

HYDRAULIC STABILITY AND WAVE OVERTOPPING OF SINGLE-LAYER CUBE ARMoured BREAKWATERS

FILIPe VIEIRA BORGES VIEIRA

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Supervisor: Professor Doctor Francisco de Almeida Taveira-Pinto

Co-Supervisor: Professor Doctor Paulo Jorge Rosa-Santos

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DEPARTAMENTO DE ENGENHARIA CIVIL

Tel. +351-22-508 1901

Fax +351-22-508 1446

✉ dec@fe.up.pt

Edited by

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>

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À minha família

Simplicity is the ultimate sophistication

Leonardo Da Vinci

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RESUMO

A utilização de blocos artificiais em mantos resistentes de quebra-mares de taludes tem evoluído continuamente desde os anos 50. O consumo reduzido de betão, o aumento da estabilidade hidráulica, a redução do espraçamento e facilidade nas técnicas de colocação são os principais fatores deste desenvolvimento. Os quebra-mares com manto resistente de cubos em camada única (SLC) com colocação regular são um conceito relativamente recente apesar de o cubo ser um dos blocos artificiais disponíveis mais antigos. Até ao presente, a investigação da estabilidade hidráulica e galgamento devido a ondas para este tipo de blocos e padrão de colocação é limitada.

Neste trabalho foram executados ensaios laboratoriais bidimensionais (2D) na Faculdade de Engenharia da Universidade do Porto, Portugal, para várias porosidades do manto resistente, declividades de onda, e cota do coroamento, para obter uma melhor compreensão do desempenho deste tipo solução para o manto resistente.

Para determinar automaticamente as posições dos cubos em cada teste foi desenvolvido um novo método recorrendo a técnicas de processamento de imagem que utilizam ferramentas avançadas de inteligência artificial. Este método reduz o tempo de processamento, pois várias imagens podem ser processadas em conjunto. Parâmetros inovadores para descrever o comportamento do manto resistente foram obtidos e propostos para a avaliação e quantificação de evolução do dano: probabilidade de excedência de movimentos do cubo, ortogonalidade média da malha, e número de dano relativo para SLC. Uma nova classificação e critérios para o início do dano são propostos para cada parâmetro.

Os galgamentos médios obtidos nos ensaios laboratoriais foram maiores do que os publicados na literatura para o mesmo tipo de manto resistente. Os métodos disponíveis para estimar galgamentos médios mostraram algumas limitações quando comparados com os ensaios realizados, sugerindo a relevância de outros parâmetros. Observou-se que, ao incluir o rácio de comprimento de onda com a largura do coroamento numa fórmula empírica alternativa, as estimativas de galgamento são melhoradas. O galgamento médio aumentou 16-42% devido a uma redução de porosidade, mas com pequenas diferenças nos coeficientes de reflexão. Verificou-se também que ao utilizar um *initial seed number* diferente, os valores de galgamento médio podem duplicar para os casos de baixas taxas médias de galgamento.

Os modelos numéricos são ferramentas valiosas para obter informação em processos de interação onda-estrutura que são difíceis de medir num modelo físico. O presente nível de precisão dos modelos numéricos é relativamente elevado, mas é necessária uma validação adequada para estabelecer os coeficientes empíricos a utilizar no modelo numérico, de modo a representar corretamente o fenómeno a ser investigado. Tradicionalmente utiliza-se a técnica de “tentativa e erro” para obter o(s) melhor(es) valor(es), o que pode resultar num elevado número de simulações. Uma nova metodologia para a determinação de coeficientes empíricos para as camadas porosas de um modelo VARANS-VOF, com base em redes neurais, foi assim proposta. O método permite uma grande redução no número de simulações, e subsequentemente tempo computacional, necessárias para a calibração do modelo numérico. O método é genérico e pode ser aplicado noutro tipo de estruturas e condições.

Este trabalho tem como principal objetivo contribuir para o conhecimento da performance de quebra-mares com SLC, considerando que atualmente existe informação limitada para esta configuração de manto resistente.

KEYWORDS: estabilidade hidráulica, galgamento, cubos em camada única, colocação regular, modelação física, modelação numérica.

ABSTRACT

The use of concrete armour units in rubble-mound breakwaters have been continuously evolving since the 1950s. Lower concrete consumption, increased hydraulic stability, reduced wave run-up, and easier placement techniques are the main drivers of this development. Single-layer cube (SLC) armoured breakwaters with a regular placement pattern are a relatively recent concept despite the cube being one of the oldest concrete units available. Limited research on the hydraulic stability and wave overtopping for this armour type is available to date.

In this work, 2D physical modelling tests were carried out at the Faculty of Engineering of the University of Porto, Portugal, for different armour porosities, wave steepnesses, and armour crest levels to gain a better understanding on the performance of this armour type and placement pattern.

A new method for automatically determining cube positions for each test was developed with image-processing techniques using advanced machine-learning tools. This method greatly reduces processing time since several images can be processed in batch mode. Innovative parameters to describe armour layer behaviour were derived and proposed for the assessment and quantification of the damage evolution: exceedance probability of cube movements, mean grid orthogonality, and relative damage number for SLC armoured breakwaters. New damage level classification and criteria for the initiation of damage are proposed for each of the parameters assessed.

The mean wave overtopping discharges obtained in the physical model tests were generally higher than the ones in literature for the same armour type. Available methods shown some limitations in predicting the measured overtopping discharges suggesting the relevance of other factors. It is observed that, by including the ratio of wavelength to the armour crest width in an alternative overtopping formula, the mean flow rate predictions are improved. It was also found that the mean overtopping discharges increase by 16-42% due to a decrease in armour porosity with negligible differences in reflection coefficients. A different initial seed number in the wave generation series can lead to differences in wave overtopping discharges up to a factor of two for lower mean overtopping discharges.

Numerical models are valuable tools to provide information on wave-structure interaction processes that are difficult to measure in a physical model. The current level of accuracy of numerical models is relatively high but an adequate validation to establish the models' empirical parameters is required to ensure a correct representation of the phenomena to be investigated. To this end, a "trial and error" approach is typically adopted, potentially resulting in many simulations. Therefore, a novel methodology for determining empirical coefficient values for the porous layers of a VARANS-VOF model for predicting mean overtopping discharges based on Artificial Neural Networks is proposed. The method enables a great reduction in the number of runs, and subsequently computational time, required for the calibration of the numerical model. The method is generic and can be used in other type of structures and forcing conditions.

This work aims to contribute to the knowledge on the performance of SLC armoured breakwaters considering the limited available information for this type of armour configuration.

KEYWORDS: hydraulic stability, wave overtopping, single-layer cube, regular placement, physical modelling, numerical modelling.

INDEX

AGRADECIMENTOS	i
RESUMO	iii
ABSTRACT	v
1 INTRODUCTION.....	1
1.1. MOTIVATION	1
1.2. OBJECTIVES	2
1.3. THESIS OUTLINE	2
2 LITERATURE REVIEW	5
2.1. INTRODUCTION.....	5
2.2. HYDRAULIC STABILITY	5
2.2.1. HYDRAULIC STABILITY OF CONCRETE ARMOUR UNITS	5
2.2.2. PREVIOUS STUDIES	6
2.2.3. KNOWLEDGE GAPS.....	7
2.3. WAVE OVERTOPPING.....	8
2.3.1. MEAN WAVE OVERTOPPING DISCHARGE.....	8
2.3.2. PREVIOUS STUDIES	9
2.3.3. KNOWLEDGE GAPS.....	9
2.4. NUMERICAL MODELLING OF WAVE OVERTOPPING	10
2.4.1. NUMERICAL MODELS.....	10
2.4.2. ARTIFICIAL NEURAL NETWORKS.....	12
3 PHYSICAL MODELLING	15
3.1. INTRODUCTION.....	15
3.2. EXPERIMENTAL FACILITIES	15
3.3. SCALING.....	17
3.4. MATERIALS.....	18
3.4.1. ARMOUR.....	18
3.4.2. FILTER	20
3.4.3. CORE	20
3.4.4. TOE	20
3.4.5. CROWN WALL	20
3.5. BATHYMETRY.....	21

3.6. CROSS-SECTION	21
3.7. MEASUREMENTS	23
3.7.1. SURFACE ELEVATION	23
3.7.2. OVERTOPPING	24
3.7.3. STABILITY	24
3.8. TEST PROGRAMME.....	25
3.9. TEST PROCEDURE.....	26
4 HYDRAULIC STABILITY	27
4.1. INTRODUCTION.....	27
4.2. METHODOLOGY FOR AUTOMATIC CUBE DETECTION.....	27
4.3. DAMAGE ASSESSMENT.....	28
4.3.1. CUBE MOVEMENT ANALYSIS.....	28
4.3.2. MEAN GRID ORTHOGONALITY	30
4.3.3. RELATIVE DAMAGE NUMBER FOR SLC.....	31
4.4. DAMAGE EVOLUTION CLASSIFICATION AND CRITERIA	33
5 WAVE OVERTOPPING	35
5.1. INTRODUCTION.....	35
5.2. PERFORMANCE OF EXISTING METHODS.....	35
5.3. NEW ESTIMATOR FOR SLC	35
5.4. INFLUENCE OF POROSITY	37
5.5. INFLUENCE OF INITIAL SEED NUMBER	38
6 NUMERICAL MODELLING OF WAVE OVERTOPPING	41
6.1. INTRODUCTION.....	41
6.2. MODEL DESCRIPTION	41
6.3. MODEL SETUP	41
6.4. MESH SENSITIVITY	43
6.5. ARTIFICIAL NEURAL NETWORK FOR MODEL CALIBRATION	43
6.6. RESULTS AND DISCUSSION	45
7 SUMMARY AND OUTLOOK.....	49
7.1. CONCLUSIONS	49
7.1.1. HYDRAULIC STABILITY	49
7.1.2. WAVE OVERTOPPING	50
7.1.3. NUMERICAL MODELLING OF WAVE OVERTOPPING	51

7.2. OUTLOOK	51
7.2.1. HYDRAULIC STABILITY	51
7.2.2. WAVE OVERTOPPING	52
7.2.3. NUMERICAL MODELLING OF WAVE OVERTOPPING	52
8 REFERENCES.....	53

APPENDIX - PUBLISHED JOURNAL PAPERS.....	63
APPENDIX I - VIEIRA <i>ET AL.</i> (2020C)	65
APPENDIX II - VIEIRA <i>ET AL.</i> (2021A)	77
APPENDIX III - VIEIRA <i>ET AL.</i> (2021B)	89
APPENDIX IV - VIEIRA <i>ET AL.</i> (2021C)	101

LIST OF FIGURES

Figure 1.1	- Sal-Rei breakwater. Courtesy of Van Gent and van der Werf (2017).....	2
Figure 2.1	- Stability number ranges obtained from the available literature for $N_{od}>0.05$ and $N_{od}>0.2$ for all (o) and filtered (x) data.....	7
Figure 2.2	- Volume-averaging process sketch for resolution of the porous flow (Losada <i>et al</i> , 2016).....	10
Figure 2.3	- Example of a two-layered ANN architecture (adapted from Vieira <i>et al</i> , 2020b).....	13
Figure 2.4	- Schematic of an ANN learning procedure (adapted from Vieira <i>et al</i> , 2020b).....	14
Figure 3.1	- HR Wallingford wavemaker installed at the wave basin of the SHRHA – FEUP.....	16
Figure 3.2	- Schematic layout of flumes inside SHRHA – FEUP wave basin.....	16
Figure 3.3	- Overview of wave basin and model (view from wavemaker area).....	17
Figure 3.4	- Silicon mould for cube production (top left), produced cubes drying (top right), sanding of cubes for achieving the target weight (bottom left) and painting of cubes (bottom right).....	19
Figure 3.5	- Sieving the core and filter material (left) and production of the crown wall (right).....	20
Figure 3.6	- Schematic of flume bathymetry.....	21
Figure 3.7	- Seabed construction sequence: Backfilling of the model area with bricks and blocks, and slope area with gravel (left); backfilling with gravel in the model area (middle); final step with concrete grout over backfilling (right).....	21
Figure 3.8	- Breakwater cross-section.....	22
Figure 3.9	- Breakwater construction sequence: Bulk placement of the core material with a shovel (left); filter placement completed (middle); placement of cube armour units (right).....	22
Figure 3.10	- Location of instrumentation in the wave flume.....	23
Figure 3.11	- Overview of wave probes at the wavemaker (left) and model (right) areas.....	23
Figure 3.12	- Overview of reservoirs (left) and model with overtopping tray and reservoirs...	24
Figure 3.13	- Overview of Faro® Focus during a scan (left), fixed markers above the structure (middle) and markers on the structure (right).....	25
Figure 4.1	- Example of Fiji/TWS flowchart for automatic detection of individual cube positions in the breakwater armour layer.....	28
Figure 4.2	- Example of the evolution of cube movements (mm) for series B ($n=0.29$) for $N_s=3.8$. The white squares represent the cube positions at the start of the test series. The black arrows show the direction of cube movement.....	29
Figure 4.3	- Exceedance probability of cube movements (mm) for all tests.....	30
Figure 4.4	- Evolution of mean grid orthogonality with stability number for all tests. MGO (top) and $MGO-MGO_0$ (bottom).....	31
Figure 4.5	- Photo after test B ($n=0.33$) for $N_s=3.5$	32
Figure 4.6	- N_{od-SLC} for all tests.....	33
Figure 5.1	- Comparison between non-dimensional mean overtopping discharge from experiments (FEUP and CLASH), VDMJ94 (top), EurOtop (middle) and ANN (bottom).....	36
Figure 5.2	- Comparison between non-dimensional mean overtopping discharge from experiments and alternative formula.....	37

Figure 5.3	- Scatter plot of H_{m0} , $T_{m-1,0}$, Kr and q for porosities 0.29 and 0.33.....	38
Figure 5.4	- Scatter plot of H_{m0} , $T_{m-1,0}$, Kr and q for different initial seed number (Series P and Q).....	39
Figure 6.1	- Model domain (top left) and mesh resolution in x (bottom left) and y (top right)	42
Figure 6.2	- Architecture of the ANN used in this study.....	45
Figure 6.3	- $nMSE$ for all ANNs tested (top) and comparison between modelled and ANN estimated mean overtopping discharges (bottom).....	46
Figure 6.4	- Comparison of mean overtopping discharges between ANN estimate and model predictions.....	46
Figure 6.5	- Comparison between model and measurements of Kr (left) and q^* (right).....	47
Figure 7.1	- Damage evolution classification for SLC armoured breakwaters.....	50

LIST OF TABLES

Table 2.1	- Range of physical model test conditions for single-layer cubes.....	6
Table 3.1	- Scaling factors according to Froude's law.....	17
Table 3.2	- Number of cubes per row for the various tests.....	19
Table 3.3	- Characteristics of the materials used in the breakwater.....	20
Table 3.4	- Range of parameter values at the toe of the structure for each test series.....	26
Table 4.1	- Stability number for IDa_{SLC} based on the proposed criteria for various parameters.....	33
Table 5.1	- Error metrics for the various overtopping prediction methods.....	37
Table 6.1	- α and β coefficients adopted for the reference cases.....	42
Table 6.2	- Characteristics of tested meshes.....	43
Table 6.3	- Mesh sensitivity results.....	43
Table 6.4	- Coefficients adopted in the simulations.....	44
Table 6.5	- Wave conditions at the toe of the structure and mean overtopping discharges from the physical model.....	44
Table 6.6	- $nMSE$ (%) for Kr and q for the various β values.....	47

SYMBOLGY AND ABBREVIATIONS

LETTERS

a_1, a_2	[-]	empirical coefficients in Van der Meer and Janssen (1994) formula
b	[-]	network biases in Artificial Neural Network
b_1, b_2, b_3	[-]	empirical coefficients in Van der Meer and Bruce (2014) formula
B	[m]	flume width
B_t	[m]	breakwater toe width
c_1, c_2	[-]	empirical coefficients in the overtopping formula proposed in this study
c_A	[-]	added mass coefficient
Cr	[-]	crest reduction factor
D_n	[m]	nominal diameter or equivalent cube length
D_{n50}	[m]	median nominal diameter or equivalent cube length
D_{50}	[m]	diameter of stone that exceeds the 50% of the sieve curve
D_{15}	[m]	diameter of stone that exceeds the 15% of the sieve curve
D_{85}	[m]	diameter of stone that exceeds the 85% of the sieve curve
e	[-]	error function in the Artificial Neural Network
f_p	[Hz]	wave peak frequency
Fr	[-]	Froude number
g	[m/s ²]	acceleration of gravity
G_c	[m]	armour crest width
h	[m]	water depth
h_t	[m]	water depth at the toe of the structure
H	[m]	wave height
H_0	[m]	significant wave height under design conditions derived from the stability number $N_s=2.2$
H_{max}	[m]	maximum wave height
H_{m0}	[m]	zero th -moment wave height in a spectrum or spectral significant wave height defined as $4m_0^{0.5}$
$H_s = H_{1/3}$	[m]	significant wave height derived from zero-crossing analysis
Ir or ξ	[-]	Iribarren's number or surf similarity parameter defined as $\tan\alpha/(H/L)^{0.5}$
Kr	[-]	reflection coefficient
L	[m]	wavelength or characteristic length measure
L_0	[m]	deepwater wavelength defined as $gT^2/2\pi$

L_{0m}	[m]	deepwater wavelength based on T_m
$L_{m-1,0}$	[m]	wavelength based on $T_{m-1,0}$
m_0	[-]	zero th -moment of wave spectrum, defined as $\int_0^\infty S(f)df$
M	[kg]	armour unit mass
M_{50}	[kg]	armour unit mass exceeded by 50% of the units
$M_{50-filter}$	[kg]	filter unit mass exceeded by 50% of the units
M_c	[kg]	cube unit mass
MGO	-	Mean Grid Orthogonality
MGO_0	-	As-built Mean Grid Orthogonality
n	[-]	porosity or number of data points
n_L	[-]	scale factor between model and prototype
$nRMSE$	[%]	normalized Root Mean Square Error
N	[-]	number of waves
N_o	[-]	number of units displaced by a distance equal to D_n
N_{od}	[-]	relative damage number
N_s	[-]	stability number defined as $H_s/(\Delta D_n)$
p	[-]	Artificial Neural Network predicted value
q	[m ³ /s/m]	mean overtopping discharge
q_{ANN}	[m ³ /s/m]	mean overtopping discharge derived by ANN
q^*	[-]	non-dimensional mean overtopping discharge defined as $q/(gH_{m0}^3)^{0.5}$
q_{SLC}^*	[-]	non-dimensional mean overtopping discharge for a single-layer cube armour
q_{VDMB}^*	[-]	non-dimensional Van der Meer and Bruce (2014) mean overtopping discharge
q_{VDMJ}^*	[-]	non-dimensional Van der Meer and Janssen (1994) mean overtopping discharge
r	[-]	correlation coefficient
R_c	[m]	structure freeboard
Re_D	[-]	Reynolds number
s	[-]	wave steepness defined as H/L
$s_{m-1,0}$	[-]	wave steepness based on $T_{m-1,0}$
s_{0m}	[-]	deepwater wave steepness based on T_m
s_{op}	[-]	deepwater wave steepness based on T_P
$S(f)$	[m ² /Hz]	Non-directional wave spectral density function

t	[-]	Artificial Neural Network target value
T	[s]	wave period
T_m	[s]	mean wave period
$T_{m-1,0}$	[s]	wave period calculated from the first negative moment of the spectrum
T_P	[s]	peak wave period
u	[m/s]	characteristic velocity
w	[-]	synaptic weight in the Artificial Neural Network

GREEK SYMBOLS

α	[-]	armour slope [rad] or empirical coefficient associated with the linear drag force in the IH2VOF model
β	[-]	empirical coefficient associated with the non-linear drag force in the IH2VOF model
Δ	[-]	relative buoyant density defined as $(\rho_c - \rho_w) / \rho_w$
γ	[-]	peak enhancement factor in JONSWAP spectrum
γ_b	[-]	berm reduction factor in overtopping formulas
γ_h	[-]	shallow foreshore reduction factor in overtopping formulas
γ_f	[-]	roughness reduction factor in overtopping formulas
γ_β	[-]	angle of wave attack reduction factor in overtopping formulas
ρ_c	[kg/m ³]	mass density of concrete or rock
ρ_w	[kg/m ³]	mass density of water
ϕ	[-]	armour packing density
ϕ_{SPM}	[-]	armour packing density recommended by USACE (1984)
ν	[N.s/m ²]	dynamic fluid viscosity
ν	[m ² /s]	kinematic fluid viscosity

ABBREVIATIONS

ANN	Artificial Neural Network
CLASH	Crest Level Assessment of coastal Structures by full-scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping
FEUP	Faculdade de Engenharia da Universidade do Porto
IDa	Initiation of damage
IDa_{SLC}	Initiation of damage in single-layer cube armour

<i>IDe</i>	Initiation of destruction
<i>IDe_{SLC}</i>	Initiation of destruction in single-layer cube armour
IH2VOF	Instituto de Hidraulica 2D Volume of Fluids model
IHRH	Instituto de Hidráulica e Recursos Hídricos
JONSWAP	Joint North Sea Wave Project
SHRHA	Secção de Hidráulica, Recursos Hídricos e Ambiente
SLC	Single-Layer Cube
SLR	Sea Level Rise
SWL	Still Water Level
VARANS	Volume-Averaged Reynolds Averaged Navier-Stokes
VOF	Volume of Fluid

1

INTRODUCTION

1.1. MOTIVATION

Rubble-mound coastal structures protect coastlines, roads, railways, industrial and residential buildings, and ports. Knowledge of a rubble-mound armour performance in terms of hydraulic stability and wave overtopping is, therefore, essential for the design of new structures and the upgrading of existing ones to cope with climate change effects on Sea Level Rise (SLR) and wave storminess. IPCC (2019) predicts an average global mean SLR by 2100 of 0.43 m to 0.84 m. This impact, together with a generally positive trend in mean and storm wave heights around the world (e.g., Aboobacker *et al.*, 2017, Reguero *et al.*, 2019, Vieira *et al.*, 2020a) will result in detrimental effects on the world's coastlines (Vieira *et al.*, 2021).

The use of single-layer cubes (SLC) with a regularly placed pattern and low porosity for the seaside armour of a breakwater is a recent concept. Previous studies indicate that this is a feasible solution with several advantages over other armour configurations. Van Gent and Luis (2013) concluded that a breakwater with cubes in a single-layer is feasible and potentially economically competitive with other single layer units despite having a lower stability number. The main advantages of a single-layer of cubes compared to other one-layer systems are: the simplicity of formwork in fabrication; high structural strength; patent-free; smaller casting yard area requirements, since the removal of the formwork is done vertically; safer handling using pressure clamps, whilst some other units make use of rope slings; efficient area utilization for stacking; simple placement grid; the lower likelihood of breaking units during construction; greater production rate and easier repair works. Disadvantages are related to the low resilience offered if some blocks are extracted or settlement occurs (as in any single-layer configuration), negligible interlocking between units, higher concrete consumption compared to other single-layer units, and difficulties during construction in the presence of waves due to a strict placement tolerance.

Studies on hydraulic stability and wave overtopping are relatively few compared to other units and, to the author's knowledge, only one prototype (Figure 1.1) with such configuration has been built to date (Van der Lem *et al.*, 2016). Furthermore, the available methods for quantification of damage are not considered adequate for use in single-layer cube armour with a regular placement pattern.

This thesis focuses on the study of the hydraulic stability and wave overtopping of SLC armoured breakwaters. Both physical and numerical models were implemented and used during this work.



Figure 1.1 – Sal-Rei breakwater. Courtesy of Van Gent and van der Werf (2017).

1.2. OBJECTIVES

The main objectives of this study are to:

- Carry out a detailed review of previous studies on the use of SLC armoured breakwaters;
- Highlight any knowledge gaps or limitations from the literature review associated with methods and techniques for analysis of results, and structure geometries and forcing conditions tested;
- Define a two-dimensional physical modelling test plan aimed at the analysis of hydraulic stability of the armour slope and overtopping discharges;
- Execute the laboratory tests and perform a detailed analysis of the results using state-of-the-art measurement techniques and software;
- Develop a new method for calibration and validation of a wave-structure interaction model to represent the mean wave overtopping discharge, using Artificial Neural Networks;
- Disseminate the findings of the study through peer-reviewed journal publications.

1.3. THESIS OUTLINE

This thesis is divided into seven chapters summarized in this section. A brief introduction to the subject and study objectives is presented in the current chapter. Chapter 2 covers the literature review on single-layer cube armoured breakwaters in terms of wave overtopping and hydraulic stability. The laboratory facilities, equipment, and physical model planning is described in detail in Chapter 3. The innovative method for automatic cube detection and the newly proposed damage classification and criteria based on different parameters are described in Chapter 4. Chapter 5 focuses on wave overtopping where results from the laboratory experiments are used to derive a new formulation to estimate mean wave overtopping discharges in SLC armoured breakwaters. The calibration of a numerical model to simulate the measured mean overtopping discharges in the experiments is included in Chapter 6. Finally, in chapter 7, the conclusions of the study and recommendations for future work are presented.

The results of the thesis were published in the following journals:

- Vieira, F., Taveira-Pinto, F., Rosa-Santos, P., 2020c, *Single-layer cube armoured breakwaters: critical review and technical challenges*, Ocean Engineering 216, 108042. <https://doi.org/10.1016/j.oceaneng.2020.108042> (no. citations: 2);
- Vieira, F., Taveira-Pinto, F., Rosa-Santos, P., 2021a, *New developments in assessment of wave overtopping on single-layer cube armoured breakwaters based on laboratory experiments*, Coastal Engineering 166, 103883. <https://doi.org/10.1016/j.coastaleng.2021.103883> (no. citations: 3);

- Vieira, F., Taveira-Pinto, F., Rosa-Santos, P., 2021b, *Damage evolution in single-layer cube armoured breakwaters with a regular placement pattern*, Coastal Engineering 169, 103943. <https://doi.org/10.1016/j.coastaleng.2021.103943>;
- Vieira, F., Taveira-Pinto, F., Rosa-Santos, P., 2021c, *Novel time-efficient approach to calibrate VARANS-VOF models for simulation of wave interaction with porous structures using Artificial Neural Networks*, Ocean Engineering 235, 109375. <https://doi.org/10.1016/j.oceaneng.2021.109375> (no. citations: 2).

2

LITERATURE REVIEW

2.1. INTRODUCTION

This chapter provides a description of the literature review on the topics covered in this thesis: hydraulic stability and wave overtopping in single-layer cube armoured breakwaters. It includes an overview of each topic covering existing studies and knowledge gaps. A more detailed description of the topics presented in this chapter was published in Vieira *et al* (2020c). Hereafter, a summary of the most relevant information is presented.

2.2. HYDRAULIC STABILITY

2.2.1. HYDRAULIC STABILITY OF CONCRETE ARMOUR UNITS

Damage in concrete armour layers is often quantified by the relative damage number, N_{od} , proposed by Van der Meer (1988), as the actual number of units displaced in a width (along the longitudinal axis of the structure) of an armour unit nominal diameter or equivalent cube length, D_n , and defined as:

$$N_{od} = \frac{N_o}{B / D_n} \quad (2.1)$$

where N_o represents the number of units displaced by a distance equal to D_n , and B the width of the tested section.

Van der Meer (1999) developed an empirical formula to estimate the stability number, N_s , for steep sloped structures with an armour of double-layer cubes in non-breaking wave conditions:

$$N_s = \frac{H_s}{\Delta D_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \left(0.40 + 0.61 \left[\frac{\phi}{\phi_{SPM}} \right] \right) \right) s_{om}^{-0.1} \cdot \left(1 + 0.17 \exp \left[\frac{R_c}{D_n} \right] \right) \quad (2.2)$$

where H_s represents the significant wave height, $\Delta = \rho_c / \rho_w - 1$ the relative buoyant density, with ρ_c the rock or concrete density and ρ_w the water density, N the number of waves, ϕ the armour packing density, ϕ_{SPM} the recommended packing density given by USACE (1984) with a value of 1.17, and s_{om} the deepwater wave steepness, defined as $s_{om} = H_s / L_{om}$, where L_{om} represents the deepwater wavelength based on the mean wave period T_m , and R_c the crest freeboard.

Categories of the different damage levels for double-layer structures were also proposed by Losada *et al* (1986) and Vidal *et al* (1991), as follows:

- Initiation of Damage (*IDa*): a certain number of armour units are displaced from their original position at a distance of or larger than a unit length and/or holes larger than average porous size are appreciable;
- Iribarren Damage (*IR*): Armour units from the lower armour layer are exposed;

- Initiation of Destruction (*IDe*): A small number of units is removed from the lower armour layer and the filter is exposed;
- Destruction: Filter layer units are removed.

The application of the above definitions of N_{od} and damage levels to single-layer cube (SLC) armoured structures present some limitations as described further in the following sections.

2.2.2. PREVIOUS STUDIES

Several physical model studies on the hydraulic stability of SLC armoured breakwaters were carried out to date (Van Gent and Spaan, 1998; d'Angremond *et al*, 1999; Van Gent *et al*, 1999; 2001; Van Buchem, 2009; Almeida, 2013; Van Gent and Luis, 2013; Van Gent, 2014; Van der Lem *et al*, 2016) (Table 2.1). The best performance in terms of stability was achieved for a structure slope of 1V:1.5H, a porosity of 0.25 to 0.30, and a filter mass larger than 1/20th of the armour unit (Vieira *et al*, 2020c).

The stability numbers for $N_{od} > 0.05$ and $N_{od} > 0.2$ range from 1.7 to 3.9 and 2.1 to 4.5, respectively, considering all test results available (Figure 2.1). The values are derived from an extensive range of test conditions with several tests aimed at performing sensitivity analysis on parameters such as filter layer and porosity. Some of the tested values are not feasible or effective in prototype cases such as porosity of 0.4 or filter mass ($M_{50-filter}/M_c$) less than 1/20. Figure 2.1 also shows filtered data that correspond to tests for an emerged structure configuration with a slope of 1V:1.5H, a porosity of 0.25 to 0.30 and a filter mass larger than 1/20th of the armour unit. The stability numbers for $N_{od} > 0.05$ and $N_{od} > 0.2$ in this case range from 2.3 to 3.9 and 2.5 to 4.5, respectively, which are larger than if all tests are considered.

Table 2.1 – Range of physical model test conditions for single-layer cubes.

Reference	Placement	n	$M_{50-Filter}/M_c$	S_{om}	h (m)	R_c/D_n	$\cot \alpha$
Van Gent <i>et al</i> (1999)	Irregular	0.25	1/10-1/5	0.03	0.30	2.1	1.5
		0.30	1/20-1/10	0.05	0.43	5.2	
		0.40	1/50	0.06	0.50	11.1	
Van Gent <i>et al</i> (2001)	Irregular	0.25	1/10	0.02	0.30	9.1	1.5
		0.30	1/20	0.04	0.50	18.2	
		0.40	1/75	0.06			
Van Buchem (2009)	Regular	0.20	1/20-1/10	0.02	0.50	6.7	1.5
		0.28		0.04			2.0
		0.35		0.06			
Almeida (2013)	Regular	0.25	1/20-1/10	0.03	0.30	2.2	1.5
		0.30	1/50	0.05	0.50	5.2	
				0.06			
Van Gent (2014)	Irregular	0.25	N/A	0.036	0.65	N/A	1.5
				0.048	0.80		
Van der Lem <i>et al</i> (2016)	Regular	0.28	1/17-1/6	0.01 0.02	N/A	4.2	1.5

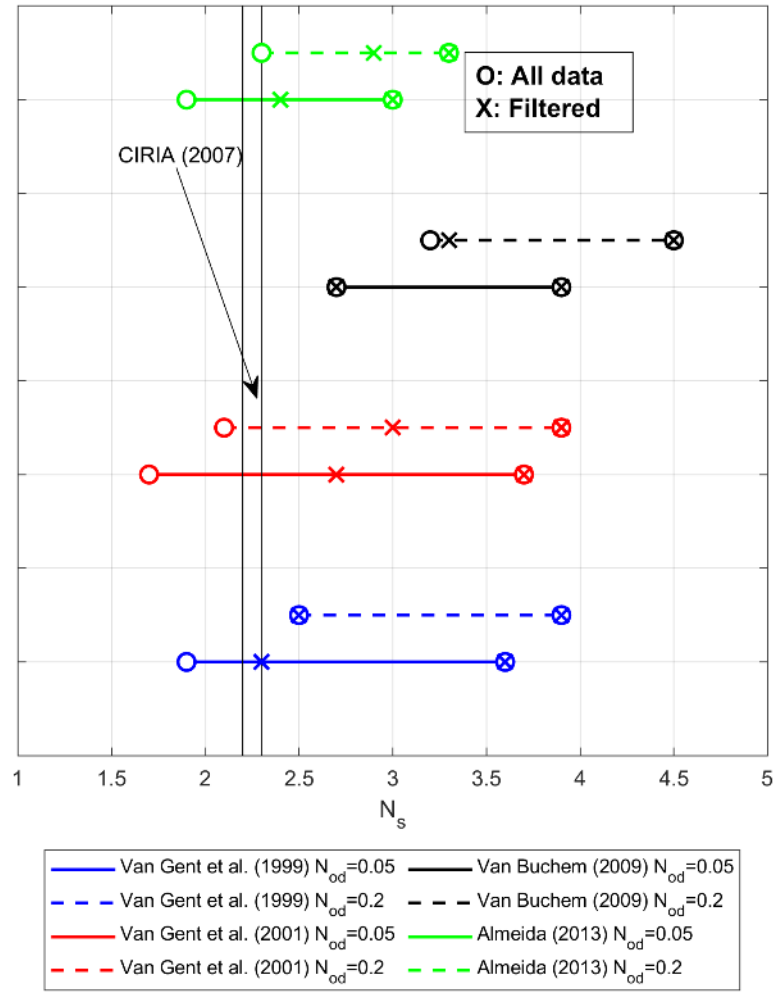


Figure 2.1 – Stability number for $N_{od} > 0.05$ and $N_{od} > 0.2$ for all (o) and filtered (x) data.

2.2.3. KNOWLEDGE GAPS

The definition of the N_{od} applied to double-layer structures is a relatively objective criterion as it provides valuable information on the damage evolution, which is a gradual process in these structures corresponding to the development of a s-shape profile until exposure of the filter layer occurs (Vieira *et al*, 2020c). The classic methodology to quantify damage does not reflect the real behaviour of a single-layer armoured structure. This methodology accounts for one-piece movement if it moves more than one D_n . Thus, they do not allow to measure the settlements and analyse the first phases of damage, which are very important in breakwaters with rigid behaviour (Sande *et al*, 2017). The stability of single-layer armoured structures can be described by a fixed stability number for “start of damage” (defined as $N_{od}=0$) and “failure” (defined as $N_{od}>0.5$) damage levels (Van der Meer, 1999). Adopting the classical definition for N_{od} , this means that units can move up to a maximum distance of D_n before N_{od} becomes larger than zero. A value of $N_{od}=0.2$ for “failure” has been adopted for SLC in previous studies. This may not be adequate since $N_{od}>0$ indicates that at least one cube has moved by a distance of D_n or extracted, which implicitly results in exposure of the filter layer (Vieira *et al*, 2020c).

2.3. WAVE OVERTOPPING

2.3.1. MEAN WAVE OVERTOPPING DISCHARGE

Mean overtopping discharges for “non-breaking”, “surging” wave conditions in relatively simple geometries can be estimated using the empirical formula of Van der Meer and Janssen (1994) (hereafter referred to as “VDMJ94”). The formula was derived from several data sets (e.g., Owen, 1980; Van der Meer and De Wall, 1990; Van der Meer and De Wall, 1993), which are included in the CLASH (CLASH, 2004) database (De Rouck *et al*, 2009), and is valid for structure slopes, $cot\alpha$, between 1 and 4 and relative freeboards, R_c/H_{m0} , between 0.5 and 3.5:

$$q_{VDMJ}^* = \frac{q}{\sqrt{gH_{m0}^3}} = a_1 \cdot \exp\left(-a_2 \frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_h \gamma_f \gamma_\beta}\right) \quad (2.3)$$

where q_{VDMJ}^* represents the non-dimensional overtopping discharge, q the mean overtopping discharge, g the acceleration due to gravity, H_{m0} the spectral significant wave height, R_c the crest freeboard (the difference between the still water level and the crest elevation), γ_b the berm reduction factor, γ_h the shallow foreshore reduction factor, γ_f the roughness reduction factor and γ_β the angle of wave attack reduction factor. a_1 and a_2 are empirical coefficients with values of 0.2 and 2.6, respectively.

Van der Meer and Bruce (2014) derived updated empirical formulas to represent better the mean overtopping discharges. These formulas give almost the same wave overtopping as the original formula, equation (2.1). Only for low and zero freeboards, these formulas will be better than VDMJ94 (Van der Meer and Bruce, 2014). For simple armoured slopes EurOtop (2018) recommends using equation (2.4) corrected with the crest width reduction factor, C_r , proposed by Besley (1999), equation (2.5).

$$q_{VDMB}^* = \frac{q}{\sqrt{gH_{m0}^3}} = b_1 \cdot \exp\left(-\left(b_2 \frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_h \gamma_f \gamma_\beta}\right)^{b_3}\right) \quad (2.4)$$

$$C_r = \min(1; 3.06 \cdot \exp(-1.5 G_c/H_{m0})) \quad (2.5)$$

where b_1 , b_2 and b_3 are empirical coefficients with values of 0.09, 1.5 and 1.3, respectively, and G_c is the armour crest width.

Van Gent *et al* (2007) developed an artificial neural network (ANN07) to predict the average overtopping discharge for a variety of wave and water level conditions and taking into account complex structure configurations based on the CLASH database. The neural network consists of a multi-layer network, with 14 dimensionless input parameters (physical conditions and structure geometry), 20 hidden neurons, and one output parameter (average overtopping discharge, q_{ANN}). ANN07 applied resampling techniques to 500 networks for the assessment of the uncertainties in the mean overtopping discharge predictions. It was shown that the agreement between the predicted overtopping discharge and the measured overtopping discharge is well acceptable and accurate when compared to the datasets used. An updated neural network (Zanuttigh *et al*, 2016), ANN16, is presented in EurOtop (2018). ANN16 was developed and trained with additional tests to the CLASH database resulting in more than 13,000 tests. More recently, Den Bieman *et al* (2020) developed a prediction method based on a specific machine learning technique, gradient boosting decision trees, and trained and tested it using the CLASH database. The results show that this method outperforms existing neural networks by reducing the error on the prediction of the CLASH database by a factor of 2.8. The accuracy of any machine learning prediction is greatly related to the number and quality of the data used to develop and train the tool. In

the case of SLC armoured breakwaters, only 18 test results from the experiments of Pearson *et al* (2004) are part of the training database.

2.3.2. PREVIOUS STUDIES

Few studies have focused on the evaluation of the performance of wave overtopping in permeable rubble mound structures with an armour of single-layer of cubes regularly placed (Vieira *et al*, 2020c). Pearson *et al* (2004) presented test results for a single-layer of cubes ($D_n=0.030$ m, $M=0.062$ kg, $\rho_{cubes}=2,361$ kg/m³) in a regularly placed pattern with a porosity of 0.35. Tests were carried out with spectral significant wave heights, H_{m0} , equal to H_0 , $0.75 H_0$ and $0.5 H_0$, where H_0 is the significant wave height under design conditions derived from the stability number $N_s=2.2$. A flat seabed was adopted in the model. Each wave height was repeated for three offshore wave steepnesses, $s_{op}=0.02$, 0.035 and 0.05 (with $s_{op} = H_{m0}/(gT_p^2/2\pi)$, where T_p represents the peak wave period) and two relative freeboard levels, $Rc/H_0=1.3$ and 0.8 . All tests resulted in non-breaking wave conditions. The structure slope, *cota*, was 1.5 and the armour crest and toe width were $3D_n$ wide. The weight of the underlayer material was in the range of $1/5^{th}$ to $1/15^{th}$ of the weight of the armour blocks. Eighteen tests for single-layer cube armoured breakwaters were carried out with two-thirds of these for $Rc/H_{m0}\leq 1.5$. The results show that generally larger overtopping discharges were obtained when compared to other single-layer armour units. Bruce *et al* (2009) re-analysed the database from Pearson *et al* (2004) experiments to derive roughness factors to be included in the VDMJ94 formula. The best fit for mean overtopping discharges between the data and the VDMJ94 formula was obtained for $\gamma_f=0.52$.

2.3.3. KNOWLEDGE GAPS

Available wave overtopping studies for regularly placed cube armoured breakwaters are limited to the tests of Pearson *et al* (2004). Although this study provides a solid base for research in this type of configuration, additional research is required to assess the influence of armour porosity on wave overtopping discharges. This was done for other concrete units (Molines *et al*, 2012). Molines and Medina (2015) presented a method for calculating the roughness factor for different armour porosities based on the CLASH database (Van der Meer *et al*, 2009) and the CLASH Neural Network method (Van Gent *et al*, 2007). Armour units placed with higher porosity tend to have a smaller roughness factor resulting in lower overtopping discharges (Molines and Medina, 2015). The recommended roughness factor for overtopping estimates with a single-layer regularly placed cube armour in EurOtop (2018) is based on the tests of Pearson *et al* (2004). However, it is unlikely that a prototype will be constructed with a porosity of 0.35, as tested in Pearson *et al* (2004), since it will result in a structure with lower hydraulic stability performance. Better hydraulic stability has been achieved for porosities between 0.25 and 0.30.

The available physical model tests also focused on a specific range of relative freeboard values (0.8 and 1.3). Further investigation on low-crested structures ($Rc/H_{m0}<0.5$) and low overtopping structures ($Rc/H_{m0}>1.5$) would be valuable for extending the range of applicability of the empirical formulations (or adjusting these formulations). In general, there is very limited information on the effect of wave obliquity on wave overtopping, particularly for single layer cube armoured breakwaters.

2.4. NUMERICAL MODELLING OF WAVE OVERTOPPING

2.4.1. NUMERICAL MODELS

Numerical modelling is a valuable tool to provide information on the processes that are difficult to measure in a physical model, despite limitations inherent to models in representing real-case situations. The level of accuracy of the present numerical models is relatively high and many applications were successful in reproducing laboratory experiments and the physical reality (prototype). Numerical modelling can also mitigate two of the biggest weaknesses of physical modelling: a) extraction of data and b) scale and laboratory effects. In numerical models, data of all kinds, such as velocity, pressure, water surface elevation, turbulent kinetic energy etc. can be extracted from any point and time in the simulation and the “virtual” probes do not interfere with the flow (Vysikas, 2014).

Most models solving the flow within a porous media are based on the Navier-Stokes (NS) equations. Two different approaches have been followed in the numerical modelling of wave propagation and interaction with structures when using the NS equations: Lagrangian and Eulerian.

The most popular Lagrangian models used in coastal engineering are based on the Smoothed Particle Hydrodynamics (SPH) method, where the fluid is discretized into a set of particles. Each computational unit, referred to as a particle, is a nodal point where physical quantities are computed as an interpolation of the values of the nearest particles (Vanneste *et al*, 2014). Equations need to be solved locally, in a discrete way, for individual particles on a meshless domain. As a drawback, if larger domains need to be covered, a large amount of individual particles/nodes needs to be considered to ensure a good definition of the free surface deformation. That implies a very high computational demand, which is a weak point when real applications are studied (Losada *et al*, 2016). Another limitation is that SPH cannot model a truly incompressible fluid.

The governing equations for Eulerian models are discretized in space with a finite difference/finite volume technique, using structured grids with staggered mesh topology. A cut-cell method is used for obstacle representation, defining cell-based area and volume fractions, which are directly incorporated in the conservation equations of mass and momentum. This method is relatively efficient in terms of computational cost and the amount of interaction needed from the user (compared to body-fitted meshing, for example), but also limited in accuracy by the mesh resolution. The Volume-of-Fluid (VOF) technique is used for free surface tracking (Vanneste *et al*, 2014). Modelling of porous media (rubble-mound structures) is highly complex since these are characterized by irregular pore shapes and sizes within a solid matrix, which are usually assumed rigid (Losada *et al*, 2016). Volume Averaged Reynolds Average NAVier-Stokes (VARANS) equations allows one to consider the porous material as continuous media when characterized by their macroscopic properties only. This eliminates the need for a detailed description of their complex internal geometry (Figure 2.2).

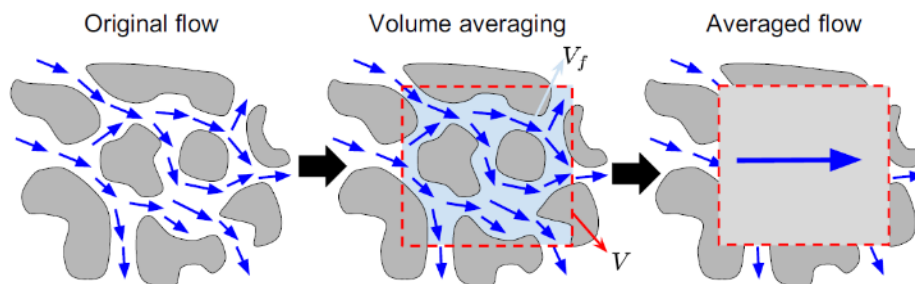


Figure 2.2 – Volume-averaging sketch for porous flow (Losada *et al*, 2016).

