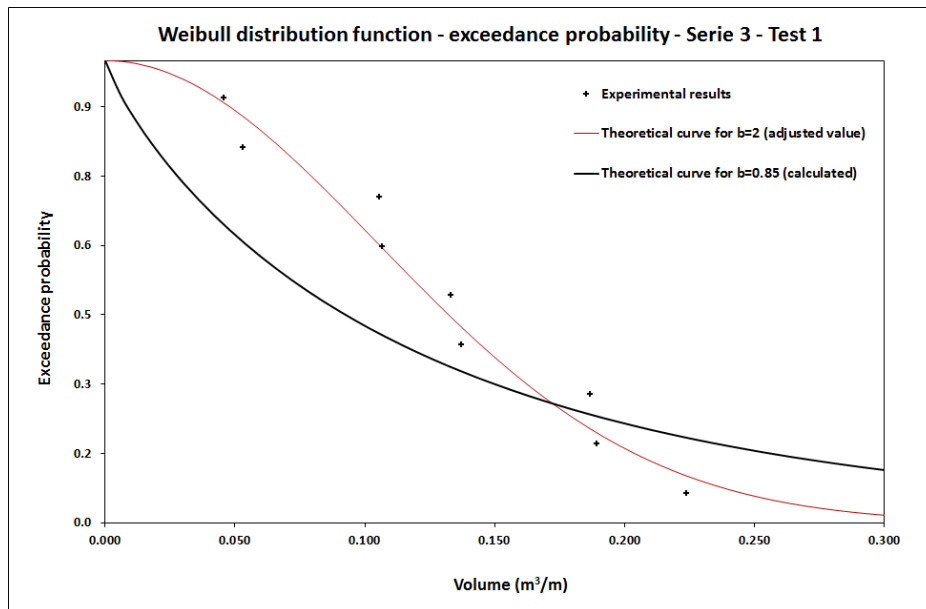


Figure 5.58 – Weibull distribution function of overtopping values, for Test 1.

Figure 5.60 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 1 of Serie 3.

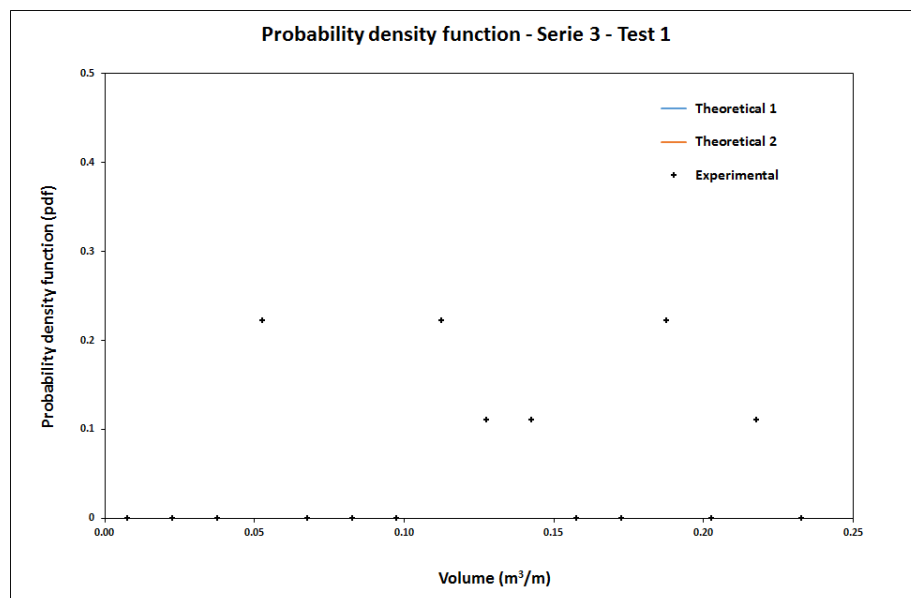


Figure 5.59 – Probability density function (pdf), for Test 1.

Figure 5.61 presents the exceedance probability function for Test 2 of Serie 3.

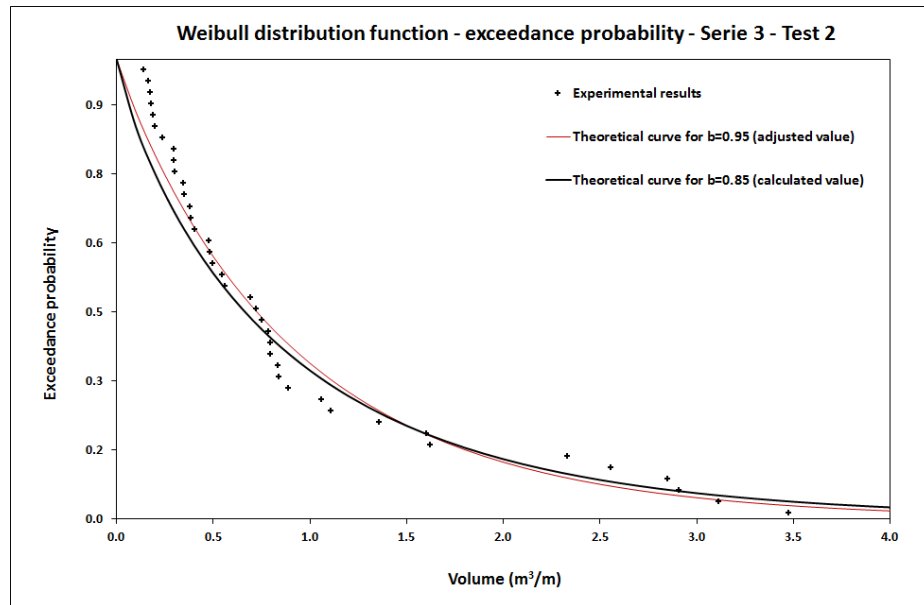


Figure 5.60 – Weibull distribution function of overtopping values, for Test 2.

Figure 5.62 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 2 of Serie 3.

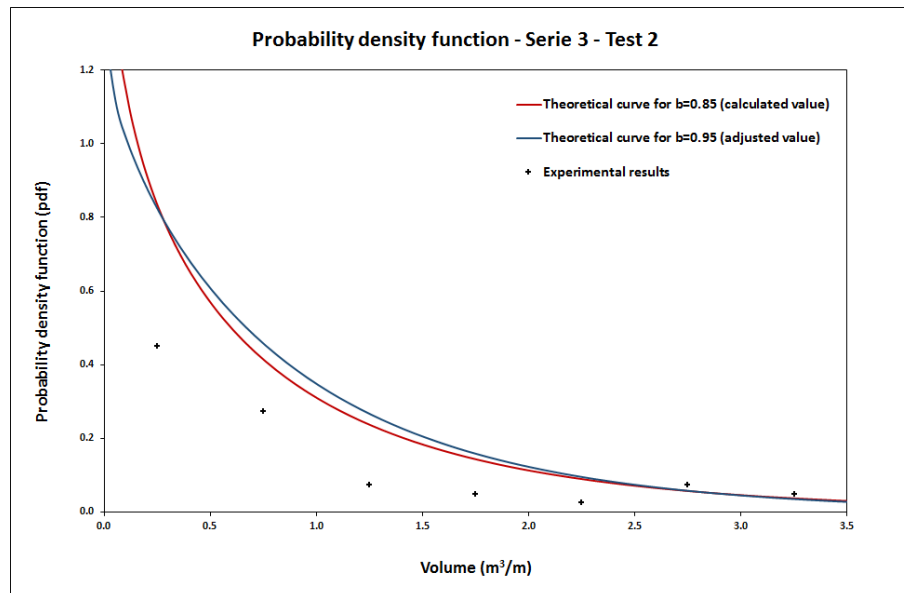


Figure 5.61 – Probability density function (pdf), for Test 2.

Figure 5.63 presents the exceedance probability function for Test 3 of Serie 3.

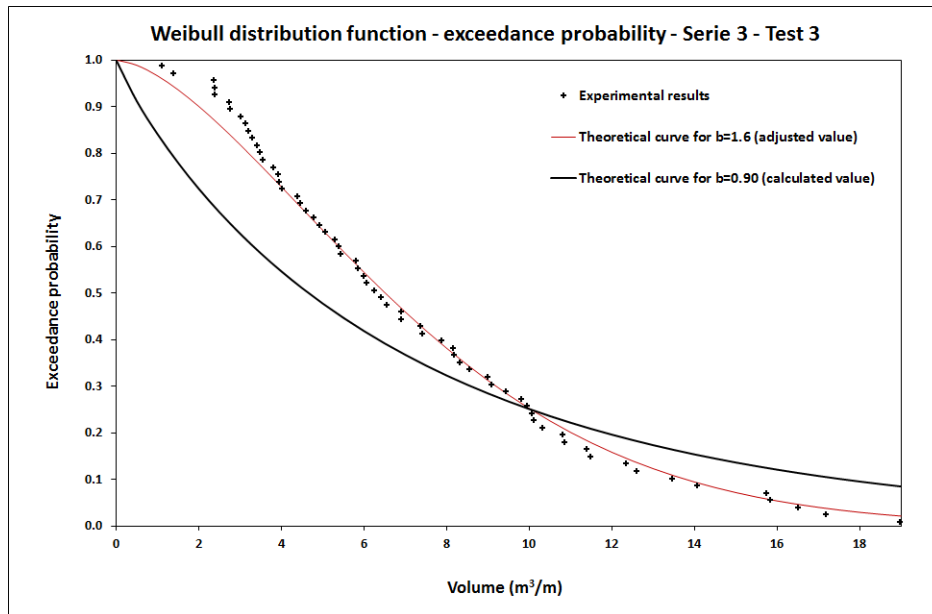


Figure 5.62 – Weibull distribution function of overtopping values, for Test 3.

Figure 5.64 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 3 of Serie 3.

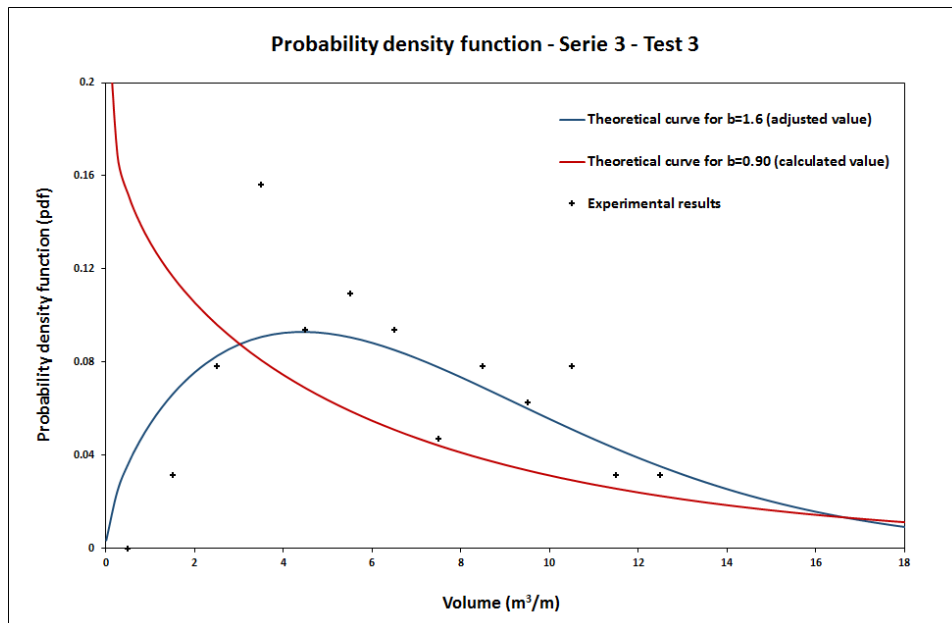


Figure 5.63 – Probability density function (pdf), for Test 3.

Figure 5.65 presents the exceedance probability function for Test 4 of Serie 3.

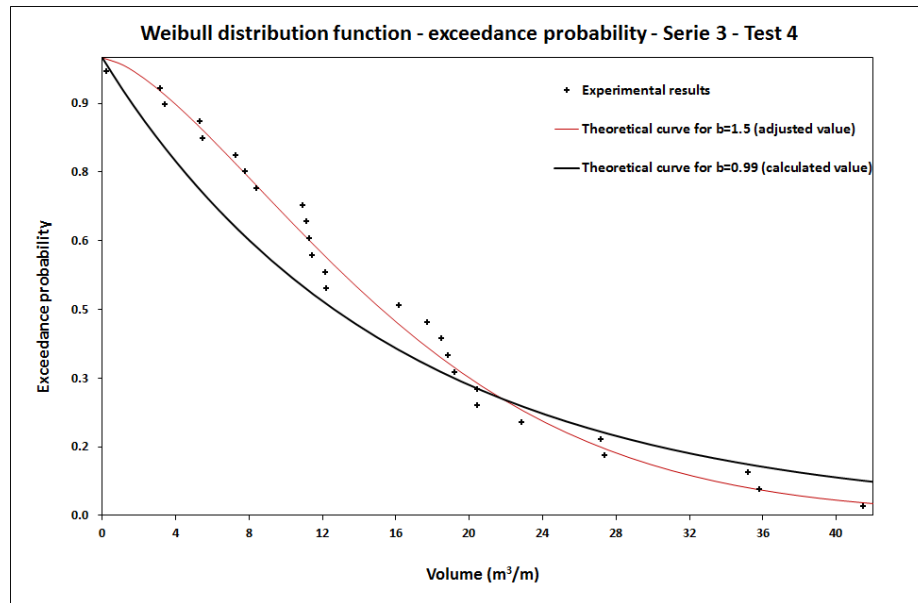


Figure 5.64 – Weibull distribution function of overtopping values, for Test 4.

Figure 5.66 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 4 of Serie 3.

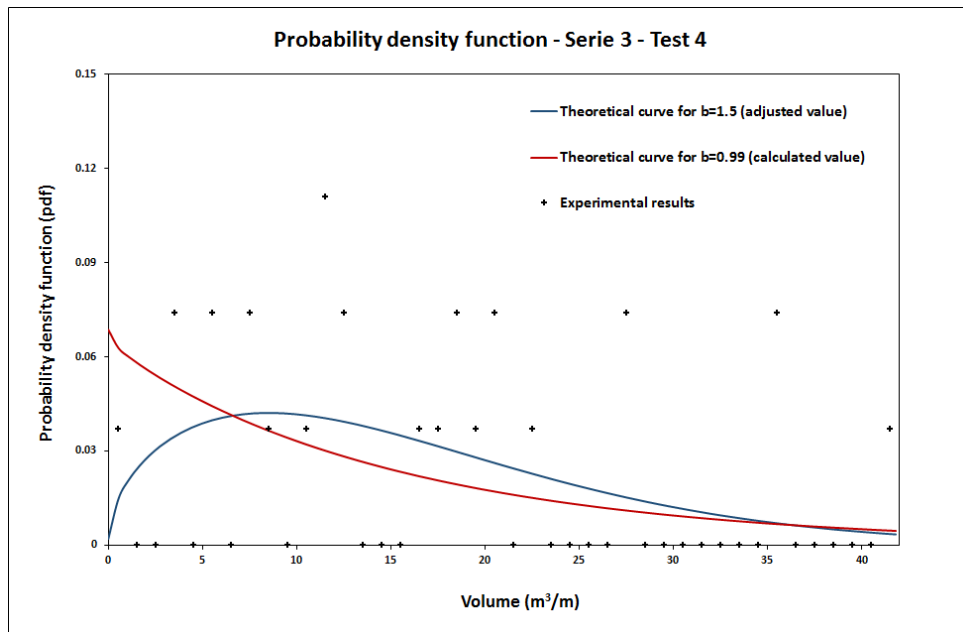


Figure 5.65 – Probability density function (pdf), for Test 4.

5.11.4. TEST SERIES 4

Figure 5.67 presents the exceedance probability function for Test 1 of Serie 4.

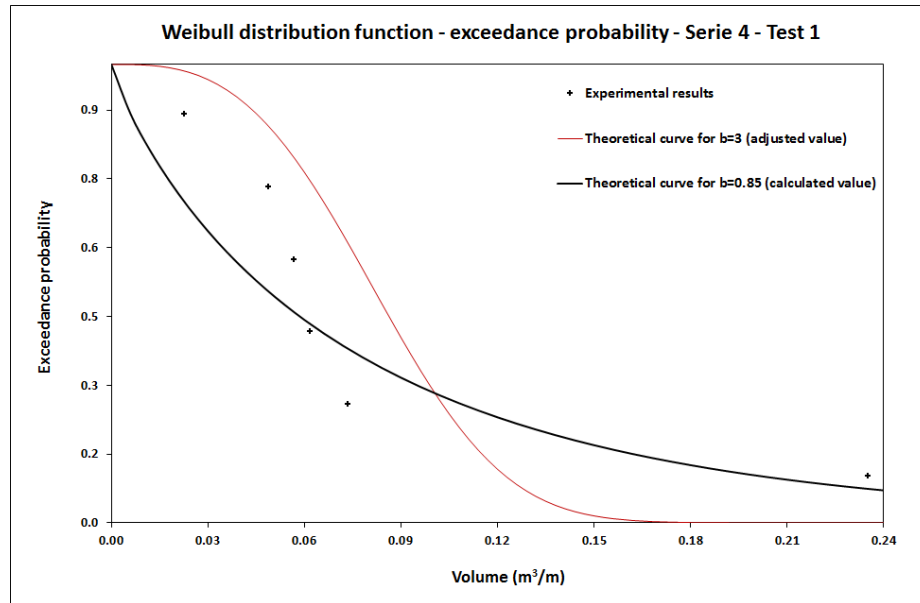


Figure 5.66 – Weibull distribution function of overtopping values, for Test 1.