The VARANS equations are derived by integrating the RANS equations over a control volume determined based on the structure porosity (n). The porous media in the VARANS equations are modelled using Forchheimer's relationship that includes two empirical coefficients associated with the linear (α) and non-linear (β) drag forces. The expression used in the model for the linear and non-linear drag forces is given by:

$$I = \frac{\alpha v(1-n)^2}{n^2 D_{50}^2} \vec{u} + \frac{\beta(1-n)}{n^2 D_{50}} \vec{u} |\vec{u}| \quad (2.6)$$

where v represents the kinematic viscosity and $\vec{u}$ the velocity vector. The added mass coefficient (c_A), included in the formulation to consider the added mass, is defined as (Van Gent, 1995):

$$c_A = 0.34 \frac{1-n}{n} \quad (2.7)$$

Losada *et al* (2008) applied a 2D model, based on the VARANS equations, to an experimental study of a breakwater. The results showed that the model was able to reproduce instantaneous and average wave overtopping discharges with a mean error of 9% and 2.5% between measurements and model results, for the mean and maximum overtopping discharge volumes, respectively. Cavallaro *et al* (2012) and Dentale *et al* (2018) present a three-dimensional Reynolds Average Navier-Stokes Equations model (FLOW-3D®) to simulate the wave-structure interactions with full simulation of the *Flow Within the Armour Units* model in an attempt to provide a detail description of the hydrodynamics around breakwater armour layer units. The model was applied to two Italian breakwaters, Port of Gela and Sant'Erasmo, for which scale model experiments results were available. The model results compared relatively well with the advantage of not needing to include any empirical coefficients for the calibration of the porous media flow. Jensen *et al* (2014) investigated the empirical coefficients applied for representation of the porous media in the VARANS equations. The model was applied and calibrated against a set of experiments for various flow regimes. The model, using the recommended empirical coefficients, was applied to the experimental results of Bruce *et al* (2009) for a rock armour layer and it was found to give a good estimation of overtopping discharges. The simulated results were distributed around the empirical relation in the same manner as the experimental results. No general over- or underestimation was seen. Pilechi *et al* (2018) applied the 2D VARANS model IH2VOF (Lara *et al*, 2011) to a rubble-mound breakwater laboratory experiment. The comparisons were conducted for a range of sea state conditions, water levels, and breakwater geometries. The average overtopping discharges predicted by the numerical model are within 20% of accuracy compared to those measured in the physical model. Considering the complex geometry of the simulated cases and the high nonlinearity introduced into the waves during their propagation in the shallow water, the consistent results of the numerical model indicated that the tool had acceptable performance in simulating the overtopping process. Palma *et al* (2016) also applied the model IH2VOF to assess the impacts of waves on the *Overtopping Breakwater for Energy Conversion* (OBREC). The simulated breakwater has a relatively complex geometry, but the overtopping results still show that the error obtained with the numerical model was relatively small when compared to the measurements. The calibrated model was then used to assess the performance of different structure geometries.

There is still large uncertainty in describing α and β coefficients correctly. Lara *et al* (2008) suggest that they depend on parameters such as the Reynolds number, shape of the stones, grade of the porous material, permeability and flow characteristics. Therefore, values for these coefficients should be determined based on the comparison of the numerical model results with experimental data. Although considerable effort has been done in the last three decades to better understand porous flow characteristics in coastal engineering applications, there are still open questions. The wide range of values found in literature is a clear indication that there is still room for improvement in the

determination of the porous parameter values and how they are linked to different flow conditions (Losada *et al*, 2016).

In most of the above-mentioned studies the determination of α and β coefficients to obtain a satisfactory model calibration was done either by taking values from the literature, which are not always applicable to the particular cases being analysed, or by trial and error. Considering that there are usually 2 to 3 layers of porous material (armour and core or armour, filter and core), this corresponds to a large number of possible combinations of parameters, which, subsequently, results in a great number of simulations and long computational times to define the best. Furthermore, the trial-and-error process is not a systematic approach. One can test different combinations, but better results could eventually be achieved. Therefore, there is a need to develop a systematic and consistent approach to determine the best combination of the porous media coefficients. Since this is a non-linear problem, it is suggested to implement Artificial Neural Networks to obtain better estimates of the coefficients for each different case in a more efficient way.

2.4.2. ARTIFICIAL NEURAL NETWORKS

Artificial Neural Networks (ANN) are data-driven algorithms that aim at simulating the structure of the human brain to identify patterns in complex data, given a set of inputs and outputs (Lippman, 1987). ANN are techniques useful to solve non-linear problems when enough experimental data are available for training. This process does not require any prior knowledge of the actual physics, although it is important to know which input variables are relevant to the problem to be solved. ANN are widely used in coastal and maritime engineering for various purposes including the estimation of wave overtopping of coastal structures (Medina, 1999; Medina *et al*, 2003a; Van Gent *et al*, 2007; Zanuttigh *et al*, 2016), wave transmission through breakwaters (Panizzo *et al*, 2003; Panizzo and Briganti, 2007), wave reflection from coastal structures (Taveira-Pinto *et al*, 2006; Garrido and Medina, 2012; Zanuttigh *et al*, 2013), wave forces on crown walls (Molines *et al*, 2018), wave forces on vertical structures (Van Gent and Van den Boogaard, 1998), stability of rubble mound breakwaters (Balas *et al*, 2010; Gedik, 2018; Iglesias *et al*, 2008; Kim *et al*, 2008; Kim *et al*, 2014; Kim and Park, 2005; Lee *et al*, 2016; Mase *et al*, 1995; Medina *et al*, 2003b), beach processes (Iglesias *et al*, 2005; Lopez *et al*, 2018), wave transformation (Browne *et al*, 2007; Santos *et al*, 2016), wave forecasting (Deka and Prahlada, 2012; Deo *et al*, 2001; Etemad-Shahidi and Mahjoobi, 2009; James *et al*, 2018; Makarynskyy and Makarynska, 2007; Mandal and Prabakaran, 2006; O'Donncha *et al*, 2018; Reikard *et al*, 2015) and data gap filling (Arena and Puca, 2004; Huang and Xu, 2009; Balas *et al*, 2004; Lee and Park, 2016; Peres *et al*, 2015; Ustoorikar and Deo, 2008; Silva *et al*, 2018; Vieira *et al*, 2020b).

ANN include processing unit elements called neurons, which transform, through a nonlinear transfer function, a linear combination of the preceding layer. The coefficients of the linear combination named synaptic weights (w), together with the network biases (b), represent the free parameters of the network model (Peres *et al*, 2015). One of the most common types of ANN is the feed-forward multi-layer perceptron, where the output of each neuron in each layer is only transmitted to the following layer. The multi-layer perceptron consists of various layers. After the input layer, one or more hidden layers, consisting of several neurons to be defined in the preparation of the ANN (Van Gent *et al*, 2007), passes information to the output layer, which involves a number of neurons equal to the number of output parameters to be predicted. This network can approximate any function with a finite number of discontinuities arbitrarily well, given a sufficient number of neurons in the hidden layer (Beale *et al*, 2019). The schematic of a two-layered (x_i -inputs, h_i -hidden layer and o -output layer) neuron model with i input parameters, j hidden nodes, k output nodes and one output parameter (p) is presented in Figure

2.3. Input values are transmitted to the hidden layer nodes, each of which sums up the received values from all input nodes, adds a “bias” to this sum and then passes them on through a nonlinear transfer function (Vieira *et al*, 2020b). The output layer nodes receive this information and operate identically to the hidden nodes. The resulting transformed output forms the network output (Deo *et al*, 2001). The sum of the contributions of all inputs to each hidden neuron (a_j) is defined as:

$$a_j = \sum_i x_i w_{i,j} + b_{1,j} \quad (2.8)$$

where x_i represent the input parameters, $w_{i,j}$ the weight matrix between input and hidden layers and $b_{1,j}$ the bias vector for the hidden layer. The values of a_j are then introduced into a sigma activation function expressed as:

$$f(a_j) = \frac{1}{1 + e^{-a_j}} \quad (2.9)$$

The values of the output layer can be defined as:

$$a_o = \sum_j f(a_j) w_{j,k} + b_{1,k} \quad (2.10)$$

where $w_{j,k}$ represents the weight matrix between hidden and output layers and $b_{1,k}$ the bias vector for the output layer.

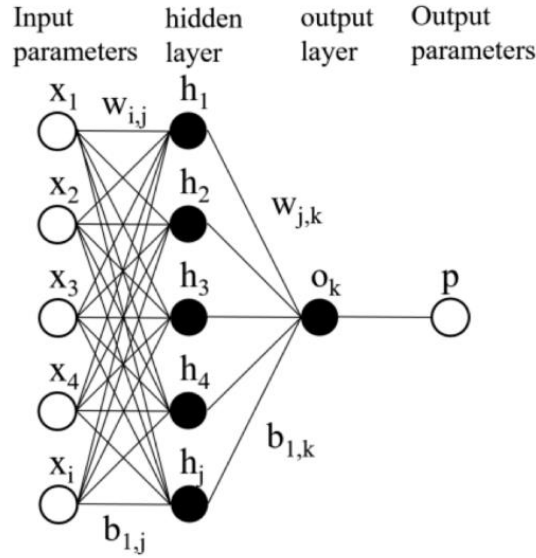


Figure 2.3 – Example of a two-layered ANN architecture (adapted from Vieira *et al*, 2020b).

The ANN learning procedure (Figure 2.4) consists of adjusting the weights and biases in each step as to minimize the errors between predictions (outputs, p) and measurements (or targets, t). This procedure is repeated until an error threshold or a maximum number of iterations is reached. The error function (e) is defined as:

$$e = \frac{1}{2} \sum_n \|p - t\|^2 \quad (2.11)$$

where n represents the number of measurements used to train the network, p the neural network predicted value or output and t the target value or measurement.

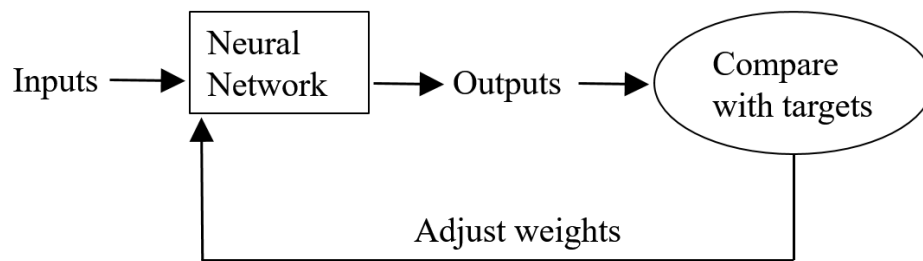


Figure 2.4 – Schematic of an ANN learning procedure (adapted from Vieira *et al* 2020b).

The training of the ANN was done using the backpropagation learning procedure of Levenberg-Marquardt (LM) algorithm (Levenberg, 1994; Marquardt, 1963) with Bayesian Regularization (BR, Mackay, 1992; Foresee and Hagan, 1997). LM was developed for faster convergence in backpropagation algorithms but is very sensitive to the initial network weights and does not consider outliers in the data that may lead to overfitting noise. Regularization is used to avoid these issues and improve generalization. BR has an objective function that includes a residual sum of squares and the sum of squared weights to minimize estimation errors and to achieve a good, generalized model (Titterton, 2004). Using this objective function causes the network to have smaller weights and biases, and these forces the network response to be smoother and less likely to overfit (Vieira *et al*, 2020b).

Several authors have applied ANN for calibration and improvement of performance for different types of numerical models for traffic simulations (Otkovic *et al*, 2013), atmospheric models (Perez and Bevilacqua, 2015), indoor building temperature (Lu *et al*, 2015), mechanical models (Mares *et al*, 2016), financial models (Liu *et al*, 2019) and near infrared spectroscopy (Chen and Wang, 2019). To the authors' knowledge, no such methods have been applied in wave-structure interaction models such as VARANS-VOF and therefore this is investigated in this study.

3

PHYSICAL MODELLING

3.1. INTRODUCTION

The experimental study performed within the scope of this thesis was carried out to produce data for investigating the damage progression and wave overtopping discharges for an armour layer composed of cubes in a regularly placed single-layer pattern, for a combination of incident wave conditions, water levels, and armour layer porosities. The test programme aimed at providing information for the following research questions:

- When do cubes start moving and by how much?
- How much do cubes move before the “*Initiation of Damage*” occurs?
- Is there an alternative descriptor or parameter useful in quantifying damage progression?
- What is the influence of the armour layer porosity on the hydraulic stability and mean overtopping discharges?
- What is the influence of the initial seed number, in the wave generation signal, on the mean overtopping discharges?

As seen in chapter 2, there is limited published information on the performance of SLC armoured breakwaters and therefore this study aims at contributing to reducing such knowledge gap.

Damage and overtopping discharges were simultaneously measured in most experimental tests. A reduced set of tests, for a relatively low N_s value (high armour stability), was specifically aimed at solely investigating wave overtopping. A summary of the physical modelling description was published in Vieira *et al* (2021a, 2021b, 2021c).

3.2. EXPERIMENTAL FACILITIES

The 2D laboratory experiments were conducted in the wave basin (28 m long, 12 m wide, and 1.2 m deep) of the Hydraulics Laboratory of the Hydraulics, Water Resources and Environment Division (SHRHA) of the Faculty of Engineering of the University of Porto, Portugal. It is equipped with HR Wallingford’s state-of-the-art wavemaker¹(Figure 3.1). The wavemaker allows for wave generation using piston-type wave paddles controlled by the HR DAQ software installed on computers located inside the control room (Figure 3.2). The wavemaker is operational for varying water depths up to 0.8 m. The wave generation system is prepared to produce regular and irregular sea-states, with a peak wave period ranging from 0.5 s to 5.0 s and wave heights up to 0.45 m depending on the water depth. The computer facilities for wave generator control, data acquisition, and data processing allow for direct control and computation of relevant wave characteristics. The wave generator is equipped with an active

¹ <http://equipit.hrwallingford.com/products/wave-generator/wavemaker-for-3d-basins>

wave absorption system. This means that the motion of the wave paddles can be modified in real-time to compensate for the reflected waves coming from the physical model preventing them to re-reflect towards the model.

A 1.6 m wide flume was constructed within the basin using guide walls supported by concrete blocks to avoid movement. An 8 mm thick acrylic glass sheet was installed between the tested models (each 0.8 m wide) from the crown wall area to approximately 2 m seaward of the toe of the structure to simultaneously test two different breakwater cross-sections with similar reflection characteristics (Figure 3.3).



Figure 3.1 – HR Wallingford wavemaker installed at the wave basin of the SHRHA – FEUP.

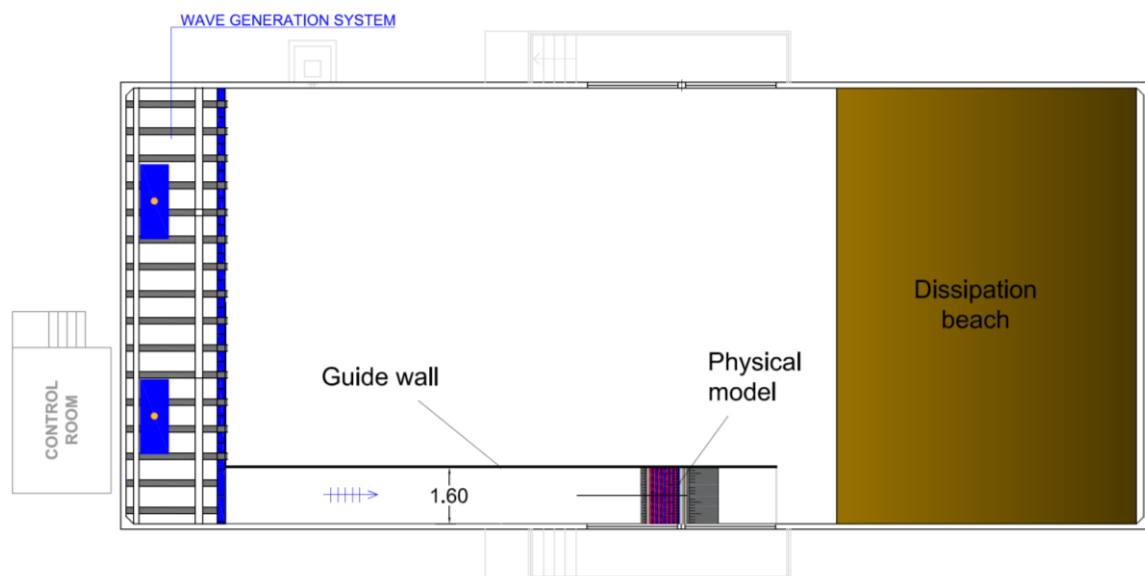


Figure 3.2 – Schematic layout of flumes inside SHRHA – FEUP wave basin.



Figure 3.3 – Overview of wave basin and model (view from wavemaker area).

3.3. SCALING

The geometric model dimensions were scaled based on the armour unit size and within the limitations of the wave basin dimensions and wave generation capabilities. A geometric scale, $\lambda_L=1:35$ is assumed for scaling purposes. Similarities according to Froude and Reynolds usually cannot be fulfilled simultaneously in a model. Since gravitational and inertial forces are dominant in the water motion of free surface waves, the physical model was scaled by preserving the similarity of the Froude number (Fr) between the model and prototype. The Froude number relates inertia with gravity forces and is described by:

$$Fr = \frac{u}{\sqrt{gL}} \quad (3.2)$$

where u represents a characteristic velocity and L a characteristic length. Scale factors can then be derived resulting in the ratios of model to prototype values presented in Table 3.1.

Table 3.1 – Scaling factors according to Froude's law

Factor	Scale	Scale Factor
Length (m)	λ_L	λ_L
Area (m ²)	λ_A	λ_L^2
Volume (m ³)	λ_V	λ_L^3
Mass (kg)	λ_G	λ_L^3
Time (s)	λ_T	$\lambda_L^{1/2}$
Velocity (m/s)	λ_U	$\lambda_L^{1/2}$
Overtopping discharge (L/s/m)	λ_q	$\lambda_L^{1.5}$

When setting up a physical model of a breakwater, the dimensions should fulfil the scaling criteria. These criteria yield (Hughes, 1993; Wolters *et al*, 2010):

- Overall structural dimensions are scaled geometrically;
- Flow hydrodynamics (waves) need to conform to the Froude similarity criterion;
- Turbulent flow conditions preferably to exist throughout the primary armour layer (satisfied reasonably by $Re_D > 30,000$, with the Reynolds number, Re_D , related to flow within the armour and defined as $Re_D = \frac{\sqrt{g \cdot H_S \cdot D_n}}{\nu}$, where ν represents the kinematic viscosity);
- It is better to operate at larger scales when possible since viscous forces can be greater if flow velocities and units are small.

Wolters *et al* (2010) recommend the following minimum modelling limits to minimize scale effects:

- Water depth: >5 cm;
- Wave height: >2-3 cm (design wave height >5 cm);
- Wave period: realistic wave steepness;
- Rock diameter: >3-5 mm;
- Rock armour: >25 mm.

3.4. MATERIALS

3.4.1. ARMOUR

The similarity in stability scaling is required and is achieved by ensuring that the stability number, N_s , is the same in the model and prototype. Differences in water density (saltwater in prototype and fresh water in model) and armour unit/rock density are accounted for in this parameter. The required armour mass and size for the model can be calculated by:

$$D_{n,m} = \frac{D_{n,p}}{\lambda_L} \cdot \frac{\Delta_p}{\Delta_m} \quad (3.3)$$

$$M_m = D_{n,m}^3 \cdot \rho_{a,m} \quad (3.4)$$

where $D_{n,p}$ is the nominal diameter of the armour in prototype, Δ_p is the relative density in prototype, Δ_m is the relative density in the model, M_m is the mass of the armour unit in the model, $D_{n,m}$ is the nominal diameter of the armour unit in the model and $\rho_{a,m}$ is the density of the armour unit in a prototype.

The cubes were produced using silicone moulds (Figure 3.5) which enabled the production of 80 cubes simultaneously. After drying, the cubes were weighted and, if needed, sanded to achieve the required mass (Figure 3.5). Due to differences in the concrete mixture between batches of cubes and deformations, not all cubes were identical. The cube units built for the structure armour have the following characteristics: $D_n=0.035$ m, $M=0.102$ kg, $\rho_c=2,382$ kg/m³ and $\Delta=1.38$. This corresponds to prototype dimensions of $D_{n-p}=1.26$ m and $M_p=4,824$ kg. Single layer armour with cubes resist mainly by gravity and lateral friction. Whilst gravity is well represented by physical models scaled according to Froude's law, this is not the case with lateral friction between blocks. All cubes were painted to make the identification of the damage areas easier (Figure 3.5). The cubes were numbered and weighted. 730 cubes were produced with an average weight of 102.1 g with a standard deviation of 1.5 g. The viscous scale effects were assumed negligible as Re_D varied between 29,900 and 41,100.

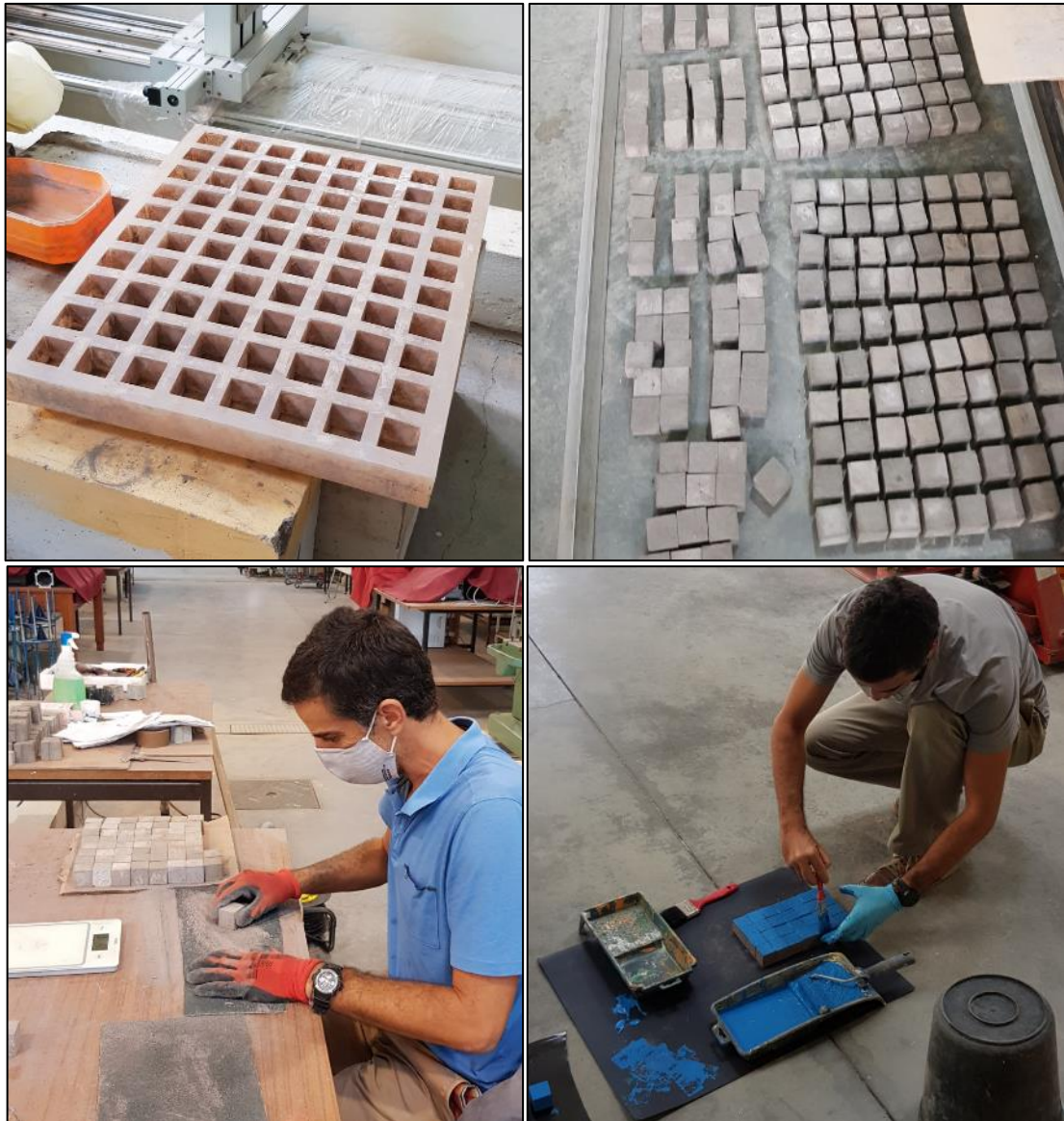


Figure 3.4 – Silicon mould for cube production (top left), cubes drying (top right), sanding (bottom left) and painting of cubes (bottom right).

Sensitivity to armour porosity was carried out by testing three different values (Table 3.2). These values are calculated to fit an exact number of cubes in each flume. Large rocks were placed in the gaps between cubes at the flume walls to limit the effects of the walls on stability. This area was not used for analysis.

Table 3.2 – Number of cubes per row for the various tests.

No. cubes per row	Gap width (cm)	Porosity
17	1.1	0.24
15	1.7	0.33
16	1.4	0.29

3.4.2. FILTER

The filter size was originally determined based on the scaling of standard rock grading. A standard rock grading of 3 to 10 kN was selected, which, at the geometric scale of 1:35, corresponds to a $D_{n50,f}=1.80$ cm and $M_{50,f}=15.0$ g (for a rock density of 2650 kg/m³). However, various sizes of material were sieved (Figure 3.6) and the final characteristics achieved were: $D_{n50,f}=1.46$ cm and $M_{50,f}=8.2$ g. This resulted in $M_c/M_{50,f}=12.4$ and $D_n/D_{n50,f}=2.4$.

3.4.3. CORE

The core material size was initially determined based on the scaling of a typical rock grading of 0.1 to 5 kN, corresponding to a $D_{n50,c}=1.02$ cm and $M_{50,c}=2.8$ g. The final characteristics for the core are $D_{n50,c}=1.13$ cm and $M_{50,c}=3.8$ g. The characteristics of all materials are summarized in Table 3.3.

Table 3.3 – Characteristics of the materials used in the breakwater.

Layer	D_n or D_{n50} (cm)	ρ_c (kg/m ³)	M_{50} (g)	D_{85}/D_{15}	D_{85} (cm)
Armour (cubes)	3.50	2380	102.1	-	-
Filter	1.46	2650	8.2	1.4	1.94
Core	1.13	2650	3.8	1.7	1.61

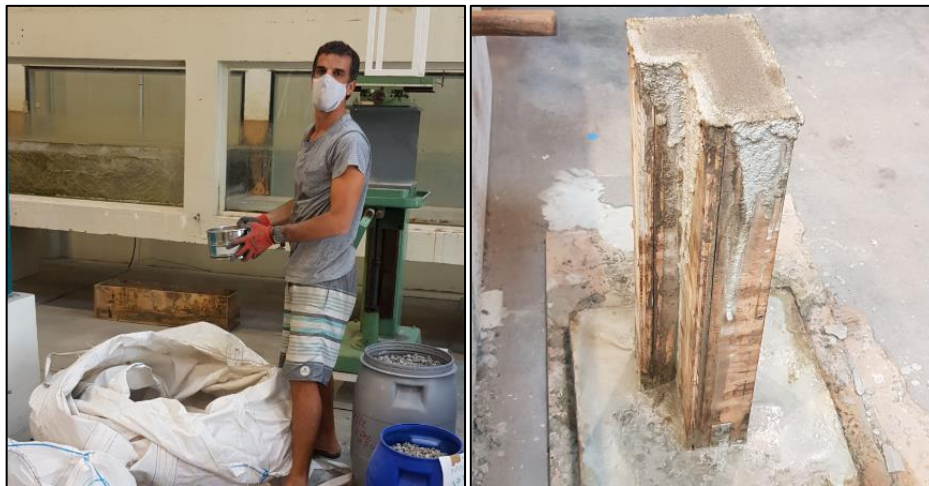


Figure 3.5 – Sieving the core and filter material (left) and production of the crown wall (right).

3.4.4. TOE

The toe consisted initially of three rows of cubes. However, after the first series of tests, it was observed that this configuration was unstable and therefore it was replaced by large blocks approximately $2D_n$ long by $2D_n$ wide by $1D_n$ high, which were already available in the laboratory.

3.4.5. CROWN WALL

The crown wall was produced using wooden moulds filled with concrete (Figure 3.6). Each section of the crown wall was 0.4 m wide (2 pieces per flume).

3.5. BATHYMETRY

A foreshore with a 4.8% slope transition between the flat seabed at the wavemaker and model areas was constructed to raise the structure and limit the volume of material required. After the initial 4.27 m of flat seabed in front of the wavemaker, required to properly generate the waves, the slope extended for 5.00 m, followed by a 2.24 m wide flat area in front of the model (Figure 3.7).

The model bathymetry was constructed with wood templates placed on the bottom of the flume. The templates were backfilled with a combination of bricks, blocks, and gravel, to a level of approximately 2 cm below the top of the templates. The 2 cm gap was filled with a skin of concrete grout to match the seabed profile defined by the templates (Figure 3.8).

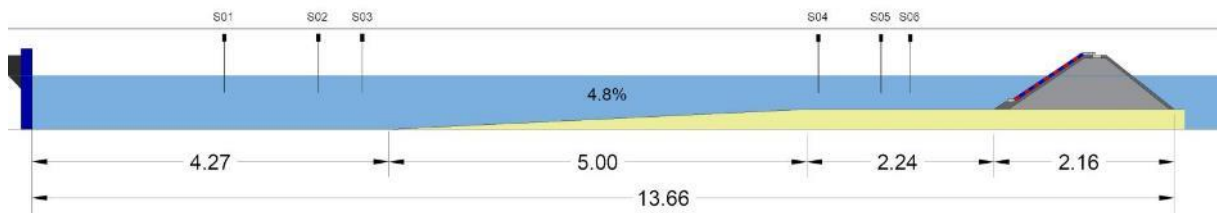


Figure 3.6 – Schematic of flume bathymetry.



Figure 3.7 – Seabed construction sequence: Backfilling of the model area with bricks and blocks, and slope area with gravel (left); backfilling with gravel in the model area (middle); final step with concrete grout over backfilling (right).

3.6. CROSS-SECTION

A breakwater cross-section with a slope of 1V:1.5H was built based on the findings of the previous studies. This slope was shown to be more stable and requires less volume of material. The water depth varied between 0.64 m and 0.74 m at the wavemaker, and between 0.40 m and 0.50 m at the toe of the structure. The original structure (series A) had a total height of 0.63 m. After the initial series, it appeared that the measured mean overtopping discharges and hydraulic stability of the armour layer were

exceeding the initial expectations (higher overtopping discharges and hydraulic stability compared to results in the literature and empirical formulations). It was therefore decided to increase the structure height to 0.68 m, whilst maintaining the water depth and wave conditions, to eventually reduce overtopping and stability (Figure 3.9).

The crest armour had a width of $3D_n$ backed by the crown wall. The filter layer consisted of rock with a thickness of $2D_{n50}$. Filter rock (with a width $3D_n$) was initially used for the toe in series A, after which it was replaced by toe blocks.

The core and filter materials were bulk-placed by shovel and hand, respectively, and then gently hand-packed to match the desired profile using wooden trowels. The cube armour units were individually placed by hand, to simulate prototype techniques of placement by crane/excavator and to achieve an appropriate amount of interlocking with neighbouring units (Figure 3.10).

During the initial tests with larger overtopping, it was observed damage to the rear side of the structure that could compromise the stability of the crest armour and crown wall. Therefore, this area was strengthened with the installation of large rocks.

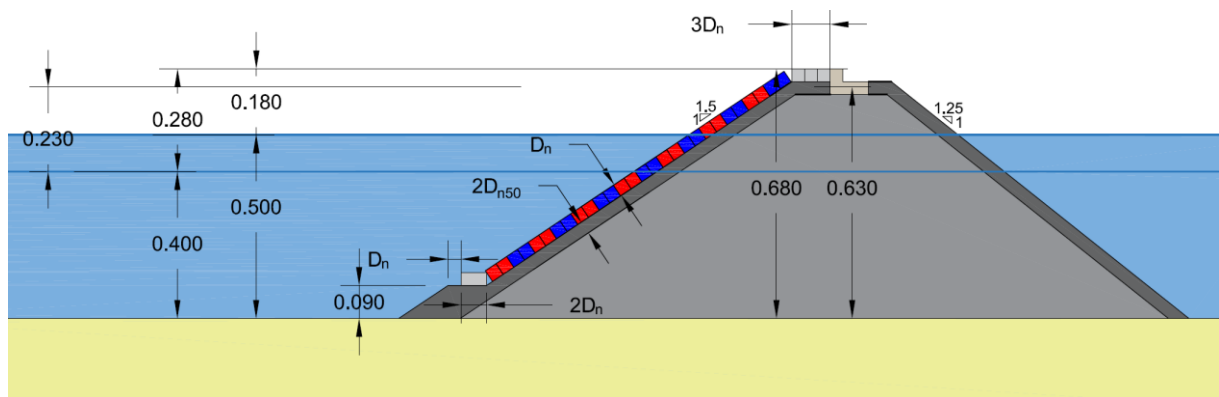


Figure 3.8 – Breakwater cross-section.

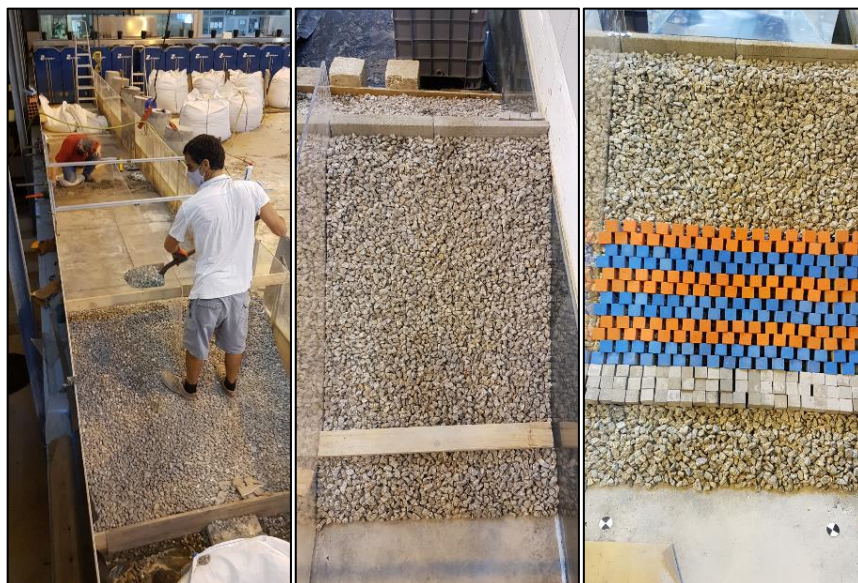


Figure 3.9 – Breakwater construction sequence: Bulk placement of the core material with a shovel (left); filter placement completed (middle); placement of cube armour units (right).

3.7. MEASUREMENTS

3.7.1. SURFACE ELEVATION

A group of four resistive type wave probes (accuracy of 0.4 mm) was placed in the deeper area, near the wavemaker, and another two groups of three probes near each of the breakwater sections (Figure 3.12 and Figure 3.13). Probe distances varied according to the incident wave conditions to capture the range of frequencies between 0.5 to 2.0 f_p ($f_p=1/T_p$). During the experiments, one of the deep-water probes stopped logging and the probe spacing in this area was adjusted accordingly. Water surface elevations were recorded at a frequency of 40 Hz and analysed with HR Wallingford's HR DAQ software². Incident and reflected waves were separated using the method of Mansard and Funke (1980), which uses simultaneous records of the free surface elevations measured by three aligned wave gauges.

Calibration of each wave gauge was carried out daily before each test. Due to restrictions in water depth and probe support structure, the calibration was carried out outside the flume to ensure an adequate range of measurements during the tests.

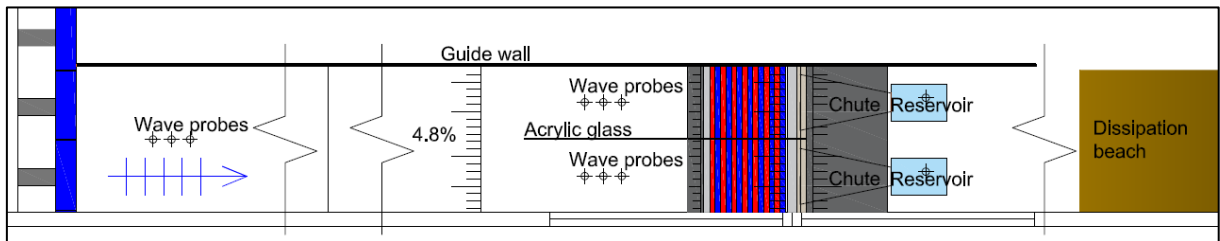


Figure 3.10 – Location of instrumentation in the wave flume.

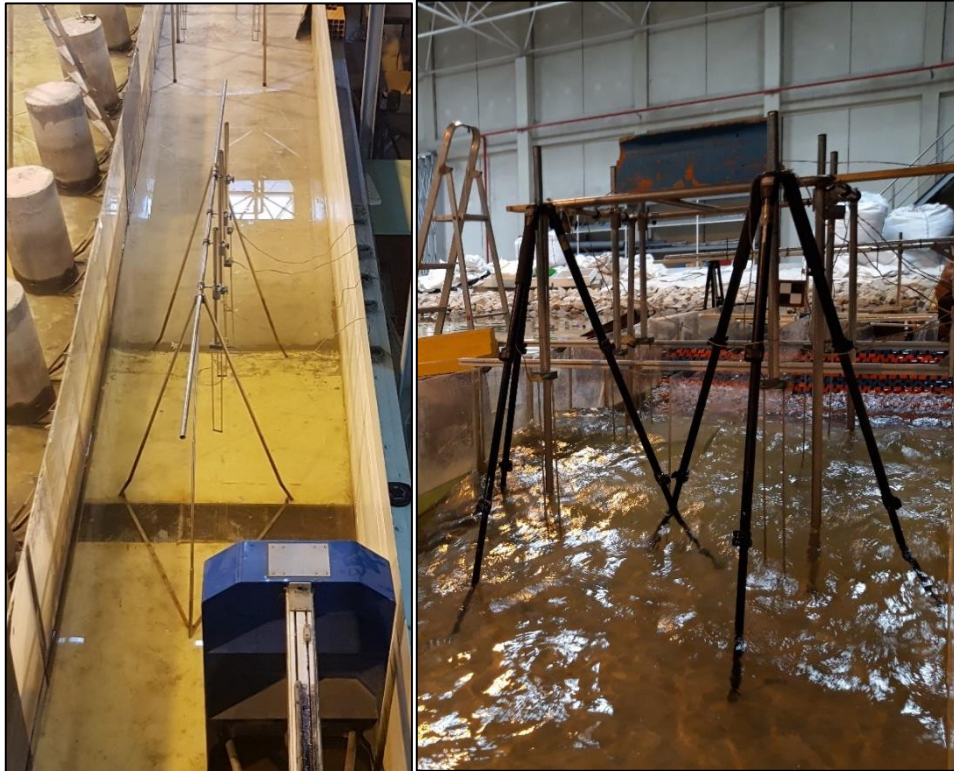


Figure 3.11 – Overview of wave probes at the wavemaker (left) and model (right) areas.

² <http://equipit.hrwallingford.com/products/software/hr-daq-software>

3.7.2. OVERTOPPING

Water was collected via a 0.6 m wide overtopping chute sloping down from the top of the crown wall into collecting reservoirs (Figure 3.14). The overtopping discharge was measured by a water surface elevation probe placed inside a perforated PVC pipe within the reservoir to minimize oscillations in the measurements. When needed, a pump, installed inside the reservoirs, was activated to empty the reservoirs during the tests, as in Cabral *et al* (2020). Two reservoir sizes were used to improve the accuracy of the measurements. For lower wave height conditions, a smaller reservoir with an area of 36 cm by 26 cm (936 cm²) was used, whilst for higher overtopping discharges the area of the large reservoirs was 94 cm by 54 cm (5,076 cm²). Concrete blocks and other weights were placed inside the reservoirs to resist buoyancy.



Figure 3.12 – Overview of reservoirs (left) and model with overtopping tray and reservoirs.

3.7.3. STABILITY

A photo camera (Canon EOS Rebel T5i) was used to capture photographs from a platform placed near the model after each test. The Faro® Focus laser scanner S70 was also used to obtain detailed information for damage assessment. The laser scanner was supported on a tripod, which was placed inside the flume after each test. The flume was drained before taking the photographs and laser scans to enable a correct acquisition of the entire armour layer. Scans were done for each flume at a resolution of 1.5 mm at 10 m, which results in an approximate resolution of 0.3 mm since the scanner was less than 2.0 m away from the structure. Each scan took around 10 minutes. Markers were placed at various locations on the structure toe and crest, and additional markers supported at a fixed position above the structure to enable correct image registration (alignment) for the various scans (Figure 3.15).

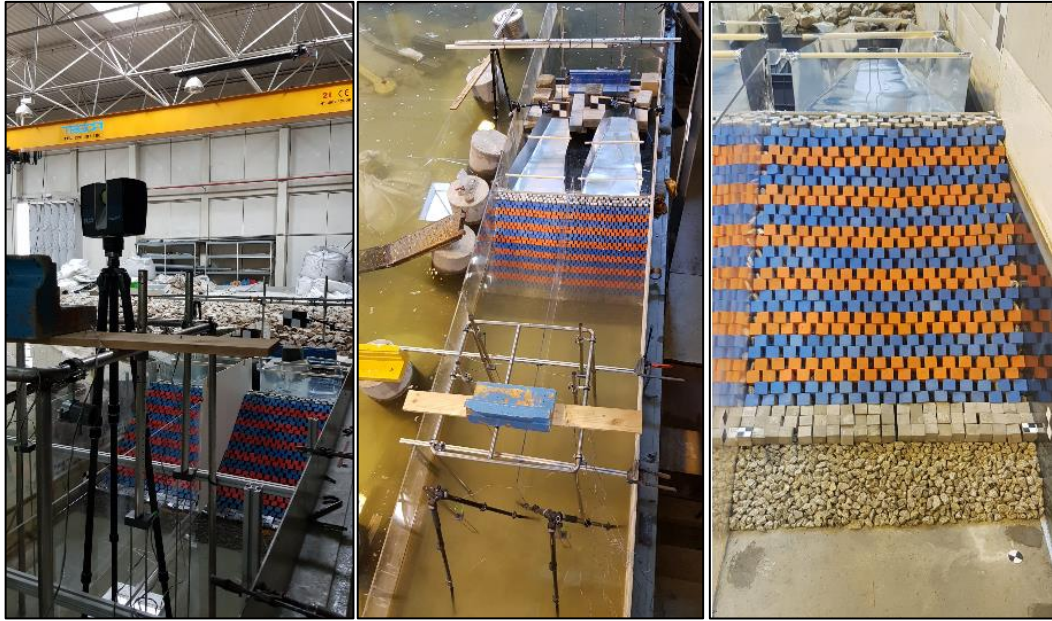


Figure 3.13 – Overview of Faro® Focus during a scan (left), fixed markers above the structure (middle) and markers on the structure (right).

3.8. TEST PROGRAMME

Three test series of eight tests for two simultaneous armour porosity values were carried out with the main focus being to measure armour stability but also capturing overtopping measurements. Three additional series of three tests each for two simultaneous armour porosity values were carried out to only measuring wave overtopping. A constant wave steepness (0.02 for series A, B, P and Q and 0.05 for series C and O) in the wave generation zone, s_{op} , was adopted with the significant wave height progressively increasing within each test series. Irregular waves defined by a JONSWAP spectrum with a peak enhancement factor $\gamma=3.3$ were used in all tests. These waves were generated by the filtered white noise technique for a sequence length of $2^{13}-1$ pulses. The same temporal sequence of waves was used in the tests having the same peak wave period and seed number. In test series Q, with similar incident wave characteristics and water level to P, a different initial seed number was imposed. The test's duration varied between 14 min and 36 min, corresponding to approximately 1,000 waves for each test. Two water depths were adopted, 0.74 m and 0.64 m at the wave generation zone, corresponding to 0.50 m and 0.40 m at the model area, respectively. The range of parameters of the test series is presented in Table 3.4.

Table 3.4 – Range of parameter values at the toe of the structure for each test series.

Series	p	s_{op}	H_s (cm)	T_P (s)	h (m)	h_t/H_s (-)	R_c (m)	No. tests	Purpose
A	0.24 0.29	0.02	7.5-17.5	1.7-2.4	0.40	2.3-5.3	0.23	16	Stability Overtopping
B	0.29 0.33	0.02	9.2-18.2	1.7-2.4	0.40	2.3-4.3	0.28	16	Stability Overtopping
C	0.29 0.33	0.05	7.4-16.7	1.1-1.6	0.40	2.4-5.4	0.28	16	Stability Overtopping
O	0.29 0.33	0.05	7.4-11.7	1.1-1.4	0.50	4.3-6.8	0.18	6	Overtopping
P	0.29 0.33	0.02	7.4-12.4	1.5-2.0	0.50	4.0-6.8	0.18	6	Overtopping
Q	0.29 0.33	0.02	7.4-12.6	1.5-2.0	0.50	4.0-6.8	0.18	6	Overtopping

3.9. TEST PROCEDURE

The test procedure consisted of several steps listed below:

1. Fill the wave basin up to the correct level using two separate pumps;
2. Calibrate wave probes outside the flume;
3. Position wave probes in the flume according to the test conditions;
4. Initiate wave paddle;
5. Prepare wave generation conditions;
6. Prepare wave probe acquisition system;
7. Set wave probes to zero;
8. Start wave generation and acquisition;
9. Run test;
10. Empty wave basin using three pumps;
11. Post-process wave probe data;
12. Take photographs of the model;
13. Switch off laboratory lights and carry out laser scan for each flume;
14. Post-process laser scan data;
15. If time was available, return to 1., otherwise, switch off systems;
16. Fill in the daily log.

Typically, one to two tests were carried out each day. The armour layer was not repaired between tests, only after completion of a test series (cumulative damage).

4

HYDRAULIC STABILITY

4.1. INTRODUCTION

The results obtained from the experimental study described in chapter 3 were used for a detailed investigation of the damage evolution on the armour layer of a rubble-mound breakwater composed of regularly placed cubes. The main objective of this chapter is to quantitatively describe the armour damage progression in SLC armoured breakwater. As mentioned previously in chapter 2, the SLC armour in a regularly placed pattern is a very particular type of arrangement for which the current available methods of damage quantification are not applicable. Therefore, in this section, an alternative damage level classification and criteria for initiation of damage in SLC armoured breakwaters are developed and presented. This work was published in Vieira *et al* (2021b) and a summary of the main methodologies applied, and findings is presented in this chapter.

4.2. METHODOLOGY FOR AUTOMATIC CUBE DETECTION

When analysing images or video material in a coastal engineering and physical modelling context, the question often is related to the identification of the position of water, sand, or the structure within the image, which changes over time. These questions can be answered with image segmentation (Den Bieman *et al*, 2020). In this research work cube positions were automatically detected in images using the “Trainable Weka Segmentation” plugin (Arganda-Carreras *et al*, 2017) in Fiji (Schindelin *et al*, 2012). Fiji, a distribution of the open-source software ImageJ, is a powerful image processing software focused on microscopy biological-image applications. The Trainable Weka Segmentation (TWS) is a machine learning tool that leverages a limited number of manual annotations to train a classifier and segment the remaining data automatically (Arganda-Carreras *et al*, 2017). The basic part of the TWS, as described in Arganda-Carreras *et al* (2017), is a classifier, which can be trained by a variety of algorithms, filter methods and other training features for binary classification of pixels. TWS classifies the images’ pixels into aggregate (or object of interest, i.e., the cubes) and background, based on pixel characteristics. The accuracy of the method is therefore associated with the image resolution, which in this case is 2000 pixels/m, or 0.5 pixels/mm. Once the training is done, the training image and its classified regions are displayed to check the quality of the segmentation. In case the segmented and real aggregate contours are matching well, the determined classifier is used for batch processing of the whole image sample. If the segmentation needs improvement, additional or different classification regions in the training image are chosen and the training is carried out again (Altenhoff *et al*, 2020).