Figure 5.68 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 1 of Serie 4.

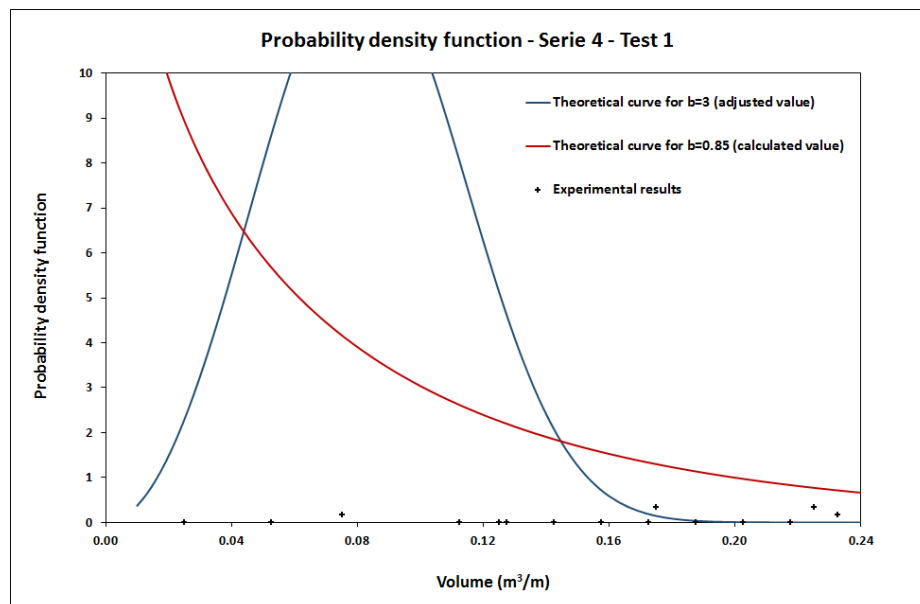


Figure 5.67 – Probability density function (pdf), for Test 1.

Figure 5.69 presents the exceedance probability function for Test 2 of Serie 4.

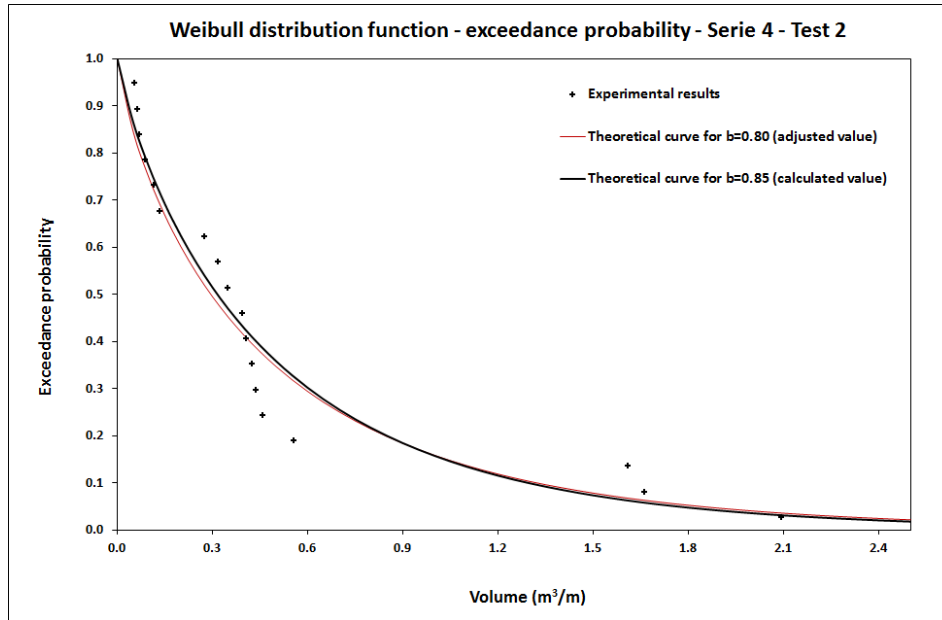


Figure 5.68 – Weibull distribution function of overtopping values, for Test 2.

Figure 5.70 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 2 of Serie 4.

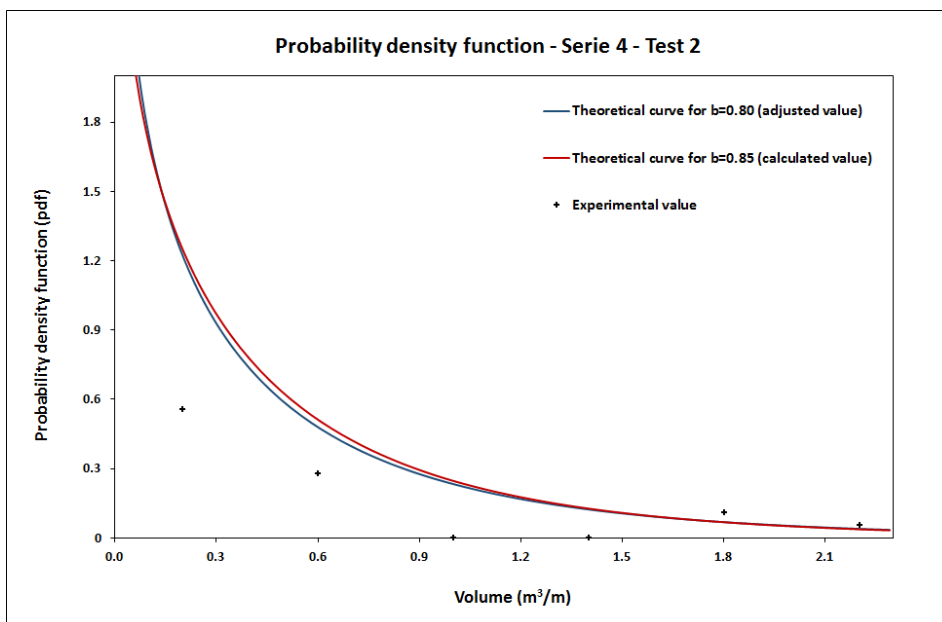


Figure 5.69 – Probability density function (pdf), for Test 2.

Figure 5.71 presents the exceedance probability function for Test 3 of Serie 4.

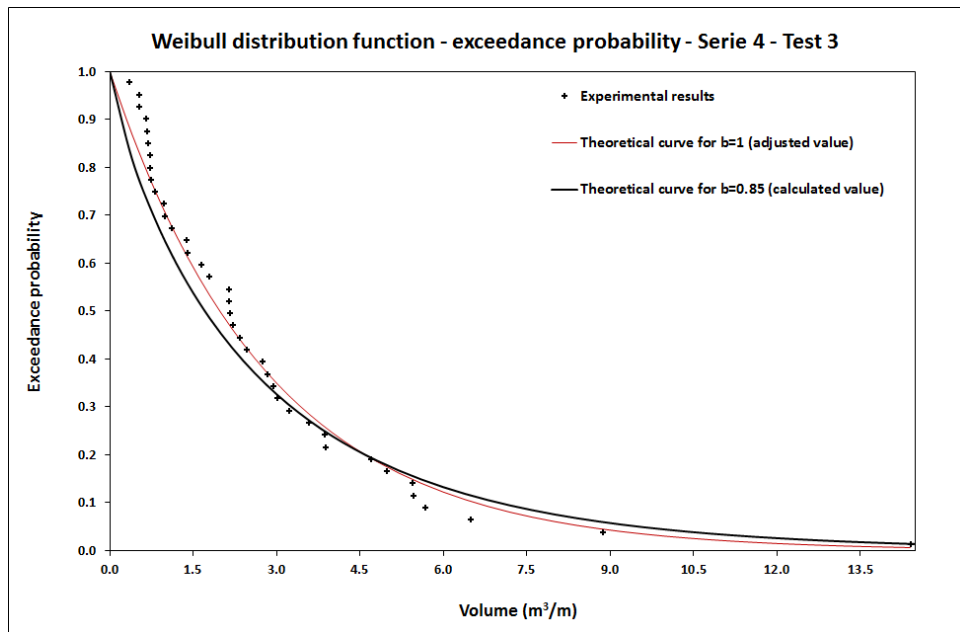


Figure 5.70 – Weibull distribution function of overtopping values, for Test 3.

Figure 5.72 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 3 of Serie 4.

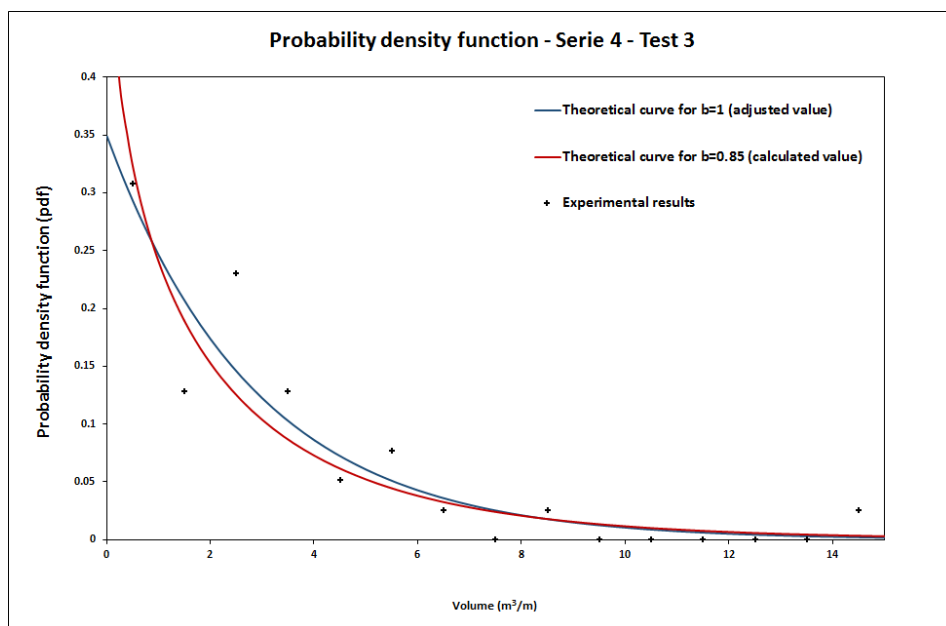


Figure 5.71 – Probability density function (pdf), for Test 3.

Figure 5.73 presents the exceedance probability function for Test 4 of Serie 4.

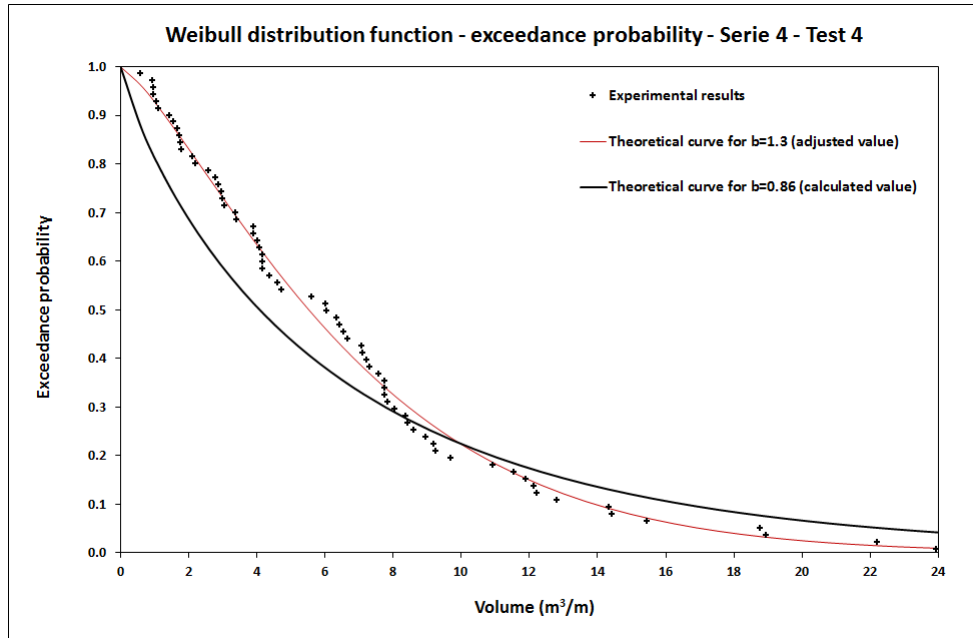


Figure 5.72 – Weibull distribution function of overtopping values, for Test 4.

Figure 5.74 presents the probability density function (pdf) of a Weibull distribution analysis, of Test 4 of Serie 4.

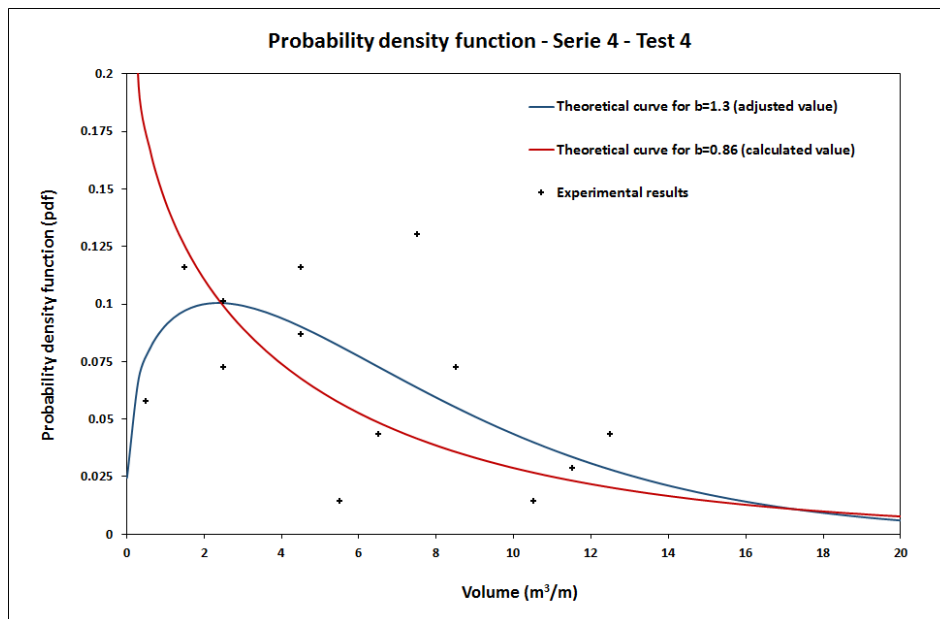


Figure 5.73 – Probability density function (pdf), for Test 4.

Table 5.24 presents the Weibull distribution parameters “a” and “b”, calculated and estimated for all the tests of this laboratorial experiment.

Table 5.24 – Weibull distribution parameters of all the tests.

Test	Scale Weibull parameter “a”		Shape Weibull parameter “b”	
	Calculated	Adjusted	Calculated	Adjusted
S1R1	0.075	0.090	0.850	1.400
S1R2	0.409	0.370	0.850	0.750
S1R3	4.110	4.866	0.863	1.400
S1R4	13.34	14.52	0.969	1.250
S2R2	0.685	0.660	0.850	0.800
S2R3	4.866	5.570	0.865	1.200
S2R4	14.01	16.16	0.946	1.800
S3R1	0.121	0.150	0.850	2.000
S3R2	0.866	0.920	0.851	0.950
S3R3	6.986	8.190	0.903	1.600
S3R4	15.87	17.68	0.986	1.500
S4R1	0.076	0.090	0.850	3.000
S4R2	0.485	0.470	0.850	0.8000
S4R3	2.629	2.860	0.852	1.000
S4R4	6.256	7.320	0.859	1.300

After the calculation of the Weibull parameters using the expressions of Eurotop II (2016), the shape parameter “b” was adjusted in order to find the most reliable theoretical curve of the Weibull distribution.

Then it was used the adjusted value for “b” to calculate the parameter “a” related with that new value for “b”. The range of values for the adjusted shape parameter “b” is comprised between 0.75 and 3, and the scale parameter “a”, between 0.09 and 17.68.

Figure 5.75 presents the Weibull shape factor, b , for rubble-mound structures, spanning a large range of relative freeboards (Eurotop II, 2016).

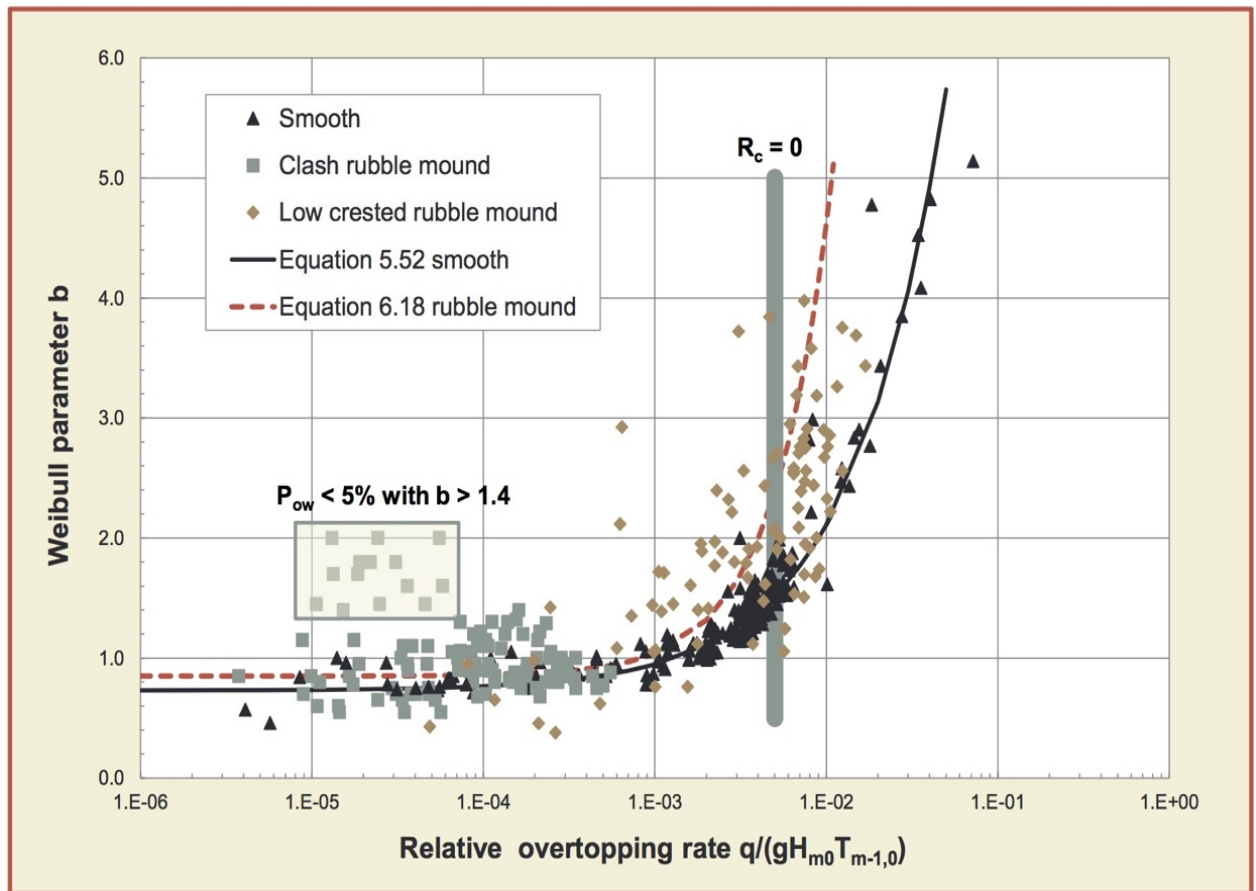


Figure 5.74 – Weibull shape factor, b , for rubble-mound structures, spanning a large range of relative freeboards (Eurotop II, 2016).

Figure 5.75 presents some range of values for different situations, one of which, the clash rubble-mound structures with approximate values for “ b ” of 1.

It is also referred that this parameter might take values out of this range, for $b > 1.4$, so for that matter the results of the Weibull distribution and probability functions are reliable.

5.12. NN-OVERTOPPING – DATA MODELLING AND VALIDATION

In this section, a results comparison between the experimental data of this work and the outputs from the NN-Overtopping software, is made.

The mean overtopping discharge from the laboratorial experiment needs to validate the results taken from a NN-Overtopping simulation, in order to confirm the tests reliability.

Recalling the parameters definition of the NN-Overtopping, made in the state-of-art of this work, Figure 5.76 presents the related parameters for a mean overtopping discharge prediction of this software.

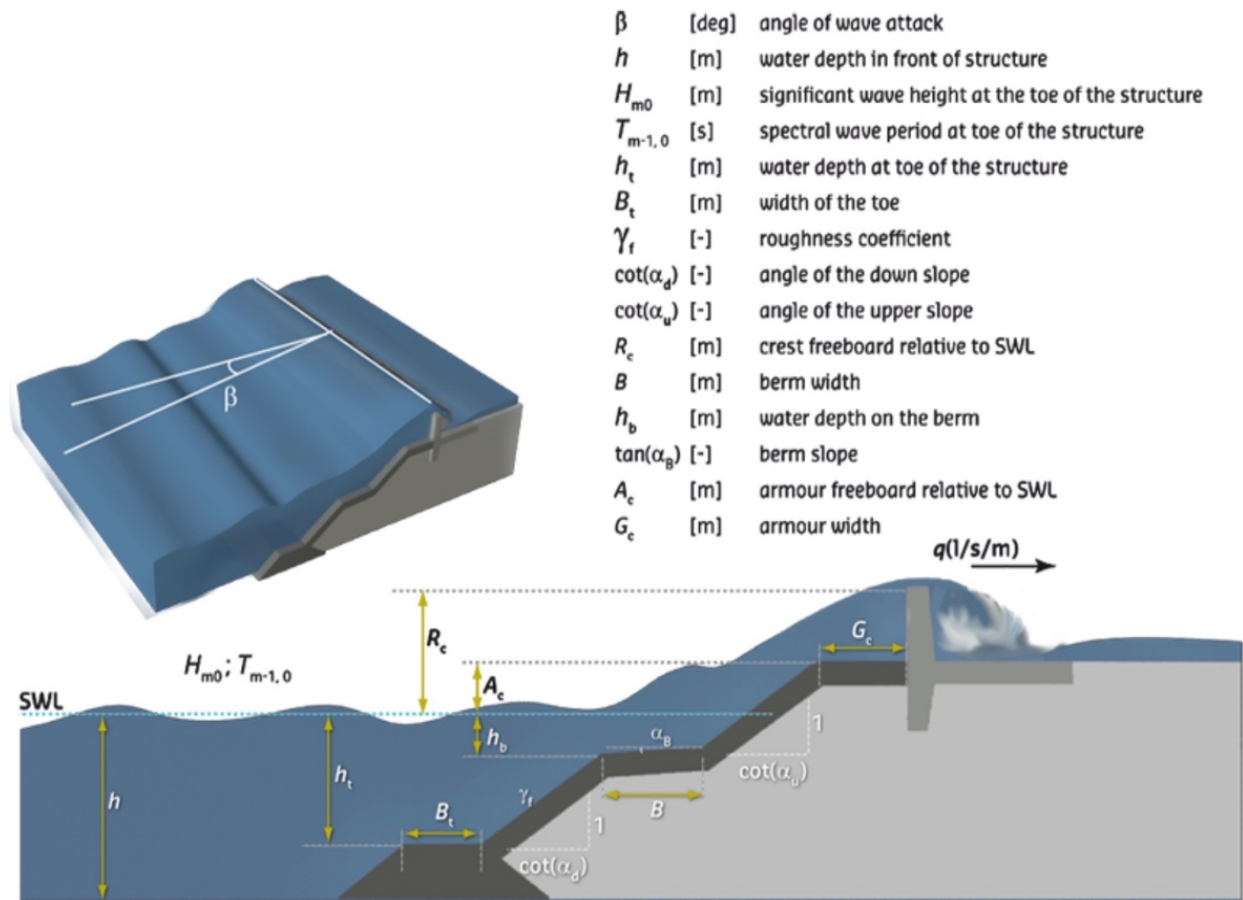


Figure 5.75 – Related parameters used in NN-Overtopping prediction tool (Deltares).

For the design of the rubble-mound breakwater of this work, the following parameters are logically considered zero:

- β (degrees);
- B_t (m);
- B (m);
- h_b (m);
- $\tan(\alpha_B)$;

For this structure, it is considered a $\gamma_r = 0.45$, for an armour layer of rocks with one layer and a permeable core.

For a slope of 1:2, the parameters $\cot(\alpha_d)$ and $\cot(\alpha_u)$ take the same calculated value of 2, approximately, because in this case scenario, the angle of the down slope is the same of the upper slope, taking into account that there is only one slope, with no toe or berm adjacent to it.

A fixed value of 0.566 m for water level is used, with the exception of Test 3 and 4 of Serie 3, where a sea level rise of 0.023 is considered. The crest of the structure is at 0.68 m from the ground.

Table 5.25 presents all the values used for the NN-Overtopping parameters simulation.

Table 5.25 – Input parameters values, for NN-Overtopping prediction tool.

Parameters	h (m)	H_{m0} (m)	$T_{m-1.0}$ (s)	h_t (m)	γ_f (-)	$\cot(\alpha_d)$ (-)	$\cot(\alpha_u)$ (-)	R_c (m)	A_c (m)
S1 - R1	0.566	0.076	1.184	0.566	0.45	2	2	0.114	0.114
S1 - R2	0.566	0.102	1.515	0.566	0.45	2	2	0.114	0.114
S1 - R3	0.566	0.126	1.646	0.566	0.45	2	2	0.114	0.114
S1 - R4	0.566	0.151	1.994	0.566	0.45	2	2	0.114	0.114
S2 - R1	0.566	0.104	1.515	0.566	0.45	2	2	0.114	0.114
S2 - R2	0.566	0.124	1.646	0.566	0.45	2	2	0.114	0.114
S2 - R3	0.566	0.153	1.994	0.566	0.45	2	2	0.114	0.114
S3 - R1	0.566	0.076	1.184	0.566	0.45	2	2	0.114	0.114
S3 - R2	0.566	0.102	1.515	0.566	0.45	2	2	0.114	0.114
S3 - R3	0.589	0.124	1.578	0.589	0.45	2	2	0.091	0.091
S3 - R4	0.589	0.154	1.994	0.589	0.45	2	2	0.091	0.091
S4 - R1	0.566	0.071	1.114	0.566	0.45	2	2	0.114	0.114
S4 - R2	0.566	0.094	1.515	0.566	0.45	2	2	0.114	0.114
S4 - R3	0.566	0.118	1.722	0.566	0.45	2	2	0.114	0.114
S4 - R4	0.566	0.142	1.891	0.566	0.45	2	2	0.114	0.114

Table 5.26 presents the output results of NN-Overtopping simulation. A prediction value for the mean overtopping discharge is given by the software, as well as the different quantiles of several orders.

As long as the mean overtopping discharge values from the experimental results are inside the intervals given by NN-overtopping quantiles, the results are validated and reliable.

Table 5.26 – NN-Overtopping prediction values for mean-overtopping discharge and different quantiles.

Output	q (L/s/m)	q (2.5%) (l/s/m)	q (5%) (l/s/m)	q (25%) (l/s/m)	q (50%) (l/s/m)	q (75%) (l/s/m)	q (95%) (l/s/m)	q(97.5%) (l/s/m)
S1 - R1	6.97×10^{-4}	5.92×10^{-5}	9.3×10^{-5}	3.22×10^{-4}	7.09×10^{-4}	0.00149	0.00464	0.007191
S1 - R2	0.00792	0.00153	0.00203	0.00480	0.00802	0.01376	0.02833	0.03762
S1 - R3	0.04380	0.01256	0.01448	0.02957	0.04444	0.06657	0.12890	0.15060
S1 - R4	0.21570	0.06501	0.07723	0.14780	0.22110	0.31960	0.54380	0.65450
S2 - R1	0.00908	0.00179	0.00238	0.00551	0.00924	0.01539	0.03109	0.04127
S2 - R2	0.03895	0.01089	0.01278	0.02596	0.04023	0.06044	0.11580	0.13910
S2 - R3	0.23490	0.06988	0.08682	0.16010	0.24080	0.34630	0.59200	0.6884
S3 - R1	0.00070	5.92×10^{-5}	9.3×10^{-5}	3.22×10^{-4}	7.09×10^{-4}	0.00149	0.00464	0.007191
S3 - R2	0.00792	0.00153	0.00203	0.00480	0.00802	0.01376	0.02833	0.03762
S3 - R3	0.09092	0.02118	0.02888	0.05858	0.09135	0.14410	0.28500	0.3731
S3 - R4	0.47750	0.12240	0.15510	0.31340	0.49450	0.74500	1.2960	1.5220
S4 - R1	4.3×10^{-4}	3.13×10^{-5}	4.7×10^{-5}	1.88×10^{-4}	4.37×10^{-4}	1.02×10^{-3}	3.54×10^{-3}	5.76×10^{-3}
S4 - R2	0.00461	0.00076	0.00106	0.00268	0.00446	0.00853	0.02041	0.02358
S4 - R3	0.02977	0.00692	0.00918	0.01920	0.03003	0.04613	0.09823	0.12390
S4 - R4	0.1305	0.03769	0.04668	0.08985	0.13440	0.19560	0.34310	0.42950

In order to compare the NN-overtopping results with the experimental results obtained from this work, a graphic with all quantiles and prediction values from NN-overtopping and from the experimental tests results for each test (presented in prototype), is made, Figure 5.77.

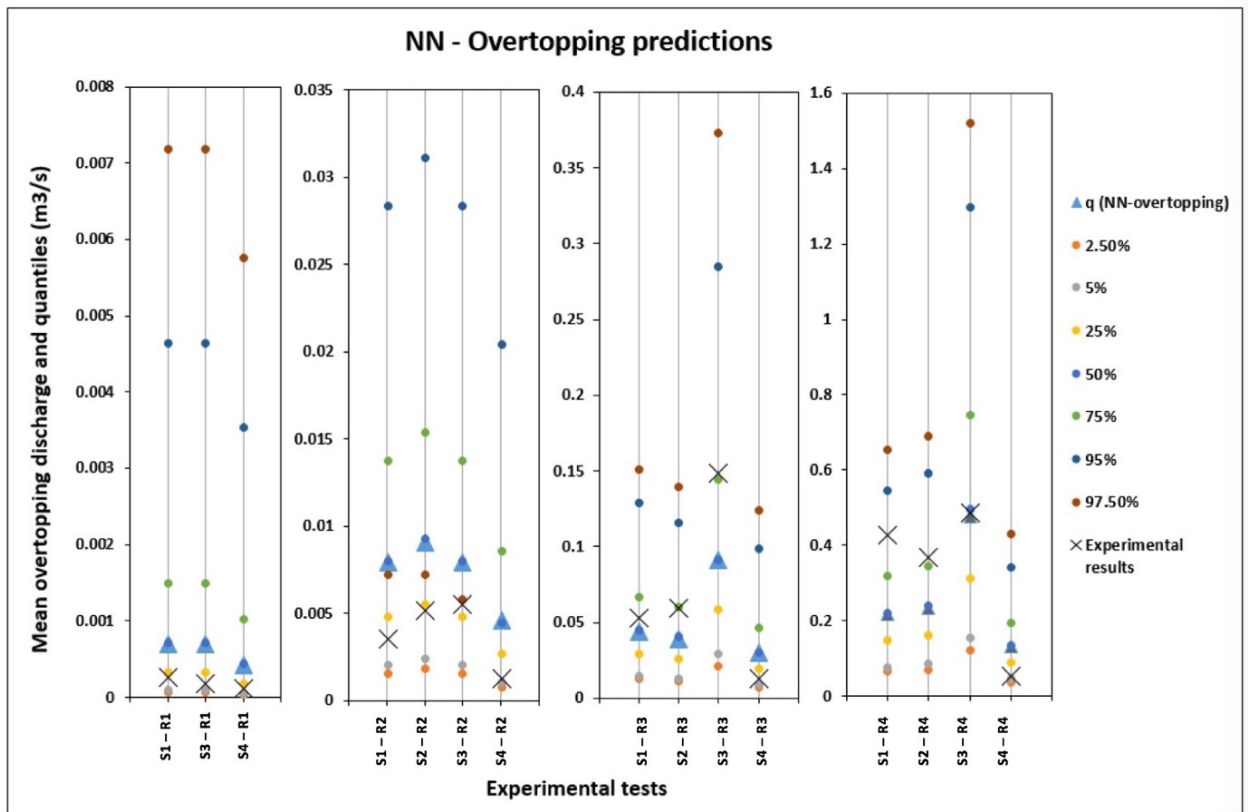


Figure 5.76 – Comparison between experimental results and the NN-overtopping prediction.

From Figure 5.77, it is visually proven that all the experimental results are inside the area of reliability, calculated by the NN-overtopping tool. The experimental results validate correctly the software prediction.

6. LABORATORY EXPERIMENTS DAMAGE EVALUATION

6.1. INTRODUCTION

For the evaluation of the damage progression in the rubble-mound breakwater, the 3D bathymetric surveys obtained from stereophotogrammetry, before and after each test, were compared. In this work, the analysis is only done for test series 1 and 3 (pre-defined decisions). For each test, the output of the survey is defined by a chromatic representation (each color represent a certain value of elevation or damage depth) of the model with initial conditions, then with the damage occurred for each test of the test series, and finally, an illustration of the cumulative damage for the four tests, to analyze the damage progression. The damage analysis is made exclusively for the area marked in red in Figure 6.1 – the inner slope and the front slope of the breakwater's trunk. For test series 1 and 3, the following pictures present the damage progression. Figure 6.1 presents the chromatic representation of the stereophotographs of the model, with initial conditions (no damage).