Images of the armour slope for tests in a series were stacked using Fiji to improve efficiency in the analysis process. Images for some of the tests with lower stability number, where less movements are expected, were unavailable. The images are then imported to the TWS plugin for training and analysis. Approximately 30 areas with pixels containing cubes are selected as “Cubes” and another 30 areas (without cubes) as “Background”. The TWS plugin classifies all images based on the training data so that cubes are isolated from the background. Cube edge and associated parameters (area, perimeter and

center) are calculated using the Fiji plugin “Analyze Particles” (Figure 4.1). ImageJ “Analyze Particles” was successfully applied in Cuthbertson *et al* (2010) for post-processing of captured sequential images to identify individual sediment flocs and determine their geometric characteristics. The “Analyze Particles” command counts and measures objects in binary or thresholded images. It works by scanning the image until it finds the edge of an object. It then performs the following actions: outlines the object, measures it, fills it to make it invisible, then resumes scanning until it reaches the end of the image. Final verification of the cube positions is carried out to ensure that all cubes were correctly detected, and any missing cubes are included.

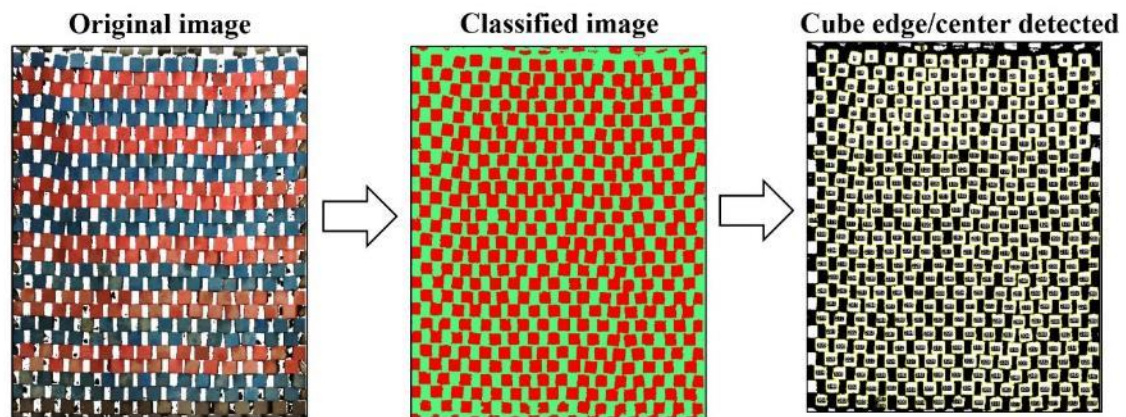


Figure 4.1 – Example of Fiji/TWS flowchart for automatic detection of individual cube positions in the breakwater armour layer.

4.3. DAMAGE ASSESSMENT

4.3.1. CUBE MOVEMENT ANALYSIS

The data generated from the method described in section 4.2 enabled the mapping of each cube position in all tests, which is subsequently used to analyse cube movements in more detail as described hereafter. An example of the cube movement for test series B ($n=0.29$) is presented in Figure 4.2. Exceedance curves for cube movements (Figure 4.3) indicate that for lower stability numbers up to $N_s=2.6-2.7$, the magnitude of movements for all series is limited, with the majority of the cubes being moved up to a distance of $D_n/7$ (5 mm). For larger stability numbers ($N_s > 3.1$) in series A, the test with lower porosity (0.24) results in a larger magnitude of movements compared to a porosity of 0.29, indicating lower hydraulic stability. In test series B, the higher porosity (0.33) causes a larger number with a larger amplitude of movements when compared to the lower porosity (0.29) and all the other series. These findings, in terms of the influence of the armour porosity on stability, are in agreement with previous studies (Van Buchem, 2009). The behaviour of series C for both porosities is similar, which suggests that for a higher wave steepness ($s_{op}=0.05$) the influence on the overall stability is relatively small. For a porosity of 0.29, series A (with a lower armour crest level) is more stable than B and C, suggesting the influence of the relative freeboard on the armour hydraulic stability. This effect was investigated in previous studies (Vidal *et al*, 1992; Van der Meer and Daemen, 1994; De Jong, 1996; Burchart *et al*, 2006) where generally a lower crest resulted in increased armour layer stability for the seaward slope because of higher wave energy transmission due to overtopping. For $N_s=3.5$, in series C ($n=0.29$) there were ~80% of cubes moved by more than $D_n/7$ (5mm) whilst for series B ($n=0.29$) only ~40% of the cubes moved more than $D_n/7$. However, in both series, 5% of the cubes moved by more than $D_n/4$ (8.75 mm). Initiation of Damage in SLC (IDa_{SLC}), based on the visual assessment during the tests when cubes

loose support from the cubes in the rows below, occur when $\sim 5\%$ of the cubes move more than $D_n/4$ (shown as a black circle in Figure 4.3). It was not possible to reach the threshold for IDa_{SLC} in Series A due to limitations in generating larger waves associated with the model dimensions chosen, the wave generator capacity and water depth at the structure. Initiation of Damage in SLC (IDe_{SLC}) was not reached during the experiments for the same reasons.

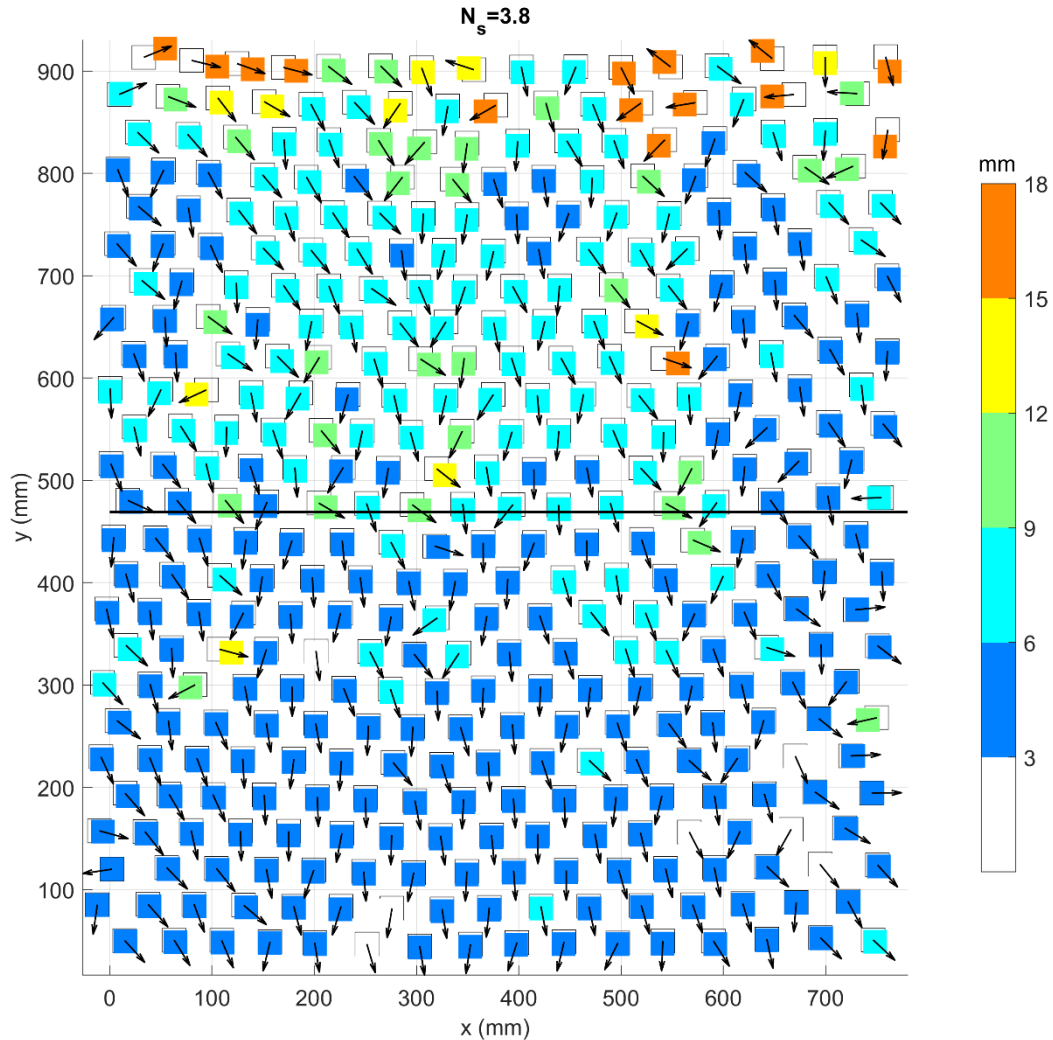


Figure 4.2 – Example of the evolution of cube movements (mm) for series B ($n=0.29$) for $N_s=3.8$. The white squares represent the cube positions at the start of the test series. The black arrows show the direction of cube movement.

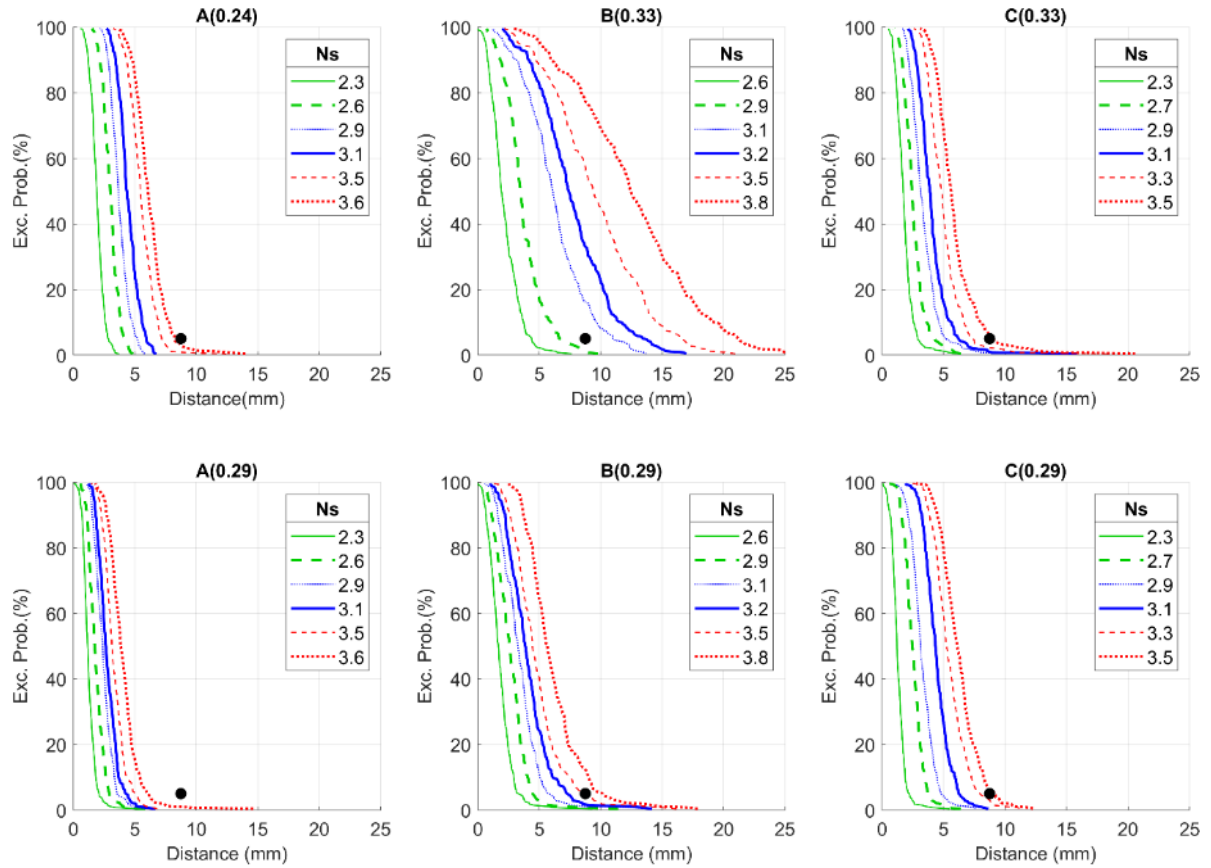


Figure 4.3 – Exceedance probability of cube movements (mm) for all tests.

4.3.2. MEAN GRID ORTHOGONALITY

Mean grid orthogonality for all tests was calculated based on the cube relative positions and is depicted in Figure 4.4. It is difficult to construct a perfectly regular armour grid ($MGO_0=0$, where MGO_0 represents the as-built mean grid orthogonality or MGO for $N_s=0$ in Figure 4.4 - top) in the laboratory, due to several factors such as the irregularity of the filter layer and manual placement of the cubes. However, it is also difficult to achieve this in a prototype due to constructability issues. The damage evolution in Figure 4.4 (bottom) takes into consideration the different MGO_0 values for each series by subtracting MGO_0 from MGO . This parameter provides a better descriptor for comparing the various test series, since there were small differences in MGO_0 values (12% between the smallest and largest). The results for series A are relatively constant, indicating that the cube positions remained stable from their original (as-built) locations up to around $N_s=3.1$ (for a porosity of 0.24) and $N_s=3.5$ (for a porosity of 0.29). For series B, the results for an armour porosity of 0.33 show that the mean grid orthogonality increases more than for the other series, in particular for higher values of N_s (>3.0). The lines for both porosities in series C have a similar trend, although the results for $n=0.33$ are generally slightly higher than those for $n=0.29$. This suggests that there is limited influence of the armour porosity in hydraulic stability for higher wave steepnesses. When comparing series A and B with a porosity of 0.29, it can be observed that, despite a similar initial mean grid orthogonality (Figure 4.4 - top), series B results in a more irregular grid as N_s increases. This indicates that the relative freeboard has an influence on the armour layer regularity and subsequently, stability, as was previously observed in Figure 4.3. The proposed threshold for IDa is a mean grid orthogonality of approximately 0.08.

IDa_{SLC} , based on the visual assessment during the tests, occurs approximately for $MGO - MGO_0 > 0.007$. As mentioned previously, it was not possible to reach IDa_{SLC} for Series A and IDe_{SLC} for any series. However, it is not essential to establish a threshold for this stage of damage since, as proposed and explained next in section 4.3.3, the relative damage number describes better the IDe_{SLC} .

The quality of the initial armour grid placement (within the tested range) appears not to have a strong impact on the damage progress. For series A and C, the lines between each of the two porosities within the same series are roughly parallel. Similar behaviour is observed for series B until approximately $N_s=3.0$, where the higher porosity case then diverges upwards (less regular grid). Further investigation is required for a more detailed evaluation of this aspect on the armour stability.

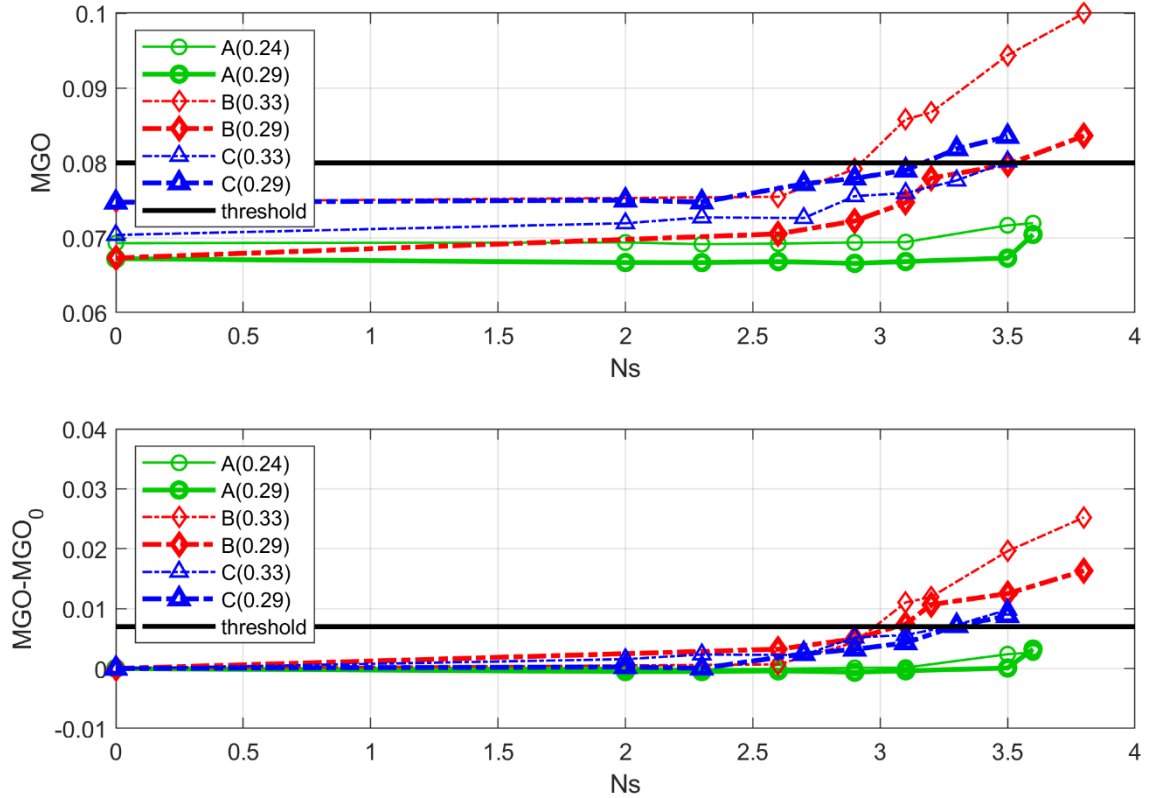


Figure 4.4 – Evolution of mean grid orthogonality with stability number for all tests. MGO (top) and $MGO - MGO_0$ (bottom).

4.3.3. RELATIVE DAMAGE NUMBER FOR SLC

The classic definition of N_{od} is not adequate for providing quantitative assessments of damage in SLC armoured breakwaters since the extraction of one unit from the armour layer, which could correspond to a very low N_{od} , results in exposure of the filter layer (Initiation of Destruction or “failure”), unlike in double-layer structures (Vieira *et al*, 2020c). The experimental test results did not show any cube being moved by a distance of D_n or extracted, resulting in a N_{od} of zero for all tests, even for relatively high values of N_s . However, it is observed in Figure 4.5, for series B ($n=0.33$), that there were substantial changes to the armour layer, with some cubes falling into the lower rows (for example on the second and third rows of blue cubes counting from the top) and others losing support from the cubes in the row below, that compromise the overall stability and therefore it is considered damage. Initiation of damage has occurred for the structure in this example, but it is still providing protection. IDe_{SLC} (defined here as

exposure of the filter layer) is eventually expected to occur if the wave conditions continue for a longer time or the wave conditions become more energetic.

An alternative relative damage number, N_{od-SLC} is proposed where cubes are considered “displaced” if they move over more than $D_n/4$ (Figure 4.6). The proposed threshold for IDa_{SLC} is $N_{od-SLC}=0.1$. This corresponds approximately to the visual assessment of the armour layer in the tests and is generally consistent with the assessment of the parameters presented in 4.3.1 and 4.3.2. It is also proposed to adopt $N_{od}>0$ (where cubes are considered “displaced” if they move by more than D_n) as a criterion for IDe_{SLC} . $N_{od} >0$ indicates that at least one cube has moved significantly (or been extracted), thus leaving a gap where the filter is exposed. Such gap will likely result in loss of support to the upper rows and subsequent collapse of the armour layer.

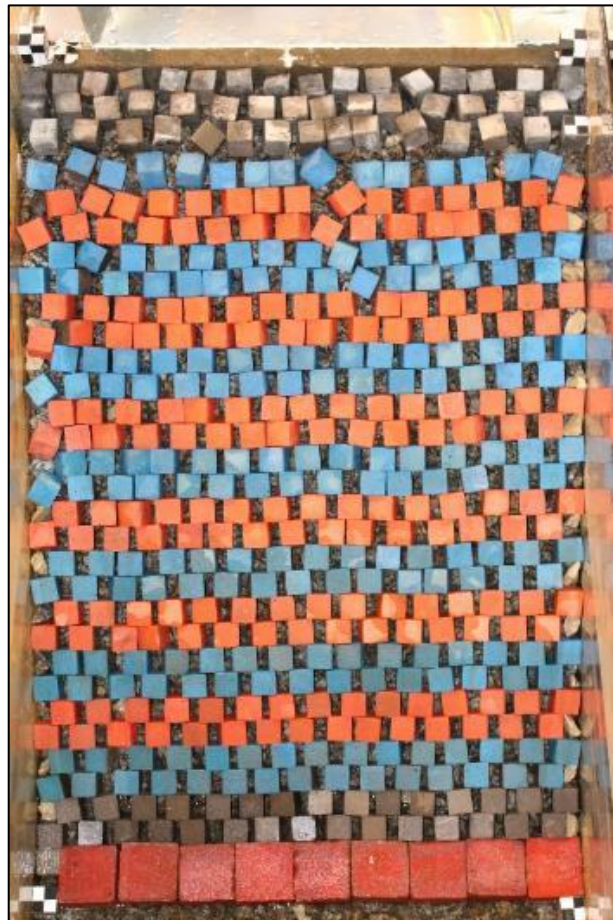
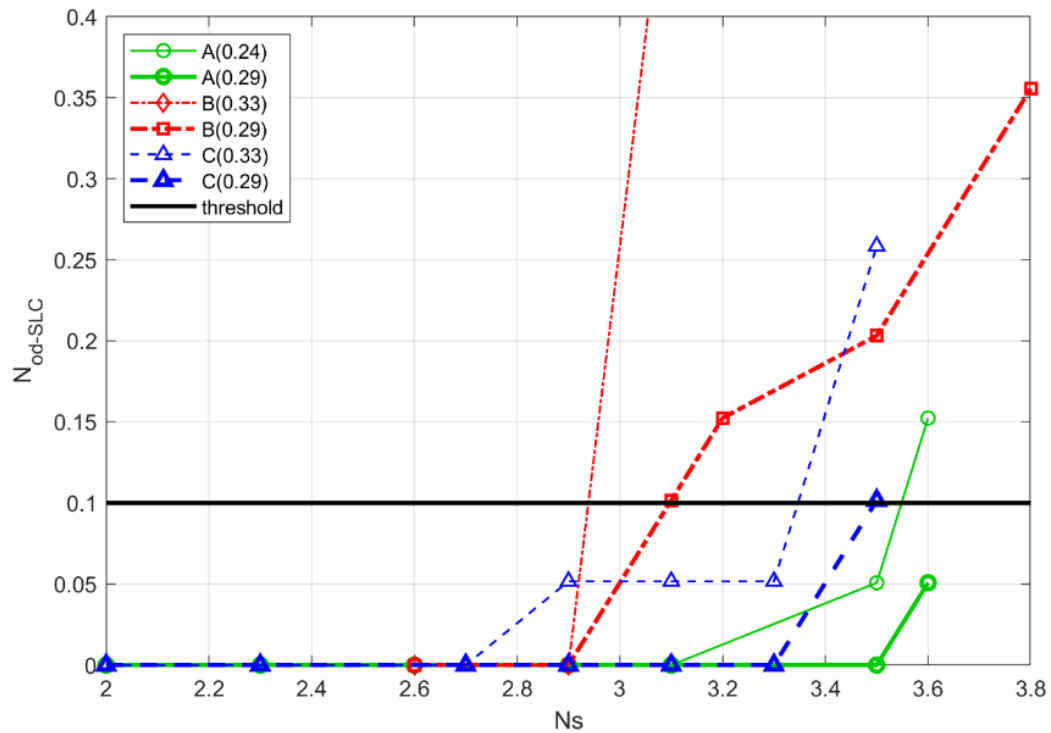


Figure 4.5 – Photo after test B ($n=0.33$) for $N_s=3.5$.


 Figure 4.6 – N_{od-SLC} for all tests.

4.4. DAMAGE EVOLUTION CLASSIFICATION AND CRITERIA

A comparison of the stability numbers for IDa_{SLC} based on the different parameters and associated criteria is presented in Table 4.1. The results are relatively consistent between the various parameters. For series A (lower armour crest) the stability number for IDa_{SLC} is higher or equal to 3.6. For series B, the range of N_s for the higher porosity ($n=0.33$) is between 2.9 and 3.1, whilst for the lower porosity ($n=0.29$) is between 3.1 and 3.5. In series C there is not a significant difference between porosities and the N_s for IDa_{SLC} is in the range of 3.2 to 3.5. Based on the literature (Van Gent *et al*, 1999; 2001; Van Buchem, 2009; Almeida, 2013) and this study, it is likely that a structure will be built with porosities around 0.29. Therefore, the range of N_s to consider in preliminary designs for this porosity, considering the different wave steepnesses and crest freeboards, is from 3.1 to more than 3.6. The lower limit is likely to be adopted for structures with higher crest freeboard and the upper limit for lower structures.

 Table 4.1 – Stability number for IDa_{SLC} based on the proposed criteria for various parameters.

Parameter	Exc. prob. movement	Mean orthogonality	N_{od-SLC}	Range
Criteria	$5\% > D_n/4$	$MGO - MGO_0 > 0.007$	> 0.1	-
Series A (0.24)	3.6	> 3.6	3.6	≥ 3.6
A (0.29)	> 3.6	> 3.6	> 3.6	> 3.6
B (0.33)	2.9-3.1	3.0	2.9	2.9-3.1
B (0.29)	3.5	3.1	3.1	3.1-3.5
C (0.33)	3.3-3.5	3.3	3.3	3.3-3.5
C (0.29)	3.3-3.5	3.3	3.5	3.2-3.5

5

WAVE OVERTOPPING

5.1. INTRODUCTION

The results obtained from the experimental study described in chapter 3 were also used for a detailed investigation of the wave overtopping in SLC armoured breakwaters. The main objectives of this chapter are to evaluate the adequacy of the current methods for estimating mean overtopping discharges and quantify the influence of the armour layer porosity and initial seed number on mean overtopping discharges. The work presented in this chapter aimed at complementing the limited laboratory test information available in the literature for this specific armour layer type. This work was published in Vieira *et al* (2021a) and a summary of the main methodologies applied, and findings is presented in this chapter.

5.2. PERFORMANCE OF EXISTING METHODS

The non-dimensional mean overtopping discharges from FEUP (chapter 3) and CLASH (Pearson *et al*, 2004) tests were compared against the VDMJ94 empirical formulation, with a roughness factor, $\gamma_f=0.52$, following the estimates from Bruce *et al* (2009) and Molines and Medina (2015), EurOtop (2018) and the Artificial Neural Network (Zanuttigh *et al*, 2016), ANN₁₆ (Figure 5.1). The FEUP overtopping discharges are generally under-estimated by VDMJ94 and EurOtop (2018) for higher values of q^* ($q^*>10^{-5}$). The ANN provides better estimates for $q^*>10^{-5}$ but over-estimates the overtopping discharges for lower values. Larger scatter in measurements from available databases typically occurs for lower values of q^* . It is suggested that the differences observed could be related to the configuration and range of parameters used in FEUP tests being beyond the range of tests used for training of the ANN. The current dataset can therefore contribute to improving the accuracy of future versions of the ANN, since the amount of data relative to SLC would be more than the double.

5.3. NEW ESTIMATOR FOR SLC

Several FEUP test conditions included various non-dimensional parameters, which are outside the range of the CLASH tests. Therefore, a combination of parameters can potentially justify the differences in the results between CLASH and FEUP (FEUP measurements are typically higher than those of CLASH). The relative importance of each non-dimensional parameter (Rc/H_{m0} , Gc/H_{m0} , h/H_{m0} and $L_{m-1,0}/Gc$) in the prediction of mean overtopping discharges was analysed using the Partial Least Squares (PLS) regression method (Wold *et al*, 1983). Two parameters, Rc/H_{m0} and $L_{m-1,0}/Gc$, were identified by the PLS method and, subsequently, used to derive an alternative wave overtopping formula for single-layer cube armoured breakwaters.

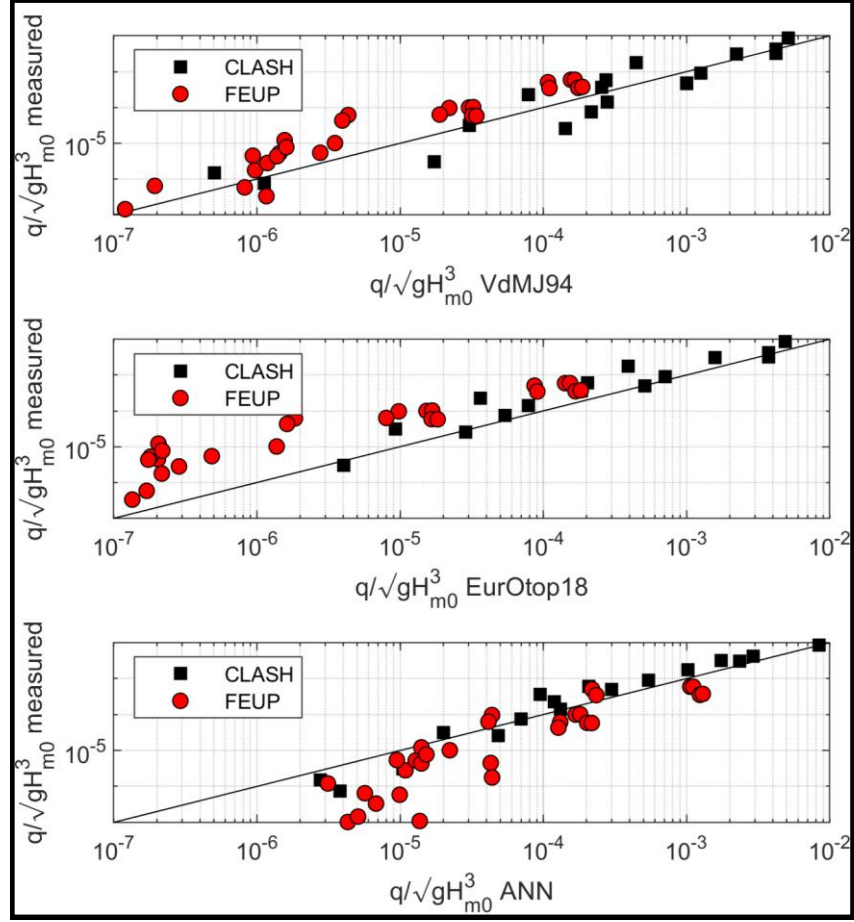


Figure 5.1 – Comparison between non-dimensional mean overtopping discharge from experiments (FEUP and CLASH), VDMJ94 (top), EurOtop (middle) and ANN (bottom).

The proposed new formula for estimation of the mean overtopping discharges for non-breaking, single-layer cube armoured breakwaters (q_{SLC}^*) is:

$$q_{SLC}^* = \frac{q}{\sqrt{gH_{m0}^3}} = c_1 \cdot \exp \left(-c_2 \frac{R_c}{H_{m0} \gamma_f^3} \frac{1}{\sqrt{L_{m-1,0}/G_c}} \right) \quad (5.1)$$

The proposed parameters c_1 and c_2 in equation (5.1) were estimated using a linear best-fit analysis, which led to values of 0.20 and 6.45, respectively. The roughness coefficient used in the equation is the same as derived by Bruce *et al* (2009) and Molines and Medina (2015), $\gamma_f = 0.52$. The error metrics used to evaluate the performance of the alternative prediction method, the normalized mean square error ($nMSE$) and the correlation coefficient (r), are defined as:

$$nMSE = \frac{\sum_{i=1}^n (y_i - x_i)^2}{Var(x)} \quad (5.2)$$

$$r = \frac{\sum_i ((x_i - \bar{x})(y_i - \bar{y}))}{\sqrt{\sum_i (x_i - \bar{x})^2 \sum_i (y_i - \bar{y})^2}} \quad (5.3)$$

where x and y represent the measured and modelled values, respectively, defined by $\log(q^*)$, where $q^* = q/(gH_{m0}^3)^{0.5}$, i is the data index and n is the number of samples. Figure 5.2 and Table 5.1 show that the proposed method can represent better the mean overtopping discharges for the entire range of values

from the laboratory experiments and shows less scatter around the line of best fit. The same r is obtained for all methods but the $nMSE$ is smaller for the proposed formula. It is important to highlight that the proposed method for SLC has currently limited applicability as it was derived from a reduced dataset when compared to VDMJ94, EurOtop and ANN16, which can be widely applied.

Table 5.1 – Error metrics for the various overtopping prediction methods.

Formula	$nMSE$ (%)	r
VDMJ94	23.7	0.92
EurOtop	79.1	0.92
ANN16	26.7	0.92
SLC (this study)	18.4	0.92

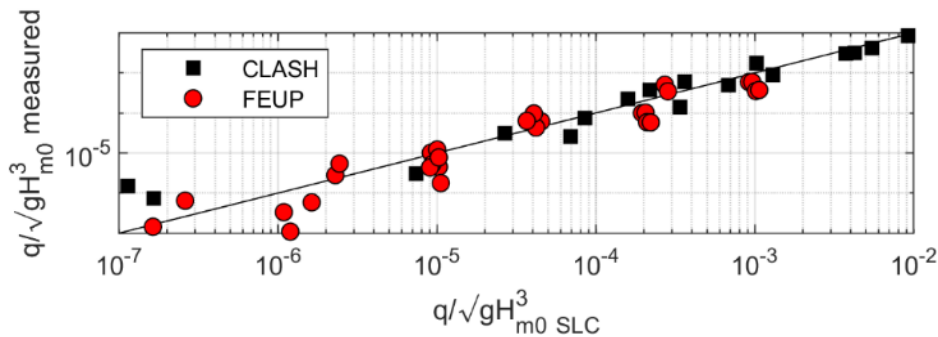
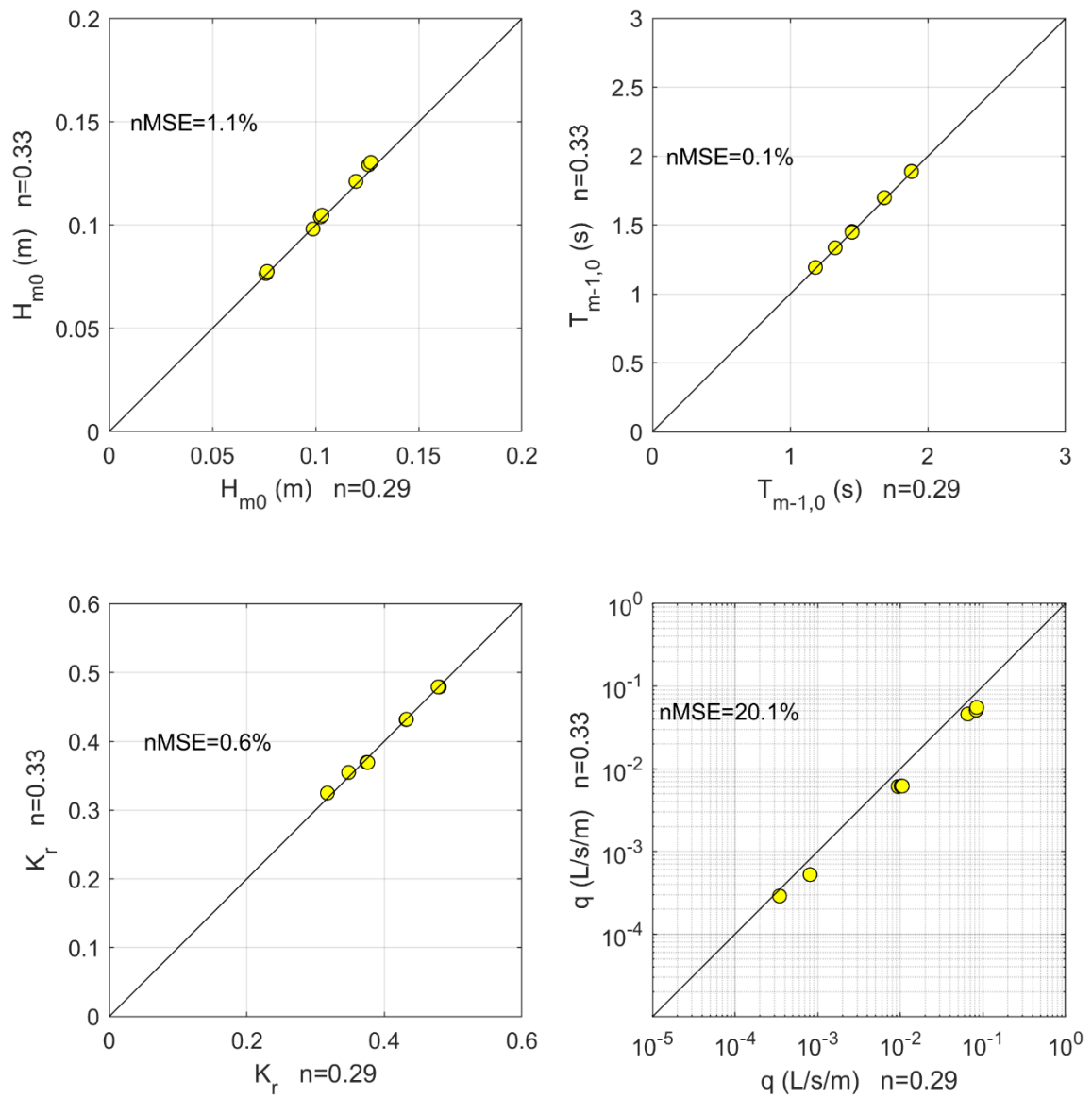


Figure 5.2 – Comparison between non-dimensional mean overtopping discharge from experiments and alternative formula.

5.4. INFLUENCE OF POROSITY

The influence of armour layer porosity on wave overtopping was investigated by testing breakwater sections with different porosity values, 0.29 and 0.33. The results (Figure 5.3) show that there is a clear trend in the overtopping discharges, where lower porosity results in larger values of q , unlike what was observed by Molines *et al* (2012). This is likely associated with the armour layer configurations tested. In Molines *et al* (2012), wave overtopping was measured for double armour layers with random placement. The differences in mean overtopping discharges based on the results of this study are within 16-42%. This difference is relatively small when compared to other factors. For example, De Rouck and Geeraerts (2005) mentions that repositioning of armour units (in a double layer anti-fer cube armour) significantly affects measured overtopping discharges and small changes in the positioning of armour elements may change the overtopping discharges by a factor of two or more.

The test results also show that the differences in armour layer porosities have a limited influence on the reflection coefficient, K_r . It is suggested that this is likely due to the following: the range of armour porosities tested is relatively small and therefore a minor influence on the wave reflection is anticipated; furthermore, although it is expected that a lower armour porosity results in increased wave reflection, the higher overtopping discharge associated with the lower armour porosity will result in slight less wave energy to be reflected from the slope.


 Figure 5.3 – Scatter plot of H_{m0} , $T_{m-1,0}$, K_r and q for porosities 0.29 and 0.33.

5.5. INFLUENCE OF INITIAL SEED NUMBER

A different initial seed number in the wave generation for series P and Q was applied, whilst the incident wave conditions, and spectrum shape remained the same. The results (Figure 5.4) show that the influence of this factor on the mean overtopping discharge is relatively small ($nMSE < 1\%$) with the larger differences occurring for lower mean overtopping discharges. This is expected since it corresponds to the region where typically there is more scatter in results. The difference for prototype mean overtopping discharges above 1 L/s/m is within the range of 0 to 8%. For the cases with smaller mean overtopping discharges (tests P01 and Q01 with $q_{prototype} < 1$ L/s/m) the maximum differences are between 80 to 130%. Romano *et al* (2015) carried out physical model tests for a rock armoured breakwater with different initial seed numbers. They found that, when the relative freeboard is larger than 1.4, the variability of the overtopping discharge may reach one order of magnitude for the same wave condition. The larger differences were observed for the higher relative crest freeboards (lower overtopping discharges).

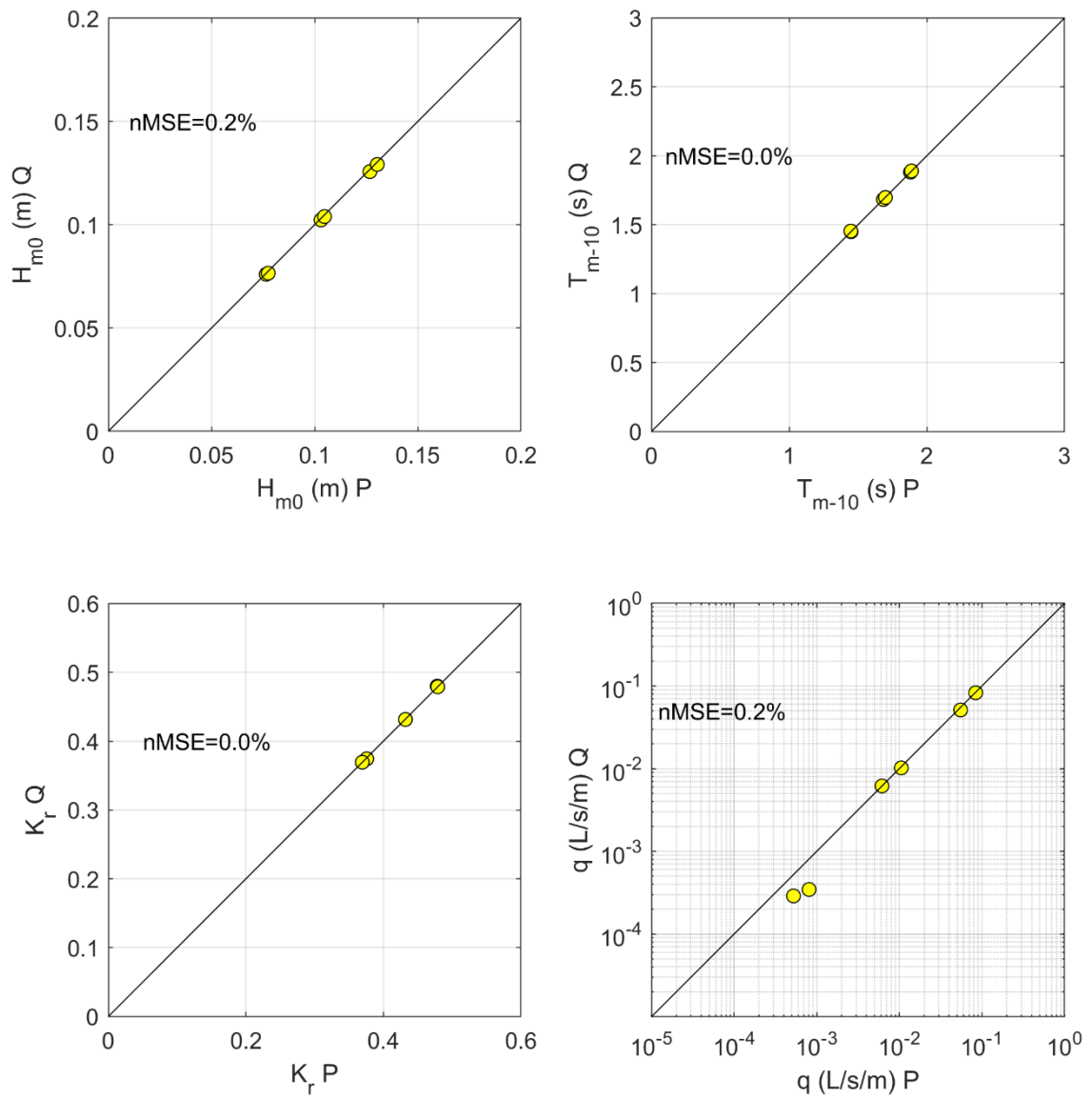


Figure 5.4 – Scatter plot of H_{m0} , $T_{m-1,0}$, K_r and q for different initial seed number (Series P and Q).

6

NUMERICAL MODELLING OF WAVE OVERTOPPING

6.1. INTRODUCTION

The results obtained from the experimental study described in chapter 3 were used to develop a novel method for calibrating porous media coefficients in a VARANS numerical model to simulate mean overtopping discharges using Artificial Neural Networks. The work presented in this chapter aimed at presenting a generic method to be used in future applications with such type of models. This work was published in Vieira *et al* (2021c) and a summary of the main methodologies applied, and findings obtained is presented in this chapter.

6.2. MODEL DESCRIPTION

The numerical model IH2VOF (Lara *et al*, 2011), based in the model COBRAS-UC (Losada *et al*, 2008) was used in this study. IH2VOF solves the two-dimensional Reynolds Averaged Navier–Stokes (RANS) equations based on the decomposition of the instantaneous velocity and pressure fields into mean and turbulent components. The effects of the turbulence field are modelled using the $k - \varepsilon$ transport equations to calculate the turbulent kinetic energy (k) and the turbulent dissipation rate (ε). The free surface movement in IH2VOF is tracked by the Volume of Fluid (VOF) method (Hirt and Nichols, 1981; Liu and Wen, 1997). The flow inside the porous media is resolved using the Volume-Averaged Reynolds Averaged Navier–Stokes (VARANS) equations. A complete mathematical formulation of the VARANS equations is presented in Liu *et al* (1999) and Hsu *et al* (2002).

The IH2VOF model was previously applied in a wide range of cases including: nearshore processes in the surf zone (Lara *et al*, 2006a; Torres-Freyermuth *et al*, 2007; Torres-Freyermuth *et al*, 2010; Lara *et al*, 2011); wave-structure interaction for low-crested breakwaters (Garcia *et al*, 2004; Losada *et al*, 2005; Lara *et al*, 2006b); composite breakwaters (Lara *et al*, 2008; Guanche *et al*, 2009); emerged rubble-mound breakwaters (Losada *et al*, 2008; Guanche *et al*, 2009); tsunami propagation (Aniel-Quiroga *et al*, 2018); and non-conventional breakwaters for wave energy conversion (Di Lauro *et al*, 2019).

6.3. MODEL SETUP

The domain modelled in this study is 14.6 m long in the x and 1.0 m high in the y direction (Figure 6.1). The rectangular mesh is divided into 3 and 2 sub-zones in the x and y directions, respectively, for refinement in the region of interest. The reference mesh size is variable with resolution of 1.0 cm to 2.0 cm in the x direction, and 0.5 cm to 1.0 cm in the y direction. The minimum and maximum $\Delta x/\Delta y$ ratio are 1 and 4, respectively. The total number of cells is 995 (x direction) by 150 (y direction) resulting in approximately 150,000 cells.

Various combinations of α and β were adopted for the porous media layers (armour, filter and core) for providing input data to the ANN. Additional simulations with the coefficients of Losada *et al* (2008) and Jensen *et al* (2014) were carried out since these studies looked specifically into wave overtopping over rubble-mound breakwaters (Table 6.1). The simulations are used as a reference for comparison of the performance of the calibrated model with the β coefficient obtained from the ANN developed in this study. Porosities of 0.29 and 0.33 were adopted for the armour (two scenarios), 0.40 for the filter and 0.38 for the core.