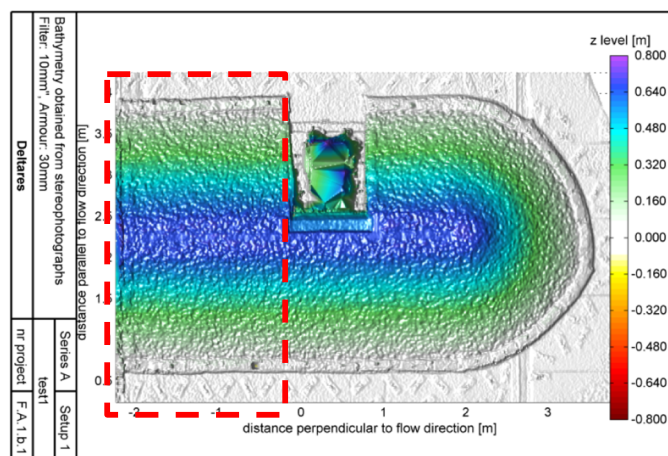


Figure 6.1 – Bathymetric survey obtained from stereophotogrammetry technique (FEUP and Delft University, 2016).

Before the damage analysis and comparison between tests, it is necessary to describe the initial conditions of each test, Table 6.1. For this analysis, the comparisons are presented in Table 6.2.

Table 6.1 – Initial conditions for each test, for damage analysis.

Series	Test	ID	Description	d (m)	H _{m0} (m)	
Serie 1	1	S1R1	Rebuilt before test	0.566	60%	0.076
	2	S1R2	Cumulative damage	0.566	80%	0.102
	3	S1R3	Cumulative damage	0.566	100%	0.126
	4	S1R4	Cumulative damage	0.566	120%	0.151
Serie 3	1	S3R1	Rebuilt before test	0.566	60%	0.076
	2	S3R2	Cumulative damage	0.566	80%	0.102
	3	S3R3	Cumulative damage + SLR	0.589	100%	0.124
	4	S3R4	Cumulative damage + SLR	0.589	120%	0.154

Table 6.2 – Comparisons definition for damage analysis.

1. Damage progression analysis, description and comparison, for equal initial conditions. Verification of the similarity of damage between the tests.	2. Damage progression analysis, description and comparison, with consideration of the sea level rise. Verification of the effects of SLR in the damage.
Serie 1 – Test 1 and Serie 3 – Test 1	Serie 1 – Test 3 and Serie 3 – Test 3
Serie 1 – Test 2 and Serie 3 – Test 2	Serie 1 – Test 4 and Serie 3 – Test 4

6.2. DAMAGE PROGRESSION ANALYSIS AND COMPARISON BETWEEN TESTS WITH THE SAME INITIAL CONDITIONS

Figure 6.2 presents the damage progression analysis and comparison between Test 1 of Series 1 and 3.

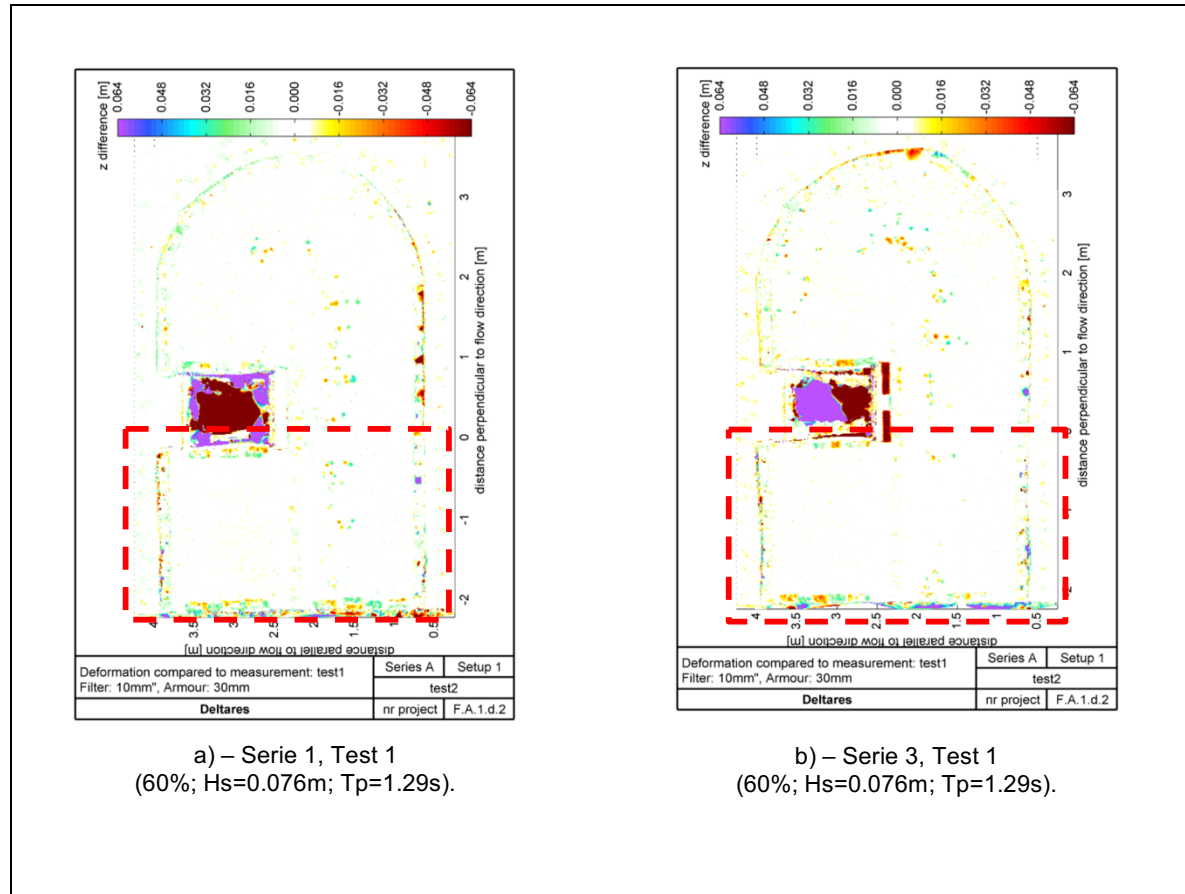


Figure 6.2 – Damage progression analysis, description and comparison between Test 1 of Series 1 and 3.
Verification of damage similarity between the two tests.

The colour white represent no damage. The area to be analyzed is marked as the red area. These figures result from a subtraction of two different bathymetric surveys (one before the test and another after the test).

If the difference between them is zero, it means no damage on that region of the breakwaters armour layer. In general, as for this test wave overtopping was very small, the damage is almost none. The inner slope almost remained intact on the two tests, with only some adjustment of the position of some blocks.

In the front slope, some points have a damage of -0.016 m and 0.016 m in test S1R1, suggesting that a little more damaged occurred in Test 1 of Serie 1, but the differences are almost none. Despite that, generally, it can be concluded that for this case, at least for equal initial conditions (same structure and wave conditions), damage will be approximately the same.

Figure 6.3 present the second comparison of tests with the same initial conditions, in this case, between Test 2 of Series 1 and 3. A verification of the damage similarity between these two tests, is also made for this case.

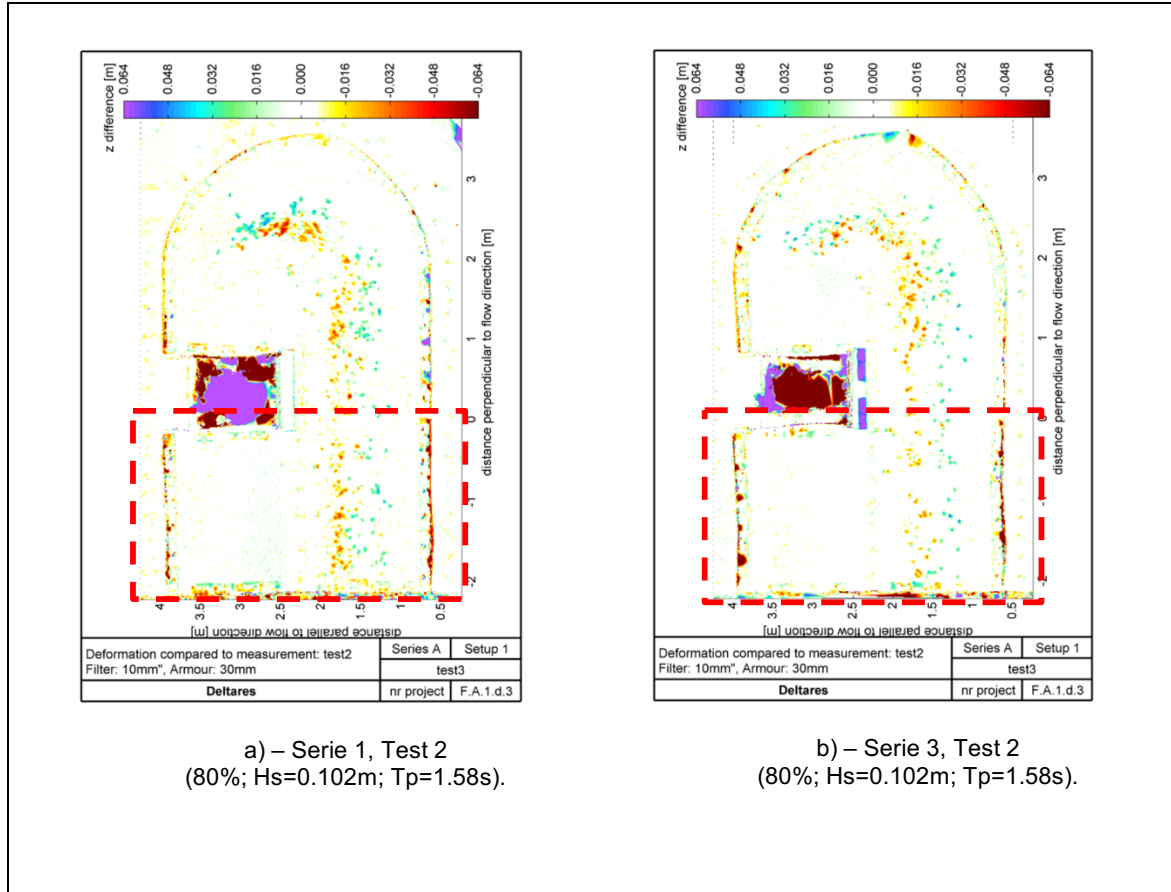


Figure 6.3 – Damage progression description and comparison, with consideration of the sea level rise.
Analysis of the effects of SLR in the damage.

Test 2 of Series 1 and 3, have higher values for the significant wave height and the registered peak period. Logically, for this case, the damage in the structure was slightly higher than for Test 1. Test 2 of the two series were initially started with cumulative damage from the previous test.

On the inner slope, the damage in the two tests was approximately zero (colour white). However in the front slope, the damage was slightly higher on Serie 1 than in Serie 3.

The front toe of the structure, had many green colors displayed in the stereophotograph, suggesting a lot of armour units movement (after sliding from the slope), during the test, with damage of about -0.016 m to -0.032 m.

Also some red points are displayed in this test, indicating armour layer damage with -0.048 m of damage. Despite the fact these two test had the same initial conditions, the damage on the front slope was slightly more intense for Serie 1.

6.3. ANALYSIS OF THE EFFECT OF SEA LEVEL RISE IN THE DAMAGE PROGRESSION

Another main objective of this dissertation, is based on sea level rise effects, not only on wave overtopping, but also in damage progression of rubble-mound structures.

For that matter, a comparison between tests with the same initial conditions, but some with a sea level rise consideration, is made, Figure 6.4.

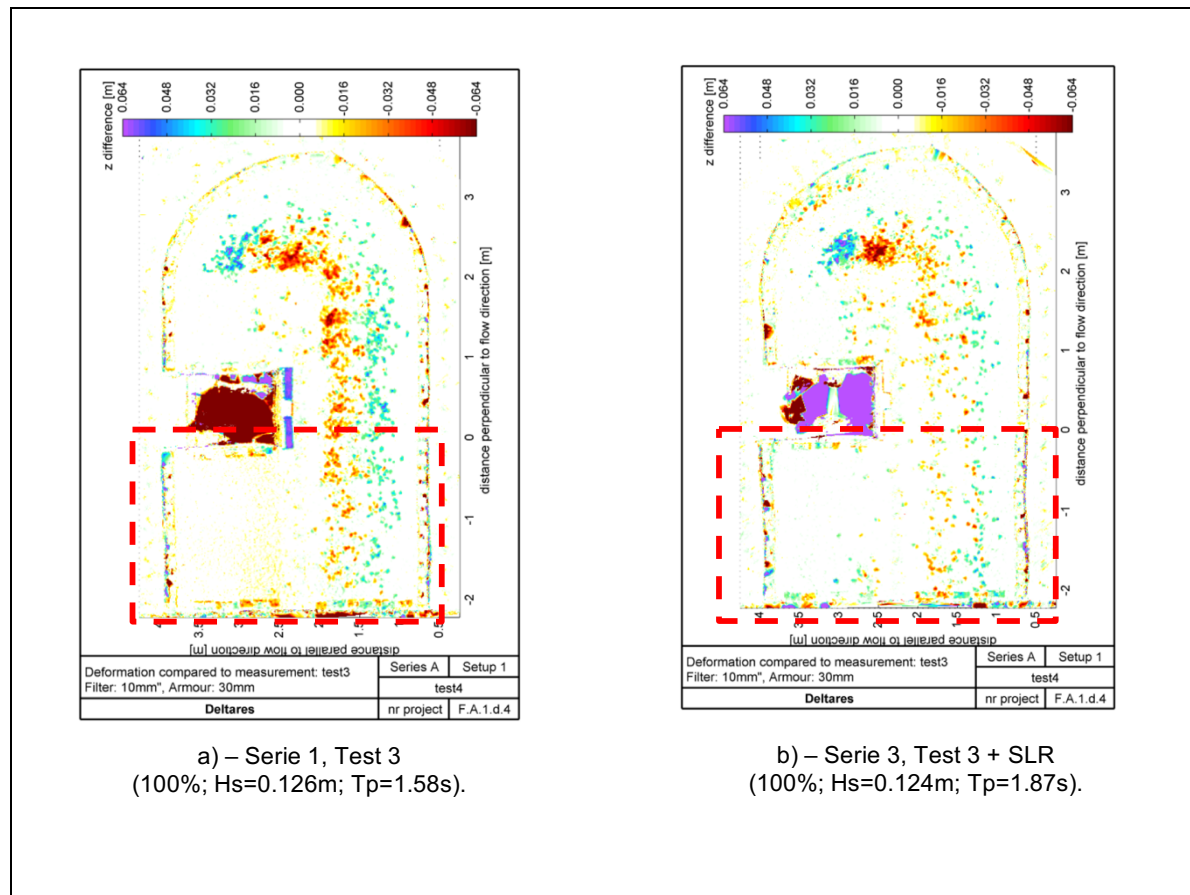


Figure 6.4 – Damage progression analysis and comparison between Test 3 of Serie 1 and 3 (SLR consideration).

For this comparison, Test 3 of Series 3 is subjected to a sea level rise of 0.023 m. It is logically expected that generally, the breakwater will have more damage with a sea level rise consideration, due to the consequent increase of water levels, and wave overtopping loads impacting the structure.

However, it appears that for this case, the damage was quite different from the expected. For Serie 1, the damage was mainly in the front slope, of -0.016 to -0.048 m, and also $+0.016$ m, suggesting armour units sliding from the upper slope and crest, to the down slope and toe of the structure. On the inner slope, the damage was generally none for the two tests.

On Series 3, green color is shown in Serie 3 (Figure 6.4 b)), on the inner slope, suggesting that some armour units crossed the crest of the structure. This event did not occurred in Serie 1 where sea level rise was not considered.

Figure 6.5 presents the second comparison of tests with the same initial conditions, although with a sea level rise consideration for one of them. In this case, comparison is made between Test 3 of Series 1 and 3.