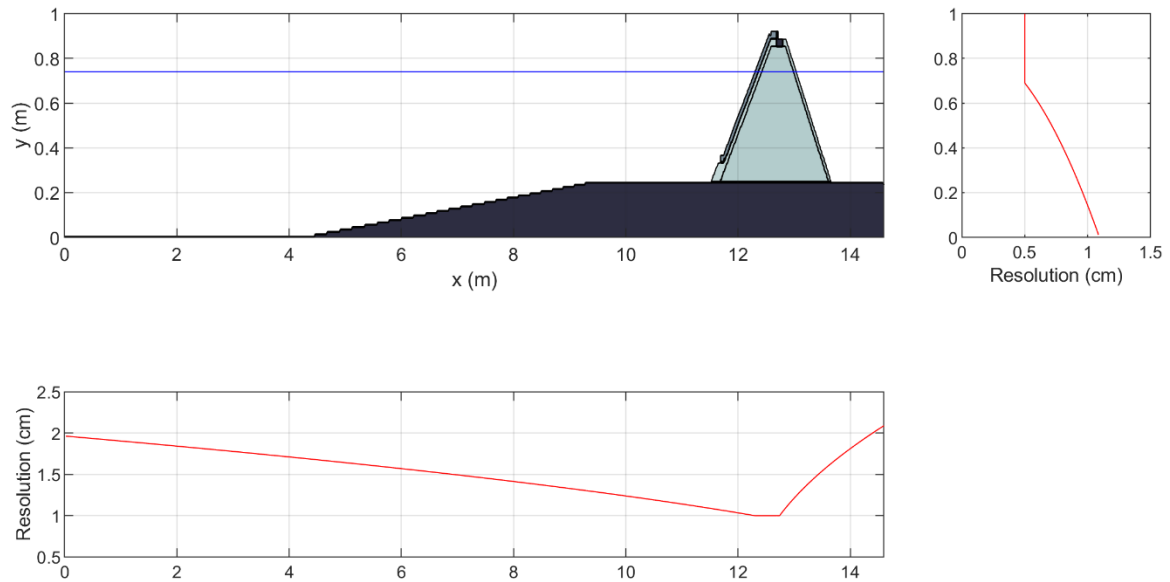


Figure 6.1 – Model domain (top left) and mesh resolution in x (bottom left) and y (top right).

Table 6.1 – α and β coefficients adopted for the reference cases.

Coef.	Losada <i>et al</i> (2008)	Jensen <i>et al</i> (2014)
α (all layers)	200	500
β_{Armour}	0.7	2
β_{Filter}	1.1	2
β_{Core}	0.8	2

A JONSWAP spectrum with a peak enhancement factor of 3.3 with the same H_{m0} and T_p as tested in the physical model, was adopted for generation of the wave series. It was not possible to impose the measured signal since paddle movements were not recorded. However, wave series with several initial seed numbers were tested to obtain wave conditions at the toe of the structure that were similar to the ones measured in the laboratory experiments. Wave absorption boundaries were implemented at the wave generation and end of the numerical flume. Numerical wave gauges were located at the same locations as in the wave flume (Figure 3.12). An additional gauge was included at the crown wall for calculation of overtopping discharges.

6.4. MESH SENSITIVITY

A mesh element size analysis was carried out to assess the sensitivity of this parameter on the results. Mesh elements with sizes 80%, 120% and 140% of the reference mesh element were tested (Table 6.2) and a sample condition applied at the model boundary, with $H_{m0}=18.0$ cm and $T_p=1.8$ s ($s_{op}=0.035$) for a duration of 60 s.

Table 6.2 – Characteristics of tested meshes.

Mesh	$\Delta x_{MIN} / \Delta x_{MAX}$ (cm)	$\Delta y_{MIN} / \Delta y_{MAX}$ (cm)	no. x / no. y
80%	0.8 / 1.6	0.4 / 0.8	1210 / 195
100%	1.0 / 2.0	0.5 / 1.0	995 / 150
120%	1.2 / 2.4	0.6 / 1.2	830 / 130
140%	1.4 / 2.8	0.7 / 1.4	705 / 110

The relative differences in comparison to the reference mesh (100%) are presented in Table 6.3. The results show that there is a relatively small decrease in the overtopping volume and increase in the reflection coefficient for a finer mesh (80%). A 20% increase in mesh resolution results in 10% higher overtopping volumes and a negligible increase in reflection coefficient. Based on these results, it was decided to adopt the reference mesh since small differences in Q and K_r are obtained with the finer mesh but the simulation time is increased by a factor of 2.3 which is considered excessive as many simulations are required in this study. The simulations were run on a workstation with Intel Xeon Platinum 8168 CPU @ 2.7 GHz processors.

Table 6.3 – Mesh sensitivity results.

Mesh	Q / Q_{REF} (%)	K_r / K_{rREF} (%)	Sim. Time (days/1000s)
80%	-3	+2	6.3
100%	-	-	2.7
120%	+10	-1	1.3
140%	+11	-4	0.7

6.5. ARTIFICIAL NEURAL NETWORK FOR MODEL CALIBRATION

The IH2VOF model was applied to a combination of incident wave conditions (similar to the conditions tested in the laboratory), armour layer porosity and model empirical coefficients to generate values of q for the development of the ANN (Figure 6.2).

The β coefficient for each of the three porous layers (β_{Armour} , β_{Filter} , β_{Core}) was chosen as the model calibration parameters. Lara *et al* (2006, 2008) suggests that numerical results are more sensitive to the non-linear drag coefficient β than the linear parameter α for turbulent flows. Therefore, α was kept constant with a value of 200 (based on values applied in literature) to limit the number of variables in the problem. A range between 0.8 and 1.2 was adopted for β based on values provided in the literature

where a reasonable calibration was achieved (Lara *et al*, 2008; Losada *et al*, 2008; Guanche *et al*, 2009; Guanche *et al*, 2015). A summary of the β coefficient combinations for each simulation is presented in Table 6.4. The model was run for cases where the mean overtopping discharges obtained in the laboratory is above 0.00 L/s/m (Table 6.5) resulting in 8 (wave conditions) x 6 (coefficient combinations) = 48 simulations which is much smaller (~5%) than if all possible combinations of values were applied (1,000). Each simulation took between 40 and 92 h.

Table 6.4 – Coefficients adopted in the simulations.

Coef.	ANN1	ANN2	ANN3	ANN4	ANN5	ANN6
β_{Armour}	1.2	1.0	1.0	1.2	0.8	0.8
β_{Filter}	1.2	1.2	0.8	0.8	1.0	1.0
β_{Core}	0.8	1.0	1.2	0.8	1.2	1.0

Table 6.5 – Wave conditions at the toe of the structure and mean overtopping discharges from the physical model.

Case.	n_{Armour}	H_{m0} (m)	$T_{m-1,0}$ (s)	q (L/s/m)	$q_{prototype}$ (L/s/m)
O01	0.29	0.08	1.0	0.00	0.1
	0.33	0.08	1.0	0.00	0.0
O02	0.29	0.10	1.2	0.01	1.9
	0.33	0.10	1.2	0.01	1.3
O03	0.29	0.12	1.3	0.07	13.6
	0.33	0.12	1.3	0.05	9.5
P01	0.29	0.08	1.5	0.00	0.1
	0.33	0.08	1.5	0.00	0.1
P02	0.29	0.10	1.7	0.01	2.1
	0.33	0.10	1.7	0.01	1.3
P03	0.29	0.13	1.9	0.08	17.1
	0.33	0.13	1.9	0.05	10.6

The MATLAB® Neural Network toolbox (Beale *et al*, 2019) was used in this study. The input parameters for the ANN are H_{m0} , $T_{m-1,0}$, n_{Armour} , β_{Armour} , β_{Filter} , β_{Core} , and the output is the mean overtopping discharge, q (Figure 6.2).

50 networks (with different initial weights) were tested, and the best performing ANN selected based on the lowest normalized Mean Square Error ($nMSE$).

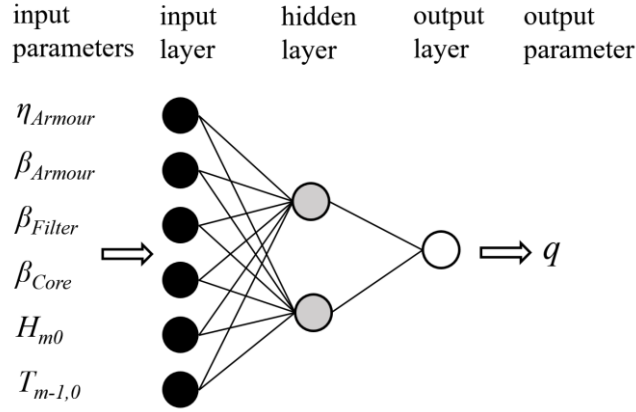


Figure 6.2 – Architecture of the ANN used in this study.

6.6. RESULTS AND DISCUSSION

Results from the 48 numerical simulations (6 combinations of β values $\times$ 8 wave conditions) were used to develop an ANN for estimation of the best combination of β values. The best performing ANN was number 41 (Figure 6.3 – top) with a $nMSE=0.5\%$. The agreement between the model results (for each of the 48 simulations) against the ANN estimate is very good (Figure 6.3– bottom).

An input matrix for the 125 possible combinations of β values for the three breakwater layers and 8 wave conditions (1,000 cases) was generated to be used as input for the ANN. The best combination of values based on the ANN estimates is $\beta_{Armour}=1.2$, $\beta_{Filter}=0.8$, $\beta_{Core}=1.2$ ($nMSE=7.9\%$). These values were applied to the numerical model.

A good agreement between the mean overtopping discharge obtained with the numerical model using the β values and those estimated by the ANN was achieved with a $nMSE$ of 1.4% (Figure 6.4). The main difference is for the cases with lower overtopping discharges (O02 tests in Table 6.5 with measured $q_{prototype}$ of 1.9 and 1.3 L/s/m), for which no overtopping was obtained in the model (Figure 6.4). For smaller average overtopping discharges in physical models ($q < 1$ L/s/m), uncertainties in measurements can become large (Troch *et al*, 2004) and therefore there is typically more scatter in the data.

Model results for the β values from Losada *et al* (2008) (β_{Los}), Jensen *et al* (2014) (β_{Jen}), and this study (β_{Vie}) are compared against the data obtained from the laboratory experiments. Kr values obtained with β_{Jen} agree better with the measurements ($nMSE$ of 14.7%). The root mean square error obtained with the model for β_{Vie} is 4.2%, which is slightly higher than the values presented in Zanuttigh *et al.* (2008) (3.7% for armour units) based on an empirical formulation, and therefore it is considered acceptable. The non-dimensional overtopping discharges obtained with β_{Vie} show a good agreement compared to the measurements and perform better than using values of β from the literature (Figure 6.5 and Table 6.6).

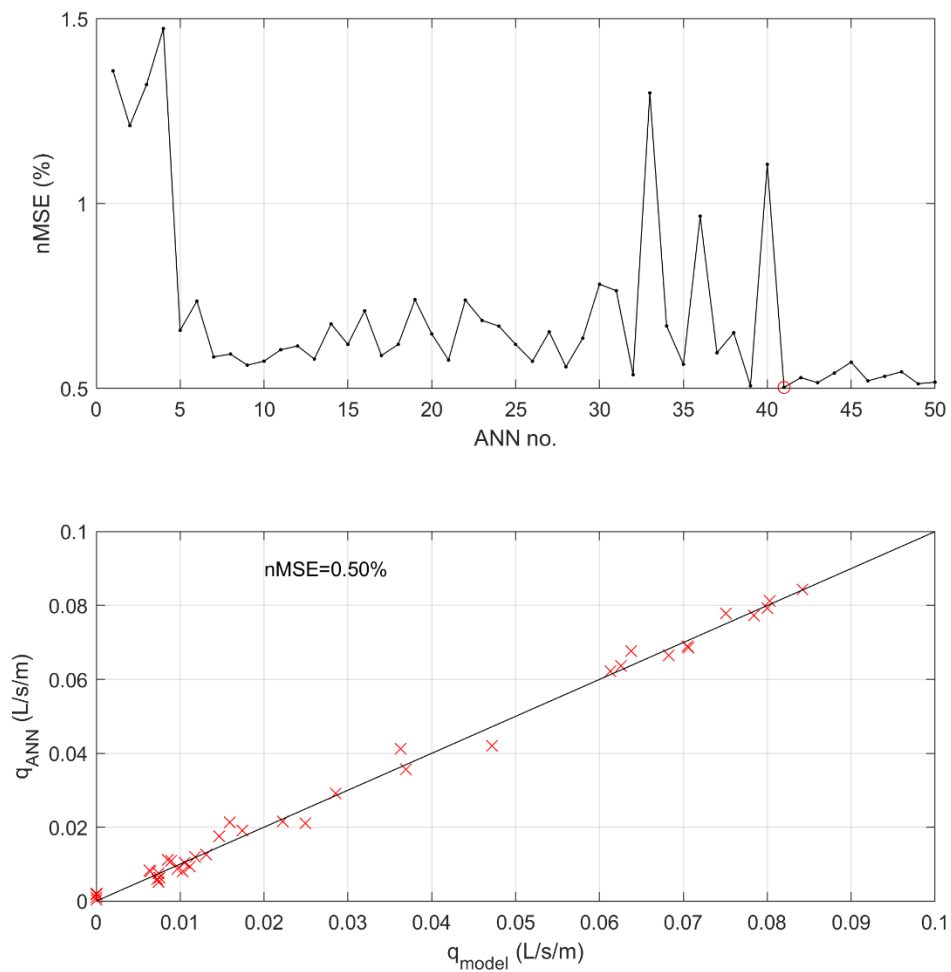


Figure 6.3 – $nMSE$ for all ANNs tested (top) and comparison between modelled and ANN estimated mean overtopping discharges (bottom).

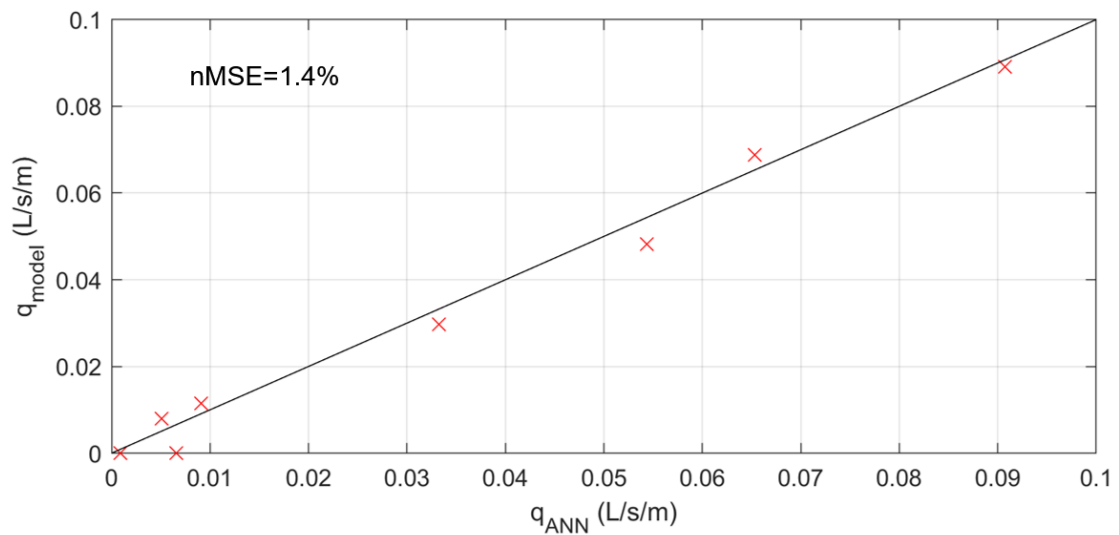
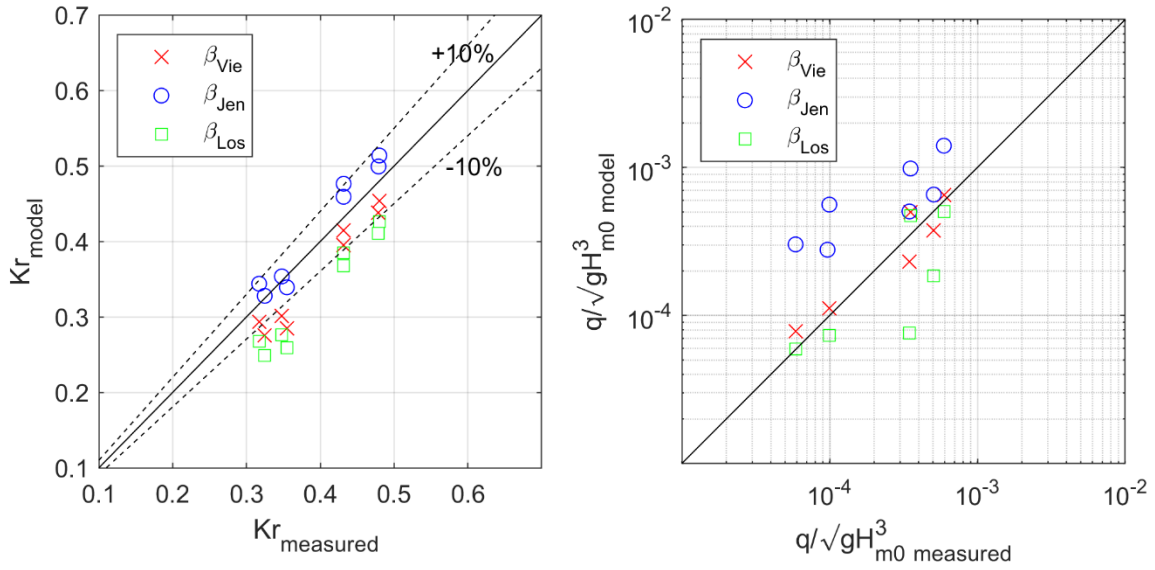


Figure 6.4 – Comparison of mean overtopping discharges between ANN estimate and model predictions.

Figure 6.5 – Comparison between model and measurements of Kr (left) and q^* (right).Table 6.6 – $nMSE$ (%) for Kr and q for the various β values.

β ref.	Kr	q^*
Losada <i>et al</i> (2008)	99.5	47.4
Jensen <i>et al</i> (2014)	14.7	301.8
This study	38.6	13.9

Differences between measurements and model results are inevitably expected for several reasons. For example, the structure slope in the 2D numerical model was imposed as a straight line whilst there will be irregularities along with the slope profile. Also, the mesh resolution, as shown in section 6.4, will result in some differences in results depending on the selected mesh size. The mesh size selection is always a compromise between accuracy and computational time constraints. Additionally, in this study, different initial seed numbers for the generation of the wave series in the numerical model and experiments were used. In Vieira *et al* (2021a) the influence of a different initial seed number for the wave generation signal was carried out based on some of the tests presented in this study. The difference for prototype mean overtopping discharges above 1 L/s/m is within the range of 0 to 8%. For the cases with smaller mean overtopping discharges ($q_{prototype} < 1$ L/s/m) the maximum differences are between 80 to 130%. Romano *et al* (2015) carried out physical model tests for a rock armoured breakwater with different initial seed numbers. They found that, when the relative freeboard is larger than 1.4, the variability of the overtopping discharge may reach one order of magnitude for the same wave condition. Based on all of the above aspects, it is considered that the validation is adequate, and the numerical model can be used for evaluating mean wave overtopping discharges for other incident wave conditions (within the range of validated values) and investigate the performance for slight geometry modifications.

7

SUMMARY AND OUTLOOK

7.1. CONCLUSIONS

Single-layer cube (SLC) armoured breakwaters with a regular placement pattern are a relatively recent concept despite the cube being one of the oldest concrete units available. Limited investigation on the hydraulic stability and wave overtopping for this armour type is available to date. 2D physical modelling tests were carried out at the Faculty of engineering of the University of Porto, Portugal, for different armour porosities, wave steepnesses, and armour crest levels to gain a better understanding of the performance of this armour type.

7.1.1. HYDRAULIC STABILITY

A novel image-processing based procedure using machine-learning techniques was applied to automatically detect individual cube positions, to enable a consistent and fast method for post-processing of the results. This method is relatively straightforward and can be implemented to process various images in a batch mode, which will reduce post-processing times.

Innovative parameters based on cube movement exceedance probability curves, mean grid orthogonality, and relative damage number, were developed to describe damage evolution and used to establish preliminary thresholds for Initiation of Damage in SLC armoured breakwaters. The suggested thresholds for the various parameters are based on the correspondent visual assessment of the damage during the tests. A summary of the proposed damage evolution classification and criteria for SLC armoured breakwaters is depicted in Figure 7.1.

Results show that porosity of around 0.29 is generally preferred for higher hydraulic stability. Structures with a lower crest height showed greater stability. Stability numbers for IDa_{SLC} range from 3.1 to more than 3.6 for armour with a porosity of 0.29. Adequate recommended safety coefficients (e.g., CIRIA, 2007) should be applied in preliminary designs to provide some margin due to the failure mechanism of this structure. Test results show that these structures are relatively resilient and therefore safety factors could potentially be reduced, subject to verification in physical model tests.

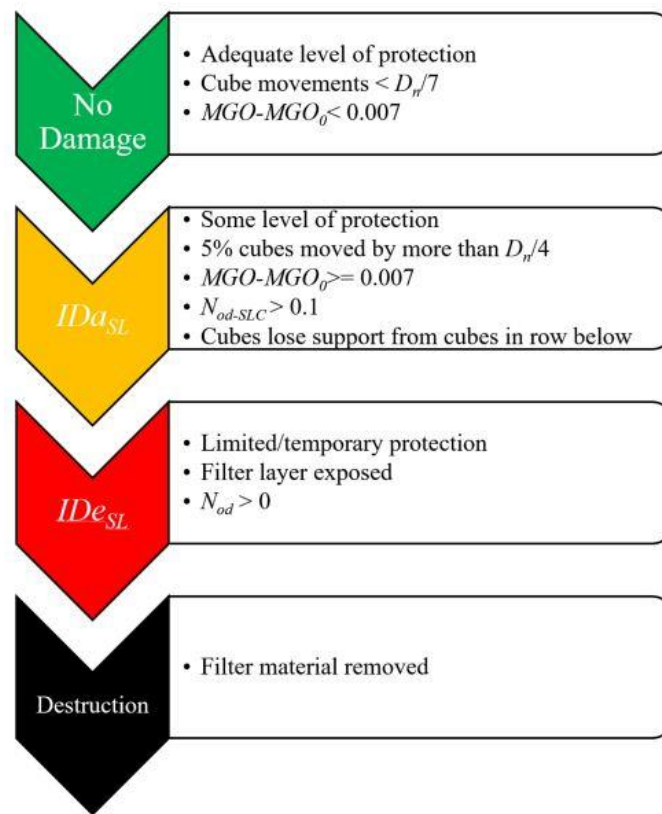


Figure 7.1 – Damage evolution classification for SLC armoured breakwaters.

7.1.2. WAVE OVERTOPPING

The obtained experimental results have shown that available overtopping prediction methods have some limitations in representing satisfactorily the measured mean overtopping discharges. Therefore, an analysis of the relative importance of the various parameters on mean overtopping discharges was carried out and it was found that Rc/H_{m0} and $L_{m-1,0}/Gc$ explain the majority of the variance in overtopping in the measurements. An alternative overtopping formula was derived and better performance, compared to other methods (Van der Meer and Janssen, 1994, EurOtop, 2018 and ANN, Zanuttigh *et al*, 2016), was achieved. The purpose of this formula is mainly to explain the variability in results compared to other studies available in the literature for the same armour configuration. In fact, despite the large relative increase of the database (from 18 tests presented in CLASH database to 49), the formula is still limited by the scarce data from which it was derived, and it will only be applicable for SLC armoured breakwaters within the geometries and incident wave and water level conditions tested.

The laboratory test results indicate that small variations in porosity (from 0.33 to 0.29) result in increased overtopping discharges in the range of 16-42%, however, no significant variations in the reflection coefficient were observed. The influence of a different initial seed number for the wave generation signal was carried out and it was found that the differences in mean overtopping discharges are up to a factor of two for structures with higher relative freeboards (corresponding to lower mean overtopping rates).

7.1.3. NUMERICAL MODELLING OF WAVE OVERTOPPING

A correct calibration and validation of numerical models against physical model tests are essential to properly represent the wave-structure interaction processes. A precise model validation provides confidence to study properties not easily measured in the laboratory, to assess different structure geometries and/or incident wave and water level conditions. In the traditional “trial-and-error” approach, it is difficult to ensure the best combination of coefficients is achieved. To this end, this study presents a novel methodology for the calibration of the porous media coefficients in a VARANS-VOF model to obtain the optimum values for the simulation of wave overtopping discharges. The new method consists of running a limited number of simulations using different combinations of coefficient values (one for each layer) to be used in the development of the ANN. The ANN is subsequently used to evaluate a large range of different combinations of coefficient values to select the best performing in terms of mean wave overtopping discharge. The selected coefficient values are then applied to the numerical model and the results are validated against the 2D physical model tests. This method is shown to perform well and results in a reduced number of simulations (48) compared to the required number to assess all possible combinations of values (1,000) considered in this study. A $nMSE$ of 13.9% was achieved for non-dimensional mean overtopping discharges between the model and laboratory test results, which is better than if values from existing literature for similar cases are applied.

7.2. OUTLOOK

7.2.1. HYDRAULIC STABILITY

The analysis performed in this thesis was limited to the data obtained in the laboratory experiments carried out in this study. It is recommended to carry out additional testing for SLC armoured breakwaters and applying the proposed damage criteria to validate its applicability to other forcing conditions and structure geometries. This analysis was focused on the slope area. Further research on other areas of the structure such as crest, rear-side, and toe are recommended. Actual safety factors between IDa_{SLC} and IDe_{SLC} were not evaluated in this study. This was not possible in these experimental tests as the wave height at the structure toe (and consequently Ns) was limited by the model scale chosen, the wave generator capacity, and water depth at the structure. Future work should be planned to cover this important aspect.

The influence of the grid placement accuracy on the armour stability was outside the scope of this work. However, the available data enabled a high-level assessment. Further investigation is recommended for a more detailed evaluation of this aspect to potentially establish adequate construction tolerances either in the model or in the prototype.

Due to the curvature of the roundhead and subsequent implications, such as deviation of unit placement from a regular grid and larger gaps between units, its stability is typically smaller than at the trunk section (Muttray and Reedijk, 2009). The roundheads are also generally subject to more complex hydrodynamic forcing, which are difficult to characterize, and are key parts of the breakwater global integrity. Therefore, it is recommended to develop specific research on damage evaluation methods adequate to these areas.

Finally, the cube shape offers the possibility of including minor features and modifications such as small holes, recesses, and striations, which increase surface complexity and promotes the growth of marine organisms (e.g., Firth *et al*, 2014; Schoonees *et al*, 2019; Evans *et al*, 2021). Such minor features are unlikely to affect the overall armour performance, but it is suggested to study them in detail.

7.2.2. WAVE OVERTOPPING

The proposed formula is an initial step for developing a better descriptor for the mean wave overtopping discharges in SLC armoured breakwaters. It is currently still limited by the relatively small number of data used. Improvements to the formula are expected as more data for this armour configuration becomes available.

The present dataset aims to contribute to extending the available wave overtopping database for single-layer cube armoured breakwaters. The data can be applied in the training of an updated version of ANN overtopping.

Additional studies considering different structure geometries and focused on areas where less data is available will be useful to improve the understanding of the overtopping process in single-layer cube armoured breakwaters. The tests were limited to 2D and therefore the influence of wave obliquity in 3D models is a topic of consideration for future research.

7.2.3. NUMERICAL MODELLING OF WAVE OVERTOPPING

The ANN developed in this study aimed at reducing the error in mean overtopping discharge estimates for the entire range of wave conditions and armour layer porosities with a unique combination of coefficient values. A better combination of values could be achieved if the objective was to focus on a single incident wave condition and armour layer porosity. For example, during the design stages, it is common to investigate the influence of structure geometry modifications on overtopping performance for the design condition (say, the 100-year return period event). Depending on the focus of the model application, an ANN to simultaneously reduce the error in both mean wave overtopping and reflection coefficient could be developed in the future.

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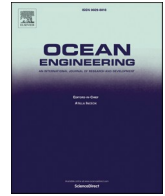
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APPENDIX – PUBLISHED JOURNAL PAPERS

APPENDIX I – Vieira et al. (2020C)



Review

Single-layer cube armoured breakwaters: Critical review and technical challenges

Filipe Vieira^{a,b,*}, Francisco Taveira-Pinto^{b,c}, Paulo Rosa-Santos^{b,c}^a Department of Biology, Chemistry and Environmental Sciences, American University of Sharjah, Sharjah, United Arab Emirates^b Department of Civil Engineering, University of Porto, Porto, Portugal^c Interdisciplinary Centre of Marine and Environmental Research of the University of Porto, Matosinhos, Portugal

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ABSTRACT

Breakwater concrete armour units have been continuously evolving since the 1950s. Lower concrete consumption, increased hydraulic stability, reduced wave run-up, and easier placement techniques are the main drivers of this development. A recent innovative solution for breakwater armour, based on one of the oldest type of units available, consists of a configuration adopting a single-layer of concrete cubes placed in a regular pattern and a porosity of 0.25–0.30. This paper presents a summary of the laboratory test results available from literature in terms of hydraulic stability and wave overtopping. Studies indicate that this is a feasible solution with several advantages over other armour configurations related to the simplicity of the cube unit and placement pattern. However, there is still some degree of uncertainty in both hydraulic stability and wave overtopping estimates for a structure with this configuration where few laboratory tests have been conducted in comparison to other armour units. Furthermore, the available methods for quantification of damage are not considered adequate for use in single-layer cube armour with a regular placement pattern. This paper discusses current knowledge gaps and provides guidelines for future research.

1. Introduction

A breakwater main function is to protect port's entrances, basins and berths to ensure safe navigation and operation in these areas. The breakwater armour stone or block weight is a critical parameter in the design of the structure. The choice between rock and concrete armour units for the armour layer in a project typically depends on the availability of an adequate rock grading, constructability or cost-related aspects. Construction costs depend on numerous factors, these being associated to design and logistic factors, including the type of armour material (unreinforced concrete, granite rock, sandstone rock, etc.), armour unit mass, personnel and material unit costs, total concrete consumption, placement equipment, casting, handling and stacking procedures (Medina et al., 2014). During the design process, once a unit is selected, guidelines for placement, hydraulic stability and wave overtopping are available for the various types of concrete units (e.g. CIRIA, 2007; EurOtop et al., 2007). The empirical formulas in the guidelines are based on laboratory research and experience with prototypes.

The use of single-layer cubes with a regularly placed pattern and low porosity (0.20–0.35) on the seaside of breakwaters is a recent concept and therefore the application of empirical formulas from available guidelines should be viewed with care. Studies on hydraulic stability and wave overtopping are relatively few compared to other units and only one prototype (Fig. 1) with such configuration has been built to date (Van der Lem et al., 2016). This review paper summarizes the state-of-the-art for single-layer cube armoured breakwaters, identifies gaps in knowledge, and presents future research challenges.

The paper is organized as follows: after a brief introduction in Section 1, a summary of the use and performance of concrete armour cubes is presented in Section 2. Section 3 describes the laboratory test results of hydraulic stability and wave overtopping for single-layer cube armoured breakwaters. Knowledge gaps identified in the literature on this breakwater armour configuration are also reported and future research directions provided. Section 4 presents the main conclusions of this study.

* Corresponding author. Department of Biology, Chemistry and Environmental Sciences, College of Arts and Sciences, American University of Sharjah, PO BOX 26666, Sharjah, United Arab Emirates

E-mail address: fvieira@aus.edu (F. Vieira).

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2. Concrete armour cube structures

2.1. Introduction

Cubes or parallelepipeds are the simplest types of concrete armour unit, which have been applied in different configurations (Table 1) in either physical models or prototypes. In regions where rock is not available, such as The Netherlands, cubes, or other types of concrete blocks are typically adopted for impermeable patterned block revetments in dike protections (Klein Breteler et al., 2014). Cubes in patterned block revetments with very low porosity mainly rely on friction between units as a stabilizing force. Concrete cubes for breakwater armour are traditionally built in a two-layer system and with a random placement pattern. The cubes of the first layer tend to slide and align with the slope thus creating a less permeable surface, which could be subject to excessive pore pressure and subsequent lifting of the blocks (Muttray and Reedijk, 2009). The second layer provides additional resilience and aids in increasing permeability, which improves hydraulic stability and reduces wave run-up. When placed randomly, the stability of the blocks is obtained primarily through their weight. Concrete (hollow) parallelepipeds in a regularly placed pattern have been used on the lee side of breakwaters (e.g., Burcharth et al., 2015; double layer). The stability of a randomly-placed double layer of cubes has also been investigated in low-crest breakwaters (Argente et al., 2018). Table 1 shows different examples of cube armoured breakwater configurations. Other types of structure configurations, common in other units, such as coastal revetments, berm breakwaters, or low-crest/submerged breakwaters using regularly placed single-layer cubes with low porosity (~ 0.3) have still not been thoroughly investigated and applied. For example, information on single-layer cube armoured low-crest or submerged breakwaters is limited to few tests (e.g. Almeida, 2013) unlike other units (e.g. Muttray et al., 2012; Argente et al., 2018) where detailed studies have been conducted. A regularly-placed cube armoured lower slope and berm was adopted in Van Gent and van der Werf (2017) but the focus of the study was to assess the stability of the upper slope built of rock and not the lower slope or the berm.

2.2. Hydraulic stability

Detailed research on hydraulic stability of double-layer cube armoured breakwaters include cube concrete density (Triemstra, 2000), density of cube placement (Van den Bosch et al., 2002), armour porosity and placement methods (Medina et al., 2010), wave obliquity (Van Gent, 2003; Wolters and van Gent, 2010; Van Gent, 2014) and round-head stability (Vidal et al., 1991; Maciñeira and Burcharth, 2007; Maciñeira and Burcharth, 2016).

Damage in concrete armour layer systems is typically defined by the relative damage number, N_{od} , proposed by Van der Meer (1988), as the number of displaced units within a strip of width D_n , defined as:

Table 1

Configurations of cube armour coastal structures in physical models or prototypes.

Type	No. layers	Placement	Porosity	Stabilizing force	Reference
Block revetment	1	Regular	Very low (<0.1)	Friction	Capel (2015)
Breakwater rear side	2	Regular	Very low (<0.1)	Friction	Burchart et al. (2015);
	1		Medium (~ 0.4)	Weight and friction	Hellingsa (2016)
Low-Crest Breakwater sea-side	2	Random	Medium (~ 0.4)	Weight	Argente et al. (2018)
Non-Overtopped Breakwater sea-side	2	Random	Medium (~ 0.4)	Weight	Burchart et al. (2015)
Non-Overtopped Breakwater sea-side	1	Regular	Low (~ 0.3)	Weight and friction	Van der Lem et al. (2016)

$$N_{od} = \frac{N_o}{B / D_n} \quad (1)$$

where N_o represents the number of units displaced out of the armour layer, B the width of the tested section, and D_n the armour unit nominal diameter or equivalent cube length.

Van der Meer (1988) developed an empirical formula, based on small-scale tests with a slope of 1 V:1.5H on a uniform 1 V:30H foreshore slope, to estimate the stability number, N_s , in double-layer cube armoured breakwaters for a fixed value of related damage number:

$$N_s = \frac{H_s}{\Delta D_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \right) s_{om}^{-0.1} \quad (2)$$

where H_s represents the significant wave height, $\Delta = \rho_c / \rho_w - 1$ the relative buoyant density with ρ_c the rock or concrete density and ρ_w the water density, N the number of waves and s_{om} the deepwater wave steepness, defined as $s_{om} = H_{m0} / L_{om}$, where L_{om} represents the deepwater wavelength based on the mean wave period T_m .

The use of single-layer armour systems has increased since the development of the Accropode unit. The main advantage of a single-layer armour is the concrete volume reduction compared to double layer systems. Van der Meer (1999) found that the start of damage in single-layer armour unit systems (Accropode and Core-Loc) occurs at very high stability numbers. However, the start of damage in single-layer armour unit systems is usually followed by a sudden failure and, therefore, additional safety factors should be applied to provide



Fig. 1. Sal-Rei breakwater. Courtesy of Van Gent and van der Werf (2017).

some margin due to their failure mechanism (Medina and Gomez-Martin, 2012).

The initial damage in double-layer cube armour structures occurs at lower stability numbers than for single-layer structures (Fig. 2). Even assuming the definition of “failure” different for double ($N_{od} = 2$) and single-layer ($N_{od} = 0.2$), as defined in Van Gent (2003) and Van Gent and Lufis (2013), available results show that the failure still occurs at higher stability numbers for single-layer structures (Fig. 2). However, the damage curves show that for double-layer structures it occurs more gradually than for single-layers.

Van Gent et al. (1999) concluded that the performance of single-layer cube armour systems is good and that the strength of single top-layers seems to be a combination of strength due to weight (as for double top-layers), strength due to contact-forces between adjacent blocks (as for placed block revetments) and less wave attack on single units due to a smoother surface of the entire slope.

Van Gent (2003) carried out small-scale tests to assess the influence of wave obliquity on the stability of double and single layer cube armoured structures. Test results show that to reach the same amount of damage (e.g. $N_{od} = 0.2$) a considerable higher wave height is required in the case of cubes in a single layer if the angle of wave attack is 30° or more. For cubes in a single layer, the wave height compared to perpendicular wave attack needs to be a factor for increased stability number, N_s , of about 2–2.5 higher for an angle of wave attack of 45° , whilst for cubes in a double layer, this factor is about 1.5.

A summary of the stability numbers recommended in CIRIA (2007) for the design of breakwaters’ trunk and roundhead sections for cube armour units is presented in Table 2. Similar stability numbers for single and double-layer are proposed for the trunk section, which would result in the same unit weight. The armour layer stability at breakwaters’ roundheads is critical with respect to the exposure of the breakwater head (Muttray and Reedijk, 2009). This is related to the location of the roundhead, usually in deeper water than the trunk, and therefore exposed to more energetic wave conditions. Additionally, due to the curvature of the roundhead and subsequent implications such as deviation of unit placement from a regular grid and larger gaps between

Table 2

CIRIA (2007) recommended stability numbers for design for cube armoured breakwaters.

Unit	No. layers	cot α	Damage level (%)	N_s (trunk)	N_s (head)	Reference
Cube	2	1 V:1.5H	5	2.0–2.4*	1.95**	*Van der Meer (1988) **SPM (CERC, 1984)
Cube	1	1 V:1.5H	0	2.2–2.3	N/A	Van Gent et al. (1999)

units, reduced stability that the trunk section is typically achieved (Muttray and Reedijk, 2009). For double-layer cube armour, CIRIA (2007) recommends a lower stability number for the roundhead than for the trunk (Table 2). To our knowledge, investigation on stability of roundheads in single-layer cube armoured breakwaters was not published to date and therefore no value is presented in Table 2). For other units, such as the Cubipod, Sande et al. (2018) show that roundheads with double-layer armour have stability values similar to that of the single ones under certain conditions.

2.3. Wave overtopping

Wave overtopping is a critical aspect of the design and performance of a breakwater. Several research works in wave overtopping in double-layer cube armoured breakwaters have been conducted (e.g., Pearson et al., 2004; Andersen and Burcharth, 2009; Molines et al., 2012b; Mares-Nassare et al., 2019).

Mean overtopping rates can be estimated with the empirical formula of Van der Meer and Janssen (1994).

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot \exp\left(-2.6 \frac{R_c}{H_{m0}} \frac{1}{\gamma_f}\right) \quad (3)$$

where q represents the mean overtopping discharge, H_{m0} the spectral significant wave height, R_c the crest freeboard (the difference between the still water level and crest elevation), and γ_f the roughness factor.

Wave overtopping in single layer systems are typically higher due to the lower structure permeability (e.g. Molines and Medina, 2015, for comparison between double and single layer armour of Cubipods). Roughness factors to be applied in Eq. (3) for different armour layer configurations were estimated in Bruce et al. (2009) based on small-scale model tests. Double-layer systems have lower roughness factors (Table 3) due to the additional armour layer, which provides increased permeability and associated lower wave run-up. Wave overtopping is also expected to be higher in the case of a single-layer of cubes than for other one-layer systems due to the lower roughness of the cube.

Several studies to assess the influence of wave obliquity on overtopping rates in rubble-mound structures have been carried out to date (e.g., Galland, 1994; Andersen and Burcharth, 2009; Nørgaard et al., 2013; Van Gent and van der Werf, 2019; Mares-Nassare and van Gent, 2020). Andersen and Burcharth (2009) show that for rubble-mound structures overtopping discharge decreases with increasing wave obliquity and propose reduction factors for short and long-crested waves. The study also indicates that two layers of randomly placed cubes give just slightly more overtopping than rock armour. To the authors’ knowledge, there is no information available on the influence

Table 3

Roughness factors for cube armoured breakwaters.

Unit	No. layers	cot α	γ_f	Reference
Cube	2	1V:1.5H	0.47	Bruce et al. (2009)
Cube	1	1V:1.5H	0.49	Bruce et al. (2009)

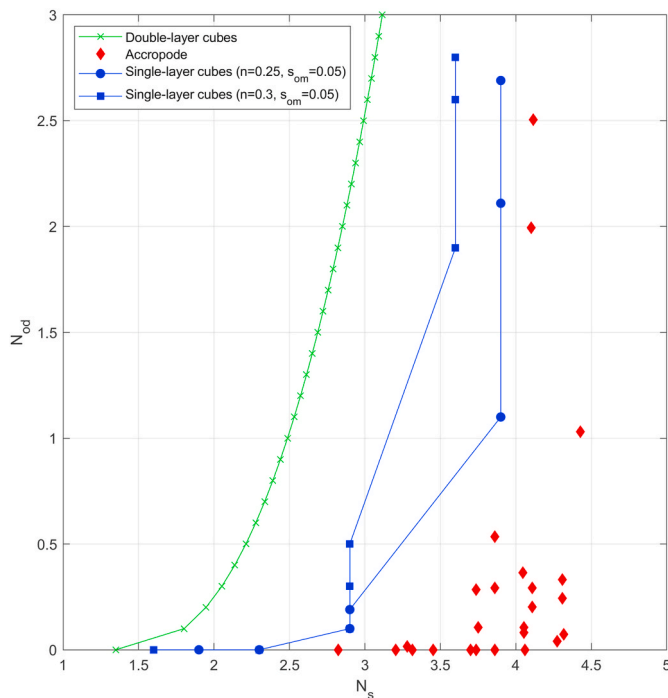


Fig. 2. Example of stability number (N_s) vs damage number (N_{od}) for double-layer cubes (empirical), single-layer cubes (data from Almeida, 2013) and Accropode (data from Van der Meer, 1999).

of wave obliquity on overtopping rates for single-layer cube armoured structures for comparison against a double-layer configuration.

3. Single-layer cube armoured breakwaters

3.1. Armour unit placement

The placement of single-layer interlocking armour units on a breakwater slope has to meet strict requirements for armour unit position and contact points with neighbouring units and slope (Reedijk et al., 2018). A regular placement pattern is understandable, simpler, and typically preferred in construction sites. Another advantage of the regular placement is the increased hydraulic stability when compared to randomly placed configurations. Hald et al. (1998) showed that higher stability was achieved when using orderly methods of placement for single-layer rock armour units for breakwaters in Norway. Safari et al. (2018) compared a regular to random placement of single-layer Star-bloc® armour units and found that higher stability numbers are achieved with the regular pattern placement.

The regular placement of cubes needs to be carefully controlled to achieve the required gap distance between cubes and subsequently the specified porosity. The gaps provide a solution for the excess pore pressure gradients when cubes are closely spaced to each other as in very low porosity block revetments. In small-scale laboratory tests, it is feasible to place a single-layer of cubes with the required precision. However, it is uncertain if a specified placing density can be achieved in prototype situations. Tests have been carried out by Verhagen et al. (2002) to determine the requirements for dropping concrete blocks from a crane onto a breakwater slope. The conclusion is that at a water depth of approximately 10 times the block size this can be achieved and, for smaller depths, this placing density can be reached without too many difficulties.

Single-layer concrete cubes with a regular placement pattern for breakwater armour is a relatively new configuration. Van Gent and Luís (2013) concluded that a breakwater with cubes in a single-layer is feasible and potentially economically competitive with other single

layer units despite having a lower stability number. The main advantages of a single-layer of cubes compared to other one-layer systems are: the simplicity of formwork in fabrication; high structural strength; patent-free; smaller casting yard area requirements, since the removal of the formwork is done vertically; safer handling using pressure clamps, whilst some other units make use of rope slings; efficient area utilization for stacking; simple placement grid; the lower likelihood of breaking units during construction; greater production rate and easier repair works. Disadvantages are related to the low resilience offered if some blocks are extracted or settlement occurs (as in any single-layer configuration), negligible interlocking between units, higher concrete consumption compared to other single-layer units, and difficulties during construction in the presence of waves due to a strict placement tolerance.

3.2. Laboratory tests on single-layer cube armoured breakwaters

3.2.1. Hydraulic stability

Experimental studies focused on the evaluation of the stability performance of a single-layer armour of cubes with low porosity values have been ongoing for the last twenty years. Several researchers (Van Gent and Spaan, 1998; d'Angremond et al., 1999; Van Gent et al., 1999, 2001) have studied the feasibility of a single layer cube armoured breakwater with porosities between 0.25 and 0.40 with an irregular placement (Fig. 3 – left). More recent research (Van Buchem, 2009; Almeida, 2013; Van Gent, 2014; Van Gent and Luís, 2013; Van der Lem et al., 2016; Van Gent and van der Werf, 2017) investigated the combination of a single-layer cube armour breakwater with a regular placement (Fig. 3 – right).

Van Gent et al. (1999) carried out small-scale physical model tests on a single armour layer of cubes ($D_n = 0.036$ m, cube mass, $M_c = 0.110$ kg, $\rho_c = 2433$ kg/m³). The cubes were placed with one face laying flat on the underlayer, but not in a regular pattern, and porosities of 0.25, 0.30 and 0.40 were considered. The tests were performed in a flume with a 1 V:30H foreshore slope and a structure slope of 1 V:1.5H. Tests were conducted for various relative freeboard (R_c/D_n , where R_c represents the

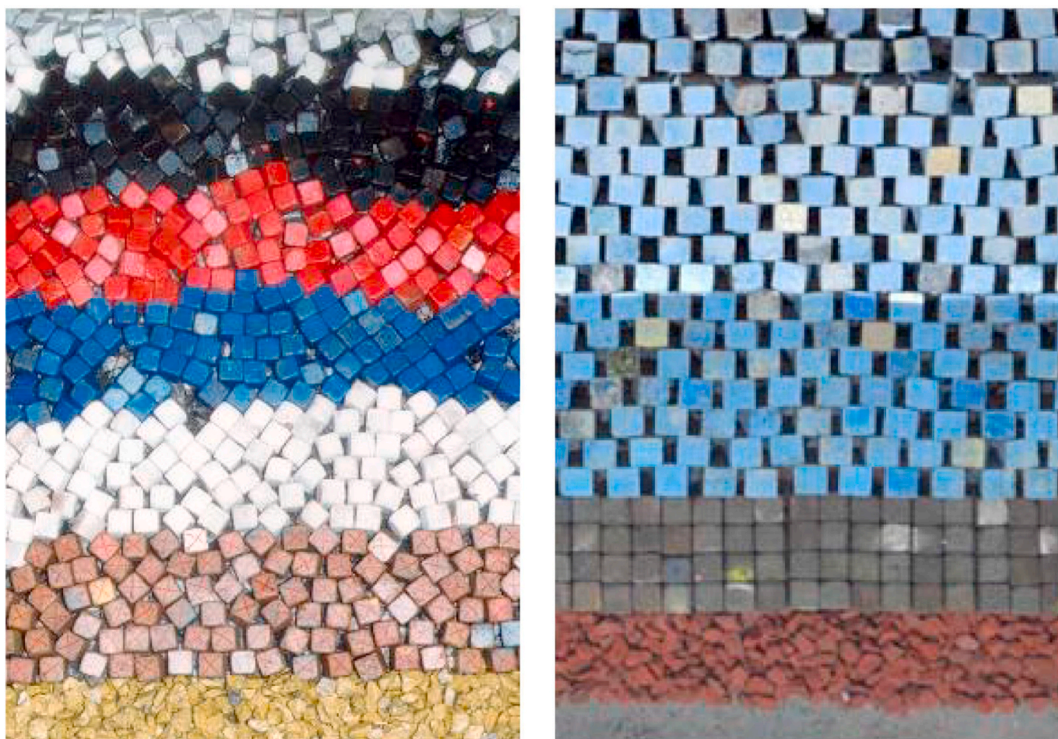


Fig. 3. Example of irregular (left) and regular (right) placement pattern. Courtesy of Van Gent and Luís (2013).

structure freeboard) conditions: 2.1, 5.2 and 11.1. Three different filter mass ranges, $M_{50\text{-filter}}/M_c$ (where $M_{50\text{-filter}}$ represents the median mass of the filter layer) of 1/10–1/5, 1/20–1/10 and 1/50, and three wave steepnesses, $s_{om} = 0.03, 0.05$ and 0.06 , were tested. The test results showed that a lower structure porosity ($p = 0.25$) corresponds to a more stable structure (larger stability number, N_s) with fewer displacements. The larger filter results in a slightly less stable top layer than the mid-filter grading. Tests with the smaller filter showed some erosion of the filter through the gaps of the armour layer. No clear conclusion could be drawn from the results on the influence of the wave steepness on the structure stability. Start of damage and failure occur for lower wave heights for the tests with larger water depth. According to Van Gent et al. (1999), this could be related to the ratio of $H_{2\%}/H_s$ (where $H_{2\%}$ represents the wave heights exceeded by 2% of all wave heights) which is higher for deeper than shallower water conditions and therefore larger waves will impact the structure. Another reason could be associated with the longer slope below still water level (SWL) for the deeper water case. For the same percentage of displaced units below SWL, N_{od} is higher for the deeper water case.

Van Gent et al. (2001) analysed the test results of Triemstra (2000) for a single-layer cube armour with normal-density ($D_n = 0.0443$ m, $M_c = 0.203$ kg, $\rho_c = 2328$ kg/m³) and high-density concrete ($D_n = 0.0221$ m, $M_c = 0.042$ kg, $\rho_c = 3907$ kg/m³). The physical model setup was similar to the one of Van Gent et al. (1999). The best performance (higher stability) was obtained, in general, for a lower structure porosity ($p = 0.25$). Washing out of filter rocks through the gaps in the armour occurred for the smaller filter ($M_{50\text{-filter}}/M_c = 1/75$). The larger filter (1/10th of the cube mass) showed better performance. The results also show that there is a tendency that lower wave steepnesses lead to initial damage at lower wave heights. Initial damage for normal and high-density cubes occurs roughly at the same N_s for a structure porosity of 0.30. For $p = 0.25$ the normal-density cubes showed initial damage at slightly higher stability numbers than the high-density cubes. Similar stability is achieved with less concrete volume when high-density is used.

Van Buchem (2009) carried out eighteen tests in a small-scale physical model for a single-layer of cubes ($D_n = 0.045$ m, $M_c = 0.171$ kg, $\rho_c = 1875$ kg/m³) regularly placed. The foreshore was flat and two structure slopes, 1 V:2H and 1 V:1.5H, were tested for structure porosities of 0.20, 0.28 and 0.35, and wave steepnesses of 0.02, 0.04 and 0.06. A filter layer with $M_{50\text{-filter}}/M_c = 1/20$ – $1/10$ and a relative freeboard of $R_c/D_n = 6.7$ was adopted in all tests. The number of waves in each test was set to 1000 and therefore, if no damage occurred during this period, no damage curves could be obtained. It was found that a gentler structure slope (1 V:2H) was less stable than a steeper slope (1 V:1.5H), which demonstrates the importance of the friction effects on the structure stability. The best results in terms of stability were obtained for a structure porosity of 0.28 (compared to 0.20 and 0.35). Larger damage occurred for higher wave steepnesses (0.04 and 0.06 compared to 0.02).

The Van Gent et al. (1999) physical model experiment was reproduced by Almeida (2013). A single-layer cube ($D_n = 0.036$ m, $M_c = 0.110$ kg, $\rho_c = 2433$ kg/m³) with regular placement and porosities of 0.25 and 0.30 were tested. The results showed that the best results (higher stability) are achieved for a lower structure porosity, $p = 0.25$. Higher porosities result in large gaps at the intersection between the slope and crest. A filter layer grading of 1/20th to 1/10th of the cube mass has shown to be more stable than the 1/50th. Some erosion of the filter layer occurred for a filter of 1/50th of the cube mass. The sensitivity to wave steepness is inconclusive as it was found that the structure is more stable for a wave steepness of 0.06 but the start of damage occurs earlier for a wave steepness of 0.05. It was also found that the failure is more sudden for higher water levels. According to Almeida (2013), this is likely because in the higher water level case, the structure is almost submerged and the initial damage occurs at the crest which then propagates down the slope. The results are generally in agreement with Van Gent et al. (1999) but higher stability numbers for failure were obtained.

The causes for these differences are not entirely clear but can be related to the fact that in Van Gent et al. (1999) the cubes were placed randomly (but with one face lying flat on the underlayer) as opposed to this experiment where a regular placement was implemented. Other factors such as model construction methods, materials used, measurement techniques, and analysis tools can also contribute to the differences in results.

Van Gent (2014) carried out physical model experiments to assess the effect of wave obliquity in various types of armour units. The structure had a single layer armour of cubes ($D_n = 0.025$ m) with a porosity of 0.25 on a 1 V:1.5H slope. The influence of oblique waves was found to be larger for cubes in a single layer than for cubes in a double layer and for rock, likely due to the smoother surface of the armour layer. A more gradual damage progression with increased wave angle was observed.

Van der Lem et al. (2016) presented the results of physical model tests for the Sal Rei breakwater in Boavista Island, Cape Verde. The 2D tests were done for assessing the stability of single layer cubes placed regularly with a porosity of 0.28 on a 1 V:1.5H slope under specific design conditions and not for developing damage curves. They found that the stability number for failure (assumed as $N_{od} > 0.2$) occurred at an approximate value of 2.5. A summary of the range of test conditions from the existing literature is presented in Table 4.

The stability numbers for $N_{od} > 0.05$ and $N_{od} > 0.2$ range from 1.7 to 3.9 and 2.1 to 4.5, respectively, considering all test results available (Fig. 4). There is a relatively wide range of stability numbers obtained in the various studies. The values are derived from an extensive range of test conditions with several tests aimed at performing sensitivity analysis on parameters such as filter layer and porosity. Some of the tested values are in reality not feasible or effective in prototype cases such as porosity of 0.4 or filter mass ($M_{50\text{-filter}}/M_c$) less than 1/20. Fig. 4 shows filtered data that correspond to tests for a structure configuration with a slope of 1 V:1.5H, emerged structure, a porosity of 0.25–0.30 and filter mass larger than 1/20th of the armour unit. The stability numbers for $N_{od} > 0.05$ and $N_{od} > 0.2$ range from 2.3 to 3.9 and 2.5 to 4.5, respectively, which are larger than if all tests are considered.

3.2.2. Wave overtopping

Regularly placed single-layer cubes with very low porosity (~ 0) are considered smooth structures in terms of wave run-up and overtopping. Several overtopping studies have been conducted for smooth-slope structures such as Owen (1980), Owen and Steele (1993), Van Gent

Table 4
Range of physical model test conditions for single-layer cubes.

Reference	Placement	p	$M_{50\text{-filter}}/M_c$	s_{om}	h_{toe} (m)	R_c/D	cot α
Van Gent et al. (1999)	Irregular	0.25	1/	0.03	0.30	2.1	1.5
		0.30	10–1/5	0.05	0.43	5.2	
		0.40	1/	0.06	0.50	11.1	
			20–1/10				
Van Gent et al. (2001)	Irregular	0.25	1/50	0.02	0.30	9.1	1.5
		0.30	1/10	0.04	0.50	18.2	
		0.40	1/75	0.06			
Van Buchem (2009)	Regular	0.20	1/	0.02	0.50	6.7	1.5
		0.28	20–1/	0.04			2.0
		0.35	10	0.06			
Almeida (2013)	Regular	0.25	1/	0.03	0.30	2.2	1.5
		0.30	20–1/10	0.05	0.50	5.2	
			1/50	0.06			
Van Gent (2014)	Irregular	0.25	N/A	0.036 0.048	0.65 0.80	N/A	1.5
Van der Lem et al. (2016)	Regular	0.28	1/	0.01	2.07	4.2	1.5
			17–1/6	0.02			

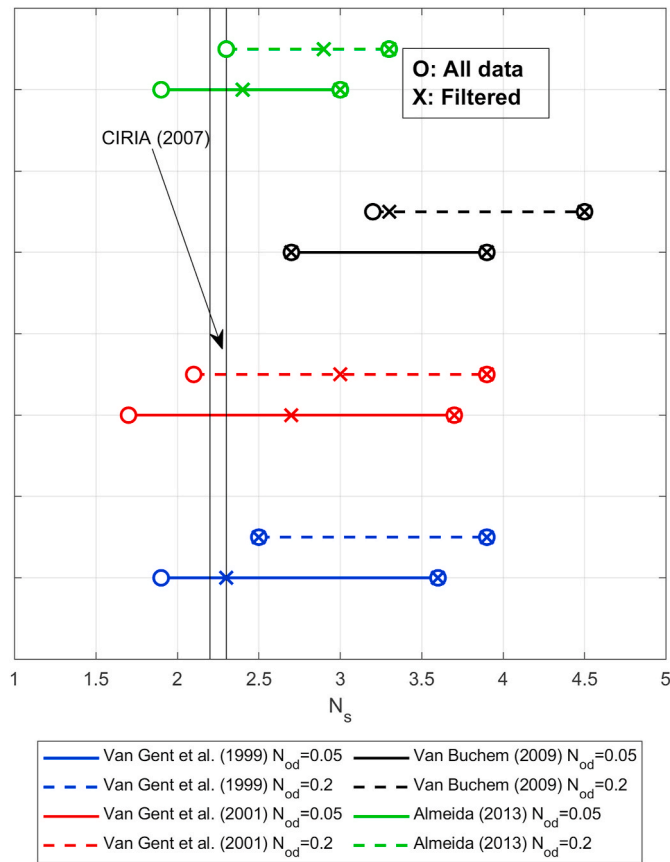


Fig. 4. Stability number ranges obtained from the available literature for $N_{od} > 0.05$ and $N_{od} > 0.2$ for all (o) and filtered (x) data.

et al. (1999). This type of structure has a roughness factor, γ_f , of 1.0 (EurOtop et al., 2007).

A limited number of studies have focused on the evaluation of overtopping performance for breakwaters with a single-layer of cubes regularly placed with low porosity values (0.25–0.35). Bruce et al. (2006) presented the results of Pearson et al. (2004) from an extensive laboratory test programme, which was part of the CLASH project. The tests for a single-layer of cubes ($D_n = 0.030$ m, $M = 0.062$ kg, $\rho_{cubes} = 2361$ kg/m³) adopted a stability number $N_s = 2.2$. A porosity of 0.30 was chosen but the actual built structure had a porosity of 0.35. Tests were carried out for a spectral significant wave height $H_{m0} = H_0$, $0.75H_0$ and $0.5H_0$, where H_0 is the wave height under design conditions derived from the stability number $N_s = 2.2$. A flat seabed was adopted in the model and therefore H_{m0} at the wavemaker is similar to H_{m0} at the toe of the structure. Each wave height was repeated for three offshore wave steepnesses, $s_{op} = 0.02$, 0.035 and 0.05 and two relative freeboard levels, $R_c/H_0 = 1.3$ and 0.8 . Water depths of $2.5H_0$ and $3.0H_0$ were imposed for all tests to avoid wave breaking. The structure slope was 1 V:1.5H and the crest and toe were $3D_n$ wide. The weight of the underlayer material was in the range of 1/5th to 1/15th of the weight of the armour blocks. The best fit of mean overtopping rate, q , between the data and the Van der Meer and Janssen (1994) formula was obtained for $\gamma_f = 0.50$. Bruce et al. (2009) re-analysed the database and obtained $\gamma_f = 0.52$. It is important to highlight that the values presented in this section are different from those of Table 3. The roughness factors in Table 3 are 5% lower as the roughness factor for a smooth slope obtained in Pearson et al. (2004) was 1.05 and therefore all values obtained for the various armour configurations are reduced by 5%. Molines and Medina (2015) re-analysed the CLASH database and derived roughness factors based on various overtopping estimators. The optimum roughness factors were estimated by minimizing the relative mean squared error between

measurements and different γ_f were obtained for the various estimators (Table 5).

There is generally a good fit for the Pearson et al. (2004) test results compared to Van der Meer and Janssen (1994) formula, in particular for $\gamma_f = 0.52$ (Fig. 5). In EurOtop (2018) it is proposed to use the Van der Meer and Bruce (2014) equation with γ_f given by Bruce et al. (2009). However, the overtopping values for a relative freeboard of $R_c/H_{m0} > 2.0$ are under-estimated in this case. A slightly better prediction is obtained with $\gamma_f = 0.53$ (Molines et al., 2015).

Chen et al. (2020) presented wave overtopping results for mild slope (1 V:3H) revetments (impermeable core) with regularly placed single-layer cubes, but these results cannot be directly applied to breakwaters (permeable core and steeper slopes). The study found that the roughness influence is considerably overestimated by the TAW method (TAW, 2002) using the roughness factor presented in Eurotop (2018) and therefore this factor may not be reasonable for dikes with an impermeable core.

3.3. Knowledge gaps and future research

3.3.1. Hydraulic stability

The ranges of stability numbers for single-layer cube armoured breakwaters obtained from available studies present a relatively large variability, which leads to a high degree of uncertainty for preliminary design purposes. This may be explained by several reasons such as cube placement, model construction methods, differences in materials and scale used, different structure configurations or measurement techniques, and analysis tools adopted in the various studies.

Model armour layer responses are usually described using a combination of quantitative and qualitative methods, including visual assessment, not all of which can be covered by generic guidelines (Garcia et al., 2013). The evolution of the armour layer is typically described by quantifying: unit extraction, rocking of units, rotation of units, settlement, and changes in packing density. One of the most common methods for quantifying stability in concrete armour units is the relative damage parameter, N_{od} . N_{od} when applied to double-layer systems, is a relatively objective criterion as it provides valuable information on the damage evolution, which is a gradual process in these structures corresponding to the development of an s-shape profile until exposure of the filter layer.

The classic methodologies to measure damage do not reflect the real behaviour of a single-layer armour system. These methodologies account for one-piece movement if it moves more than one D_n . Thus, they do not allow to measure the settlements and analyse the first phases of damage, which are very important in breakwaters with rigid behaviour (Sande et al., 2018).

N_{od} is not an adequate method for providing quantitative assessments of damage in single-layer cube armoured breakwaters since the extraction of one unit from the armour layer, which could correspond to a very low N_{od} , results in exposure of the filter layer ("failure"), unlike double-layer systems. For example, assuming a $D_n = 0.04$ m, a flume

Table 5

Roughness factors for regularly placed single-layer cubes estimated by various authors.

Reference	Overtopping formula			
	Van der Meer and Jensen (1994)	EurOtop et al. (2007)	CLASH Neural Network	Van der Meer and Bruce (2014)
Bruce et al. (2006)	0.50	–	–	–
Bruce et al. (2009)	0.52	–	–	–
Molines et al. (2015)	0.52	0.59	0.54	0.53

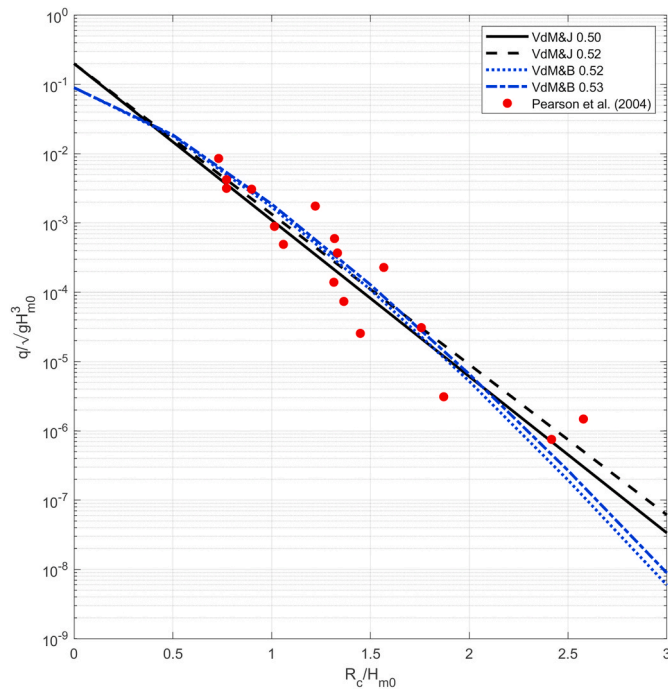


Fig. 5. Mean overtopping discharge (red dots), Pearson et al. (2004), CLASH experiments and lines of best fit for various overtopping estimators. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

width of 1.0 m, the extraction of 1 unit corresponds to a $N_{od} = D_n = 0.04$. This means that the start of damage and failure for this structure occurs for any $N_{od} > 0$ and information on the armour layer evolution leading to the unit extraction is unknown. This method also does not provide detailed information on full armour layer evolution and associated damage as it does not quantify unit rocking, rotation, or changes in packing density along the slope which eventually may lead to extraction or large displacement of armour units.

Small armour displacements (without unit extraction) may lead to exposure of the filter layer and subsequent damage phases may follow up quickly (brittle failure). Failure (exposure of underlayer) can occur for very low values of N_{od} (only one gap of length $D_{n50-filter}$ is sufficient to be considered *failure*). For example, let us consider a cube size of D_n a porosity of 0.25 and a rock filter layer with $M_{cube}/M_{50-filter} = 15$ and $\rho_r = 2650 \text{ kg/m}^3$ ($D_{n50-filter}/D_n = 0.4$). The horizontal gap distance is then $gap/D_n = 0.33$ or $gap/D_{n50-filter} = 0.85$ (Fig. 6). Assuming one cube moves horizontally by a distance of $0.1D_n$, it will create a gap between adjacent cubes of $gap/D_n = 0.43$ or $gap/D_{n50-filter} = 1.10$. In this case, the cube has only been displaced from the original position by $0.1D_n$ (that is typically considered no displacement), but this displacement has caused exposure of the filter layer with the potential consequences of extraction of filter through the gaps in the armour (Fig. 6). Whilst, in this case, it is assumed that a gap larger than $D_{n50-filter}$ could result in extraction of the filter layer, in reality, given the grading curve of the filter layer and random placement of rock, it would be difficult to confirm if all gaps larger than $D_{n50-filter}$ would result in actual extraction of filter rocks or if this would only occur after a certain number of waves. Perhaps the extraction of some of the smaller rocks does not cause any significant impact on the overall slope integrity. Additional testing specifically aiming at understanding this issue would provide essential information on defining the appropriate gap tolerances and the maximum admissible

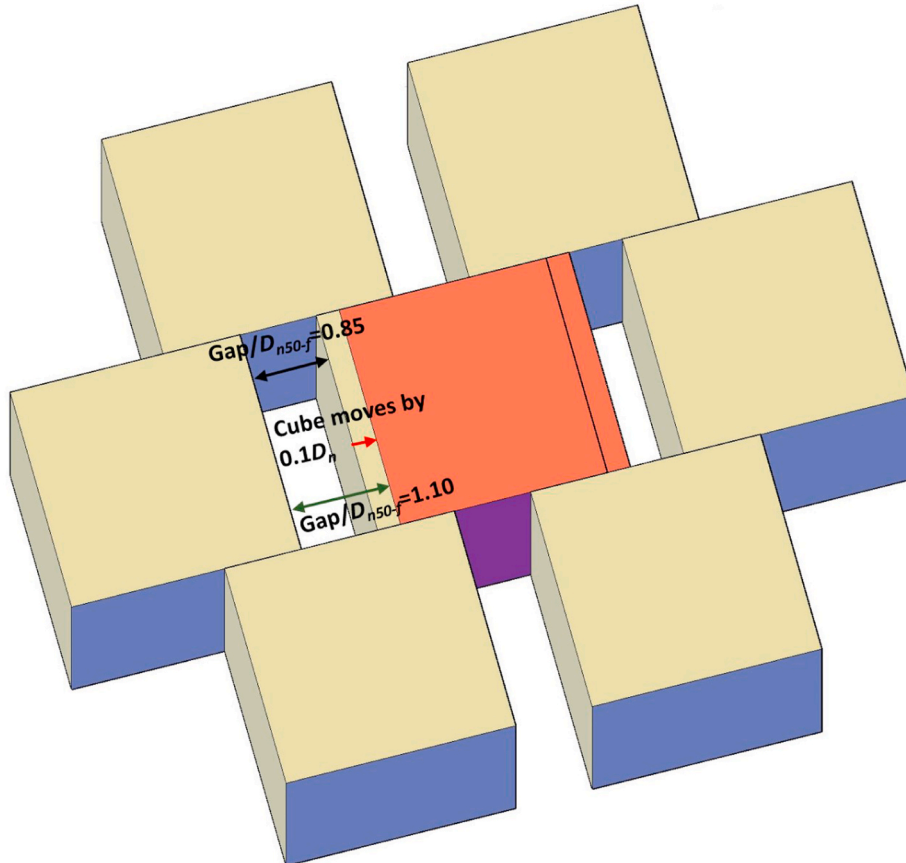


Fig. 6. Example of gap distance between cube armour units and displacement.

limits for the motion of the armour layer blocks, which are supposed to be related to the block placement pattern, selected armour porosity (and subsequent gap distances between units) and grading of the filter material. The gap size in the analysis needs to be defined in terms of size/area but also taking into consideration the vertical/horizontal (V/H) ratio. In an ideal structure, the vertical gap is close to D_n , which, if the horizontal gap distance is small, will unlikely result in the extraction of filter layer rocks. However, rearrangement of armour units may lead to different H/V gap ratios.

N_{od} is defined differently by various researchers. A unit can be considered “displaced” if it moves out of its original location by a certain factor of the distance D_n . This factor is typically assumed as one ($1D_n$ for being considered displacement). Hellinga (2016) used lower factors for the assessment of the damage of single-layer cubes on the rear side of breakwaters (cubes with large displacement (higher than $0.3D_n$) were considered for the calculation of armour damage). The method used in Loman et al. (2012) to quantify damage in a single layer armour of cubes in a revetment for the sea defence of Maasvlakte 2 is more adequate since it considers gaps between cubes exceeding the size of one cube, D_n , in addition to cube displacements of more than $1D_n$, for calculation of N_{od} . This method also accounts for gaps created by settlement or heterogeneous packing. However, gaps smaller than D_n , which are not accounted for, may still result in the exposure of the underlayer and subsequently be considered “failure”. Another limitation in Loman et al. (2012) method is that one gap width of D_n or one cube displaced by a distance of $1D_n$ will most likely result in the exposure of the underlayer (despite N_{od} corresponding to a small value). Niels (2018) highlights an important aspect for regularly placed single layer cube armour units, where the lack of overlay between cubes in the upper and lower rows may displace the cube in the upper row and cause settlement of the rows



Fig. 7. Example of test results showing a cube (marked in red) that fall into the gap of the lower row due to lack of overlapping. Courtesy of Almeida (2013).

above, resulting in possible damage/failure to the structure (Fig. 7). Although the methods described previously are an improvement in comparison to the description of damage levels for double-layer systems, which only take into account the number of displaced/extracted units, it still lacks consistency and generalization to broadly be applied in single-layer cube systems with a regular placement pattern. It also does not provide information on the evolution of the armour unit movement and associated damage level.

Another aspect still not fully investigated and most relevant in construction sites where wave conditions are relatively rough, is the influence of the underlayer profile irregularities on the displacement of units from the armour layer. Van den Berg et al. (2020) analysed the effect of the underlayer irregularities on the stability of the Xbloc^{PLUS} concrete armour unit. It was found that large-scale convex (protruding outwards) undulations on the underlayer influenced the stability of the armour layer and recommendations on underlayer profile tolerances are provided for that type of block. This could be also a relevant aspect to take into account when designing regularly placed single-layer cube armour structures that have relatively strict placement tolerances. On one hand, a larger $M_{50-filter}$ provides more porosity thus increasing hydraulic stability and also allows for larger cube displacements (as gaps can be slightly larger before rocks from the filter layer get extracted through the gaps), but on the other hand the irregularities of the filter layer will likely be larger which can affect the cube placement pattern and consequently reducing the stability of the armour layer.

The above-mentioned aspects are mainly aimed at the trunk slope of the structure, which typically corresponds to the longest section of a breakwater and therefore receives the most attention and focuses on research studies. However, other components of a breakwater such as crest and roundhead are crucial for the overall integrity of a breakwater. Detailed research on crest stability of regularly-placed single layer cube armour structures is not available. In the available literature there is an indication that, unless the breakwater is subject to limited overtopping, the crest units may have to be larger than those of the slope or a different configuration should be implemented to ensure adequate stability. In terms of roundheads, the only information available is in Van der Lem et al. (2016) where it is mentioned that, in contrast to normal practice, the steepness of the breakwater slope at the roundhead as well as the cube size was kept similar to the breakwater trunk.

It is important to quantify the evolution of movements that lead to exposure of the filter layer. Some options proposed for a detailed evaluation of the armour damage evolution are (to be analysed for the top, center, and bottom areas of the slope):

- Relate the number of units displaced (for the various displacement classes, to be defined according to test results) with stability number: this would provide evidence of whether there are many small movements before a large displacement occurs. An analysis of the results of Almeida (2013) show that a cube was only removed from the armour layer after 28% of the armour units moved and a corresponding $N_s = 3.9$ (Fig. 8). Results for other test series (with different porosity and wave steepness) did not show any cube removed from the armour layer for higher relative number of displaced units (32%);
- Relate the number and magnitude of gap distances (or distance between adjacent cubes) between all cubes in the slope through a “regularity index” (RI) and its evolution with the stability number. Being a regular placement, it is relatively straightforward to identify how much the constructed (and subsequent damaged) armour unit placement grid is modified from the ideal (perfect) placement grid. Initial placement may also play an important role in the damage evolution of the armour layer. This “index” could provide information on how well the structure is built and what is the influence on the damage evolution.
- Relate settlement with stability number: Hofland and van Gent (2016) developed a method for quantification of settlement in single-layer systems. Aspects like rocking, toe support, changes in

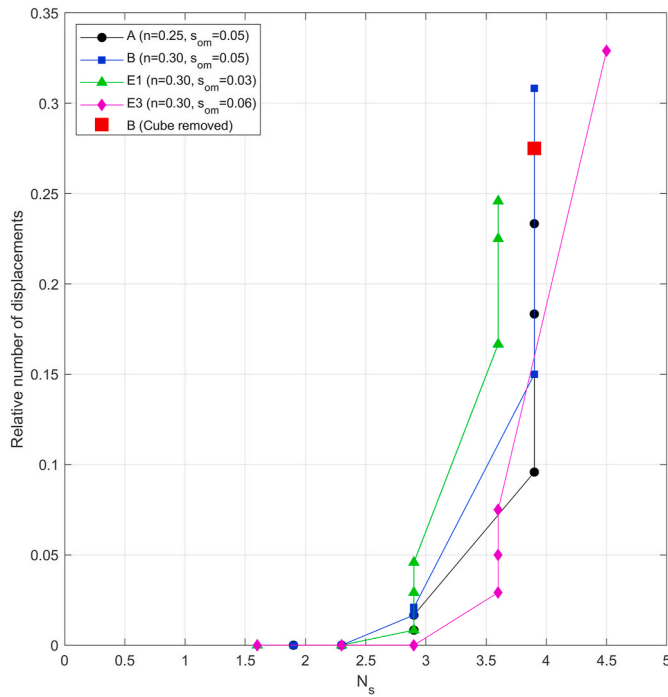


Fig. 8. Evolution of relative number of displacements in the armour layer with stability number (N_s) based on the tests of Almeida (2013).

packing density, movement of the entire armour layer will result in settlement after a test. These results can provide an insight into potential settlement thresholds before a unit is extracted from the armour layer.

Categories of the different damage levels for double-layer systems were proposed by Losada et al. (1986) and Vidal et al. (1991) as follows:

- Initiation of Damage (IDa): a certain number of armour units are displaced from their original position at a distance of or larger than a unit length and/or holes larger than average porous size are clearly appreciable;
- Iribarren Damage (IR): Armour units from the lower armour layer are exposed;
- Initiation of Destruction (IDe): A small number of units is removed from the lower armour layer and the filter is exposed;
- Destruction: Filter layer units are removed.

These definitions are not directly applicable to single-layer systems and therefore a new classification is proposed for IDa. IDa for single-layer (IDa_{SL}) can be defined when units are displaced from their original position (considered as displacements of armour units by less than a unit length) but the filter layer is not exposed (for example, if gaps between units are smaller than filter size, $D_{n50-filter}$) and therefore the structure still provides some level of protection. IR is not applicable for single-layer systems. The definition of IDe for single-layer systems (IDe_{SL}) can be adopted as when the filter layer is exposed due to large holes/gaps in the armour layer. Destruction will occur when the filter material starts to be removed. A flowchart of the proposed levels of damage and associated damage parameters and structure behaviour is presented in Fig. 9, where RI_0 represents the regularity index of the as-built structure. It is recommended that future physical model tests adopt the proposed damage levels definition to try to identify thresholds for stability number, percentage of displaced units and regularity index for the different classes. Sensitivity to the thresholds should be assessed for various structure porosities, wave steepnesses, number of waves and all other parameters that may influence the stability of the armour layer.

Knowledge of the damage evolution and failure processes in this type of structure needs to be investigated more exhaustively using modern techniques that can provide detailed information on individual armour movements as well as the definition of overall parameters that characterize the condition of the whole slope or structure. The studies mentioned in section 3.2.1 made use of photographic overlay methods

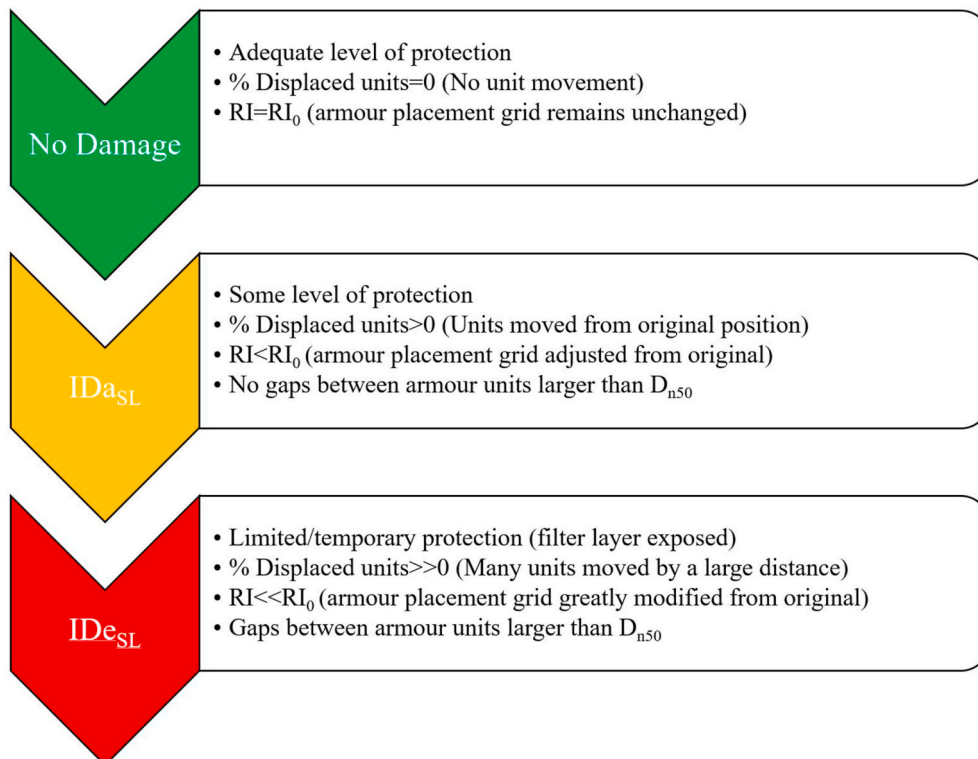


Fig. 9. Alternative damage evolution classification for single-layer cube armoured structures.

for quantification of unit displacements. Recent high-resolution technologies, such as digital stereo photography (DSP) or laser scanners, with typical sub-millimetre accuracy, have been successfully applied to assess damage in rock and concrete armoured breakwaters (Rigden and Stewart, 2012; Molines et al., 2012a; Puente et al., 2014; Shen et al., 2018; De Almeida et al., 2019). These techniques would provide additional information on the damage areas, settlements, variation in porosity along the slope, crest and roundhead, identification of position and configuration of units at trigger points for the initiation of damage.

3.3.2. Wave overtopping

Available wave overtopping studies for regularly placed cube armoured breakwaters are limited to the tests of Pearson et al. (2004). Although this study provides a solid base for research in this type of configuration, additional research is required to assess the influence of armour porosity on wave overtopping rates. This was done for other concrete units (Molines et al., 2012b). Molines and Medina (2015) presented a method for calculating the roughness factor for different armour porosities based on the CLASH database (Van der Meer et al., 2009) and the CLASH Neural Network method (Van Gent et al., 2007). Armour units placed with higher porosity tend to have a smaller roughness factor resulting in lower overtopping rates (Molines and Medina, 2015). The recommended roughness factor for overtopping estimates with a single-layer regularly placed cube armour in EurOtop (2018) is based on the tests of Pearson et al. (2004). However, it is unlikely that a prototype will be constructed with a porosity of 0.35, as tested in Pearson et al. (2004), since it will result in a structure with lower hydraulic stability performance. Better hydraulic stability has been achieved for porosities between 0.25 and 0.30.

The available physical model tests also focused on a specific range of relative freeboard values (0.8 and 1.3). Further investigation on low-crested structures ($R_c/H_{m0} < 0.5$) and low overtopping structures ($R_c/H_{m0} > 1.5$) would be valuable for extending the range of applicability of the empirical formulations (or adjusting these formulations). In general, there is very limited information on the effect of wave obliquity on wave overtopping, particularly for single layer cube armoured breakwaters.

More detailed information on wave overtopping could be obtained through recently developed techniques, which combine video imagery with deep learning algorithms (e.g. Den Bieman et al., 2020). These imagery-based techniques provide additional information in terms of the spatial and temporal domains without the use of any sensors that can have interference with the physical processes. In addition to providing detailed information on wave overtopping, the measurements based on imagery can be useful to understand better the relation between overtopping and rear-side damage.

4. Conclusions

The sizing of armour units and crest levels in rubble-mound structures is typically based on empirical formulations specifically developed for the different types of units. These formulations are based on many laboratory tests and experience with prototypes.

A review of available studies on hydraulic stability and wave overtopping for single-layer cube armoured breakwaters is presented. This is an innovative solution obtained by combining cube units in a single-layer with a regular placement pattern with medium porosity (0.25–0.30). Test results show that this configuration is relatively stable and provides several advantages over other units such as the simple formwork and placement grid, which can be major constraints on construction sites. However, it also has some disadvantages such as the difficult regular cube placement pattern under wave action, the brittle failure after the start of damage associated with single-layer armour behaviour and higher concrete volumes compared to other single-layer systems.

The review of available literature indicates a relatively wide range of stability numbers that lead to a high degree of uncertainty for

preliminary design purposes. This raises the need for additional research similar to what is available for double-layer cube armoured structures. Another limitation associated with this type of structure is related to the inappropriate methods currently used for quantification of damage evolution that are similar to the ones applied in double-layer concrete armour unit structures. Regularly placed single-layer cube armoured structures are not as “flexible” as double-layer armour structures, but are also not as “rigid” as friction type concrete armour unit (such as Seabee and Shed) structures. Therefore, one of the key research questions related to hydraulic stability is “how much is the structure allowed to move between the initiation of damage and initiation of destruction?” Knowledge of the damage evolution and failure processes for this configuration needs to be investigated in more detail using modern techniques, such as laser scanners, that can provide additional information on armour unit movements and overall armour layer behaviour. Various parameters proposed in this paper could be useful to provide a better insight into the damage evolution processes. Additional research aspects relate to other components of the structure, such as the crest and roundhead, or other configurations, where little or no investigation has been carried out to date.

Limited research was conducted for wave overtopping in single-layer cube armoured breakwaters. Roughness factors from empirical formulas in the available design guidelines for wave overtopping estimates are based on a limited number of tests for a single structure configuration with a porosity of 0.35. Porosities lower than 0.30 are typically preferred to increase the structure hydraulic stability and therefore additional information on the influence of porosity on wave overtopping is essential.

Further research and physical model testing will provide more confidence in the evaluation of hydraulic stability and wave overtopping for single-layer cube armour with a regularly placed pattern. The tests should aim at assessing other structure configurations, shallow foreshores (depth-limiting conditions), seabed steepness, toe/berm height/width and armour and filter placement tolerances.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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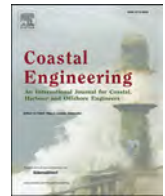
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APPENDIX II – Vieira *et al.* (2021A)



New developments in assessment of wave overtopping on single-layer cube armoured breakwaters based on laboratory experiments

Filipe Vieira^{a,b,*}, Francisco Taveira-Pinto^{b,c}, Paulo Rosa-Santos^{b,c}

^a Department of Biology, Chemistry and Environmental Sciences, American University of Sharjah, Sharjah, United Arab Emirates

^b Department of Civil Engineering, Faculty of Engineering of the University of Porto, Porto, Portugal

^c Interdisciplinary Centre of Marine and Environmental Research of the University of Porto, Matosinhos, Portugal

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ABSTRACT

Wave overtopping is a critical aspect to take into consideration when designing a coastal structure. An adequate crest geometry (a combination of height and width) is required to maintain the required acceptable safety levels for the population and infrastructure behind the structure. This paper presents the wave overtopping results of laboratory experiments for single-layer cube (SLC) armoured breakwaters with a regular placement pattern. The experiments considered different incident wave conditions and two armour layer porosities, 29% and 33%. Mean wave overtopping discharges obtained were generally higher than the ones observed in previous experiments for the same armour configuration. Available methods shown some limitations in predicting the measured overtopping discharges suggesting the relevance of other factors. It is observed that, by including the ratio of wavelength to the armour crest width in an alternative overtopping formula, the mean flow rate predictions are improved. It was also found that mean overtopping discharges increase by 16–42% due to a decrease in armour porosity with negligible differences in reflection coefficients. A different initial seed number in the wave generation series can lead to differences in wave overtopping discharges up to a factor of two. This work aims to contribute to the knowledge of wave overtopping in SLC armoured breakwaters considering the limited available information for this type of armour configuration.

1. Introduction

Wave overtopping is an important aspect in the design and performance assessment of a rubble mound structure. An appropriate structure crest configuration (the combination of height, width and/or crest elements) is essential to ensure adequate levels of safety for people and infrastructure. This requires that the level of hazard, and its probability of occurrence, is assessed, allowing appropriate action plans to be devised to ameliorate risks arising from overtopping. Such plans can be to limit wave overtopping during severe storm events, but plans can sometimes also have to be made to exclude people and/or vehicles during severe storm events when overtopping may occur (EurOtop, 2018).

IPCC (2019) predicts an average global mean sea level rise by 2100 of 0.43 m–0.84 m. This impact, together with a generally positive trend in mean and storm wave heights around the world (e.g., Aboobacker et al., 2017; Reguero et al., 2019; Vieira et al., 2020b) will result in

detrimental effects to the world's coastlines. It is therefore crucial to better understand wave-structure interactions to improve wave overtopping predictions.

Coastal structures are typically protected by an outer armour that consists of a double or single layer system. Single-layer concrete armour units have been developed over the last six decades to reduce concrete consumption and construction time, without compromising hydraulic stability and wave overtopping flow rates. The use of single-layer cubes (SLC) with a regularly placed pattern and low porosity on the seaside of the breakwater is a recent concept. Studies on hydraulic stability and wave overtopping are relatively few compared to other units and, to the authors knowledge, only one prototype (Fig. 1) with such configuration has been built to date (Van der Lem et al., 2016; Vieira et al., 2020a). Several physical model studies for single-layer cube armoured breakwaters with irregular and regular placement patterns have been conducted to date (Van Gent and Spaan, 1998; d'Angremond et al., 1999; Van Gent et al., 1999, 2001; Van Buchem, 2009; Almeida, 2013; Van

* Corresponding author. Department of Biology, Chemistry and Environmental Sciences, College of Arts and Sciences, American University of Sharjah, PO BOX, 26666, Sharjah, United Arab Emirates.

E-mail address: fvieira@aus.edu (F. Vieira).

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Fig. 1. Sal-Rei breakwater. Courtesy of Van Gent and van der Werf (2017).

Gent, 2014; Van Gent and Luís, 2013; Van der Lem et al., 2016; Van Gent and van der Werf, 2017), but most of them were focused on assessing the hydraulic stability of the armour layer and do not provide information on overtopping discharges.

1.1. Mean wave overtopping discharges

Mean overtopping discharges for “non-breaking”, “surging” wave conditions in relatively simple geometries can be estimated using the empirical formula of Van der Meer and Janssen (1994) (hereafter referred to as “VDMJ94”). The formula was derived from several data sets (e.g., Owen, 1980; Van der Meer and De Wall, 1990; Van der Meer and De Wall, 1993), which are included in the CLASH (CLASH, 2004) database (De Rouck et al., 2009), and is valid for structure slopes, $\cot\alpha$, between 1 and 4 and relative freeboards, R_c/H_{m0} , between 0.5 and 3.5:

$$q_{VDMJ}^* = \frac{q}{\sqrt{gH_{m0}^3}} = a_1 \cdot \exp\left(-a_2 \frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_h \gamma_f \gamma_\beta}\right) \quad (1)$$

where q_{VDMJ}^* represents the non-dimensional overtopping discharge, q the mean overtopping discharge, H_{m0} the spectral significant wave height, R_c the crest freeboard (the difference between the still water level and the crest elevation), γ_b the berm reduction factor, γ_h the shallow foreshore reduction factor, γ_f the roughness reduction factor and γ_β the angle of wave attack reduction factor. a_1 and a_2 are empirical coefficients with values of 0.2 and 2.6, respectively.

Van der Meer and Bruce (2014) derived updated empirical formulas [Eq. (2)] to represent better the mean overtopping discharges. These formulas give almost the same wave overtopping as the original formula, Eq. (1). Only for low and zero freeboards, these formulas will be better than VDMJ94 (Van der Meer and Bruce, 2014). For simple armoured slopes EurOtop (2018) recommends using Eq. (2) corrected with the crest width reduction factor, C_r , proposed by Besley (1999), Eq. (3).

$$q_{VDMB}^* = \frac{q}{\sqrt{gH_{m0}^3}} = b_1 \cdot \exp\left(-\left(b_2 \frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_h \gamma_f \gamma_\beta}\right)^{b_3}\right) \quad (2)$$

$$C_r = \min(1; 3.06 \cdot \exp(-1.5G_c / H_{m0})) \quad (3)$$

where b_1 , b_2 and b_3 are empirical coefficients with values of 0.09, 1.5 and 1.3, respectively, and G_c is the armour crest width.

Van Gent et al. (2007) developed an artificial neural network (ANN₀₇) to predict the average overtopping discharge for a variety of wave and water level conditions and taking into account complex structure configurations based on the CLASH database. The neural network consists of a multi-layer network, with 14 dimensionless input parameters (physical conditions and structure geometry), 20 hidden neurons, and one output parameter (average overtopping discharge, q_{ANN}). ANN₀₇ applied resampling techniques to 500 networks for the assessment of the uncertainties in the mean overtopping discharge predictions. It was shown that the agreement between the predicted overtopping discharge and the measured overtopping discharge is well acceptable and accurate when compared to the datasets used. An

updated neural network (Zanuttigh et al., 2016), ANN₁₆, is presented in EurOtop (2018). ANN₁₆ was developed and trained with additional tests to the CLASH database resulting in more than 13,000 tests. More recently, Den Bieman et al. (2020) developed a prediction method based on a specific machine learning technique, gradient boosting decision trees, and trained and tested it using the CLASH database. The results show that this method outperforms existing neural networks by reducing the error on the prediction of the CLASH database by a factor of 2.8. The accuracy of any machine learning prediction is greatly related to the number and quality of the data used to develop and train the tool. In the case of SLC armoured breakwaters, only 18 test results from the experiments of Pearson et al. (2004) are part of the training database.

1.2. Influence of armour porosity on overtopping

The packing density of an armour layer is frequently a key factor affecting construction cost, concrete consumption and breakwater safety (Medina et al., 2014). Armour porosity affects not only armour roughness and overtopping, but also armour hydraulic stability (Molines and Medina, 2015). Armour porosity, p , can be defined as the volume of voids within an armour layer.