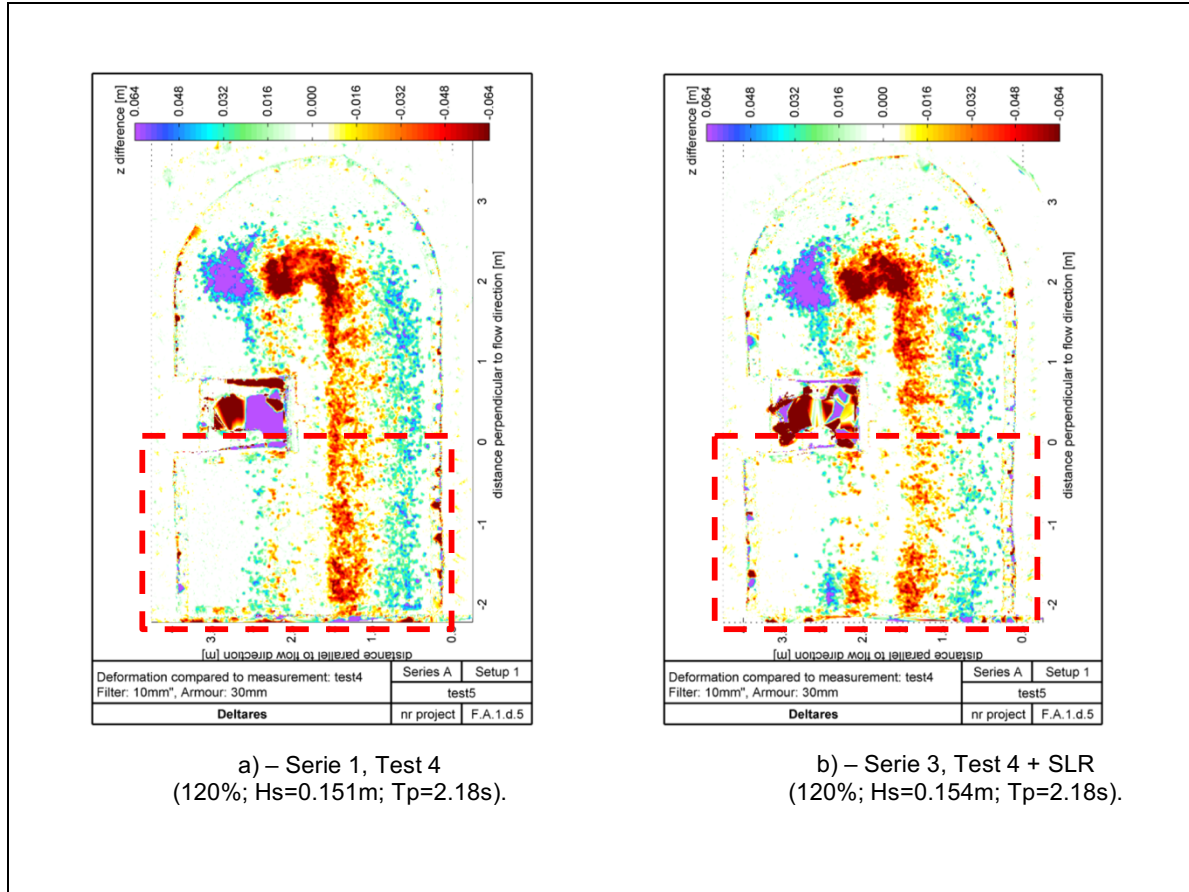


Figure 6.5 – Damage progression analysis and comparison between Test 4 of Series 1 and 3 (SLR consideration)..

On this comparison, the initial test conditions are the most critical, with significant wave height of 120%. In Serie 3 a consideration of a sea level rise was made. In Test 4 of Serie 1, the damage was mainly in the front slope, close to the crest of the structure (waves impact zone), with damage of - 0.048 to - 0.064 m. Many armour units displacement, with severe rock sliding from the front slope to the front toe, and some units crossing over the crest from the front slope to the inner slope.

On Test 4 of Serie 3, the main damage also occurred in the front slope, but with lesser intensity than in Serie 1. On the other hand, in the inner slope, near the area against the wall, strong damage occurred, more intense than in Test 4 of Serie 1. Damage of about - 0.048 to - 0.064 m. On these two tests, the waves impact damaged the armour layer, and in some zones, it pierced the armour to the surface of the core.

From the observation of the stereophotographs, it can be concluded, that with a sea level rise consideration, more damage occurs on the inner slope, and lesser on the front slope. Another conclusions is the fact that with SLR consideration, there is more armour units crossing the crest of the structure.

For a total damage analysis and comparison between the total damage occurred in Serie 1 and the total damage occurred in Serie 3, an overlapping of all the tests stereophotographs of each serie, is made, Figure 6.6.

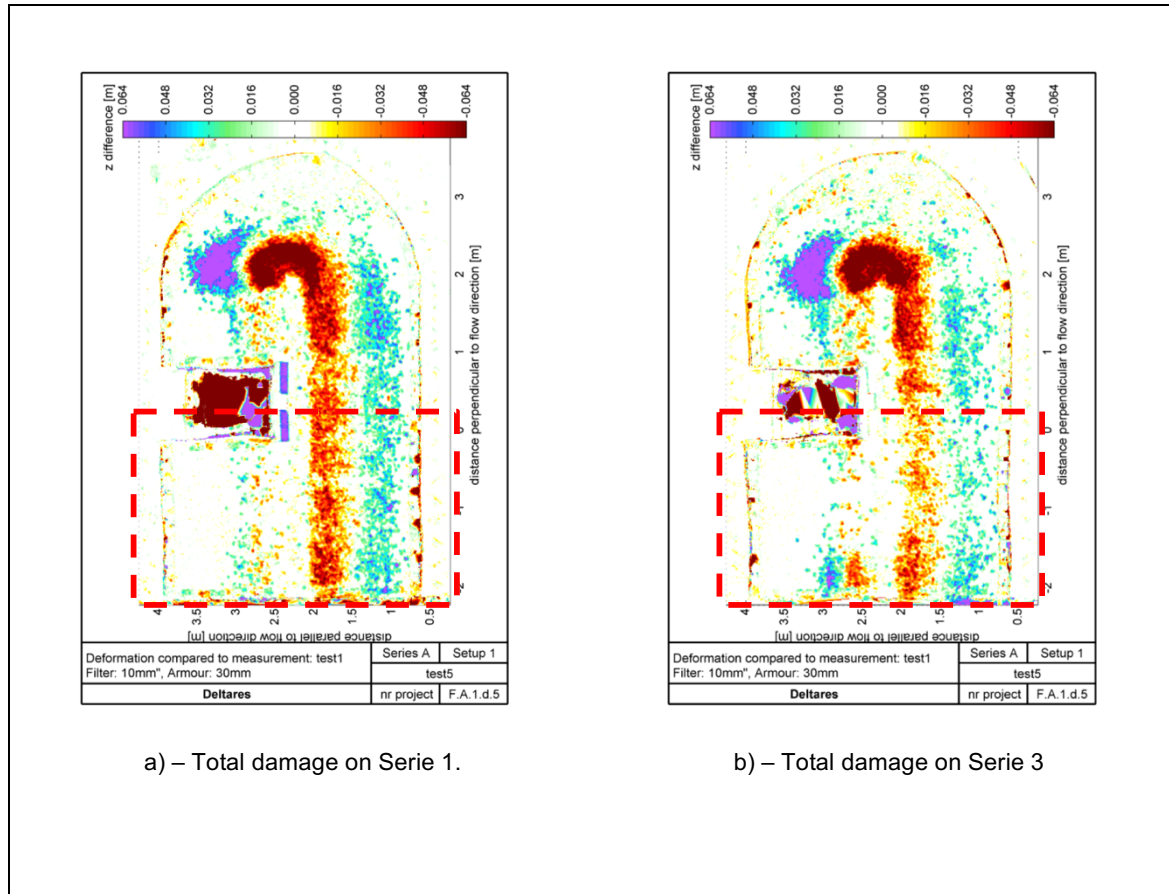


Figure 6.6 – Total damage description and comparison between Serie 1 and Serie 3.

Concerning only the red marked zone, this means, the inner slope, the front slope, and part of the crest of the structure, a comparison of the total damage occurred in Serie 1 and Serie 3, is made. It appears that in Serie 1, the front slope suffered mainly sliding of the armour units, and some few other armour units crossed the crest of the structure. The damage in the front slope appears to be homogeneous and severe.

On the other hand, Serie 3 (with SLR) presents visually a total damage less intense in the front slope. In the front slope, the damage was not so homogeneous, and not so deep as in Serie 1. However in the inner slope, the total damage was more intense in Serie 3 with the consideration of sea level rise.

Another aspect, is the fact that Serie 1 had a higher damage level than Serie 3, it does not mean the movement of armour units was lesser in this case. The fact that in Serie 3, the colors are more disperse, means that a lot of different types of movement occurred. That might be explained by the fact of a sea level rise consideration.

Concluding, Considering the sea level rise consideration, more damage will occur in the inner slope, and lesser in the front slope, suggesting that with an increase of water levels, more armour units of the

armour layer cross the crest of the structure, and less sliding events, which is logic. From the observation of the tests in the hydraulic tank, if a wave crosses the crest of the structure, it is probable that some armour units cross the crest too. Although, if a wave impacts the front slope but overtopping does not occur, it is more likely that the armour units suffer sliding from the front slope to the front toe of the structure.

7. CONCLUSIONS AND FUTURE DEVELOPMENTS

7.1. CONCLUSIONS

This work had the main goal of studying the damage progression and study of wave overtopping phenomena on rubble-mound structures. Also a part of this work includes these analyses considering climate change effects.

In order to enable these studies, advanced laboratory techniques were applied in this work, using physical models with a proper scale. After a long and complex data processing, the experimental results were compared with those obtained using the prediction tools for the range of values given by Eurotop II (2016), and some mathematical distributions were also applied. Various conclusions can be taken from this work, concerning wave overtopping and damage analysis and sea level rise effects.

From the wave-by-wave overtopping analysis, it can be concluded that for tests with severe wave conditions, the total overtopped volume and mean overtopping discharge were larger, and for tests with lower significant wave heights, that volume and mean overtopping discharge were lower.

From the analysis of wave overtopping using normalized overtopping volumes, it was concluded that tests with the lower value for the significant wave height, were the tests that needed less overtopping events to produce the total measured overtopping volume.

An increase of the significant wave height, causes a decrease on the number of incident waves, and an increase on the number of overtopping waves. Therefore, the probability of overtopping per wave increases with an increase of the significant wave height.

For the short-crested waves analysis, it was concluded that despite the fact the number of incident waves in the tests were almost the same, for tests with short-crested waves, a much lower number of overtopping waves was recorded. For short-crested waves, the probability of overtopping per wave is much lower, such as for the total overtopped volume and mean overtopping discharge.

The analysis and comparisons between tests for the study of the sea level rise influence on wave overtopping led also to some conclusions. For the two tests with a consideration of SLR, the total overtopped volume was larger than in the tests with the same initial conditions except the sea level rise

consideration. The volume of each wave overtopping event was approximately double in the tests with SLR. The same conclusion was made for the mean overtopping discharge. On the two comparisons, the number of incident waves were the same with or without SLR, but the number of overtopping waves in the tests with SLR recorded 10 more overtopping waves than the other two tests, for the same initial conditions.

The damage evaluation led to the conclusion that for tests with the same initial conditions, the damage might be slightly different, but identical in general. For the analysis on the influence of Sea level Rise in damage, it was concluded that tests without SLR had most of the damage in the front slope, and tests with SLR, had slightly smaller damage in the front slope, and more damage in the inner slope. Another conclusion is that for tests without SLR, the displacement of units was characterized only as sliding of the armour units. On the other hand, for tests with SLR, there was sliding of armour units but also armour units crossing over the crest of the structure, from the front slope to the inner slope, due to the bigger waves and number of overtopping waves caused by SLR effects.

The analysis of this experimental work led to the conclusion that climate change effects on wave overtopping and damage on rubble-mound breakwaters, is severe and may affect the functionality of the current coastal protection structures. For that matter, and taking into account the increasing effects of climate Change, there is an urgent need for upgrading and reinforce the actual structures whose functionalities were designed for a maritime environment without the consideration of Sea Level Rise.

7.2. FUTURE DEVELOPMENTS

Despite the increasing use of numerical models, the reduced-scale physical models continue to be an effective tool in supporting the design of coastal and port structures, as well as analyses concerning wave overtopping and damage progression. Taking into account the increasing effects of climate change, it is important to continue these works in order to pursue better results, and better solutions for the mitigation of these effects.

Concerning the work of this thesis, several studies and different analyses can be made, some of which are referred in Eurotop II (2016).

In this work, the wave-by-wave overtopping analysis was a very long and complex process, due to the difficulty in discovering what water level variations inside the reservoirs that corresponded to wave overtopping events. Several fluctuations appeared in the wave probes record, that represented just turbulence caused by the entrance of overtopping waves, and in some cases those fluctuations remained for some time.

Another possible development to this work, is the Weibull distribution and probability density function analysis, where in some tests, and for unknown reasons, it was impossible to find values for the shape and scale factor, in order to adjust the theoretical curve to the experimental results, on the probability density function.

BIBLIOGRAPHIC REFERENCES

- AMAZON: Hu, K., Mingham, C.G. and Causon D.M. 2000. Numerical simulation of wave overtopping of coastal structures using the non-linear shallow water equations. *Coastal Engineering*, 41, pp. 433-465.
- Baird, W. F., Caldwell, J. M., Edge, B. L., Magoon, O. T., & Treadwell, D. D. (1980). Report on the damages to the Sines Breakwater, Portugal. In *Coastal Engineering 1980* (pp. 3063-3077).
- Bairrão, Miguel (2014). *Galgamento de estruturas portuárias protegidas por um quebra-mar submerso*. Masters dissertation, FCT.
- Burcharth, Hans F (2007). *Aspects related to design and construction of breakwaters in deep water*. 2007.
- Burcharth, Hans F, Hughes, Steven A (2003) – Types and functions of coastal structures. In *Coastal Engineering Manual*.
- CEM (2011). *Coastal Engineering Manual, Chapter 5*.
- EurOtop 2007. European Manual for the Assessment of Wave Overtopping. Pullen, T, Allsop, N.W.H. Bruce, T. Kortenhaus, A. Schüttrumpf, H. and Van der Meer, J.W. At: www.overtopping-manual.com.
- Eurotop II, 2016. *Manual on Wave Overtopping of Sea Defences and Related Structures – An Overtopping Manual largely Based on European Research, but for Worldwide Application*. Second edition (2016). www.overtopping-manual.com, in press.
- Fielding, C. R., Browne, G. H., Field, B., Florindo, F., Harwood, D. M., Krissek, L. A., ... & Pekar, S. F. (2011). Sequence stratigraphy of the ANDRILL AND-2A drillcore, Antarctica: A long-term, ice-proximal record of Early to Mid-Miocene climate, sea-level and glacial dynamism. *Palaeogeography, Palaeoclimatology, Palaeoecology*, 305(1), 337-351.
- Gutierrez-Serret Ramón, *Centro de Estudios de Puertos y Costas – Centro de Estudios y Experimentación de Obras Públicas (CEDEX), Ministry of Public Works Madrid, Spain*.
- IPCC, (2007). *Climate Change 2007: Impacts, Adaptation and Vulnerability. Working Group II Contribution to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change*.
- Lopes, Hugo Guedes (2005). *Ensaio em modelo físico do comportamento hidráulico e estrutural do quebra-mar norte do porto de Leixões*. Masters dissertation, FEUP.
- PIANC 1992. Analysis of Rubble mound breakwaters. *Report Working Group 12*.
- Queirós, Beatriz (2016). *Estudo de Dimensionamento de Cabeças de Quebramares Portuários. Aplicação ao Porto de Leixões*. Masters dissertation, FEUP.
- Romano *et al.*, 2015. Romano, A. Bellotti, G. Briganti, R. Franco, L. *Uncertainties in the physical modelling of the wave overtopping over a rubble mound breakwater: the role of the seeding number and of the test duration*. *Coastal Engineering*, 103 (2015), (pp. 15-21)
- Santos-Ferreira, A., Cabral, M., & Santos, C. (2015). The rehabilitation of north breakwater of Nazaré harbor, Portugal. *Procedia Engineering*, 116, 755-762.
- Sigurdarson, S, *et al.*, 2010. *Coasts, marine structures and breakwaters: Adapting to change*. January 1th, 2010 (1: 156-168).

SPM (1984) – *Shore Protection Manual*.

TAW, 2002. *Technical Report on Wave Run-up and Wave Overtopping Dikes*. Technical Advisory Committee on Flood Defence, Delft (2002).

Teixeira, Ricardo André Fernandes (2012). *Quebramares portuguesas. Inventário e análise comparativa de soluções*. Masters dissertation, FEUP.

The Rock Manual. *The use of rock in hydraulic engineering (second edition)*. CIRIA, CUR, CETMEF. 978-0-86017-683-1. June, 2007.

Thomas Telford, 2015. *Breakwater development in Spain. The last ten years*. Thomas Telford Limited 2010, 2556636. VAT Reg. No: 240877747, July 02, 2015.

Van Der Meer, Jentsje W (1992). *Conceptual design of rubble mound breakwaters*.

Van Gent, 2001 M.R.A. *Wave run-up on dikes with Shallow foreshore*. J. Waterw. Port Coast. Ocean Eng. ASCE, 127 (5) (2001), (pp. 254-262)

Veloso-Gomes, F. Taveira-Pinto, F. Rosa-Santos, P. Coastal Structures 2007, Volume I, 187-198, World Scientific Publishing.

Williams, R. Briganti, R. Pullen, T. Dodd, N. *The uncertainty in the prediction of the distribution of individual wave overtopping volumes using a non-linear Shallow water equation solver*. Journal Coastal Research. 2015. <http://www.bioone.org/doi/10.2112/JCOASTRES-D-14-00241.1> (in press)

Zanuttigh, B., Van der Meer, J.W., Bruce, T. and Hughes, S. 2013. Statistical characterisation of extreme overtopping wave volumes. *Proc. ICE, Coasts, Marine Structures and Breakwaters 2013*, Edinburgh, UK.

http://www.huffingtonpost.com/2015/01/15/human-impacts-danger_n_6480782.html. 08/01/2017.

<https://www.deltares.nl/en/software/overtopping-neural-network/>. 01/02/2017.

<https://www.pik-potsdam.de/news/press-releases/archive/2012/projektionen-zum-meeresspiegelanstieg-koennten-unterschaetzt-worden-sein>. 06/01/2017.

ANNEXE I

(Work Environment in APDL)

A1.1. Work environment in APDL

During the development of this thesis, a work environment experience has been made in APDL (Administration of Ports of Douro, Leixões and Viana do Castelo, S.A.). The main goals of this professional environment experience, were to learn the dynamics underlying the activity of this company, as well as the observation of interventions made by APDL, that are related to this thesis subject matter.

1. Recognition of company premises and workers: On the first day in APDL, a full recognition of the company premises, and many departments and presentation to all the persons working in this enterprise, were made by Engineer Hugo Lopes.
2. A follow-up of a quay reinforcement by introducing a slab in the ground to increase the strength of this quay. A full presentation to all the staff team has been made.
3. Visit to the Oil Terminal of Leixões: In this visit, many areas of the breakwater have been analyzed. For instance, in the Post A, an observation and visual damage observation was made for the exterior slope with an armour layer composed by tetrapods and for the roundhead of the breakwater:
 - This visit gave an idea of the amount of concrete volume needed to build this structure. The tetrapods are a better solution in terms of the structure's resistance to wave overtopping. The Antifer cubes are also stable but they boost the waves.
 - After the crest rise of the initial breakwater, and the tetrapods introduction on the armour layer, a deposit zone for the reserve blocks was made, with the capability to store 25 to 30 blocks, that can be used in the need of armour units replacement or reinforcement of the armour layer.
 - On this visit it could be seen how strong the overtopped waves impact is on that blocks, for they initially were placed in the deposit zone, far from the waves impact, but the overtopping waves moved these blocks of several tons a few meters, through time.
 - Every year, some tetrapods have to be replaced, for in some cases the armour layer is uncovered during winter's season.
 - Related to the breakwater's roundhead, the diffraction and refraction phenomena are very visible. This structural element has been damaged and reconstructed several times. The last intervention took place in 2013 after a total destruction of the head.
 - A diversion work in order to avoid the passage under the Titã, was made on the past year. The Titan structure was unstable so there was a need for a new passage. Some

tetrapods and rockfill from the excavations were used on the armour layer of this new passage, and also concrete and *tout venant*, that have been ordered for this intervention.

This work environment experience at APDL, allowed many important aspects:

- A complete external and internal view of a national and international enterprise, and how trade is a very complex business;
- To understand the dynamics underlying the activity of a public company with the scope of the APDL;
- To grow the knowledge in terms of maritime projects;
- Acknowledging the constant need for rehabilitation and interventions on coastal and port protection projects, which should be carried out during the summer season;
- Observation in situ of wave overtopping consequences in structural units such as tetrapods.

ANNEXE II

A2.1. Wave-by-wave Overtopping Analysis

On this attachment is presented all the graphics of wave-by-wave overtopping analysis, for all the tests.

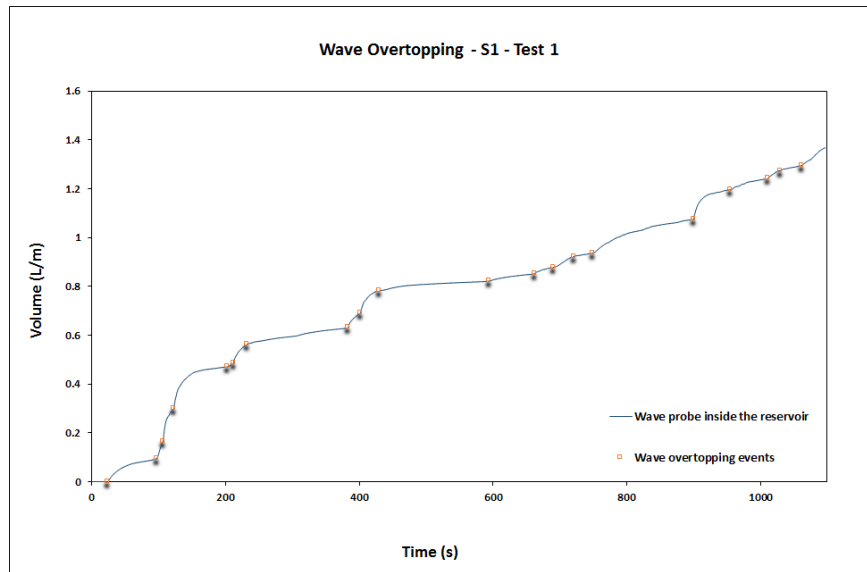


Fig. A.2.1 – Wave-by-wave overtopping.
Individual identification of the events for Test 1, Serie 1.

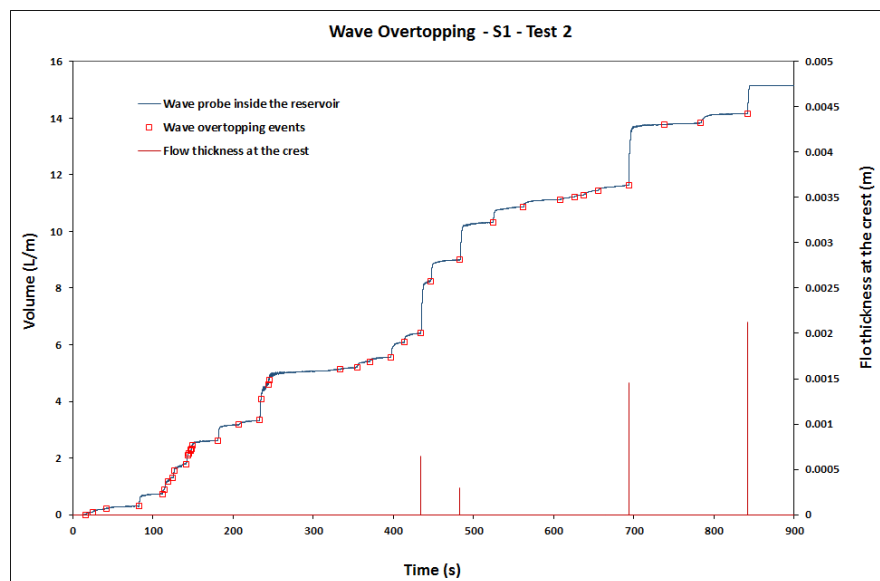


Fig. A.2.2 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Serie 1.

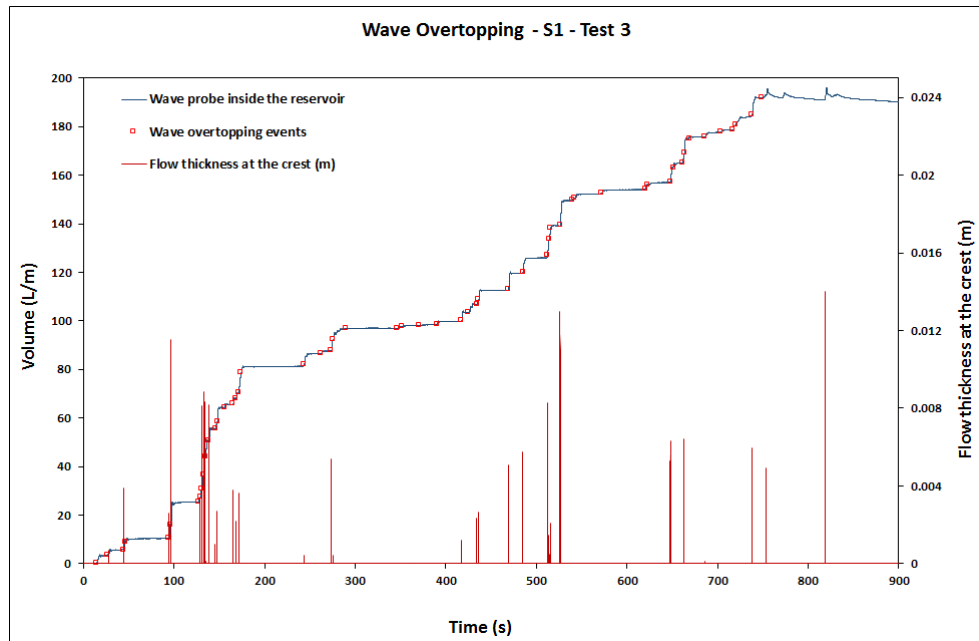


Fig. A.2.3 – Wave-by-wave overtopping.
Individual identification of the events for Test 3, Serie 1.

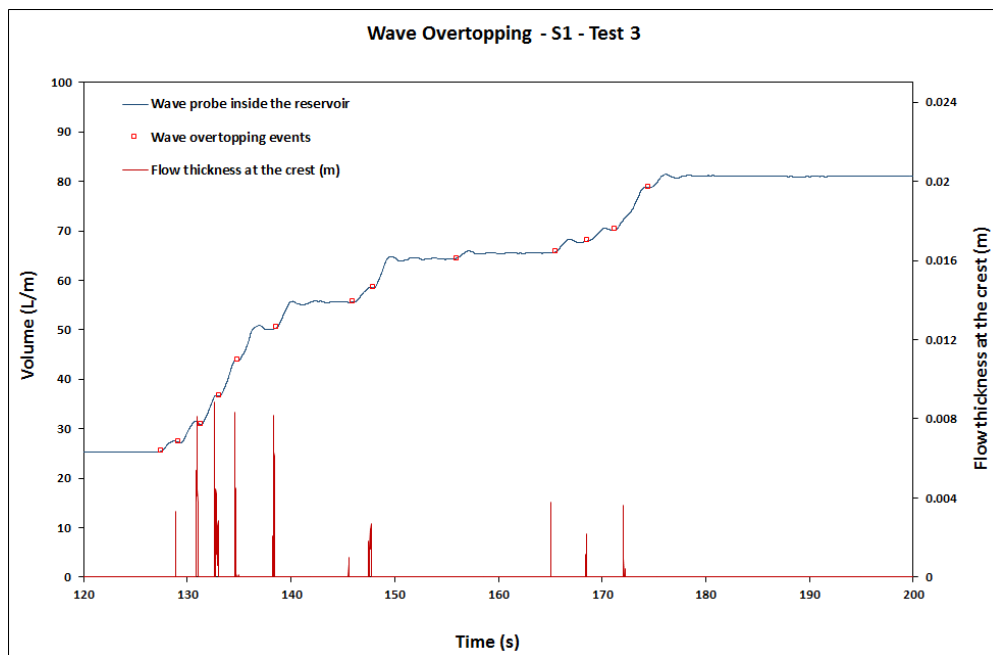


Fig. A.2.4 – Zoom-In of Figure 5.15 from Test 3, Serie 1.

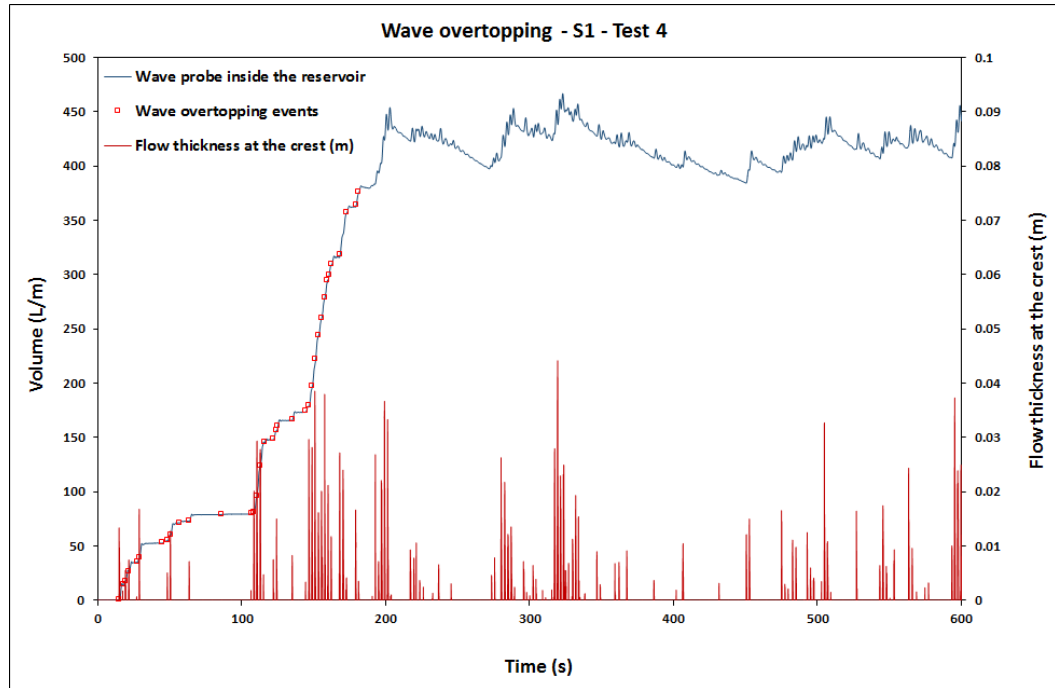


Fig. A.2.5 – Wave-by-wave overtopping.
Individual identification of the events for Test 4, Serie 1.

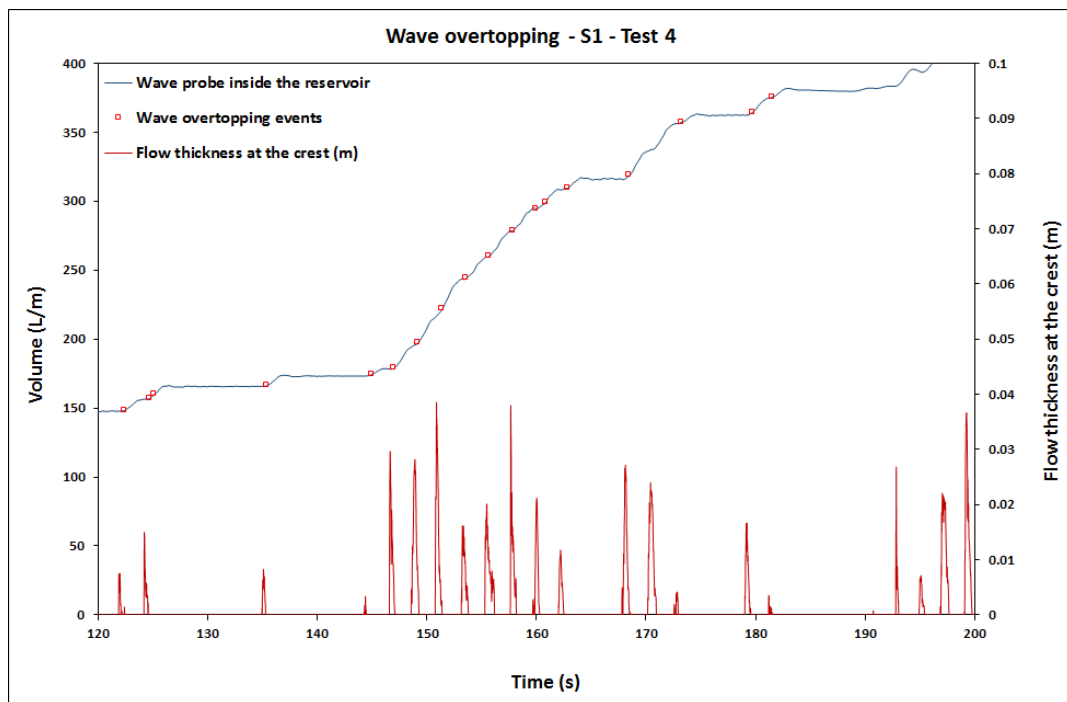


Fig. A.2.6 – Zoom-In of Figure 5.17 from Test 4, Serie 1.

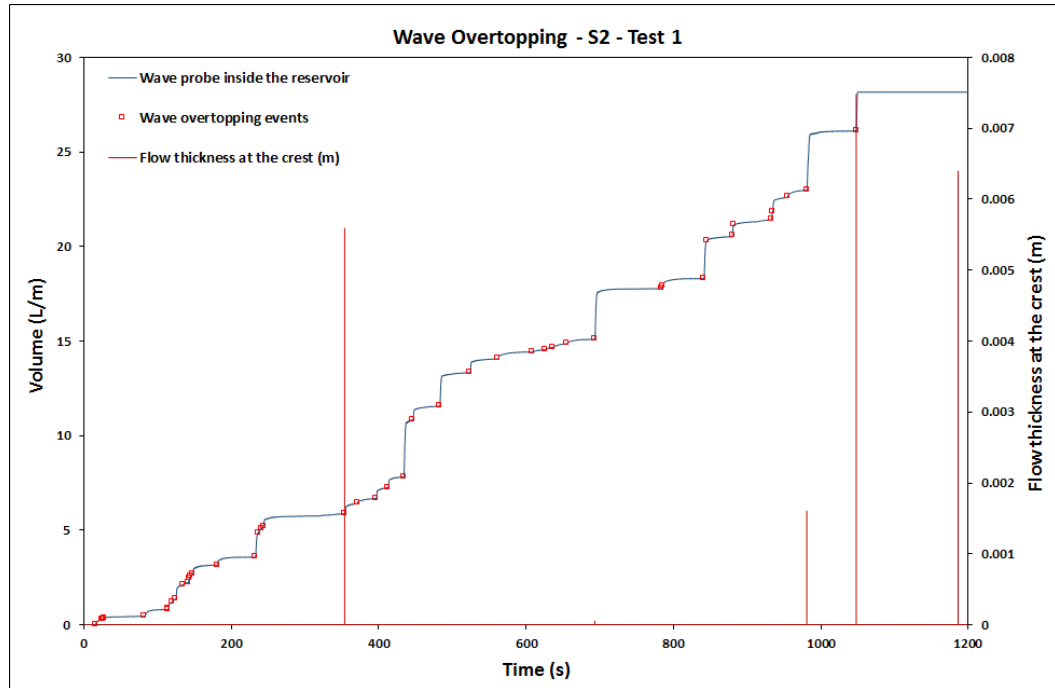


Fig. A.2.7 – Wave-by-wave overtopping.
Individual identification of the events for Test 1, Serie 2.