Molines et al. (2012) presented the influence of armour porosity on wave overtopping in cube and Cubipod armoured breakwaters. Test results indicate that the armour porosity did not significantly affect overtopping discharges for standard construction porosities (37%–46% for cubes and 37%–42% for Cubipods). The study concluded that the tested porosity range might be too small to assess properly the influence on overtopping.

Molines and Medina (2015) used the CLASH database to fit a linear model for estimating the roughness factor based on armour porosity without considering the armour unit geometry or number of layers. The linear model $\gamma_f = 1 - 1.25p$ was shown to fit reasonably well in all cases except the double-layer rock armour which has a lower than expected roughness factor.

Whilst it may not be easy to calculate armour porosity in single or double interlocking armour configurations, it is relatively straightforward in single layer cube armour layers with a regular placement grid.

1.3. Wave overtopping in single-layer cube armoured breakwaters

Few studies have focused on the evaluation of the performance of wave overtopping in permeable rubble mound structures with an armour of single-layer of cubes regularly placed (Vieira et al., 2020b). Pearson et al. (2004) presented test results for a single-layer of cubes ($D_n = 0.030$ m, $M = 0.062$ kg, $\rho_{cubes} = 2,361$ kg/m³) in a regularly placed pattern with a porosity of 0.35. Tests were carried out with spectral significant wave heights, H_{m0} , equal to H_0 , $0.75H_0$ and $0.5H_0$, where H_0 is the significant wave height under design conditions derived from the stability number $N_s = 2.2$. A flat seabed was adopted in the model. Each wave height was repeated for three offshore wave steepnesses, $s_{op} = 0.02$, 0.035 and 0.05 (with $s_{op} = H_{m0}/(gT_p^2/2\pi)$, where g represents the acceleration due to gravity and T_p the peak wave period) and two relative freeboard levels, $R_c/H_0 = 1.3$ and 0.8 . All tests resulted in

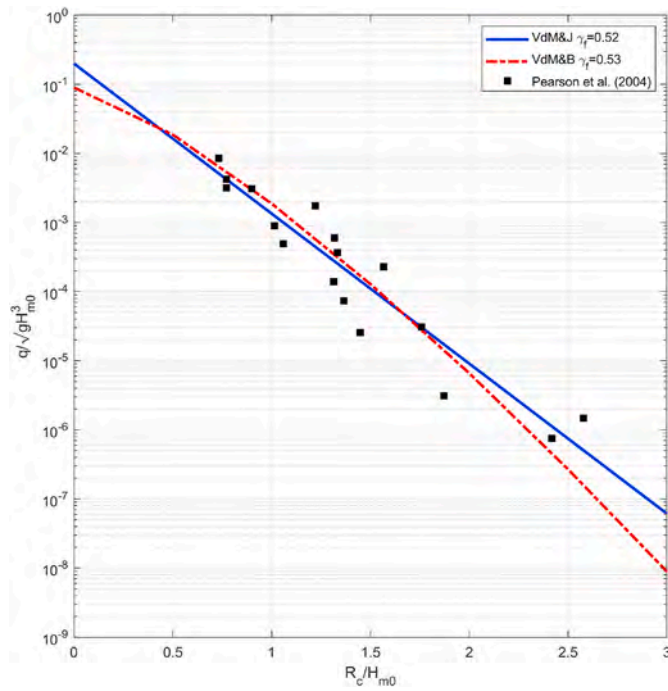


Fig. 2. Mean overtopping discharge from Pearson et al. (2004) (black squares), and lines of best fit for various overtopping formulas.

Table 1

Estimated roughness factors for regularly placed single-layer cubes.

Reference	Overtopping method		
	Van der Meer and Janssen (1994)	Neural Network (Van Gent et al., 2007)	Van der Meer and Bruce (2014)
Bruce et al. (2009)	0.52	–	–
Molines and Medina (2015)	0.52	0.54	0.53

non-breaking wave conditions. The structure slope, $\cot\alpha$, was 1.5 and the armour crest and toe width were $3D_n$ wide. The weight of the underlayer material was in the range of 1/5th to 1/15th of the weight of the armour blocks. Eighteen tests for single-layer cube armoured breakwaters were carried out with two-thirds of these for $R_c/H_{m0} \leq 1.5$ (Fig. 2). The results show that generally larger overtopping discharges were obtained when compared to other single-layer armour units.

Bruce et al. (2009) re-analyzed the database from Pearson et al. (2004) experiments to derive roughness factors to be included in the VDMJ94 formula [Eq. (1)]. The best fit for mean overtopping discharge between the data and the VDMJ94 formula was obtained for $\gamma_f = 0.52$ (Fig. 2). Roughness factors for various overtopping prediction methods for single-layer cube armoured breakwaters are presented in Molines and Medina (2015) (Table 1 and Fig. 2).

This paper presents the experimental results of wave overtopping for single-layer cube armoured breakwaters with a regular placement

pattern. Vieira et al., 2020b; Vieira et al., 2020a highlighted a knowledge gap in research of wave overtopping for this armour configuration. The main limitations identified are the reduced number of data available when compared to other armour units, and that the data was derived from a single structure geometry with constant porosity. The objective of this study is therefore to partially fill this knowledge gap, contribute to a better description of the overtopping process for this type of armour configuration, and to quantify the effects of armour porosity and initial seed number on mean overtopping discharges. The paper is organized as follows: after an introduction and literature review on wave overtopping in Section 1, Section 2 describes the laboratory experiments conducted in this study. The analysis and discussion of test results are provided in Section 3. Section 4 presents the main conclusions of the study.

2. Materials and methods

2.1. Physical model tests

The physical model tests were carried out at the Hydraulics Laboratory of the Hydraulics, Water Resources and Environment Division (SHRHA) of the Faculty of Engineering of the University of Porto. A 1.6 m wide flume was constructed with guide walls within the wave basin (28 m long, 12 m wide, and 1.2 m deep) to test simultaneously two different cross-sections (0.8 m wide each) with similar reflection characteristics, at a geometric scale of 1:35 (Fig. 3). An 8 mm thick acrylic glass sheet was installed between the tested models from the crown wall area to a distance of approximately 2 m seaward of the toe of the structure. Two 0.75 m wide piston-type paddles of the 12 m wide HR Wallingford wave generation system were used to generate and absorb reflected waves, making use of its active wave absorption module. A flat seabed followed by a 4.8% slope and another flat foreshore were built in front of the breakwater model (Fig. 4). The breakwater slope is 1V:1.5H ($\cot\alpha = 1.5$) (Fig. 5) and the armour layer was built with porosities (p) of 0.29 and 0.33. The structure consisted of a permeable rock core ($D_{n50} = 1.1$ cm), a filter layer ($D_{n50} = 1.5$ cm) and a single layer of cubes ($D_n = 3.5$ cm) placed by hand in a regularly placed pattern. A crown wall was placed behind a $3D_n$ (10.5 cm) wide armoured crest. The top of the armour crest (A_c) and crown wall (R_c) are at the same level. The presence of an impermeable structure on the lee side of the armour crest results in lower overtopping discharge rates compared to structures without this element. This is because part of the overtopping waves will then penetrate through the crest armour (EurOtop, 2018). Van Doorslaer et al. (2018) proposes a method to account for different armour and crown wall relative levels in the overtopping formulas. The characteristics of the materials used in the breakwater model construction are presented in Table 2. An overview of the model is depicted in Fig. 6.

The water surface elevation was measured using resistive type wave probes (accuracy of 0.4 mm). Three probes were placed in the deeper area, near the wavemaker, and another two sets of three probes near each of the breakwater sections. Probe distances varied according to the incident wave conditions in order to capture the range of frequencies between 0.5 and $2.0 f_p$ (where $f_p = 1/T_p$). Water surface elevations were recorded at a frequency of 40 Hz and incident and reflected waves were separated using the method of Mansard and Funke (1980), which uses simultaneous records of the free surface elevations measured by three aligned wave gauges. Water was collected via a 0.6 m wide overtopping chute sloping down from the top of the crown wall into overtopping

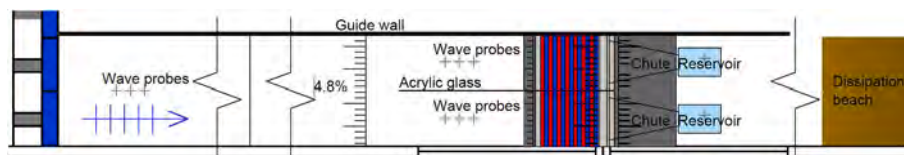


Fig. 3. Plan view of wave flume created within the FEUP wave basin.

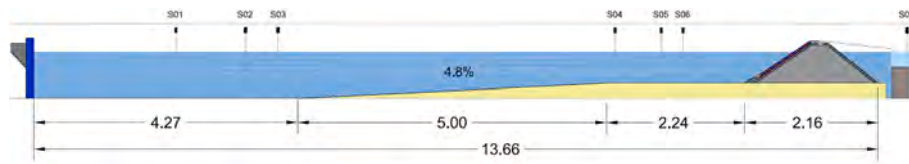


Fig. 4. Wave flume longitudinal cross-section (dimensions in m, model scale).

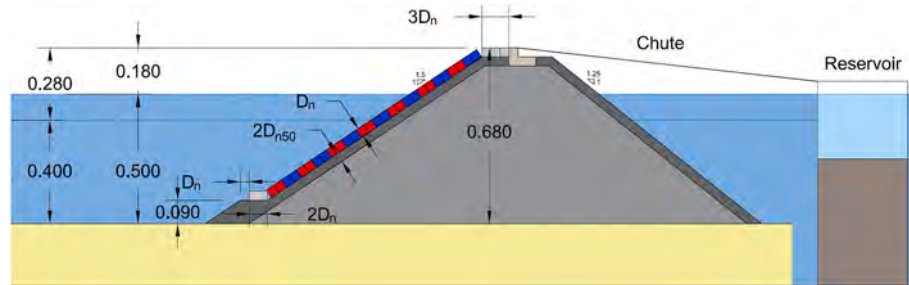


Fig. 5. Breakwater model cross-section (dimensions in m, model scale).

Table 2
Characteristics of the breakwater layers.

Layer	D_{n50} (cm)	ρ_r (kg/cm ³)	M_{50} (g)
Armour (cubes)	3.50	2.38	102.1
Filter	1.46	2.65	8.2
Core	1.13	2.65	3.8

reservoirs. The overtopping discharge was measured by a water surface elevation probe placed inside a perforated PVC pipe within the overtopping reservoir to minimize oscillations in the measurements. When needed, a pump, installed inside the reservoir, was activated to empty the reservoirs during the tests, as in [Cabral et al. \(2020\)](#).

2.2. Test programme

Five test series (B, C, O, P and Q) were completed for two simultaneous armour layer porosities resulting in thirty-one data points. A constant wave steepness (0.02 and 0.05) in the wave generation zone, s_{op} , was adopted with the significant wave height progressively increasing within each test series ([Table 3](#)). Irregular waves defined by a JONSWAP spectrum with a peak enhancement factor $\gamma = 3.3$ were used

in all tests. These waves were generated by the filtered white noise technique for a sequence length of $2^{13}-1$ pulses. The same temporal sequence of waves was used in the tests having the same peak wave period and seed number. In test series Q, with similar incident wave characteristics and water level to P, a different initial seed number was imposed. The tests duration varied between 14 min and 28 min, corresponding to approximately 1,000 waves for each test. Two water depths were tested, 0.74 m and 0.64 m at the wave generation zone, and 0.50 m and 0.40 m at the model area, respectively, resulting in crest freeboards, R_c , of 0.18 m and 0.28 m. Significant wave heights at the structure varied between 7.5 and 13.0 cm; mean energy wave periods, $T_{m-1,0}$, between 1.02 and 1.89 s; relative water depths, h/H_{m0} (where h represents the water depth in front of the structure), between 3.1 and 6.6; relative crest freeboards, R_c/H_{m0} , between 1.4 and 3.7 and surf similarity parameters, defined as $\xi_{1,0} = \tan\alpha / (H_{m0}/L_{m-1,0})^{0.5}$ (where $L_{m-1,0}$ is the wavelength calculated with $T_{m-1,0}$ at the structure), between 2.8 and 4.4. The full details for each test are provided in [Table 5](#).

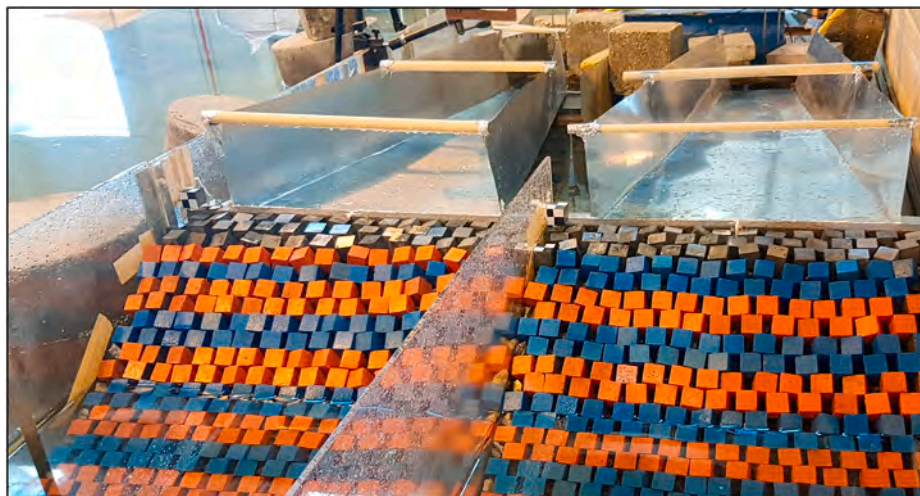


Fig. 6. Overview of the breakwater models with armour porosities of 0.33 (left) and 0.29 (right).

Table 3

Range of parameter values for each test series. The range of values from Pearson et al. (2004) for single-layer cubes are presented in the last row for comparison.

Series	P	s_{op}	H_{m0} (cm)	$T_{m-1,0}$ (s)	h/H_{m0}	R_c/H_{m0}	$\xi_{-1,0}$	No. tests
B	0.29 0.33	0.02	9.6–13.0	1.60–1.84	3.1–4.2	2.2–2.9	4.2–4.3	6
C	0.29 0.33	0.05	7.5–12.9	1.04–1.28	3.1–5.3	2.2–3.7	2.8–3.2	7
O	0.29 0.33	0.05	7.5–12.1	1.02–1.34	4.1–6.6	1.5–2.4	3.0–3.2	6
P	0.29 0.33	0.02	7.6–12.9	1.45–1.89	3.9–6.6	1.4–2.4	4.4	6
Q	0.29 0.33	0.02	7.7–13.0	1.45–1.89	3.8–6.5	1.4–2.4	4.3–4.4	6
CL (Pearson et al. (2004))	0.35	0.02 0.035 0.05	4.5–9.7	0.79–1.54	2.3–5.5	0.7–2.6	2.8–4.2	18

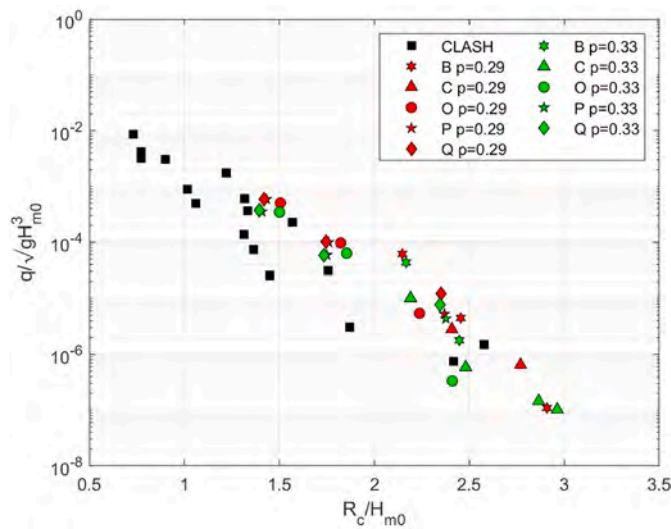


Fig. 7. Non-dimensional mean overtopping discharges as a function of the relative freeboard from CLASH and FEUP tests.

3. Results and discussion

3.1. Mean overtopping discharge

The non-dimensional mean overtopping discharges, $q^* = q/\sqrt{gH_{m0}^3}$, obtained in the current experiments (series B, C, O, P and Q, hereafter referred to as “FEUP”) were compared against the results from Pearson et al. (2004) (hereafter referred to as “CLASH”) for single-layer cube armour structures in terms of relative freeboard (Fig. 7). Despite the expected variability associated with overtopping measurements, it is observed that in general the mean overtopping flow rates obtained in FEUP tests are higher than those from CLASH, with differences of one order of magnitude in some cases. The difference in predictions to the CLASH tests can be attributed to various factors such as the different model geometry, the range of tested values, the model construction effects, and the measurement and analysis procedures. Bakker et al. (2018) also found that several overtopping results for tests using the single-layer Xbloc® armour unit sometimes deviate from the results of Pearson et al. (2004). They found that overtopping flow rates are influenced to some extent by the seabed slope, wave steepness, and breakwater crest width. It was also suggested that the relatively large waves in some tests could contribute to larger than expected overtopping flow rates due to the reduced relative armour layer roughness and permeability of filter and core layers. This may occur when measuring overtopping while carrying out stability tests with increasing wave heights until damage of the armour layer occurs.

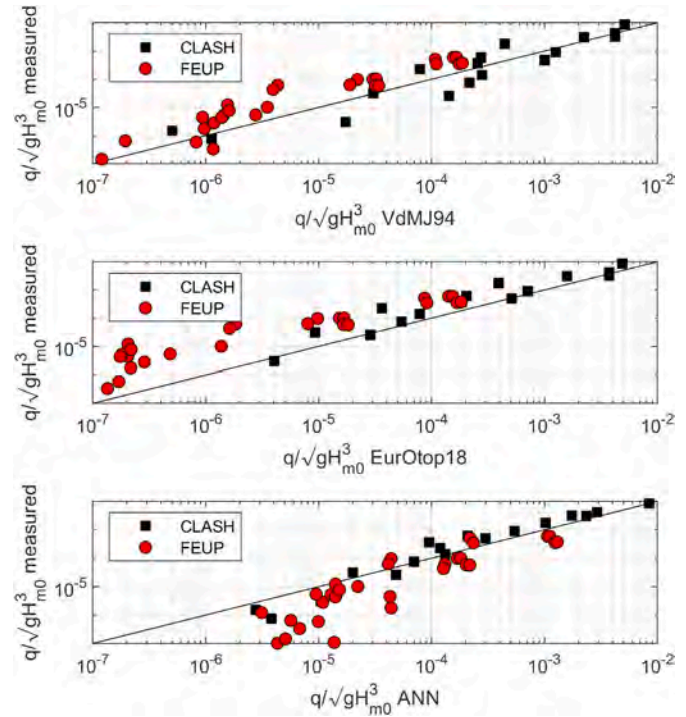


Fig. 8. Comparison between non-dimensional mean overtopping discharge from experiments (FEUP and CLASH), VDMJ94 (top), EurOtop (middle) and ANN (bottom).

Fig. 7 only depicts the relation between q^* and R_c/H_{m0} . Other parameters may also play a relevant role in the estimation of the mean overtopping discharge. It is important to highlight that the CLASH data for single-layer cube armoured breakwaters is relatively limited (18 tests for a single model configuration) and therefore variability in overtopping discharges associated with other model geometries are expected. The current tests contribute to extending the range of incident conditions and structure geometry for SLC armoured breakwaters, in particular for relative freeboards larger than 1.4.

The non-dimensional mean overtopping discharges from FEUP and CLASH were compared against the VDMJ94 empirical formulation, with a $\gamma_f = 0.52$ following estimates from Bruce et al. (2009) and Molines and Medina (2015), EurOtop (2018) and the Artificial Neural Network, ANN₁₆ (Fig. 8). The FEUP overtopping discharges are generally under-estimated by VDMJ94 and EurOtop (2018) in particular for higher values of q^* ($q^* > 10^{-5}$). The ANN provides better estimates for $q^* > 10^{-5}$ but over-estimates the overtopping discharges for lower values. Larger scatter in measurements from available databases typically occurs for lower values of q^* . It is suggested that the differences

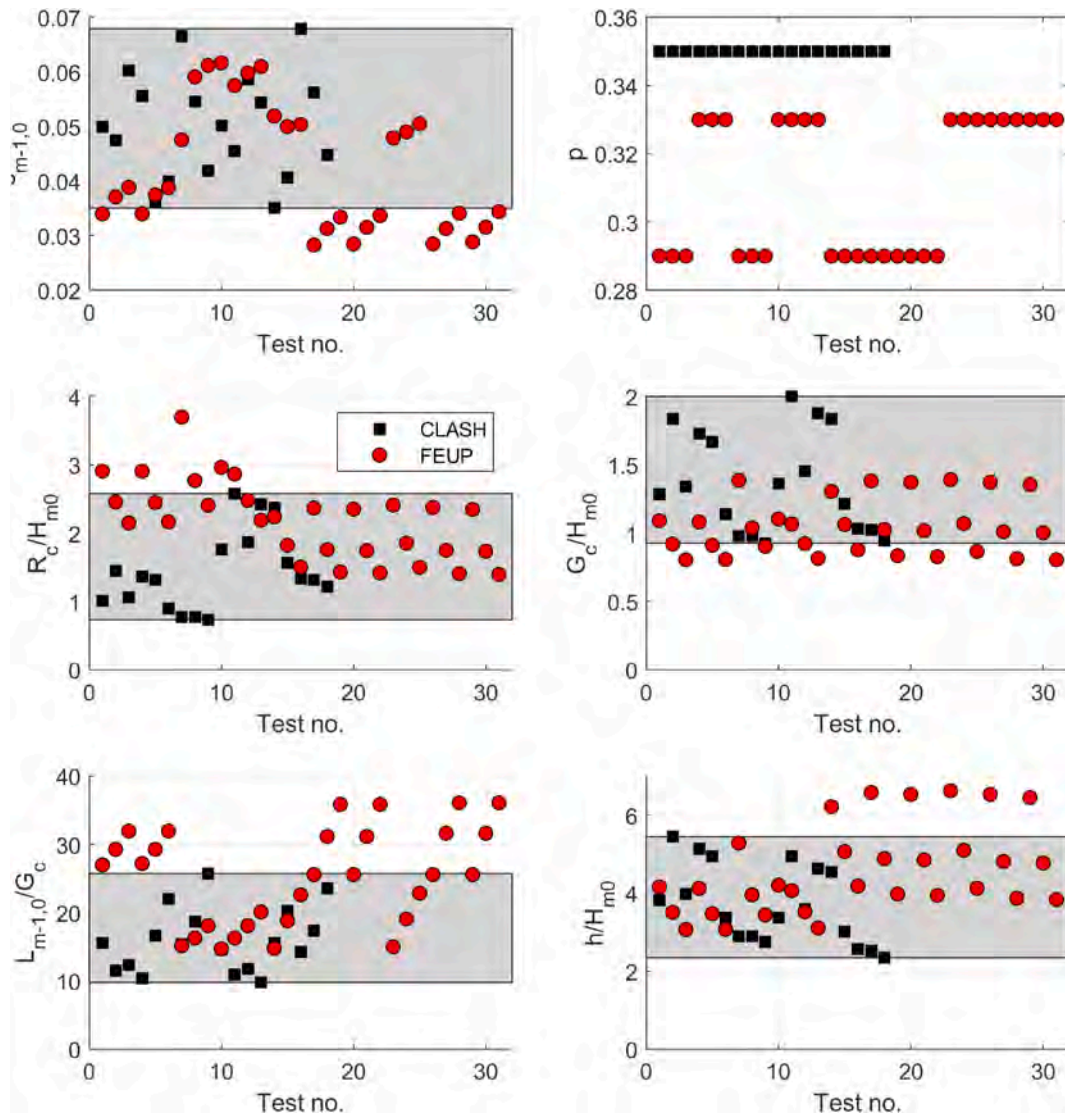


Fig. 9. Range of non-dimensional parameters for the CLASH (black squares) and FEUP (red circles) experiments. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

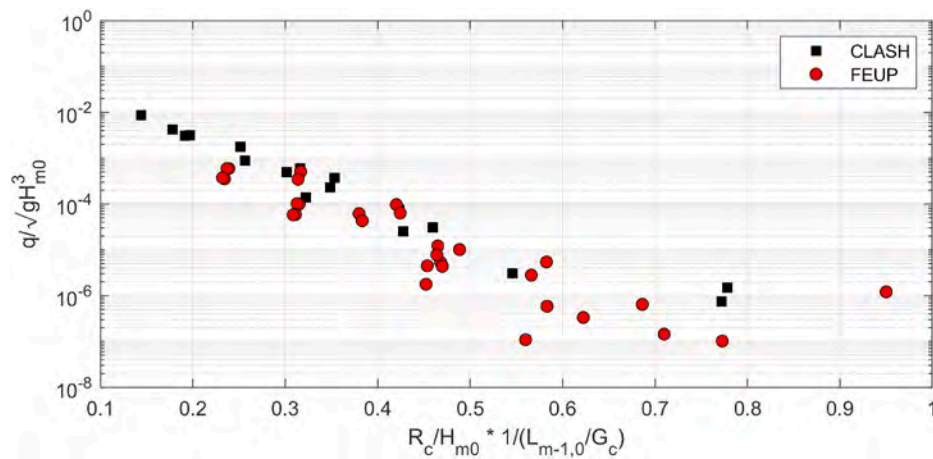


Fig. 10. Non-dimensional overtopping rate as a function of the modified non-dimensional freeboard.

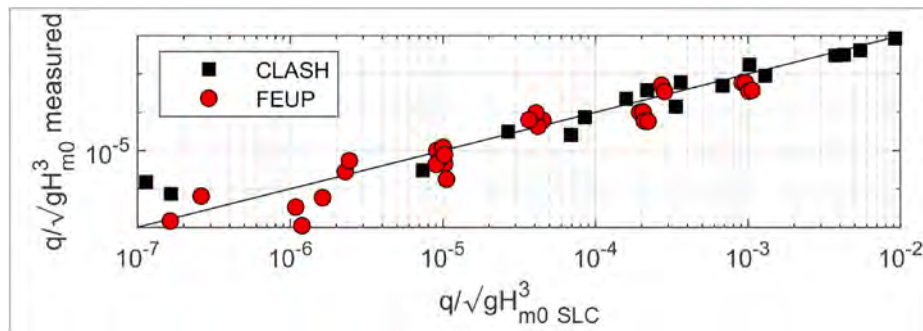


Fig. 11. Comparison between non-dimensional mean overtopping discharge from experiments and alternative formula.

Table 4

Error metrics for the various overtopping prediction method.

Formula	nMSE (%)	r
VDMJ94	23.7	0.92
EurOtop	79.1	0.92
ANN ₁₆	26.7	0.92
SLC (this study)	18.4	0.92

observed could be related to the configuration and range of parameters used in FEUP tests being beyond the range of tests used for training of the ANN. The current dataset can therefore contribute to improving the

accuracy of future versions of the ANN, since the amount of data relative to SLC would more than double.

The analysis of Fig. 8 shows that the overtopping prediction methods applied (EurOtop and ANN) have limitations in predicting mean overtopping discharges for the FEUP tests, likely due to the small number of tests available for single-layer cube armoured structures. The results obtained with VDMJ94 are relatively accurate despite some scatter.

3.2. Alternative mean wave overtopping formula

3.2.1. Assessment of relevant parameters

The range of non-dimensional parameters for each of the

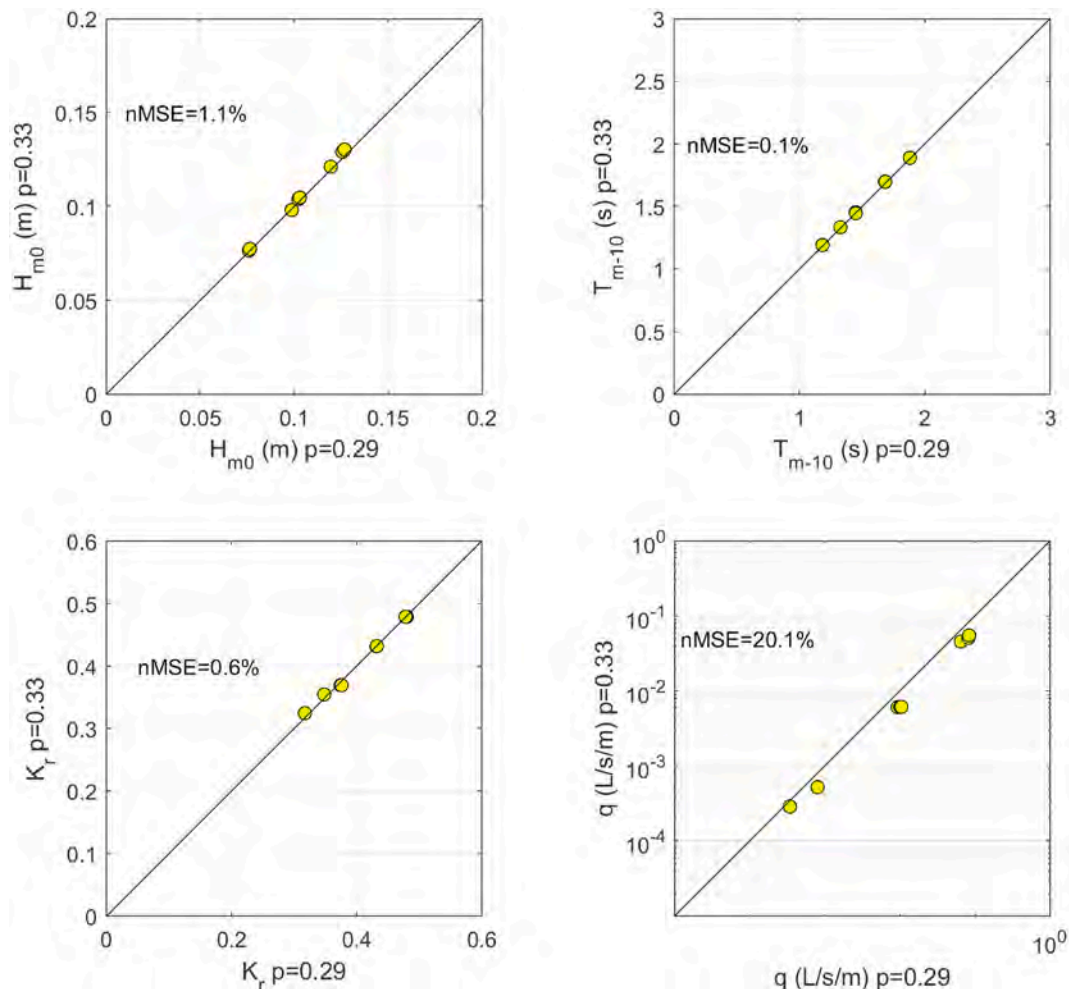


Fig. 12. Scatter plot of H_{m0} , $T_{m-1,0}$, K_r and q for porosities 0.29 and 0.33.

Table 5

Wave conditions and mean overtopping discharge obtained in FEUP tests.

Test	p	H_{m0} (m)	$T_{m-1,0}$ (s)	T_p (s)	R_c	Q (L)	Logtime (s)	q (L/s/m)
B01	0.29	0.096	1.60	1.65	0.28	0.01	1325	1.02E-05
B02	0.29	0.114	1.71	1.77	0.28	0.49	1503	5.42E-04
B03	0.29	0.130	1.84	1.90	0.28	9.16	1684	9.07E-03
B01	0.33	0.097	1.61	1.65	0.28	0.00	1325	0.00E+00
B02	0.33	0.115	1.71	1.83	0.28	0.20	1503	2.18E-04
B03	0.33	0.130	1.84	1.90	0.28	6.41	1684	6.34E-03
C01	0.29	0.076	1.06	1.07	0.28	0.04	880	7.87E-05
C02	0.29	0.101	1.11	1.14	0.28	0.04	969	6.50E-05
C03	0.29	0.116	1.19	1.22	0.28	0.22	1065	3.48E-04
C01	0.33	0.095	1.04	1.07	0.28	0.00	880	9.39E-06
C02	0.33	0.098	1.11	1.14	0.28	0.01	969	1.40E-05
C03	0.33	0.114	1.19	1.22	0.28	0.04	1065	7.04E-05
C04	0.33	0.129	1.26	1.28	0.28	0.99	1129	1.47E-03
O01	0.29	0.080	1.02	1.07	0.18	0.20	859	3.88E-04
O02	0.29	0.099	1.18	1.25	0.18	5.60	992	9.41E-03
O03	0.29	0.120	1.33	1.38	0.18	43.64	1109	6.56E-02
P01	0.29	0.076	1.45	1.51	0.18	0.25	1215	3.46E-04
P02	0.29	0.102	1.68	1.77	0.18	8.59	1403	1.02E-02
P03	0.29	0.126	1.88	1.97	0.18	77.90	1568	8.28E-02
Q01	0.29	0.077	1.45	1.51	0.18	0.59	1215	8.08E-04
Q02	0.29	0.103	1.68	1.77	0.18	8.89	1403	1.06E-02
Q03	0.29	0.127	1.88	1.97	0.18	79.09	1568	8.41E-02
O01	0.33	0.075	1.03	1.07	0.18	0.01	859	2.16E-05
O02	0.33	0.098	1.19	1.25	0.18	3.63	992	6.10E-03
O03	0.33	0.121	1.34	1.38	0.18	30.49	1109	4.58E-02
P01	0.33	0.077	1.45	1.51	0.18	0.21	1215	2.89E-04
P02	0.33	0.104	1.70	1.77	0.18	5.20	1403	6.18E-03
P03	0.33	0.129	1.89	1.97	0.18	48.25	1568	5.13E-02
Q01	0.33	0.078	1.45	1.51	0.18	0.38	1215	5.24E-04
Q02	0.33	0.105	1.70	1.77	0.18	5.19	1403	6.17E-03
Q03	0.33	0.130	1.89	1.97	0.18	51.94	1568	5.52E-02

experiments with SLC (CLASH and FEUP) is depicted in Fig. 9 to better understand the potential influence of each parameter on the overtopping results. It can be seen that for several FEUP tests the various non-dimensional parameters are outside the range in CLASH. Therefore, a combination of several parameters can potentially justify the differences in the results between CLASH and FEUP. This is further analyzed using the Partial Least Squares (PLS) regression (Wold et al., 1983). The regression problem, i.e., how to model one or several dependent variables, responses, Y , by means of a set of predictor variables, X , is one of the most common data-analytical problems in science and technology (Wold et al., 2001). PLS focuses on maximizing the variance of the dependent variables explained by the independent ones instead of reproducing the empirical covariance matrix (Haenlein and Kaplan, 2004). PLS can analyse data with strongly collinear (correlated), noisy, and numerous predictor variables, and simultaneously model several response variables (Wold et al., 2001). More details on the PLS method can be found in (Kuhn and Johnson, 2013). The VIP (Variable Importance in the Projection) scores obtained by the PLS regression is an important measure of each explanatory variable or predictor. A VIP score greater than one is generally used as a criterion for variable selection (Chong and Jun, 2001).

The relative importance of each parameter (or dependent variables, Y) R_c/H_{m0} , G_c/H_{m0} , h/H_{m0} and $L_{m-1,0}/G_c$ in the prediction of mean overtopping discharge (or predictor variable, X) was assessed by applying the partial least squares regression (de Jong, 1993) function in MATLAB®. In this application of the PLS, two variables, R_c/H_{m0} and $L_{m-1,0}/G_c$, obtained VIP scores greater than 1 and therefore these variables are used to derive an alternative wave overtopping formula for single-layer cube armoured breakwaters. The parameter $L_{m-1,0}/G_c$ relates horizontal dimensions and takes into account the effect of the wave period at the structure location, which is derived from spectra that were subject to transformations due to shoaling and friction, from the wave generation to the nearshore areas. Both dimensional parameters, wave period and crest width, were previously found to have an impact on wave overtopping. Owen (1980) proposed the inclusion of the wave

period in the wave overtopping estimation for mildly sloping dikes ($1 \leq \cot \alpha \leq 5$), and Besley (1999) derived a reduction factor, Cr , to take into account the relative armour crest width (G_c/H_{m0}). The influence of the parameter $L_{m-1,0}/G_c$ can be explained as a longer wavelength (for the same wave height) will reach further leeward over the crest of a breakwater than shorter wavelengths, thus resulting in more water overtopping the structure. The dependence of the non-dimensional overtopping rate with the modified relative crest freeboard (including $L_{m-1,0}/G_c$) depicted in Fig. 10 indicates a relative small scatter in the data except for low overtopping rates (or high relative freeboards). Although based on a larger set of data than the one used in CLASH, the conclusions of the present analysis may need to be adjusted as additional datasets for single-layer cube armoured breakwaters become available.

3.2.2. Overtopping formula

The proposed formula for estimation of mean overtopping discharges for non-breaking, single layer cube armoured breakwaters (q_{SLC}^*) is:

$$q_{SLC}^* = \frac{q}{\sqrt{gH_{m0}^3}} = a \cdot \exp \left(-b \frac{R_c}{H_{m0}\gamma_f} \frac{1}{\sqrt[3]{L_{m-1,0}/G_c}} \right) \quad (4)$$

The proposed parameters a and b in Eq. (2) were estimated using a linear best-fit analysis and result in 0.20 and 6.45, respectively. The roughness coefficient used in the equation is the same as derived by Bruce et al. (2009) and Molines and Medina (2015), $\gamma_f = 0.52$. The error metrics to evaluate the performance of the alternative prediction method, normalized mean square error ($nMSE$) and correlation coefficient (r), are defined as:

$$nMSE = \frac{\sum_{i=1}^n (y_i - x_i)^2}{Var(x)} \quad (5)$$

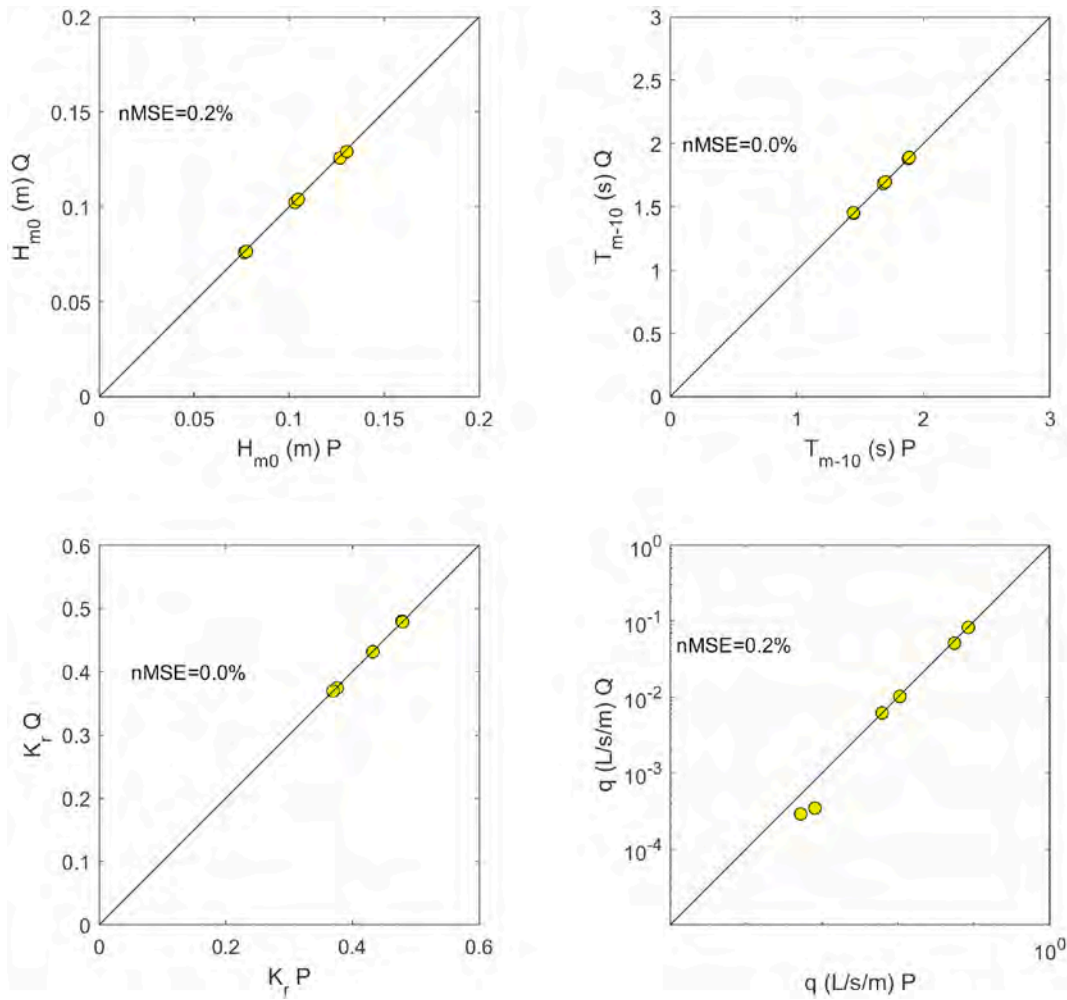


Fig. 13. Scatter plot of H_{m0} , $T_{m-1,0}$, K_r and q for different initial seed number (Series P and Q).

$$r = \frac{\sum_i (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_i (x_i - \bar{x})^2 \sum_i (y_i - \bar{y})^2}} \quad (6)$$

where x and y represent the measured and modelled values, respectively, defined by $\log(q^*)$, where $q^* = q/(gH_{m0}^3)^{0.5}$, i is the data index and n is the number of samples.

Fig. 11 and Table 4 show that the proposed method can represent better the mean overtopping discharges for the entire range of values from the laboratory experiments and shows less scatter around the line of best fit. The same r is obtained for all methods but the $nMSE$ is smaller for the proposed formula. It is important to highlight that the proposed method for SLC has currently limited applicability as it was derived from a reduced dataset when compared to VDMJ94, EurOtop and ANN₁₆, which can be widely applied.

3.3. Influence of armour porosity

The influence of armour layer porosity on wave overtopping was investigated by testing breakwater sections with different porosity values, 0.29 and 0.33. The results (Figs. 7 and 12 and Table 5) show that there is a clear trend in the overtopping discharges, where lower porosity results in larger values of q , unlike what was observed by Molines et al. (2012). This is likely associated with the armour layer configurations tested. In Molines et al. (2012), wave overtopping was measured for double armour layers with random placement. By comparing test series O, P and Q for the two porosities, and excluding

test O01, where the difference in incident wave heights between the two porosities exceeds 3%, the differences in mean overtopping discharges are within 16–42%. This difference is relatively small when compared to other factors. For example, De Rouck and Geeraerts (2005) mentions that repositioning of armour units (in a double layer antifer cube armour) significantly affects measured overtopping discharges and small changes in the positioning of armour elements may change the overtopping discharges by a factor of two and more.

The test results also show that the differences in armour layer porosities tested have a limited influence on the reflection coefficient, K_r . It is suggested that this is likely due to the following: the range of armour porosities tested is relatively small and therefore a minor influence on the wave reflection is anticipated; furthermore, although it is expected that a lower armour porosity results in increased wave reflection, the higher overtopping discharge associated with the lower armour porosity will result in slight less wave energy to be reflected from the slope.

3.4. Influence of initial seed number

A different initial seed number in the wave generation for series P and Q was applied, whilst the incident wave conditions and spectrum shape remained the same. The results (Fig. 13 and Table 5) show that the influence of this factor on the mean overtopping discharge is relatively small ($nMSE < 1\%$) with the larger differences occurring for lower mean overtopping rates. This is expected since it corresponds to the region where typically there is more scatter in results. The difference for prototype mean overtopping rates above 1 L/s/m is within the range of

0–8%. For the cases with smaller mean overtopping rates (tests P01 and Q01 with $q_{prototype} < 1$ L/s/m) the maximum differences are between 80 and 130%. Romano et al. (2015) carried out physical model tests for a rock armoured breakwater with different initial seed numbers. They found that, when the relative freeboard is larger than 1.4, the variability of the overtopping discharge may reach one order of magnitude for the same wave condition. The larger differences were observed for the higher relative crest freeboards (lower overtopping rates).

4. Conclusions

Wave overtopping data for single layer cube armoured breakwaters with regular placement is sparse. Therefore, thirty-one tests were carried out, for a combination of incident wave and water level conditions and armour layer porosities, to study wave overtopping for this type of armour layer configuration.

The obtained experimental results have shown that available overtopping prediction methods have some limitations in representing satisfactorily the measured mean overtopping discharges. Therefore, an analysis of the relative importance of the various parameters on mean overtopping discharges was carried out and it was found that R_c/H_{m0} and $L_{m-1,0}/G_c$ explain the majority of the variance in overtopping in the measurements. An alternative overtopping formula was derived and better performance, compared to other methods (Van der Meer and Janssen, 1994; EurOtop, 2018; ANN, Vieira et al., 2020b; Vieira et al., 2020a), was achieved. The purpose of this formula is mainly to explain the variability in results compared to other studies available in the literature for the same armour configuration. In fact, in spite of the large relative increase of the database (from 18 tests presented in CLASH database to 49), the formula is still limited by the scarce data from which it was derived and it will only be applicable for SLC armoured breakwaters within the geometries and incident conditions tested. Improvements to the formula are expected as more data for this armour configuration becomes available.

The laboratory test results indicate that small variations in porosity (from 0.33 to 0.29) result in increased overtopping discharges in the range of 16–42%, however, no significant variations in the reflection coefficient were observed. The influence of a different initial seed number for the wave generation signal was carried out and it was found that the differences in mean overtopping discharges are up to a factor of two for higher relative freeboards.

The present dataset aims to contribute to extending the available wave overtopping database for single-layer cube armoured breakwaters. The data can be applied in the training of an updated version of ANN overtopping.

Additional studies considering different structure geometries and focused on areas where fewer data is available will be useful to improve the understanding of the overtopping process in single-layer cube armoured breakwaters. The tests were limited to 2D and therefore the influence of wave obliquity in 3D models is a topic of consideration for future research.

CRedit author contribution statement

F. Vieira: Funding acquisition, Conceptualization, Methodology, Investigation, Visualization, Formal Analysis, Writing - original draft.