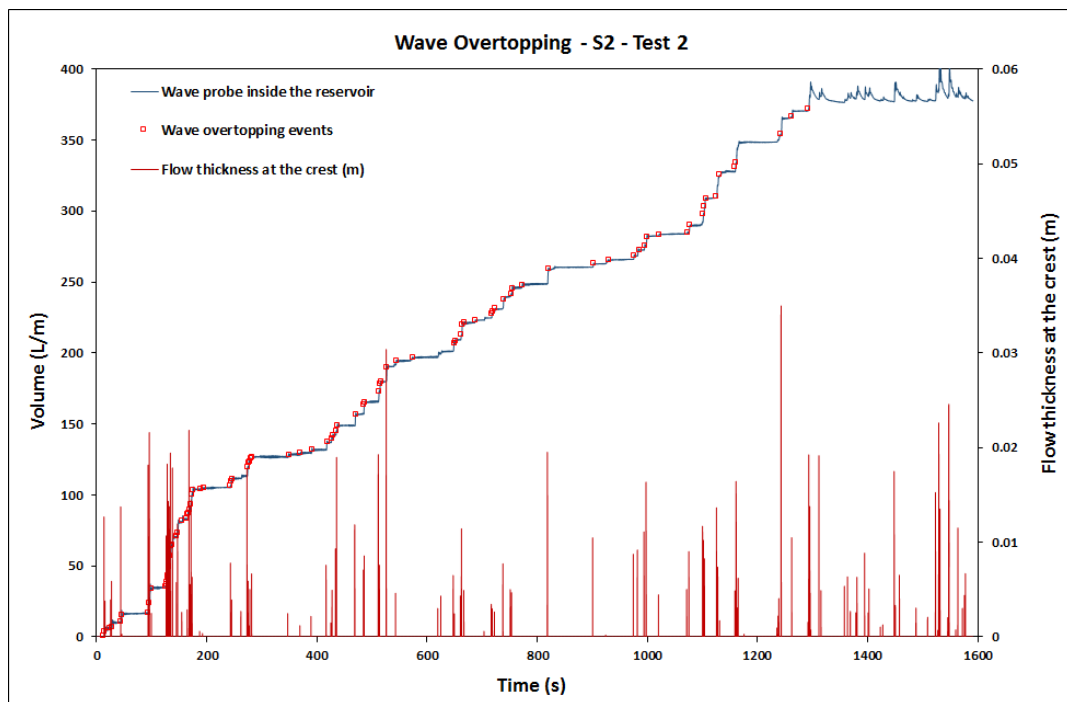


Fig. A.2.8 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Serie 2.

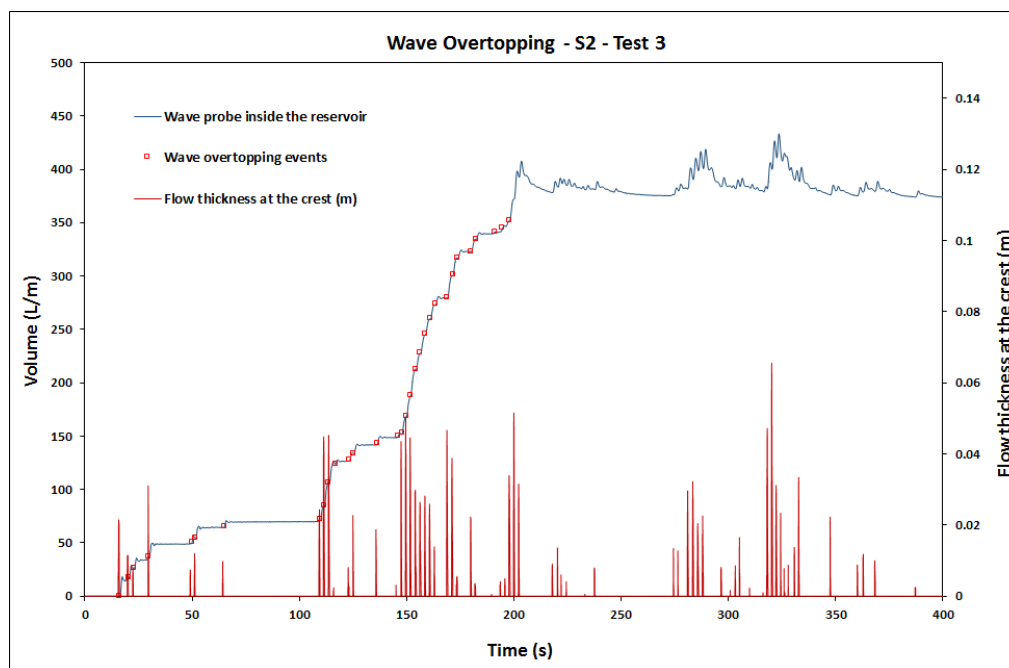


Fig. A.2.9 – Wave-by-wave overtopping.
Individual identification of the events for Test 3, Serie 2.

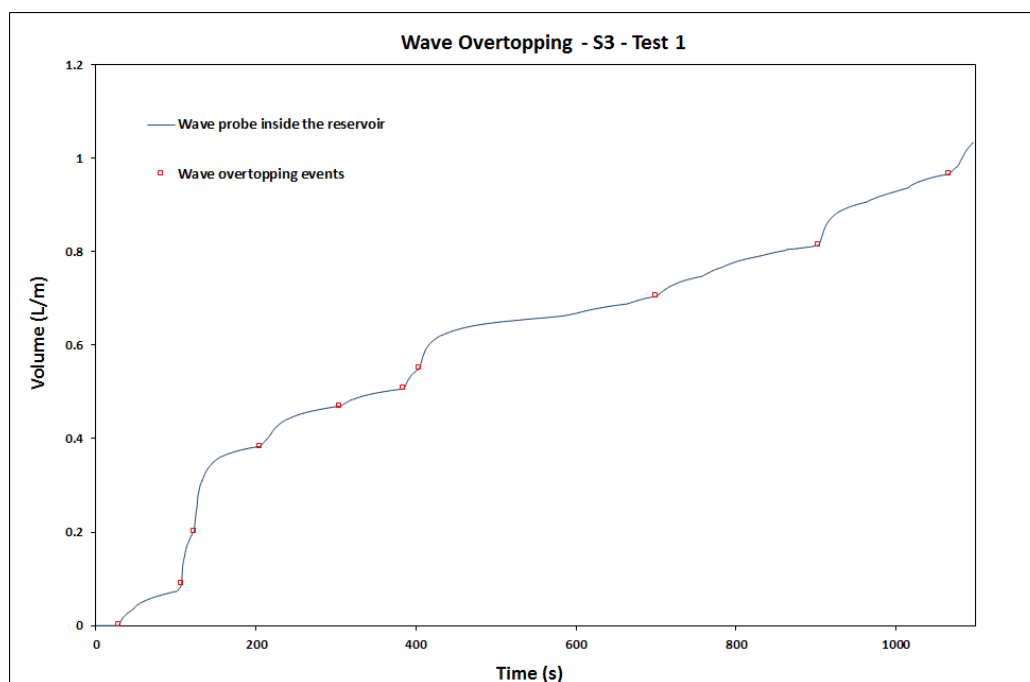


Fig. A.2.10 – Wave-by-wave overtopping.
Individual identification of the events for Test 1, Serie 3.

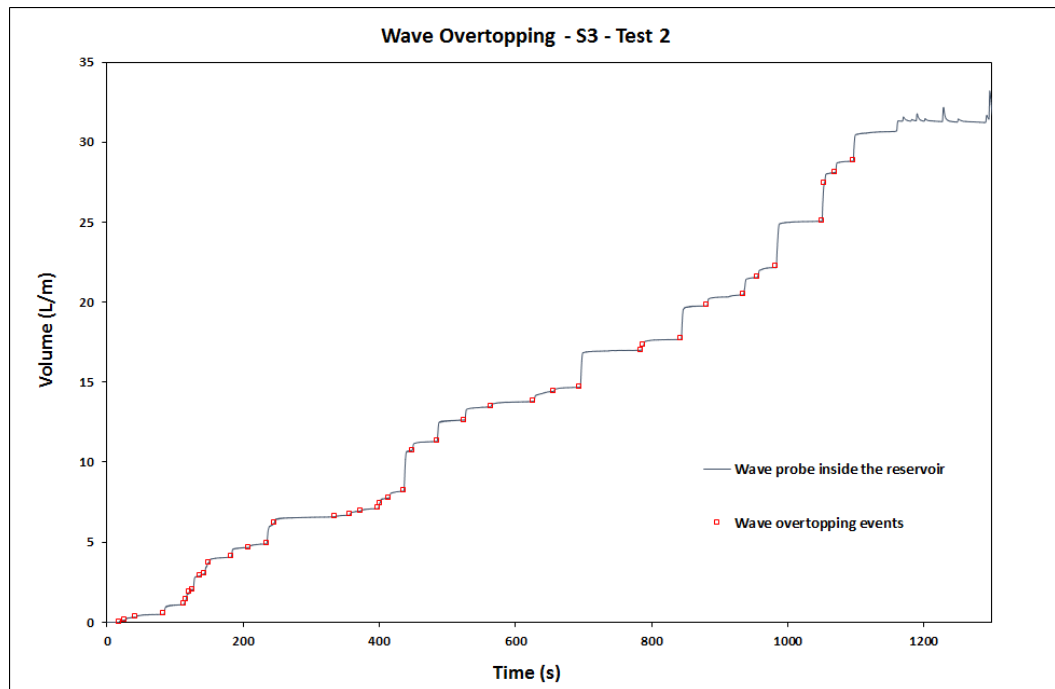


Fig. A.2.11 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Serie 3.

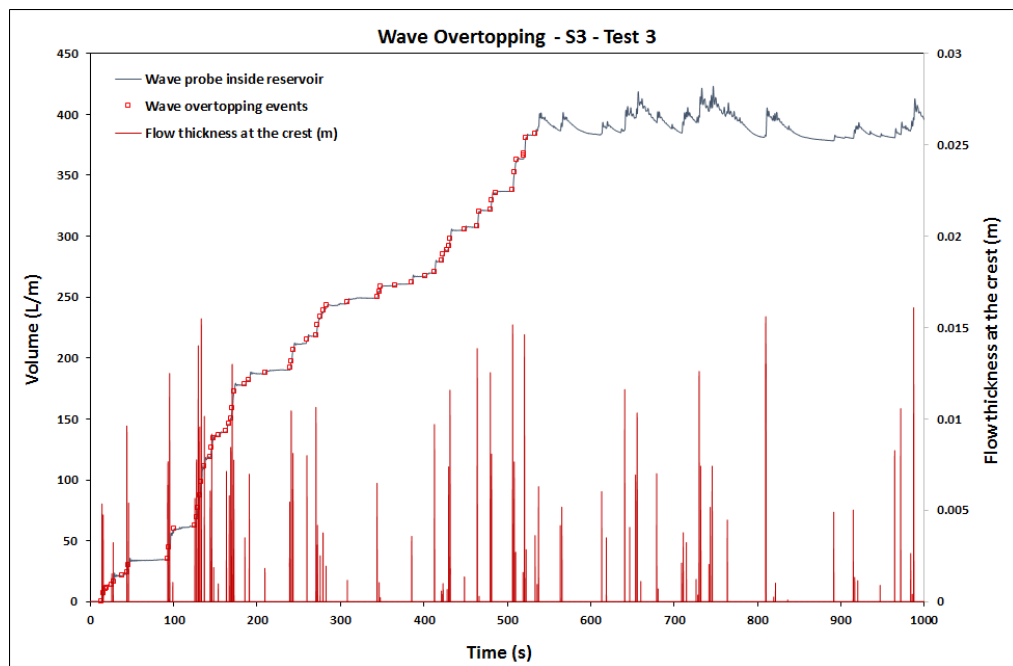


Fig. A.2.12 – Wave-by-wave overtopping.
Individual identification of the events for Test 3, Serie 3.

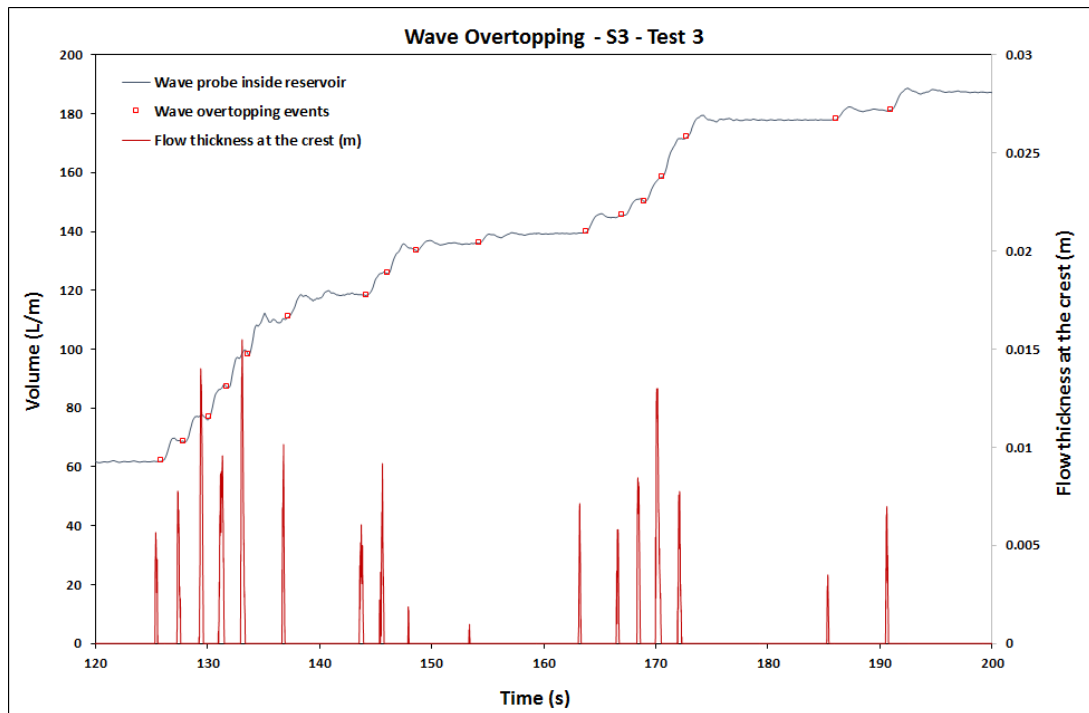


Fig. A.2.13 – Zoom-in of the wave-by-wave overtopping analysis, from Test 3, Serie 3.

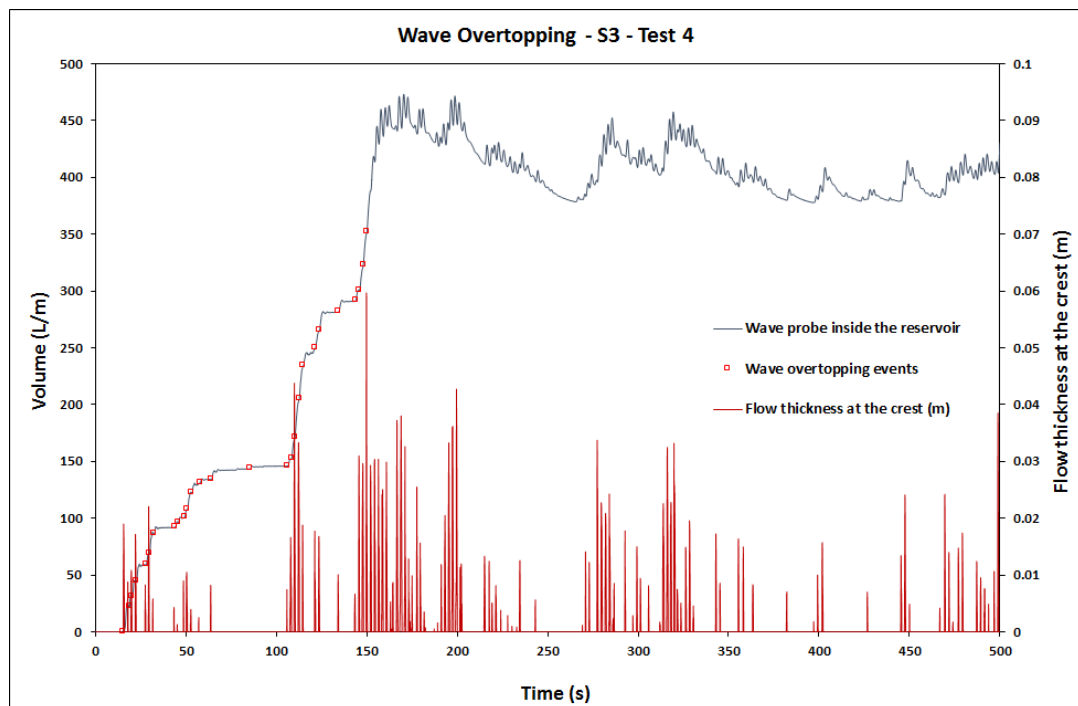


Fig. A.2.14 – Wave-by-wave overtopping.
Individual identification of the events for Test 4, Serie 3.