P. Rosa-Santos: Methodology, Resources, Writing - Review & Editing.

F. Taveira-Pinto: Funding acquisition, Conceptualization, Writing - Review & Editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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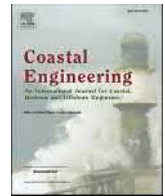
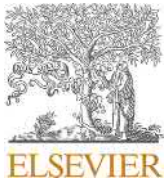
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APPENDIX III – Vieira *et al.* (2021B)



Damage evolution in single-layer cube armoured breakwaters with a regular placement pattern

Filipe Vieira^{a,b,*}, Francisco Taveira-Pinto^{b,c}, Paulo Rosa-Santos^{b,c}

^a Department of Biology, Chemistry and Environmental Sciences, American University of Sharjah, Sharjah, United Arab Emirates

^b Department of Civil Engineering, Faculty of Engineering of the University of Porto, Porto, Portugal

^c Interdisciplinary Centre of Marine and Environmental Research of the University of Porto, Matosinhos, Portugal

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ABSTRACT

Single-layer cube (SLC) armoured breakwaters with a regular placement pattern are a relatively recent concept despite the cube being one of the oldest concrete units available. Limited investigation on the performance and hydraulic stability of this armour type is available to date. This work presents a quantitative assessment of the damage evolution for SLC armoured breakwaters with a regular placement pattern based on 2D physical modelling tests carried out at the University of Porto, Portugal. Tests were carried out for different armour porosities, wave steepnesses, and armour crest levels.

A new method for automatically determining cube positions for each test was developed with image-processing techniques using advanced machine-learning tools. This method greatly reduces processing time since several images can be processed in batch mode.

Innovative parameters to describe armour layer behaviour were derived and proposed for assessment and quantification of the damage evolution: exceedance probability of cube movements, mean grid orthogonality, and relative damage number for SLC. New damage level classification and criteria for initiation of damage are proposed for each of the parameters assessed. This analysis is focused on the initial stages of the damage, which are critical in designing new structures.

1. Introduction

Rubble-mound coastal structures protect coastlines, roads, railways, industrial and residential buildings, and ports. These structures are composed of several layers of rock with an outer layer of rock or concrete units. Knowledge of a rubble-mound armour performance in terms of hydraulic stability is therefore essential for the design of new structures and the upgrading of existing ones to cope with climate change effects on Sea Level Rise (SLR) and wave storminess. IPCC et al. (2019) predicts an average global mean SLR by 2100 of 0.43 m–0.84 m. This impact, together with a generally positive trend in mean and storm wave heights around the world (e.g., Aboobacker et al., 2017; Reguero et al., 2019; Vieira et al., 2020a) will result in detrimental effects on the world's coastlines (Vieira et al., 2021).

This paper presents the experimental results of hydraulic stability for single-layer cube armoured breakwaters with a regular placement pattern. The objective of this study is to contribute to a better

description of the armour damage evolution for SLC and propose new methods and thresholds for damage evaluation and classification. The analysis is focused on the initial stages of damage as failure and destruction of the breakwater was not achieved due to limitations in generating larger waves under the adopted experimental set-up. The paper is organized as follows: after an introduction and literature review on hydraulic stability and SLC in Section 1, Section 2 describes the laboratory experiments conducted in this study. The analysis and discussion of test results is provided in Section 3. Section 4 presents the main conclusions and recommendations of this study.

1.1. Hydraulic stability of concrete armour units

Damage in concrete armour layers is often quantified by the relative damage number, N_{od} , proposed by Van der Meer (1988), as the actual number of units displaced in a width (along the longitudinal axis of the structure) of armour unit nominal diameter or equivalent cube length,

* Corresponding author. Department of Biology, Chemistry and Environmental Sciences, College of Arts and Sciences, American University of Sharjah, PO BOX 26666, Sharjah, United Arab Emirates.

E-mail address: fvieira@aus.edu (F. Vieira).

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D_n , and defined as:

$$N_{od} = \frac{N_o}{B / D_n} \quad (1)$$

where N_o represents the number of units displaced by a distance equal to D_n , and B the width of the tested section.

For double-layer structures, no damage at all, $N_{od} = 0$, is a very strict criterion and armour layers designed with this criterion will result in large concrete units. For rock layers, some settlement and small displacement is considered in the “start of damage” definition. For $N_{od} = 0.5$ a similar situation is found and this is a more economical criterion than no damage at all (Van der Meer, 1999).

Categories of the different damage levels for double-layer structures were also proposed by Losada et al. (1986) and Vidal et al. (1992), as follows:

- Initiation of Damage (*Ida*): a certain number of armour units are displaced from their original position at a distance of or larger than a unit length and/or holes larger than average porous size are appreciable;
- Iribarren Damage (*IR*): Armour units from the lower armour layer are exposed;
- Initiation of Destruction (*Idc*): A small number of units is removed from the lower armour layer and the filter is exposed;
- Destruction: Filter layer units are removed.

This provides a better description of the damage evolution in a double-layer structure compared to the “start of damage” or “failure” classification. The application of the above definitions of N_{od} and damage levels to single-layer cube (SLC) armoured structures present some limitations as described further in the following section.

1.2. Single layer cube armoured breakwaters

Single-layer concrete armour units have been developed over the last decades to reduce concrete consumption and construction time, without compromising the hydraulic stability of the structure and tolerable wave overtopping discharge (Vieira et al., 2021). Single-layer units such as Accropode™, Core-loc™, Xbloc®, and Cubipod® mainly rely on interlocking, whilst Cob, Diahitis, Seabee and Shed units rely on friction between units to provide stability.

Van Gent and Luís (2013) presented a summary of findings for breakwaters with cubes in a single-layer, which rely on both weight and friction as stabilizing forces. They concluded that it is a feasible solution and maybe economically competitive against other single-layer units. The main advantages of a single-layer of cubes compared to other one-layer structures are: the simplicity of formwork in fabrication, high structural strength, patent-free, smaller casting yard area requirements, safer handling using pressure clamps instead of rope slings, simple placement grid, lower likelihood of breaking units during construction; greater production rate and easier repair works. Disadvantages are associated with the low resilience offered if some blocks are extracted or settlement occurs, negligible interlocking between units, higher concrete consumption and overtopping discharge (Bruce et al., 2009; Molines et al., 2015; Vieira et al., 2021), and difficulties during construction in the presence of waves. Van Gent et al. (1999) concluded that the performance of SLC armoured structures is good and that the strength of single top-layers seems to be a combination of strength due to weight (as for double top-layers), strength due to contact-forces between adjacent blocks (as for placed block revetments) and less wave attack on single units due to a smoother surface of the entire slope.

A regular placement pattern of concrete armour units is simpler and typically preferred in construction sites (Reedijk et al., 2018). Several studies showed that a regular placement generally increases hydraulic stability when compared to randomly placed configurations (e.g., Hald

et al., 1998; Frens et al., 2008; Safari et al., 2018; Yuksel et al., 2020).

An armour of SLC with a regularly placed pattern on the seaside of the breakwater was recently implemented in prototype (Fig. 1, Van der Lem et al., 2016).

Several physical model studies on hydraulic stability of SLC armoured breakwaters were carried out to date (Van Gent and Spaan, 1998; d'Angremond et al., 1999; Van Gent et al., 1999; 2001; Van Buchem, 2009; Almeida, 2013; Van Gent and Luís, 2013; Van Gent, 2014; Van der Lem et al., 2016; Van Gent and van der Werf, 2017). The best performance in terms of stability was achieved for a structure slope of 1 V:1.5H, a porosity of 0.25–0.30, and a filter mass larger than 1/20th of the armour unit (Vieira et al., 2020b).

N_{od} applied to double-layer structures is a relatively objective criterion as it provides valuable information on the damage evolution, which is a gradual process in these structures corresponding to the development of an s-shape profile until exposure of the filter layer (Vieira et al., 2020b). The classic methodology to quantify damage does not reflect the real behaviour of a single-layer armoured structure. This methodology accounts for one-piece movement if it moves more than one D_n . Thus, they do not allow to measure the settlements and analyze the first phases of damage, which are very important in breakwaters with rigid behaviour (Sande et al., 2017). The stability of single-layer armoured structures can be described by a fixed stability number for “start of damage” (defined as $N_{od} = 0$) and “failure” (defined as $N_{od} > 0.5$) damage levels (Van der Meer, 1999). Adopting the classical definition for N_{od} , this means that units can move up to a maximum distance of D_n before N_{od} becomes larger than zero. A value of $N_{od} = 0.2$ for “failure” has been adopted for SLC in previous studies. This may not be adequate since $N_{od} > 0$ indicates that at least one cube has moved by a distance of D_n or extracted, which implicitly results in exposure of the filter layer (Vieira et al., 2020b).

The damage level definitions described in section 1.1 are also not applicable to single-layer structures (Vieira et al., 2020b) and therefore the following damage level definitions for SLC are proposed:

- Ida_{SLC} : units are displaced from their original position (considered as the movement of armour units by less than a unit length) but the filter layer is not exposed and therefore the structure still provides some level of protection;
- *IR*: not applicable for single-layer structures;
- Idc_{SLC} : the filter layer is exposed due to large holes/gaps in the armour layer;
- *Destruction*: the filter material starts to be removed.

2. Materials and methods

2.1. Physical model tests

The 2D laboratory experiments were conducted in the wave basin (28 m long, 12 m wide, and 1.2 m deep) of the Hydraulics Laboratory of the Hydraulics, Water Resources and Environment Division (SHRHA) of the Faculty of Engineering of the University of Porto. Guiding walls were used to construct a 1.6 m wide flume within the wave basin. An 8 mm thick acrylic glass sheet was installed between the tested models (each 0.8 m wide) from the crown wall area to a distance of approximately 2 m seaward of the toe of the structure to simultaneously test two different breakwater cross-sections with similar reflection characteristics (Fig. 2). The model was built according to the Froude similarity. Turbulent flow conditions should occur through the armour layer (satisfied reasonably by Reynolds number, $Re_D > 30,000$ (Hughes, 1993; Wolters et al., 2010), with Re_D related to flow within the armour and defined as $Re_D = \sqrt{gH_s D_n} / \nu$, where g represents the acceleration of gravity and ν the kinematic viscosity) to avoid viscous scale effects. The viscous scale effects were assumed negligible as the Re_D , varied between 29,900 and 41,100.



Fig. 1. Sal-Rei breakwater, Cape Verde. Courtesy of Van Gent and van der Werf (2017).

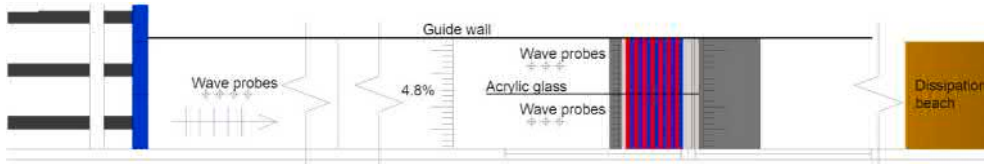


Fig. 2. Plan view of wave flume created within the wave basin of FEUP.

Two 0.75 m wide piston-type paddles from HR Wallingford wave system were used to generate and absorb reflected waves through its active wave absorption system. The water depth at the wave generation and model areas was 0.64 and 0.40 m, respectively, with a 4.8% slope transition between the flat seabed areas (Fig. 3). The model bathymetry was constructed with wood templates placed along the slope and model area. The templates were backfilled with gravel to a level approximately 2 cm below the top of the templates. The 2 cm gap was filled with a skin of concrete grout to match the seabed profile defined by the templates. The breakwater model had a slope ($\cot\alpha$) of 1 V:1.5H (Fig. 4). The armour layer was built with porosities (p) of 0.24, 0.29, and 0.33. Two structure heights were tested in the model (0.63 and 0.68 m) for a fixed water level of 0.40 m in front of the breakwater. The structure consisted of a permeable rock core ($D_{n50} = 1.13$ cm, where D_{n50} represents the nominal stone diameter with a density (ρ_c) of 2,650 kg/m³ and defined by $D_{n50} = (M_{50}/\rho_c)^{1/3}$, and $M_{50} = 3.8$ g, where M_{50} represents the armour unit mass), a filter layer ($D_{n50} = 1.46$ cm and $M_{50} = 8.2$ g) and a single layer of cubes ($D_n = 3.5$ cm and $M_{50} = 102.1$ g) placed by hand in a regularly placed pattern. Approximately 730 cubes were fabricated for this study. The cubes were placed in coloured bands to enable an easier visualization of movements and damage during the experiments. A crown wall was placed behind a $3D_n$ wide armoured crest. The characteristics of the materials used in the breakwater model construction are presented in Table 1, where D_{85} and D_{15} represent the equivalent cube size corresponding to the 85% and 15% percentile, respectively.

The water surface elevation was measured along the flume using resistive type wave probes with an accuracy of 0.4 mm. A group of three probes was placed in the deeper area, near the wavemaker, and another two groups of three probes near each of the breakwater sections (Fig. 2). Probe distances varied according to the incident wave conditions to capture the range of frequencies between 0.5 and $2.0 f_p$ ($f_p = 1/T_p$, where T_p represents the peak wave period). Water surface elevations were recorded at a frequency of 40 Hz and incident and reflected waves were separated using the method of Mansard and Funke (1980), which uses simultaneous records of the free surface elevations measured by three

aligned wave gauges.

2.2. Test programme

Three test series (A, B and C) were completed for two simultaneous armour layer porosities resulting in 48 tests. A constant wave steepness (0.02 for series A and B and 0.05 for series C) in the wave generation zone, $s_{op} = H_s/L_{op}$, where H_s represents the significant wave height, L_{op} the deepwater wavelength associated to the sea state peak period, $L_{op} = gT_p^2/2\pi$, was adopted with the significant wave height progressively increasing within each test series (Table 2). Irregular waves defined by a JONSWAP spectrum with a peak enhancement factor $\gamma = 3.3$ were used in all tests. The waves were generated by the filtered white noise technique for a sequence length of $2^{13}-1$ pulses. Test durations varied between 16 min and 36 min, corresponding to approximately 1,000 waves for each test. A fixed water level of 0.40 m at the structure was adopted resulting in crest freeboards, R_c , of 0.23 m (series A) and 0.28 m (series B and C). Significant wave heights for all series varied between 7.5 and 18.2 cm, peak wave periods, T_p , between 1.07 and 2.44 s, relative water depths at the toe of the structure, h/H_s (where h represents the water depth in front of the structure), between 2.2 and 5.3 and relative crest freeboards, R_c/H_s , between 1.3 and 3.8. A cumulative damage approach was followed where damage was only repaired at the end of each series of tests. The conditions tested are not representative of shallow-water conditions, defined here as $h/H_s < 2$. Therefore, the results presented in this study may not be applicable for such conditions.

2.3. Automatic detection of cube position

When analysing images or video material in a coastal engineering and physical modelling context, the question often is related to the position of water, sand, or the structure within the image, which changes over time. These questions can be answered with image segmentation (Den Bieman et al., 2020). Cube positions were automatically detected in images using the “Trainable Weka Segmentation” plugin

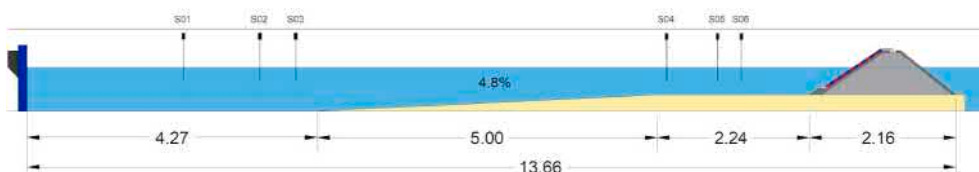


Fig. 3. Wave flume longitudinal cross-section (dimensions in m, model scale).

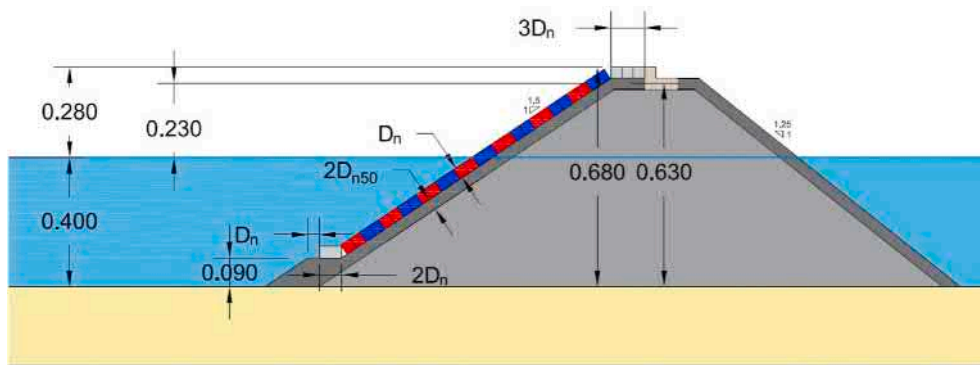


Fig. 4. Breakwater model cross-section (dimensions in m, model scale).

Table 1

Characteristics of the breakwater layers.

Layer	D_{n50} (cm)	ρ_c (kg/cm ³)	M_{50} (g)	D_{85}/D_{15}	D_{85} (cm)
Armour (cubes)	3.50	2.38	102.1	–	–
Filter	1.46	2.65	8.2	1.3	1.9
Core	1.13	2.65	3.8	1.6	1.6

Table 2

Range of parameter values at the toe of the structure for each test series.

Series	p	s_{op}	h/H_s	R_c/H_s	No. tests
A	0.24 0.29	0.02	2.3–5.3	1.3–3.0	16
B	0.29 0.33	0.02	2.2–4.3	1.5–3.0	16
C	0.29 0.33	0.05	2.4–5.4	1.7–3.8	16

(Arganda-Carreras et al., 2017) in Fiji (Schindelin et al., 2012). Fiji, a distribution of the open-source software ImageJ, is a powerful image processing software focused on microscopy biological-image applications. The Trainable Weka Segmentation (TWS) is a machine learning tool that leverages a limited number of manual annotations to train a classifier and segment the remaining data automatically (Arganda-Carreras et al., 2017). Other techniques for quantifying settlement on single-layer concrete armour units were presented in Hofland and van Gent (2016).

Images of the armour slope for tests in a series were stacked using Fiji to improve efficiency in the analysis process. Images for some of the tests with lower stability number where less movements are expected, were unavailable. The images are then imported to the TWS plugin for training and analysis. Approximately 30 areas with pixels containing

cubes are selected as “Cubes” and another 30 areas (without cubes) as “Background”. The TWS plugin classifies all images based on the training data so that cubes are isolated from the background. Cube edge and associated parameters (area, perimeter and center) are calculated using the Fiji plugin “Analyze Particles” (Fig. 5). ImageJ “Analyze Particles” was successfully applied in Cuthbertson et al. (2010) for post-processing of captured sequential images to identify individual sediment flocs and determine their geometric characteristics. The “Analyze Particles” command counts and measures objects in binary or thresholded images. It works by scanning the image until it finds the edge of an object. It then performs the following actions: outlines the object, measures it, fills it to make it invisible, then resumes scanning until it reaches the end of the image. Final verification of the cube positions is carried out to ensure that all cubes were correctly detected and any missing cubes are included.

2.4. Armour grid orthogonality

The regular placement of this type of armour implies a regularity (cubes in alternate rows and all lines are theoretically aligned), which can be quantified through its “cell orthogonality” similarly to a computational grid (e.g., Deltares, 2021). In this case, the center of the cubes define a vertex of a cell (Fig. 6). Cell orthogonality is calculated as the cell-centred cosine value between the intersecting axis of horizontal and vertical neighbours (Fig. 6). The cell orthogonality for a perfectly regular grid is zero (all angles equal to 90°, $\cos 90^\circ = 0$), whilst for a grid where cubes are not aligned this value is higher. Although a grid could theoretically move as a whole between tests and maintain its orthogonality, it is unlikely that this will happen. This parameter should be analysed in conjunction with cube movements to provide a better overview of the armour damage evolution.

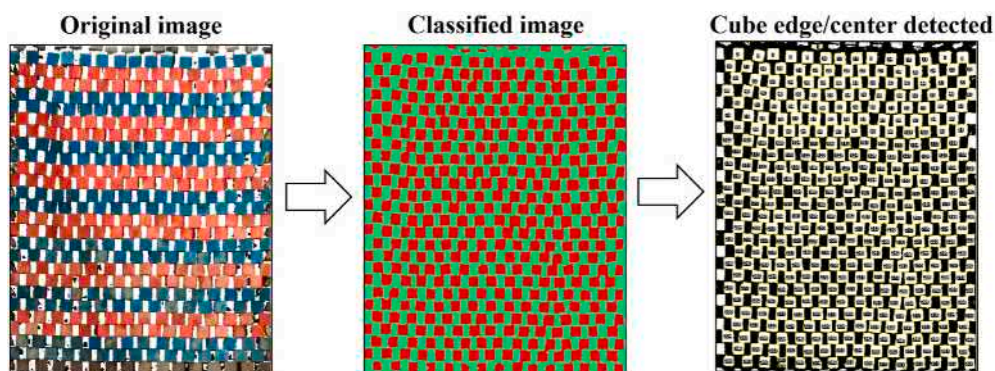


Fig. 5. Example of Fiji/TWS flowchart for automatic detection of individual cube positions.

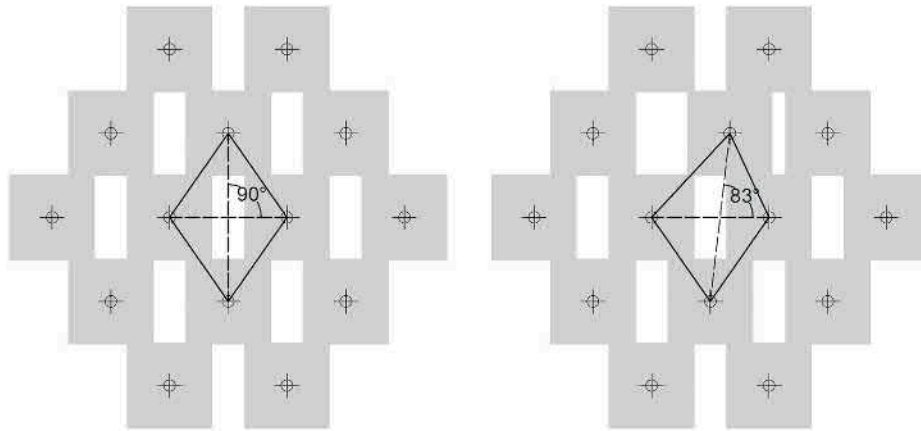


Fig. 6. Example of a perfectly regular grid cell (left) and a cell where the upper cube moved to the right (right).

2.5. Damage analysis area

The damage analysis for all parameters was carried out excluding the top two lines of cubes (near the crest) and the two columns of cubes adjacent to the flume walls. The first line of cubes located at the intersection between the slope and crest are subject to the lack of support from the cubes in the crest and therefore may influence the results for the slope, which is the area of interest for this study. Additionally, the cubes adjacent to the walls of the flume only have support from a cube above and another below, unlike cubes in the other areas that are in contact with two cubes above and another two below, and therefore their behaviour will not be representative. For SLC in a regular grid, despite the lack of interlocking between units, the strong friction between adjacent cubes in the top and lower rows provides support to each other. The entire armour layer can be compromised in case one cube is extracted, therefore requiring consideration of the armour layer as a whole. A one-layer structure reacts as an integral layer, where a two-

layer structure reacts on the stability of individual units (Van der Meer, 1999).

3. Results and discussion

3.1. Cube movement analysis

The cube position identification using the methods described in 2.3 enables the mapping of all cubes for each test. As an example, Fig. 7 depicts the evolution of cube movements (for all cubes in the armour slope) for test series B with a porosity of 0.29. The damage evolution and magnitude of movements are clear with few units being moved for lower stability numbers. An increase in stability number results in additional movements of up 15–18 mm ($\sim D_n/2$). It is also observed that more movements occur above the waterline, in particular near the crest. The cubes in the first line of the slope, immediately below the crest, are indeed less stable than the remaining of the slope since there is little

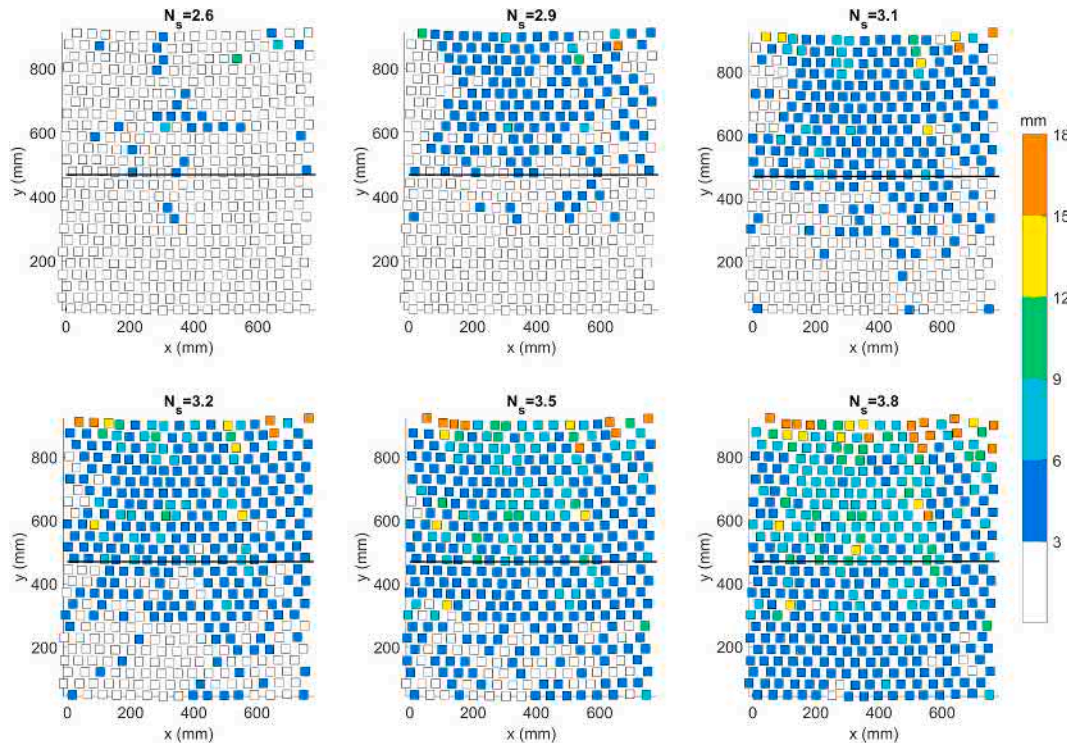


Fig. 7. Example of the evolution of cube movements (mm) for series B ($p = 0.29$). The top corresponds to the crest of the structure. The black line indicates the still water level during the tests.

support from the units in the row above to counteract the forces during wave runup. This was also observed in other studies (Almeida, 2013; Van Gent and Luís, 2013) and some solutions have been proposed to mitigate the transition between slopes and horizontal areas (e.g., Van Gent and van der Werf, 2017). As an example, the direction of cube movements direction (and magnitudes) for test series B with a porosity of 0.29 (for a stability number $N_s = 3.8$, with $N_s = H_s/\Delta D_n$, where $\Delta = \rho_c/\rho_w - 1$ represents the relative buoyant density and ρ_w the water density) is depicted in Fig. 8, where it is observed that there is a general down-slope movement.

Exceedance curves for cube movements (Fig. 9) indicate that for lower stability numbers up to $N_s = 2.6$ – 2.7 , the magnitude of movements for all series is limited, with the majority of the cubes being moved up to a distance of $D_n/7$ (5 mm). For larger stability numbers ($N_s > 3.1$) in series A, the test with lower porosity (0.24) results in a larger magnitude of movements compared to a porosity of 0.29, indicating lower hydraulic stability. In test series B, a higher porosity (0.33) causes a larger number and magnitude of movements when compared to a lower porosity (0.29) and all the other series. These findings, in terms of the influence of the armour porosity on stability, are in agreement with previous studies (Van Buchem, 2009). The behaviour of series C for both porosities is similar, which suggests that for a higher wave steepness ($s_{op} = 0.05$) the influence on the overall stability is relatively small. For a porosity of 0.29, series A (with a lower armour crest level) is more stable than B and C, suggesting the influence of the relative freeboard on the armour hydraulic stability. This effect was investigated in previous studies (Vidal et al., 1992; Van der Meer and Daemen, 1994; De Jong, 1996; Burchart et al., 2006) where generally a lower crest resulted in increased armour layer stability for the seaward slope because of higher wave energy transmission due to overtopping. For $N_s = 3.5$, in series C ($p = 0.29$) there were $\sim 80\%$ of cubes moved by more than $D_n/7$ (5 mm) whilst for series B ($p = 0.29$) only $\sim 40\%$ of the cubes moved more than $D_n/7$. However, in both series, 5% of the cubes moved by more than $D_n/4$ (8.75 mm). IDa_{SLC} , based on the visual assessment during the tests when cubes lose support from the cubes in the rows below, occur when $\sim 5\%$ of the cubes move more than $D_n/4$ (shown as a black circle in Fig. 9). It was not possible to reach the threshold for IDa_{SLC} in Series A

due to limitations in generating larger waves associated with the model dimensions chosen, the wave generator capacity and water depth at the structure. IDe_{SLC} was not reached during the experiments for the same reasons.

3.2. Grid orthogonality

It is difficult to construct a perfectly regular armour grid ($MGO_0 = 0$, where MGO_0 represents the as-built mean grid orthogonality or MGO for $N_s = 0$ in Fig. 10 - top) in the laboratory, due to several factors such as the irregularity of the filter layer and manual placement of the cubes. The damage evolution in Fig. 10 (bottom) takes into consideration the different MGO_0 values for each series by subtracting MGO_0 from MGO . This parameter provides a better descriptor for comparing the various test series, since there were small differences in MGO_0 values (12% between the smallest and largest). The results for series A are relatively constant, indicating that the cube positions remained stable from their original (as-built) locations up to around $N_s = 3.1$ (for a porosity of 0.24) and $N_s = 3.5$ (for a porosity of 0.29). For series B, the results for an armour porosity of 0.33 show that the mean grid orthogonality increases more than for the other series, in particular for higher values of N_s (> 3.0). The lines for both porosities in series C have a similar trend, although the results for $p = 0.33$ are generally higher than those for $p = 0.29$. When comparing series A and B with a porosity of 0.29, it can be observed that, despite a similar initial mean grid orthogonality (Fig. 10 - top), series B results in a more irregular grid as N_s increases. This indicates that the relative freeboard has an influence on the armour layer regularity and subsequently, stability, as was previously observed in Fig. 9. IDa_{SLC} , based on the visual assessment during the tests, occurs approximately for $MGO - MGO_0 > 0.007$. As mentioned previously, it was not possible to reach IDa_{SLC} for Series A and IDe_{SLC} for any series. However, it is not essential to establish a threshold for this stage of damage since, as proposed and explained in section 3.3, the relative damage number better describes IDe_{SLC} .

The quality of the initial armour grid placement (within the tested range) appears not to have a strong impact on the damage progress. For series A and C, the lines between each of the two porosities within the same series are roughly parallel. Similar behaviour is observed for series B until approximately $N_s = 3.0$, where the higher porosity case then diverges upwards (less regular grid). Further investigation is required for a more detailed evaluation of this aspect on the armour stability.

3.3. Relative damage number for SLC

The classic definition of N_{od} is not an adequate method for providing quantitative assessments of damage in SLC armoured breakwaters since the extraction of one unit from the armour layer, which could correspond to a very low N_{od} , results in exposure of the filter layer (Initiation of Destruction or “failure”), unlike in double-layer structures (Vieira et al., 2020b). The test results did not show any cube being moved by a distance of D_n or extracted resulting in a N_{od} of zero for all tests even for relatively high values of N_s . However, it is observed in Fig. 11, for series B ($p = 0.33$), that there were substantial changes to the armour layer, with some cubes falling into the lower rows (for example on the second and third rows of blue cubes from the top) and others losing support from the cubes in the row below, that compromise the overall stability and therefore it is considered damage. Initiation of damage has occurred for the structure in this example but it is still providing protection. IDe_{SLC} (defined here as exposure of the filter layer) is eventually expected to occur if the wave conditions continue for a longer time or the waves become more energetic.

An alternative relative damage number, N_{od-SLC} is proposed where cubes are considered “displaced” if they move over a distance of more than $D_n/3$ (Fig. 12). The proposed threshold for IDa_{SLC} is $N_{od-SLC} = 0.1$. This corresponds approximately to the visual assessment of the armour layer in the tests (cubes falling into the lower rows and/or losing support

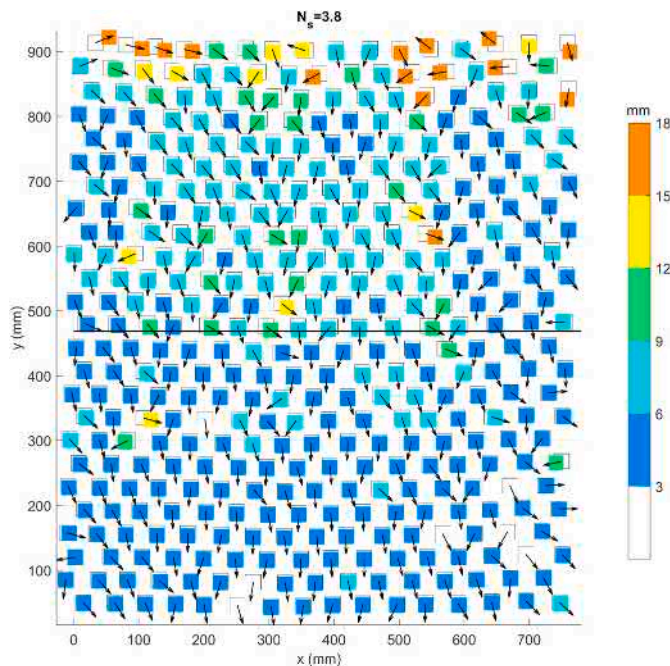


Fig. 8. Example of the evolution of cube movements (mm) for series B ($p = 0.29$) for $N_s = 3.8$. The white squares represent the cube positions at the start of the test series. The black arrows show the direction of cube movement.

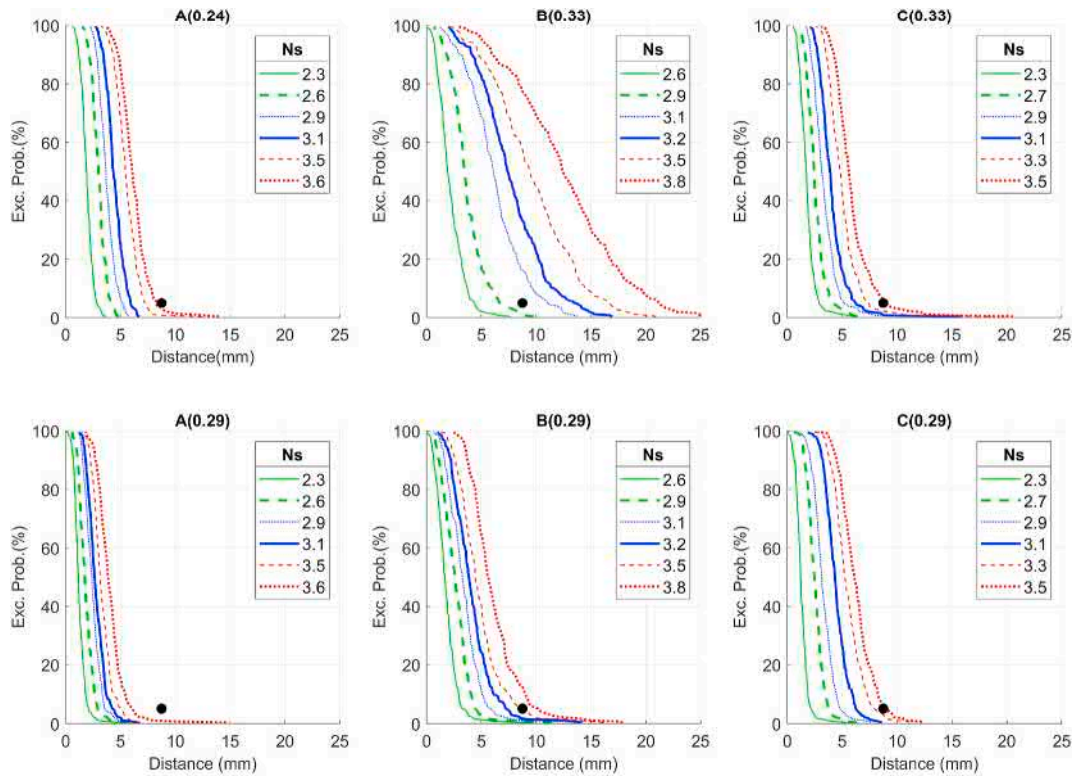


Fig. 9. Exceedance probability of cube movements (mm) for all tests.

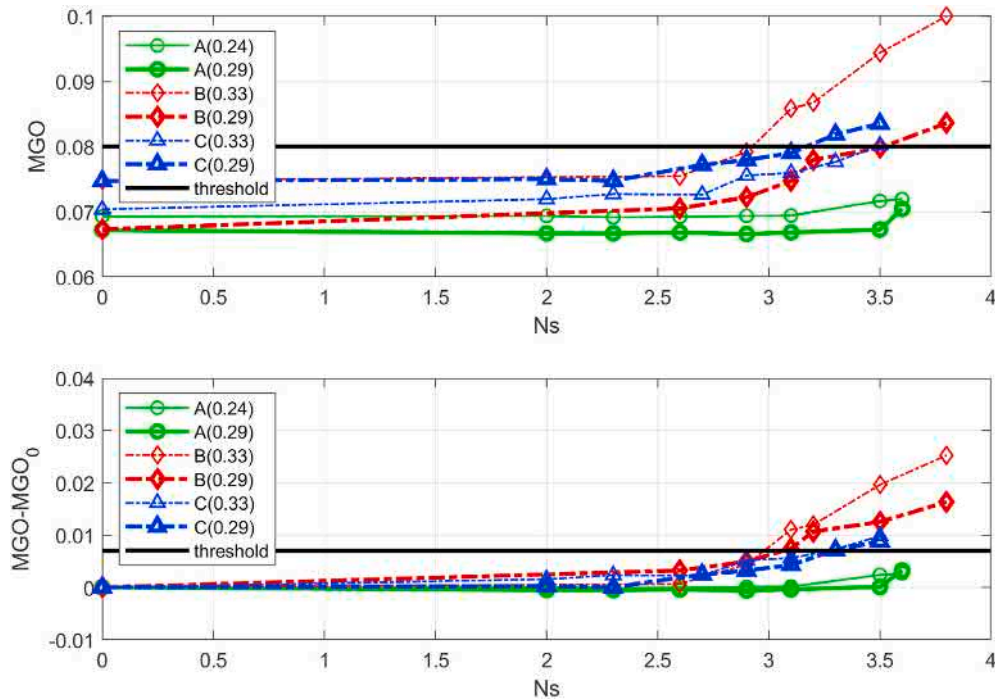


Fig. 10. Evolution of mean grid orthogonality with stability number for all tests. MGO (top) and $MGO - MGO_0$ (bottom).

from the cubes in the row below) and is generally consistent with the assessment of the parameters presented in 3.1 and 3.2 (Table 3). It is also proposed to adopt $N_{od} > 0$ (where cubes are considered “displaced” if they move by a distance of more than D_n or are removed from the armour layer) as a criterion for ID_{eSLC} . $N_{od} > 0$ indicates that at least one cube has moved significantly (or been extracted), thus leaving a gap where the filter is exposed. Such gap will likely result in loss of support

to the upper rows and subsequent collapse of the armour layer.

3.4. Damage evolution classification and criteria

A comparison of the stability numbers for ID_{eSLC} based on the different parameters and associated criteria is presented in Table 3. The results are relatively consistent between the various parameters. For

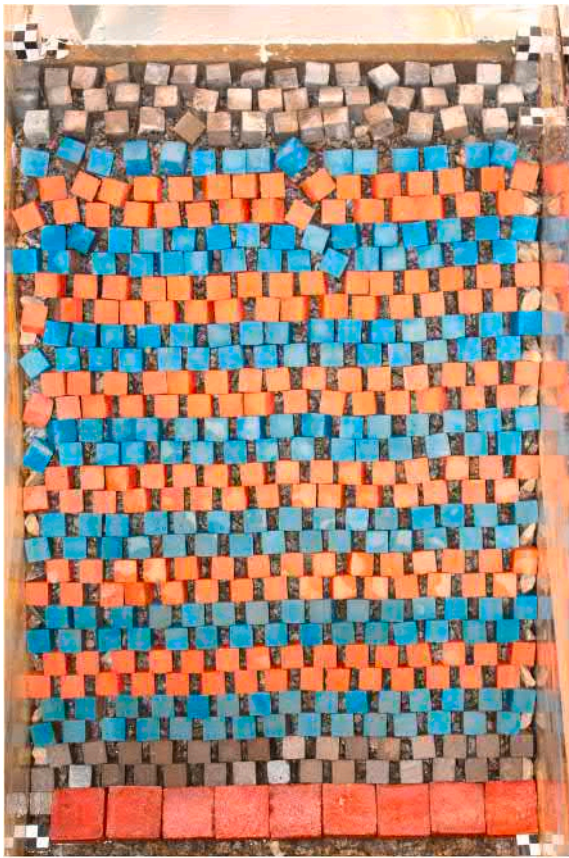


Fig. 11. Photo after test B ($p = 0.33$) for $N_s = 3.5$.

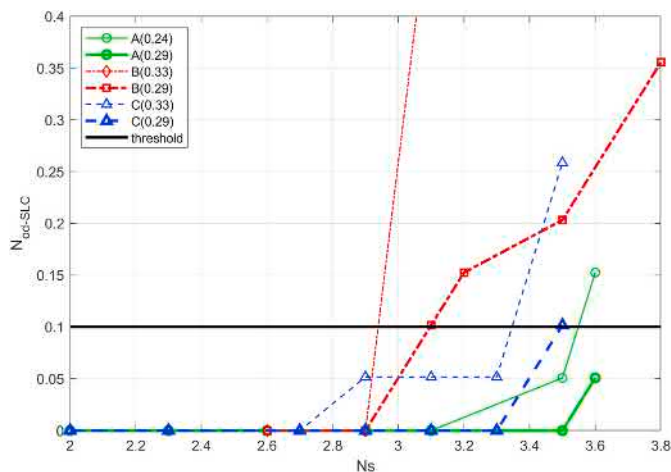


Fig. 12. N_{od-SLC} for all tests.

series A (lower armour crest) the stability number for IDa_{SLC} is ≥ 3.6 . For series B, the range of N_s for the higher porosity ($p = 0.33$) is 2.9–3.1, whilst for the lower porosity ($p = 0.29$) is 3.1–3.5. In series C there is not a significant difference between porosities and the N_s for IDa_{SLC} is in the range of 3.2–3.5. Based on the literature (Van Gent et al., 1999, 2001; Van Buchem, 2009; Almeida, 2013) and this study, it is likely that a structure will be built with porosities around 0.29. Therefore, the range of N_s to consider in preliminary designs for this porosity, considering the different wave steepnesses and crest freeboards, is from 3.1 to more than 3.6. The lower limit is likely to be adopted for structures with higher crest freeboard and the upper limit for lower structures.

Table 3

Stability number for IDa_{SLC} based on the proposed criteria for various parameters.

Parameter	Exc. prob. movement	Mean orthogonality	N_{od-SLC}	Range
Criteria	$5\% > D_n/4$	$MGO - MGO_0 > 0.007$	> 0.1	
SERIES A				
A (0.24)	3.6	> 3.6	3.6	≥ 3.6
A (0.29)	> 3.6	> 3.6	> 3.6	> 3.6
B (0.33)	2.9–3.1	3.0	2.9	2.9–3.1
B (0.29)	3.5	3.1	3.1	3.1–3.5
C (0.33)	3.3–3.5	3.3	3.3	3.3–3.5
C (0.29)	3.3–3.5	3.3	3.5	3.3–3.5

4. Conclusions and recommendations

4.1. Conclusions

2D physical model study results for single-layer cube (SLC) armoured breakwaters with a regular placement pattern are presented in this work. Different combinations of armour porosity, crest level, and wave steepness were tested.

A novel image-processing based procedure using machine-learning techniques was applied to automatically detect individual cube positions, to enable a consistent and fast method for post-processing of the results. This method is relatively straightforward and can be implemented to process various images in a batch mode, which will reduce post-processing times.

Innovative parameters based on cube movement exceedance probability curves, mean grid orthogonality, and relative damage number, were developed to describe damage evolution and used to establish preliminary thresholds for Initiation of Damage in SLC armoured breakwaters. The suggested thresholds for the various parameters are based on the correspondent visual assessment of the damage during the tests. A summary of the proposed damage evolution classification and criteria for SLC armoured breakwaters is depicted in the graphical abstract.

Results show that a porosity of around 0.29 is generally preferred for higher hydraulic stability. Structures with a lower crest height showed greater stability. Stability numbers for IDa_{SLC} range from 3.1 to more than 3.6 for armours with a porosity of 0.29. Adequate recommended safety coefficients (e.g., CIRIA, 2007) should be applied in preliminary designs to provide some margin due to the failure mechanism of this structure. Test results show that the structures are relatively resilient and therefore safety factors could potentially be reduced, subject to verification in physical model tests.

4.2. Final considerations and recommendations

The analysis was limited to the data obtained in the laboratory experiments carried out in this study. It is recommended to carry out additional testing for SLC armoured breakwaters and apply the proposed damage criteria to assess its adequacy and to validate its applicability to other forcing conditions and structure geometries. This analysis was focused on the slope area. Further research on other areas of the structure such as crest, rear-side and toe are recommended. Actual safety factors between IDa_{SLC} and IDe_{SLC} were not evaluated in this study. This was not possible in these experimental tests as the wave height at the structure (and consequently N_s) was limited by the model scale chosen, the wave generator capacity and water depth at the structure. Future work should be planned to cover this important aspect.

The influence of the grid placement accuracy on the armour stability was outside the scope of this work. However, the available data enabled a high-level assessment. Further investigation is recommended for a more detailed evaluation of this aspect to potentially establish adequate construction tolerances either in the model or in the prototype.

Due to the curvature of the roundhead and subsequent implications, such as deviation of unit placement from a regular grid and larger gaps between units, its stability is typically smaller than at the trunk section (Muttray and Reedijk, 2009). The roundheads are also generally subject to more complex hydrodynamic forcing, which are harder to characterize, and are key parts of the breakwater global integrity. Therefore, it is recommended to develop specific research on damage evaluation methods adequate to these areas.

Finally, the cube shape offers the possibility of including minor features and modifications such as small holes, recesses and striations, which increase surface complexity and promotes the growth of marine organisms (e.g., Firth et al., 2014; Schoonees et al., 2019; Evans et al., 2021). Such minor features are unlikely to affect the overall armour performance but it is suggested to study them in detail.