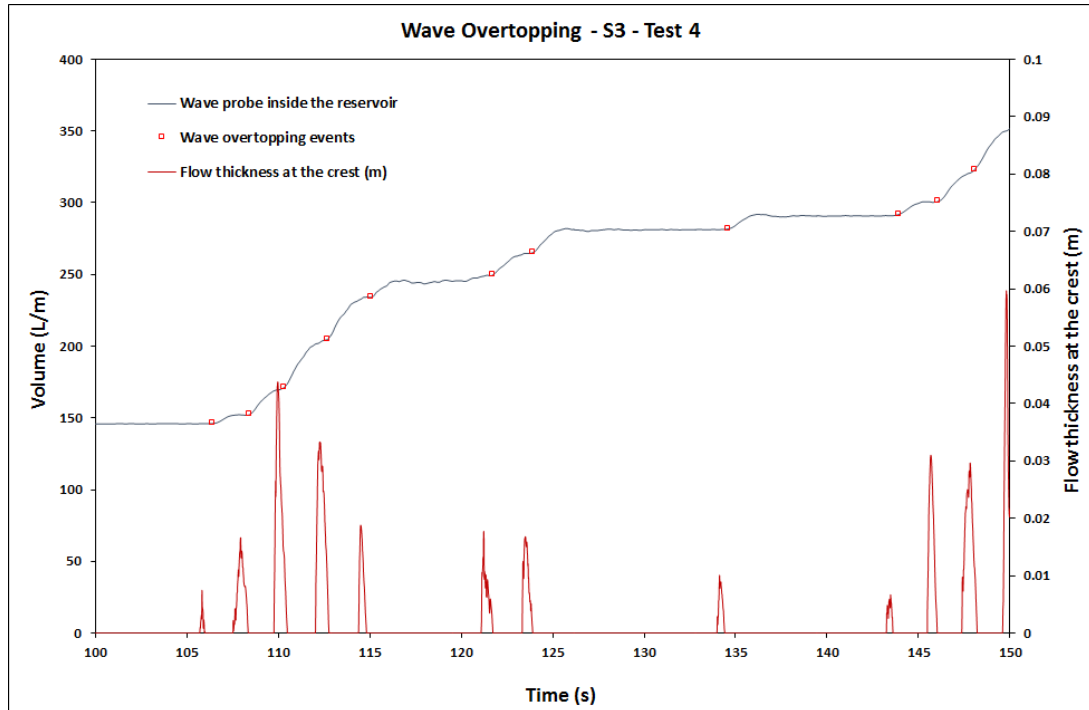


Fig. A.2.15 – Zoom-in of the wave-by-wave overtopping analysis, from Test 4, Serie 3.

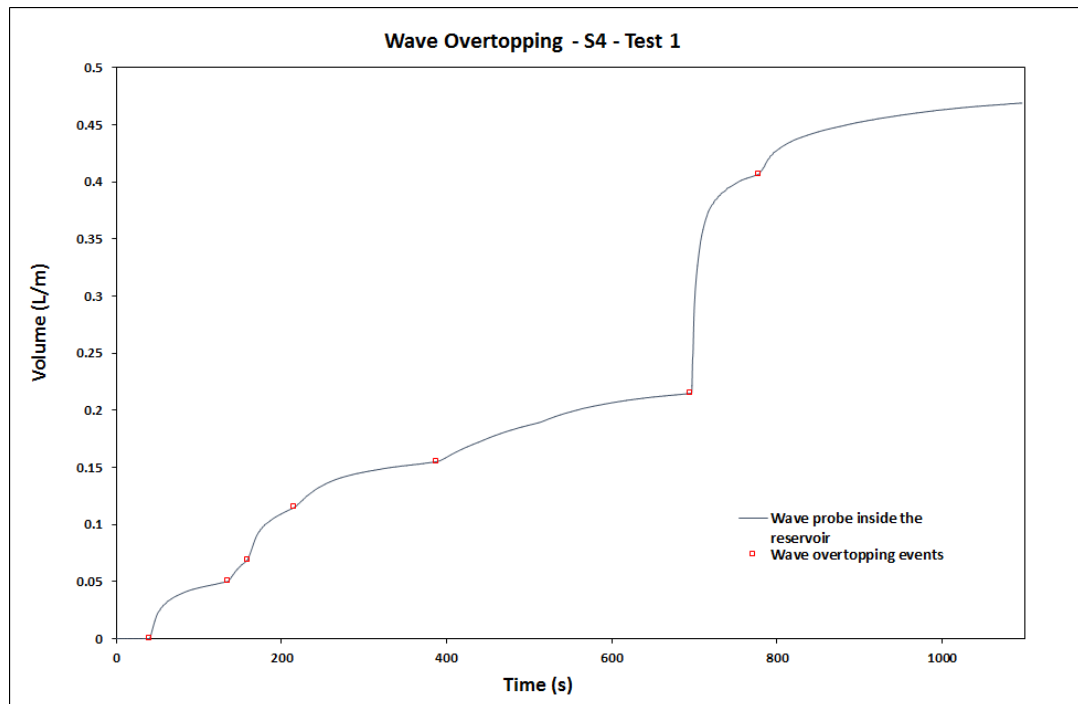


Fig. A.2.16 – Wave-by-wave overtopping.
Individual identification of the events for Test 1, Serie 4.

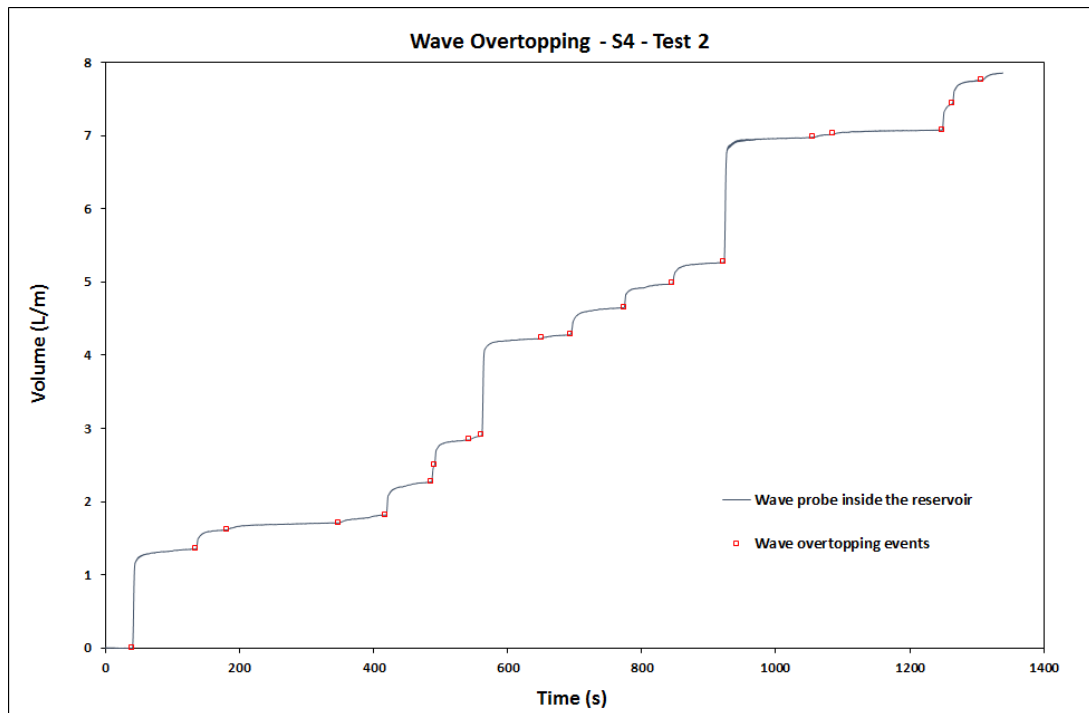


Fig. A.2.17 – Wave-by-wave overtopping.
Individual identification of the events for Test 2, Serie 4.

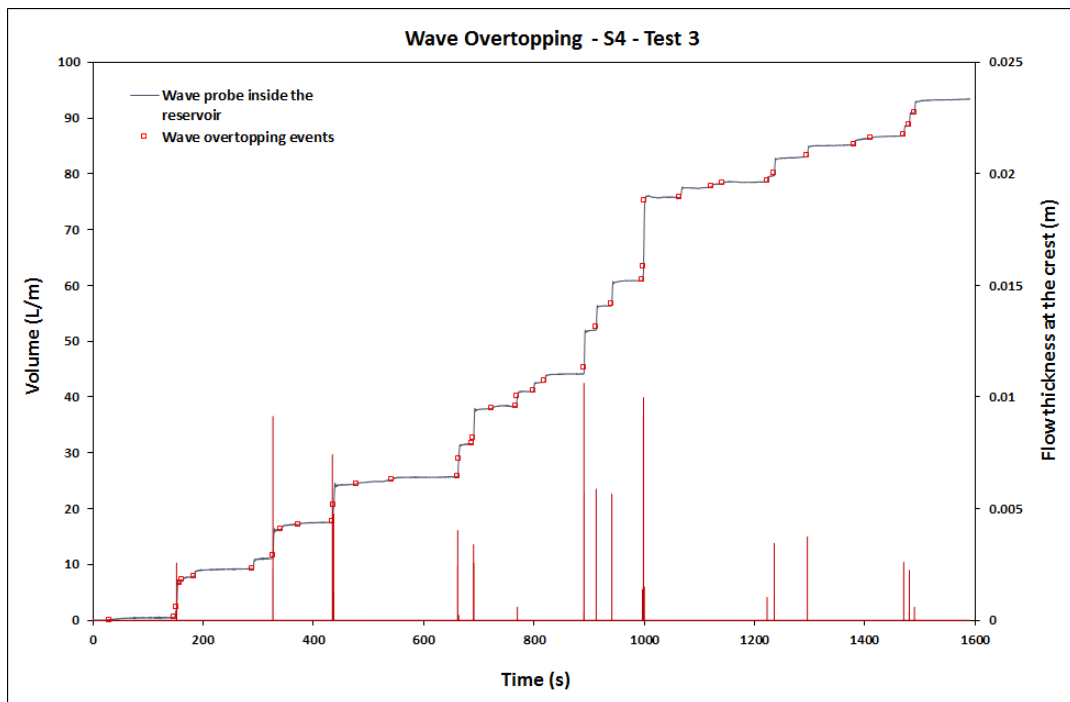


Fig. A.2.18 – Wave-by-wave overtopping.
Individual identification of the events for Test 3, Serie 4.

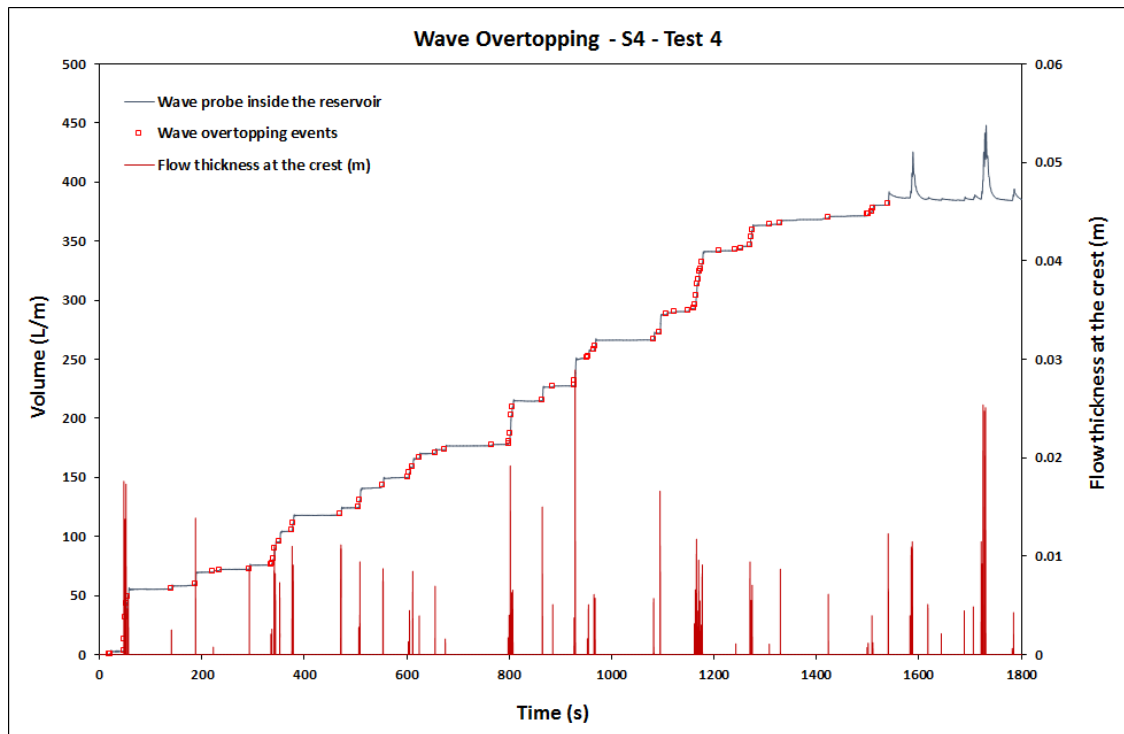


Fig. A.2.19 – Wave-by-wave overtopping.
Individual identification of the events for Test 4, Serie 4.

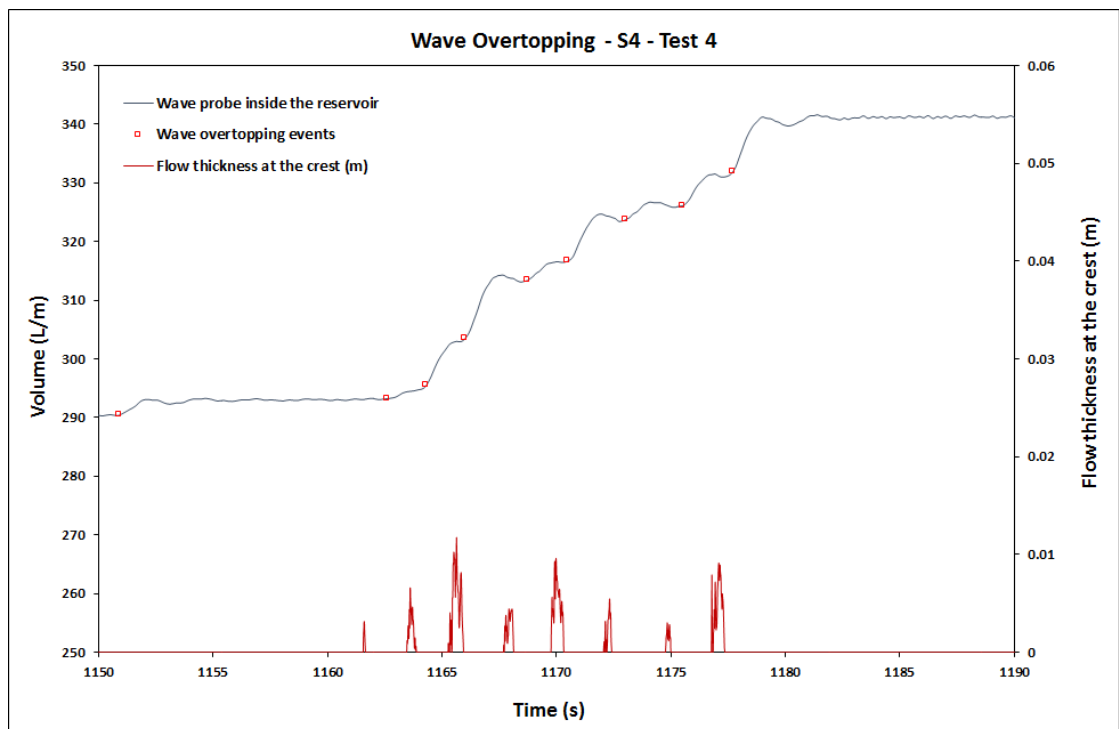


Fig. A.2.20 – Zoom-in of the wave-by-wave overtopping analysis, from Test 4, Serie 4.