CRedit authorship contribution statement

Filipe Vieira: Funding acquisition, Conceptualization, Methodology, Investigation, Visualization, Formal analysis, Writing – original draft. **Francisco Taveira-Pinto:** Funding acquisition, Conceptualization, Writing – review & editing. **Paulo Rosa-Santos:** Methodology, Resources, Writing – review & editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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APPENDIX IV – Vieira et al. (2021C)



Novel time-efficient approach to calibrate VARANS-VOF models for simulation of wave interaction with porous structures using Artificial Neural Networks

Filipe Vieira^{a,b,*}, Francisco Taveira-Pinto^{b,c}, Paulo Rosa-Santos^{b,c}

^a Department of Biology, Chemistry and Environmental Sciences, American University of Sharjah, Sharjah, United Arab Emirates

^b Department of Civil Engineering, Faculty of Engineering of the University of Porto, Porto, Portugal

^c Interdisciplinary Centre of Marine and Environmental Research of the University of Porto, Matosinhos, Portugal

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ABSTRACT

Numerical models are valuable tools to provide information on wave-structure interaction processes that are difficult to measure in a physical model. The current level of accuracy of numerical models is relatively high but an adequate validation to establish the models' empirical parameters is required to ensure a correct representation of the phenomena to be investigated. To this end, a "trial and error" approach is typically adopted, potentially resulting in a large number of simulations.

In this study, a methodology based on Artificial Neural Networks (ANN) is presented, to obtain the optimal combination of values for the empirical coefficients that characterize the porous media in a VARANS-VOF model for predicting mean overtopping discharges. From an initial reduced set of simulations, input and output data are obtained for the training of the ANN. The ANN is then used to estimate the best values for the combination of coefficients that describe the three breakwater layers (armour, filter, and core), which are subsequently applied in the numerical model. The method was successfully applied to a set of laboratory experiments aimed at obtaining overtopping discharges for a Single-Layer Cube armoured breakwater. It resulted in a large reduction of computational effort when compared to the simulation of all possible combinations of values.

1. Introduction

Rubble-mound structures protect a variety of critical facilities for populations along the coastline, from transportation infrastructure to industrial and residential buildings. It is, therefore, crucial to understand in detail wave-structure interaction processes for the design of new structures or upgrading of existing ones to cope with climate change effects on Sea Level Rise (SLR) and wave storminess. IPCC (2019) predicts an average global mean SLR by 2100 of 0.43 m–0.84 m. This impact, together with a generally positive trend in mean and storm wave heights around the world (e.g., Aboobacker et al., 2017; Reguero et al., 2019; Vieira et al., 2020a) will result in detrimental effects on the world's coastlines (Vieira et al., 2021).

Physical modelling is considered one of the most adequate tools to investigate wave-interaction processes. Extensive research to address the impact of climate change based on such tool has been carried out to

date (e.g., Norgaard et al., 2012; Eldrup et al., 2019; Fortes et al., 2019; Long and Murphy, 2019; Franco et al., 2020; Stagnitti et al., 2020). The main advantages of a physical model are the inclusion of all physics associated with phenomena that are difficult to describe or are not entirely understood and the fact that it is easily controlled and modified as required. Some disadvantages are associated with laboratory and scale effects as well as the impact of limited accuracy, precision, and sampling rates of measurement equipment on the quality of experimental data (Hughes, 1993; Wolters et al., 2010). Numerical modelling is a valuable tool to provide information on the processes that are difficult to measure in a physical model. The level of accuracy of the present numerical models is relatively high and many applications were successful in reproducing wave-structure interaction processes measured in laboratory experiments. Numerical modelling can also mitigate two of the biggest weaknesses of physical modelling: extraction of data and scale and laboratory effects. In numerical models, various

* Corresponding author. Department of Biology, Chemistry and Environmental Sciences, College of Arts and Sciences, American University of Sharjah, PO BOX 26666, Sharjah, United Arab Emirates.

E-mail address: fvieira@aus.edu (F. Vieira).

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types of data, such as velocity, pressure, water surface elevation, turbulent kinetic energy, can be extracted from any point and time in the simulation, and the “virtual” probes do not interfere with the flow (Vyzikis et al., 2014). Several studies using numerical models, calibrated and validated against physical model tests and subsequently used for assessment of the sensitivity of coastal structures to different scenarios of incident conditions and/or geometry, are available in the literature (Raosa et al., 2012; Altomare et al., 2014; Molines et al., 2019; Palma et al., 2019; De Finis et al., 2020; Chen et al., 2021). The calibration phase for the establishment of the models’ empirical parameters is often time-consuming, but this process can be facilitated by the use of Artificial Neural Networks (ANN) as investigated in the present study. To the authors’ knowledge, the application of ANN to determine the optimum values for the models’ empirical coefficients that best represent the wave-structure interaction processes have not been implemented to date and, therefore, is investigated in this study. The method is developed and validated against a set of laboratory experiments for Single-Layer Cube (SLC) armoured breakwaters described in detail in Vieira et al. (2021).

The paper is organized as follows: after an introduction and literature review on numerical modelling of wave overtopping and ANN in Section 1, Section 2 describes the laboratory experiments and numerical modelling conducted in this study. The analysis and discussion of results are provided in Section 3. Section 4 presents the main conclusions and recommendations of this study.

1.1. Numerical modelling of wave overtopping

Most models solving the flow within a porous media are based on the Navier-Stokes (NS) equations. Two different approaches have been followed in numerical modelling of wave propagation and interaction with structures when using the NS equations: Lagrangian and Eulerian (Losada et al., 2016). A detailed description of both approaches is described in detail in Losada et al. (2016). The governing equations for Eulerian models are discretized in space with a finite difference/finite volume technique, using structured grids with a staggered mesh topology. The Volume-of-Fluid (VOF) technique is used for free surface tracking (Vanneste et al., 2014). Modelling of porous media (rubble-mound structures) is highly complex since these are characterized by irregular pore shapes and sizes within a solid matrix, which are usually assumed rigid (Losada et al., 2016). Volume Averaged Reynolds Average Navier-Stokes (VARANS) equations allow one to consider the porous material as continuous media when characterized by their macroscopic properties only, eliminating the need for a detailed description of their complex internal geometry (Fig. 1).

The VARANS equations are derived by integrating the RANS equations over a control volume determined based on the structure porosity (n). The porous media in the VARANS equations are modelled using Forchheimer’s relationship that includes two empirical coefficients associated with the linear (α) and non-linear (β) drag forces. The expression used in the model for the linear and non-linear drag forces is given by:

$$I = \frac{\alpha\nu(1-n)^2}{n^2D_{50}^2}\vec{u} + \frac{\beta(1-n)}{n^2D_{50}}\vec{u}|\vec{u}|. \quad (1)$$

where n represents the structure porosity, D_{50} the nominal mean stone diameter, ν the kinematic viscosity and $\vec{u}$ the velocity.

Losada et al. (2008) applied a 2D VARANS model to a rubble-mound breakwater that had previously been the subject of an experimental study. The results showed that the model was able to reproduce instantaneous and average wave overtopping discharge with a mean error of 9% and 2.5% between measurements and model results for mean and maximum overtopping discharge volumes, respectively. Jensen et al. (2014) investigated the empirical coefficients applied for representation of the porous media in the VARANS equations using OpenFOAM®. The model was applied and calibrated against a set of experiments for various flow regimes. The model, using the recommended empirical coefficients, was applied to the experimental results reported in Bruce et al. (2009) for a rock armour layer. The model was found to give a good estimation of overtopping discharges. The simulated results were distributed around the empirical relationship in the same manner as the experimental results. Pilechi et al. (2018) applied the 2D VARANS model IH2VOF (Lara et al., 2011) to a rubble-mound breakwater laboratory experiment. The comparisons were carried out for a range of sea state conditions, water levels, and breakwater geometries. The average overtopping discharges predicted by the numerical model are within 20% of accuracy compared to those measured in the physical model. Palma et al. (2019) also applied the model IH2VOF to assess the impacts of waves on an Overtopping Breakwater for Energy Conversion. This breakwater has a relatively complex geometry, but the overtopping results still show that the error obtained with the numerical model was relatively small when compared to the measurements. Molines et al. (2019) also applied OpenFOAM®, to a rubble-mound breakwater to evaluate the influence of the crown wall geometry on wave overtopping discharges. They obtained relatively good calibration results when compared to physical model tests.

Nevertheless, there is still large uncertainty in describing α and β coefficients in VARANS models. Lara et al. (2008) suggest that they depend on parameters such as the Reynolds number, shape of the stones, grade of the porous material, permeability, and flow characteristics. Therefore, values for these coefficients should be determined based on the comparison between numerical model results and experimental data. Although considerable effort has been done over the last three decades to better understand porous flow characteristics in coastal engineering applications, there are still open questions. The wide range of values found in the literature is a clear indication that there is still room for improvement in the determination of the porous parameter values and how they are linked to the different flow conditions (Losada et al., 2016). In most of the above-mentioned studies, the determination of α and β coefficients to obtain a satisfactorily model calibration was either taken from literature, which is not always applicable to generic cases, or by trial-and-error. Considering that there are usually 2 to 3 layers of porous material (armour and core or armour, filter and core), this corresponds to a large number of possible combinations of parameters,

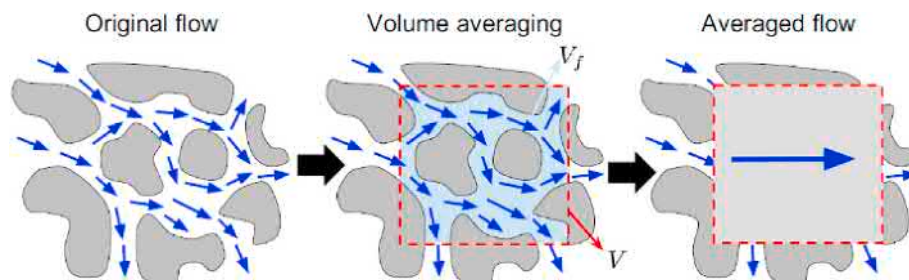


Fig. 1. Volume-averaging process sketch for resolution of the porous flow. Republished from Losada et al. (2016), Copyright (2021), with permission from ASCE.

which, subsequently, results in a great number of simulations and long computational time. Furthermore, the trial-and-error process is not a systematic and efficient approach. One can test different combinations of values, but better results could eventually be achieved for combinations that were not considered. Therefore, there is a need to develop a systematic and consistent approach to determine the best combination of values for the porous media coefficients. Since this is a non-linear problem, this study presents the implementation of Artificial Neural Networks to obtain better and quicker estimates of the empirical coefficients for the porous media in VARANS models.

1.2. Artificial Neural Networks

Artificial Neural Networks (ANN) are data-driven algorithms that aim at simulating the structure of the human brain to identify patterns in complex data, given a set of inputs and outputs (Lippman, 1987). ANN are techniques useful to solve non-linear problems when enough experimental data are available for training. This process does not require any prior knowledge of the actual physics, although it is important to know which input variables are relevant to the problem to be solved (Vieira et al., 2020b). ANN are widely used in coastal and ocean engineering in various fields including wave overtopping of coastal structures (Medina, 1999; Medina et al., 2002; Van Gent et al., 2007; Zanuttigh et al., 2016), wave transmission through breakwaters (Panizzo et al., 2003; Panizzo and Briganti, 2007), wave reflection from coastal structures (Taveira-Pinto et al., 2006; Garrido and Medina, 2012; Zanuttigh et al., 2013); wave forces on crown walls (Molines et al., 2018), wave forces on vertical structures (Van Gent and Van den Boogaard, 1998), stability of rubble mound breakwaters (Mase et al., 1995; Medina et al., 2003; Kim and Park, 2005; Kim et al., 2008, 2014; Iglesias et al., 2008; Balas et al., 2010; Gedik, 2018; Lee et al., 2016), beach processes (Iglesias et al., 2005; Lopez et al., 2018), wave transformation (Browne et al., 2007; Santos et al., 2016), wave forecasting (Deo et al., 2001; Mandal and Prabakaran, 2006; Etemad-Shahidi and Mahjoobi, 2009; Deka and Prahlada, 2012; Reikard et al., 2015; James et al., 2018; O'Donncha et al., 2018; Bento et al., 2021) and data gap filling (Arena and Puca, 2004; Huang and Xu, 2009; Balas et al., 2004; Lee and Park, 2016; Peres et al., 2015; Ustoorikar and Deo, 2008; Silva et al., 2018; Vieira et al., 2020b).

ANN include processing unit elements called neurons, which transform, through a nonlinear transfer function, a linear combination of the preceding layer. The coefficients of the linear combination named synaptic weights (w), together with the network biases (b), represent the free parameters of the network model (Peres et al., 2015). The schematic of a two-layered (x_i -inputs, h_i -hidden layer, and o -output layer) neuron model with i input parameters, j hidden nodes, k output nodes and one output parameter (p) is presented in Fig. 2. Input values are transmitted to the hidden layer nodes, each of which sums up the received values from all input nodes, adds a "bias" to this sum, and then passes them on through a nonlinear transfer function (Vieira et al., 2020b). The output layer nodes receive this information and operate identically to the hidden nodes.

The ANN learning procedure (Fig. 3) consists of adjusting the weights and biases in each step to minimize the errors between predictions (outputs, p) and measurements (or targets, t). This procedure is repeated until an error threshold or a maximum number of iterations is reached. A more detailed description of ANN can be found in Vieira et al. (2020b).

Several authors have applied ANN for calibration and improvement of the performance of different types of numerical models for traffic simulations (Otkovic et al., 2013), atmospheric modelling (Perez and Bevilacqua, 2015), indoor building temperature modelling (Lu et al., 2015), mechanical modelling of materials (Mares et al., 2016), financial models (Liu et al., 2019) and near-infrared spectroscopy (Chen and Wang, 2019).

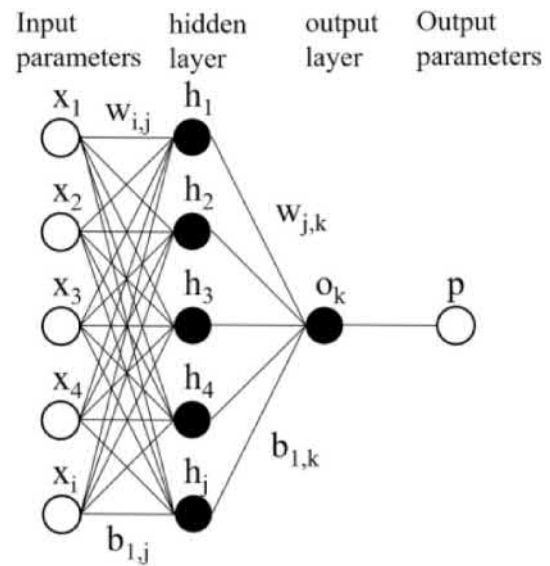


Fig. 2. Example of a two-layered ANN architecture. Republished from Vieira et al. (2020b), Copyright (2021), with permission from Elsevier.

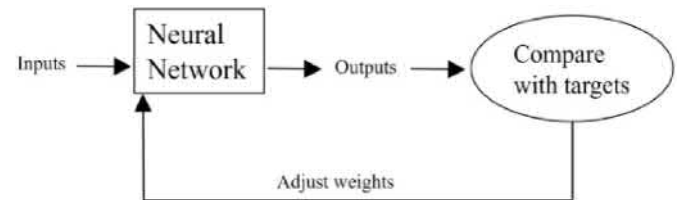


Fig. 3. Schematic of an ANN learning procedure. Republished from Vieira et al. (2020b), Copyright (2021), with permission from Elsevier.

2. Materials and methods

2.1. Physical model tests

The 2D laboratory experiments were conducted in the wave basin (28 m long, 12 m wide, and 1.2 m deep) of the Hydraulics Laboratory of the Hydraulics, Water Resources and Environment Division (SHRHA) of the Faculty of Engineering of the University of Porto, on a geometrical scale of 1/35. A 1.6 m wide flume was constructed within the wave basin. The flume was divided in two by an acrylic glass sheet installed between the tested models from the crown wall area to a distance of approximately 2 m seaward of the toe of the structure, to test two cross-sections with similar reflection characteristics simultaneously. The wave flume has a piston-type wavemaker with an active wave absorption system, which was activated to minimize wave reflections in the model. The water depth at the wave generation and model areas was 0.74 and 0.50 m, respectively, with a 4.8% slope transition between the flat seabed areas (Fig. 4). The breakwater model had a slope of 1 V:1.5H (Fig. 5). The armour layer was built with porosities (n) of 0.29 and 0.33. The structure consisted of a permeable rock core ($D_{n50} = 1.1$ cm), a filter layer ($D_{n50} = 1.5$ cm), and a single layer of cubes ($D_n = 3.5$ cm) placed by hand in a regular pattern. A crown wall was placed behind a $3D_n$ wide armoured crest. The water surface elevation was measured along the flume using resistive type wave probes with an accuracy of 0.4 mm. A group of three probes was placed in the deeper area, near the wave-maker, and two other groups of three probes near each of the breakwater sections (Fig. 4). Probe distances varied according to the incident wave conditions to capture the range of frequencies between 0.5 and $2.0 f_p$ ($f_p = 1/T_p$, where T_p represents the peak wave period). Water surface elevations were recorded at a frequency of 40 Hz and incident and reflected

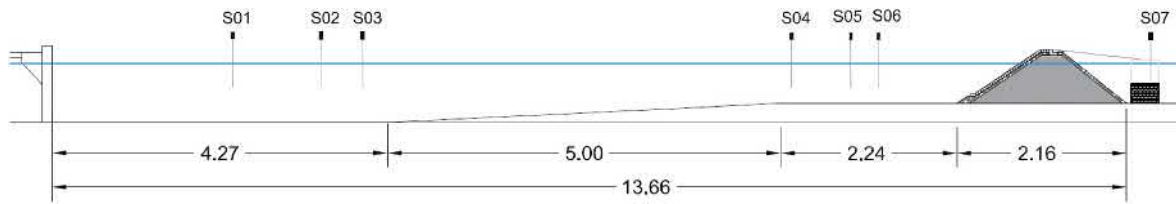


Fig. 4. Wave flume longitudinal cross-section (dimensions in m, model scale).

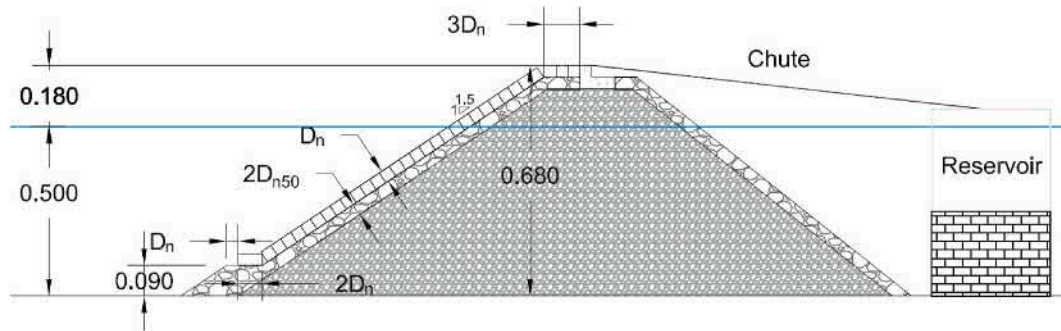


Fig. 5. Breakwater model cross-section (dimensions in m, model scale).

waves were separated using the method of [Mansard and Funke \(1980\)](#), which uses simultaneous records of the free surface elevations measured by three aligned wave gauges. Water was collected via a 0.6 m wide overtopping chute sloping down from the top of the crown wall into reservoirs located on the lee side of the structure. The overtopping discharge was measured by a water surface elevation probe placed inside a perforated PVC pipe within the reservoirs to minimize oscillations in the measurements. When needed, a pump, installed inside the reservoir, was activated to empty the reservoir during the tests.

Two armour porosities (0.29 and 0.33) and constant wave steepness (0.02 and 0.05) in the wave generation zone, $s_{op} = H_{m0}/L_{op}$ (where H_{m0} represents the spectral significant wave height, L_{op} the wavelength based on the peak wave period, T_p) were adopted with the significant wave height progressively increasing within each test series ([Table 1](#)). Irregular sea states defined by a JONSWAP spectrum with a peak enhancement factor $\gamma = 3.3$ were used in all tests. The test durations varied between 14 min and 26 min, corresponding to the generation of approximately 1000 waves in each test. A fixed water depth, h , of 0.50 m at the structure was adopted resulting in a crest freeboard, R_c , of 0.18 m.

2.2. Numerical model

2.2.1. Description

The physical model tests were simulated using the numerical model IH2VOF ([Lara et al., 2011](#)), based on the model COBRAS-UC ([Losada et al., 2008](#)). IH2VOF solves the two-dimensional Reynolds Averaged Navier–Stokes (RANS) equations based on the decomposition of the instantaneous velocity and pressure fields into mean and turbulent components. The effects of the turbulence field are modelled using the $k - \epsilon$ transport equations to calculate the turbulent kinetic energy (k) and the turbulent dissipation rate (ϵ). The free surface movement in IH2VOF

is tracked by the Volume of Fluid (VOF) method ([Hirt and Nichols, 1981](#); [Liu and Wen, 1997](#)). The flow inside the porous media is resolved using the Volume-Averaged Reynolds Averaged Navier–Stokes (VARANS) equations. A complete mathematical formulation of the VARANS equations is presented in [Liu et al. \(1999\)](#) and [Hsu et al. \(2002\)](#). The IH2VOF model was previously applied in a wide range of cases including nearshore processes in the surf zone ([Lara et al., 2006a, 2011](#); [Torres-Freyermuth et al., 2007, 2010](#)); wave-structure interaction for low-crested breakwaters ([Garcia et al., 2004](#); [Losada et al., 2005](#); [Lara et al., 2006b](#)); composite breakwaters ([Lara et al., 2008](#); [Guanche et al., 2009](#)); emerged rubble-mound breakwaters ([Losada et al., 2008](#); [Guanche et al., 2009](#)); tsunami propagation ([Aniel-Quiroga et al., 2018](#)) and non-conventional breakwaters for wave energy conversion ([Di Lauro et al., 2019](#)).

2.2.2. Model setup

The domain is 14.6 m long in the x and 1.0 m in the y directions, respectively ([Fig. 6](#)). The rectangular mesh is divided into 3 and 2 sub-zones in the x and y directions, respectively, for refinement in the region of interest ([Fig. 7](#)). The reference mesh size is variable with a resolution of 1.0 cm–2.0 cm in the x -direction, and 0.5 cm–1.0 cm in the y -direction. The minimum and maximum $\Delta x/\Delta y$ ratio are 1 and 4, respectively. The $H_{m0}/\Delta y_{MIN}$ and $L_p/\Delta x_{MIN}$ ratios for the smallest wave condition simulated (see [Table 5](#)) are 21 and 216, respectively. The total number of cells is 995 (x -direction) by 150 (y -direction) resulting in approximately 150,000 cells. A JONSWAP spectrum with a peak enhancement factor of 3.3 with the same H_{m0} and T_p as specified in the physical model was adopted for the generation of the wave series but different initial seed numbers were used in the physical and numerical models. Wave absorption boundaries were implemented at the wave generation and the end of the numerical flume. Numerical wave gauges were placed at the same locations as those in the physical model. An additional gauge was included at the crown wall for the calculation of overtopping discharges. The model was run for the same duration as in the physical model.

Various combinations of β were adopted for the porous media layers (armor, filter, and core) in the simulations for providing data to the ANN. [Lara et al. \(2006, 2008\)](#) suggest that numerical results are more sensitive to the non-linear drag coefficient β than the linear parameter α for turbulent flows. Therefore, α was kept constant with a value of 200 to

Table 1
Range of parameter values for each test series.

Series	n	s_{op}	H_{m0} (cm)	$T_{m-1,0}$ (s)	h/H_{m0}	R_c/H_{m0}	No. of tests
O	0.29	0.05	7.5–12.1	1.02–1.34	4.1–6.6	1.5–2.4	6
	0.33						
P	0.29	0.02	7.6–12.9	1.45–1.89	3.9–6.6	1.4–2.4	6
	0.33						

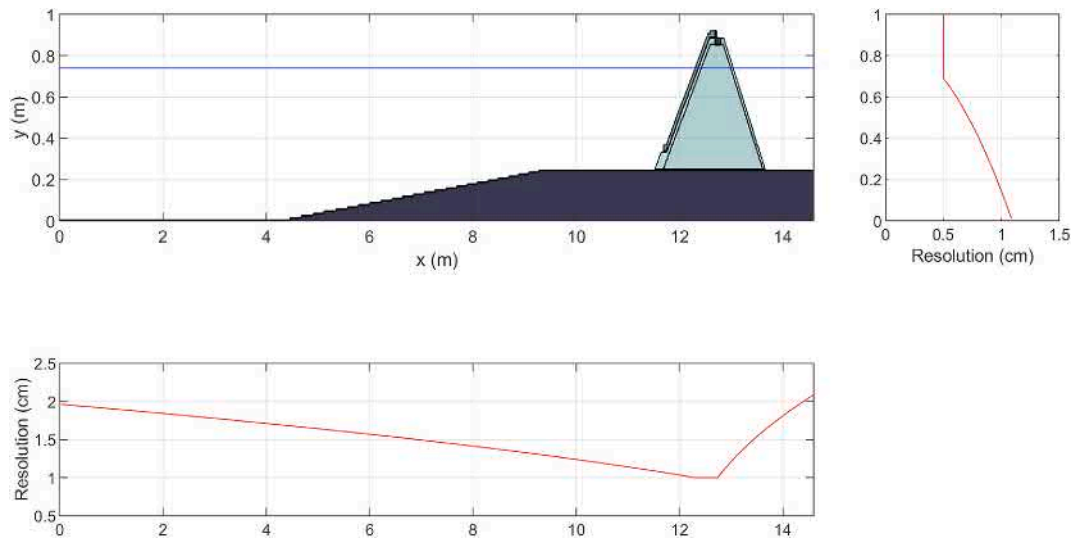


Fig. 6. Model domain (top left) and mesh resolution in x (bottom left) and y (top right).

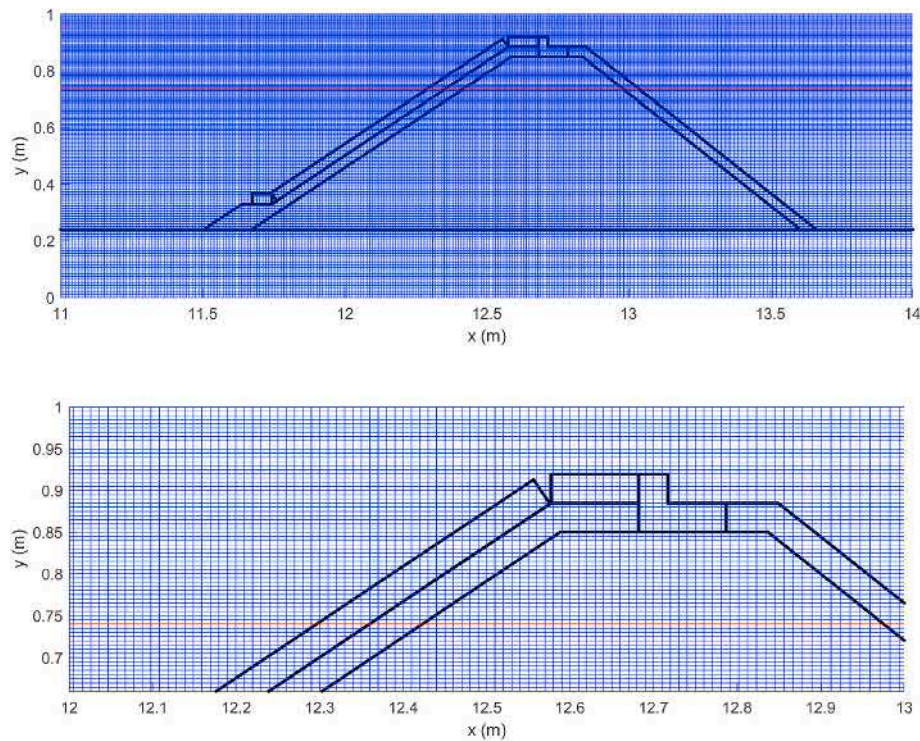


Fig. 7. Computational mesh at the breakwater area (top) and zoomed to the crest area of interest (bottom). The red line depicts the still water level. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

limit the number of variables in the problem. The value of 200 is based on various applications of IH2VOF for similar wave-structure interaction problems (Lara et al., 2008; Losada et al., 2008; Guanche et al., 2009, 2015). Additional simulations with the coefficients of Losada et al. (2008) and Jensen et al. (2014) were carried out since these studies looked specifically into wave overtopping over rubble-mound breakwaters (Table 2). The simulations are used as a reference for comparison of the performance of the calibrated model with the β obtained from the ANN developed in this study. Porosities of 0.29 and 0.33 were adopted for the armour layer (two scenarios), 0.40 for the filter, and 0.38 for the core.

Table 2
 α and β adopted for the reference cases.

Coefficient	Losada et al. (2008)	Jensen et al. (2014)
α (all layers)	200	500
β_{Armour}	0.7	2
β_{Filter}	1.1	2
β_{Core}	0.8	2

2.2.3. Mesh sensitivity

A sensitivity analysis to the mesh element size was carried out to assess the influence of this parameter on the results. Mesh elements with sizes equal to 80%, 120%, and 140% of the reference mesh element were

tested (Table 3) and a sample condition applied at the model boundary, with $H_{m0} = 18.0$ cm and $T_p = 1.8$ s ($s_{op} = 0.035$) for a duration of 60 s.

The relative differences in comparison to the reference mesh (100%) are presented in Table 4. The results show that there is a relatively small decrease in the cumulative overtopping volume, Q , and an increase in the reflection coefficient, Kr , for a finer mesh (80%). A 20% increase in mesh size results in 10% higher overtopping volumes and a negligible decrease in the reflection coefficient. The results for the 140% mesh give similar cumulative overtopping discharges to the 120% mesh but with a reduced value of Kr . Based on these results, it was decided to adopt the reference mesh since small differences in Q and Kr are obtained with the finer mesh but the simulation time is increased by a factor of 2.3, which is considered excessive, as many simulations are required in this study. The simulations were run on a workstation with Intel Xeon Platinum 8168 CPU @ 2.7 GHz processors.

2.2.4. Artificial Neural Network for estimation of β values

The IH2VOF model was applied to a combination of incident wave conditions, armour layer porosities, and β coefficient values to generate values of mean overtopping discharges, q , for the development of the ANN (Fig. 8).

The model was run for cases where the mean overtopping discharge obtained in the laboratory is above 0.00 L/s/m (Table 5) resulting in 8 wave conditions. The values of the β coefficient for each of the three porous layers (β_{Armour} , β_{Filter} , β_{Core}) were chosen as model calibration parameters. A range between 0.8 and 1.2 was selected for β , based on values provided in the literature where a reasonable calibration was achieved. The total number of possible combinations for the three β coefficients assuming a range of 0.8–1.2 with a 0.1 spacing is 5 (β -values)³(β -coefficients) = 125, which would result in a very large number of simulations (125×8 wave conditions = 1000 simulations). To limit the number of simulations needed to provide enough information for the development of the ANN for prediction of the best estimates of β , six representative combinations of β values were used (Table 6) resulting in 8 (wave conditions) $\times$ 6 (coefficient combinations) = 48 simulations ($\ll 1,000$, around 5%). The six combinations of values were chosen to represent enough variability such that the ANN could identify trends in the non-linear behaviour of the three coefficients. The simulation time for each case varied between 40 and 92 h. The 48 runs were done simultaneously in a workstation since IH2VOF runs on a single processor.

The MATLAB® Neural Network toolbox (Beale et al., 2019) was used in this study. A feed-forward backpropagation multilayer perceptron type of ANN with a sigmoid transfer function in the hidden layer and a linear transfer function in the output layer was applied. The input parameters for the ANN are H_{m0} , $T_{m-1,0}$, n_{Armour} , β_{Armour} , β_{Filter} , β_{Core} , and the output is the mean overtopping discharge, q . In this study, 4 input layers, 2 hidden layers, and 1 output layer (Fig. 8) were used. The early stopping criterion was used in the ANN to prevent overlearning. In this technique, the available data is divided into three subsets. The first subset (70% of the data) is the training set, which was used for computing the gradient and updating the network weights and biases. The second subset (15% of the data) is the validation set. The error on the validation set was monitored during the training process. The validation error normally decreases during the initial phase of training, as does the training set error. However, when the network begins to overfit the data, the error on the validation set typically begins to rise. When the

Table 3
Characteristics of tested meshes.

Mesh	$\Delta x_{MIN}/\Delta x_{MAX}$ (cm)	$\Delta y_{MIN}/\Delta y_{MAX}$ (cm)	no. x/no. y
80%	0.8/1.6	0.4/0.8	1210/195
100%	1.0/2.0	0.5/1.0	995/150
120%	1.2/2.4	0.6/1.2	830/130
140%	1.4/2.8	0.7/1.4	705/110

Table 4
Mesh sensitivity results.

Mesh	Q/Q_{REF} (%)	Kr/Kr_{REF} (%)	Sim. Time (days/1000s)
80%	−3	+2	6.3
100%	−	−	2.7
120%	+10	−1	1.3
140%	+11	−4	0.7

Table 5
Wave conditions at the toe of the structure and mean overtopping discharges from the physical model. $T_{m-1,0}$ represents the mean energy wave period.

Case.	n_{Armour}	H_{m0} (m)	$T_{m-1,0}$ (s)	q (L/s/m)	$q_{prototype}$ (L/s/m)
O01	0.29	0.08	1.0	0.00	0.1
	0.33	0.08	1.0	0.00	0.0
O02	0.29	0.10	1.2	0.01	1.9
	0.33	0.10	1.2	0.01	1.3
O03	0.29	0.12	1.3	0.07	13.6
	0.33	0.12	1.3	0.05	9.5
P01	0.29	0.08	1.5	0.00	0.1
	0.33	0.08	1.5	0.00	0.1
P02	0.29	0.10	1.7	0.01	2.1
	0.33	0.10	1.7	0.01	1.3
P03	0.29	0.13	1.9	0.08	17.1
	0.33	0.13	1.9	0.05	10.6

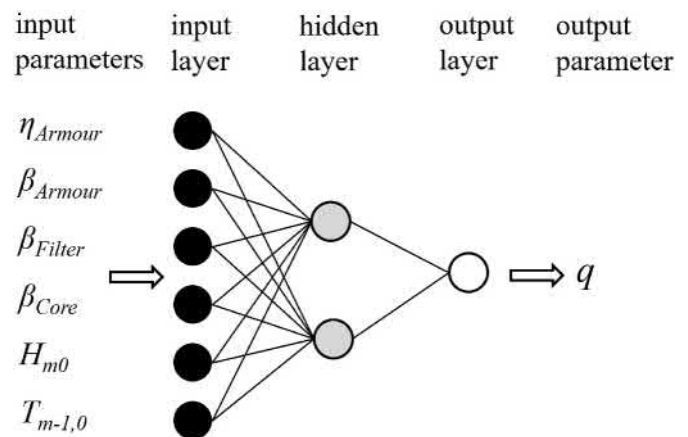


Fig. 8. Architecture of the ANN used in this study.

Table 6
Values for the β coefficients adopted in simulations.

Coefficient	ANN1	ANN2	ANN3	ANN4	ANN5	ANN6
β_{Armour}	1.2	1.0	1.0	1.2	0.8	0.8
β_{Filter}	1.2	1.2	0.8	0.8	1.0	1.0
β_{Core}	0.8	1.0	1.2	0.8	1.2	1.0

validation error increases, the training is stopped, and the weights and biases at the minimum of the validation error are returned (Beale et al., 2019). The testing subset (15% of the data) was used as cross-validation. Feed-forward backpropagation networks may have drawbacks; among them is the results' sensitivity to the initialization weights, which are randomly set at the beginning of the training process (Iglesias et al., 2008). It is therefore recommended to train the ANN several times (Beale et al., 2019) to ensure good accuracy of the results. In this study, 50 networks (with different initial weights) were tested and the best performing ANN selected based on the lowest normalized Mean Square Error ($nMSE$) defined as:

$$nMSE = \frac{\sum_{i=1}^n (y_i - x_i)^2}{Var(x)} \quad (2)$$

where Var represents the variance of the data, N the number of data, y_i the estimates, and x_i the observations. A lower value of $nMSE$ corresponds to better prediction.

3. Results and discussion

3.1. Selection of β coefficient values

Results from the numerical 48 simulations (6 combinations of β values $\times$ 8 wave conditions) were used to develop an ANN for estimation of the best combination of β values. The $nMSE$ was calculated for each of the 50 ANNs (each with different initial weights). The best performing ANN was number 41 (Fig. 9 – top) with a $nMSE = 0.5\%$. The agreement between the model results (for each of the 48 simulations) against the ANN estimate is very good (Fig. 9 – bottom).

An input matrix for the 125 possible combinations of β values for the three breakwater layers and 8 wave conditions (1000 cases) was generated to be used as input for the ANN. The $nMSE$ for the mean overtopping discharges (ANN-estimated vs. actual-measured) was calculated for each set of β values. Higher β_{Armour} and β_{Core} and lower β_{Filter} seems to result in a lower $nMSE$ (Fig. 10). The best combination of values based on the ANN estimates is $\beta_{Armour} = 1.2$, $\beta_{Filter} = 0.8$, $\beta_{Core} = 1.2$ ($nMSE = 7.9\%$). These values were then used in the numerical model and the results are presented in the following section.

3.2. Model validation

The comparison between the measured incident wave conditions and the numerical model results for the selected β values show a good agreement, with a $nMSE$ of 2.6% and 1.5% for H_{m0} and $T_{m-1,0}$, respectively (Fig. 11). A snapshot of an overtopping event for one of the simulations is depicted in Fig. 12 for illustrative purposes.

A good agreement between the mean overtopping discharge obtained with the numerical model using the β values and those estimated by the ANN was achieved with a $nMSE$ of 1.4% (Fig. 13). The main difference is for the cases with lower overtopping rates (O02 tests in Table 5 with measured $q_{prototype}$ of 1.9 and 1.3 L/s/m), for which no overtopping was obtained in the model (Fig. 13). For smaller average overtopping rates in physical models ($q < 1$ L/s/m), uncertainties in measurements can become large (Troch et al., 2004) and therefore there is typically more scatter in the data. The reliability of the 90% confidence interval prediction with the empirical formula of EurOtop (2018) is between a factor of 2.5 and 20 (for very small overtopping) (Van der Meer et al., 2016). It was also noted in Zanuttigh et al. (2016) that despite the zero values in the datasets of physical model experiments, the trained ANN in that study could predict relatively small non-zero values.

Model results for the β values from Losada et al. (2008) (β_{Los}), Jensen et al. (2014) (β_{Jen}), and this study (β_{Vie}) are compared against the data obtained from the laboratory experiments. The comparison between the modelled and measured reflection coefficients and non-dimensional mean overtopping rates, $q^* = q/\sqrt{gH_{m0}^3}$, between the measurements and model results with β_{Los} , β_{Jen} , and β_{Vie} is depicted in Fig. 14 and

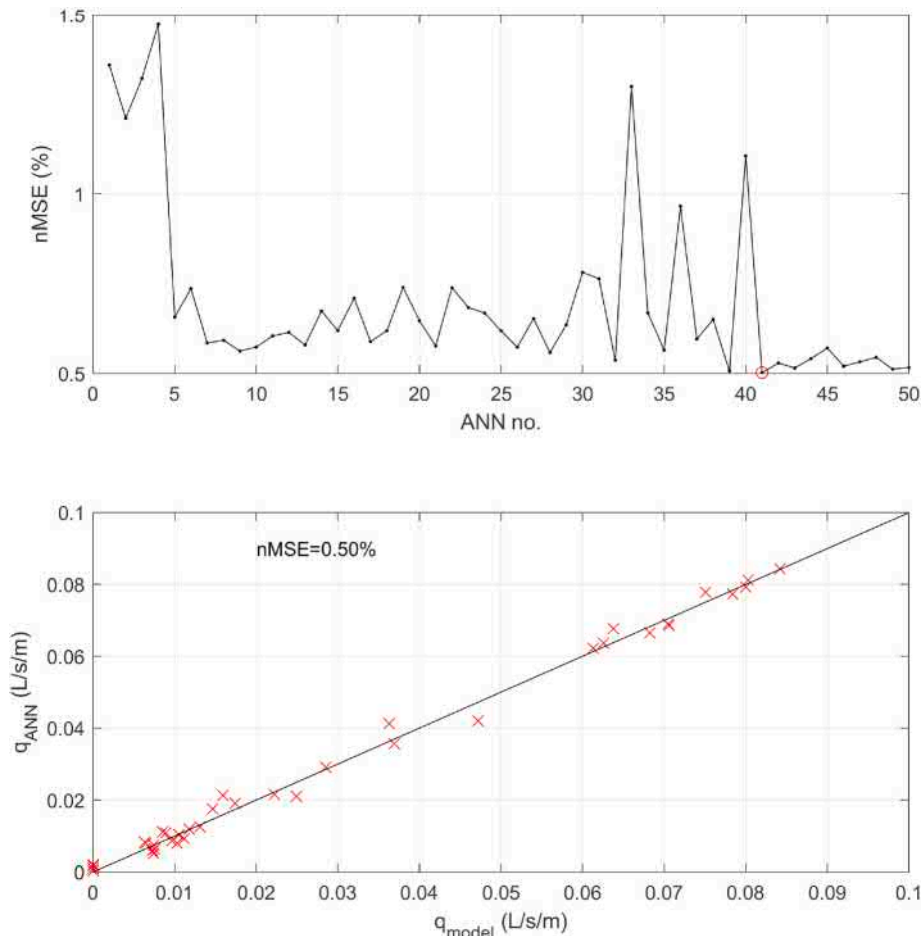


Fig. 9. $nMSE$ for all ANNs tested (top) and comparison between modelled and ANN estimated mean overtopping discharges (bottom).

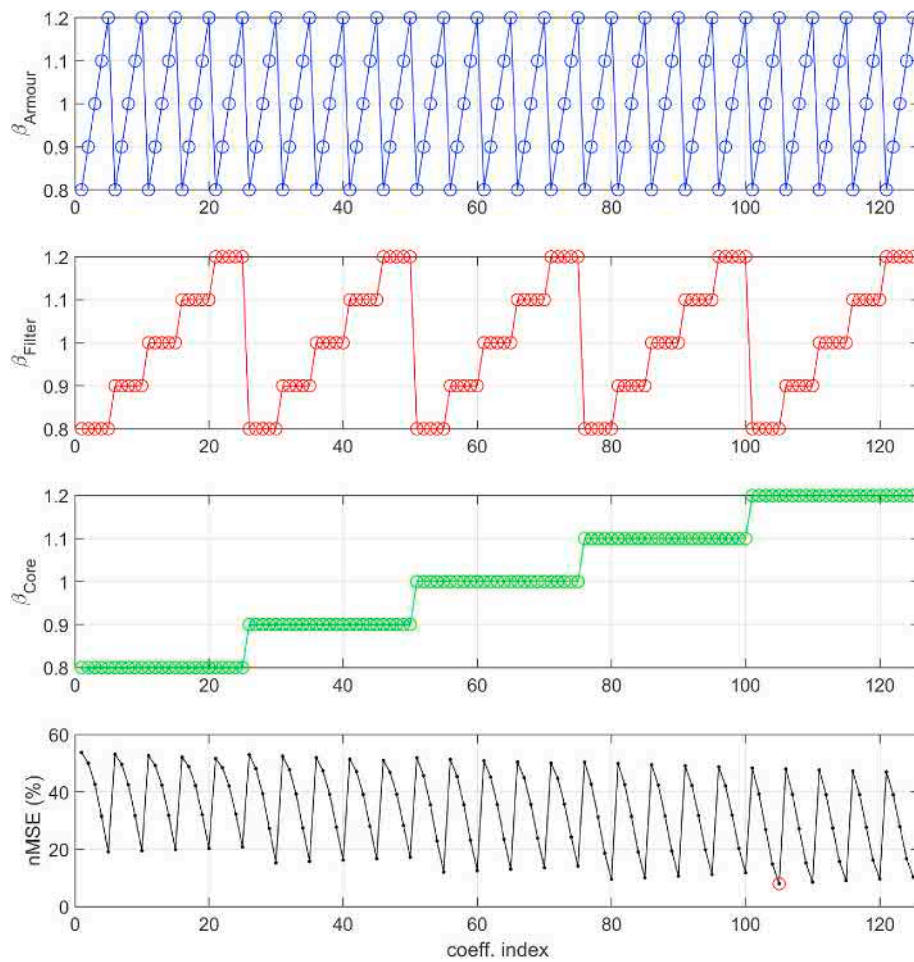


Fig. 10. β values (top three figures) and corresponding nMSE between ANN-estimated and measured mean overtopping discharges (bottom).

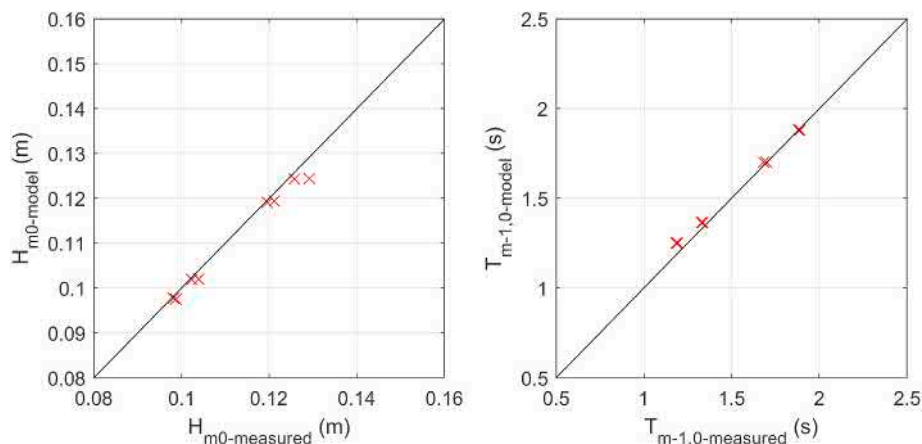


Fig. 11. Comparison between incident modelled and measured H_{m0} (left) and $T_{m-1,0}$ (right) at the toe of the structure (wave probe S06 – see Fig. 4).

presented in Table 7. Model results with β_{Los} and β_{Vie} appear to underestimate the reflection coefficient obtained in the laboratory experiments (nMSE of 99.5% and 38.6%). K_r values obtained with β_{Jen} agree better with the measurements (nMSE of 14.7%). The root mean square error obtained with the model for β_{Vie} is 4.2%, which is slightly higher than the values presented in Zanuttigh and van der Meer (2008) (3.7% for armour units) based on an empirical formulation, and therefore it is considered acceptable. The non-dimensional overtopping rates obtained with β_{Vie} show a good agreement compared to the measurements and

perform better than using values of β from the literature (Fig. 14 and Table 7). This combination of β values results in the best estimates of q^* which was the main objective of this work. Similar performance could eventually be obtained with different combinations of α and β values.

Differences between measurements and model results are inevitably expected for several reasons. For example, the structure slope in the 2D numerical model was imposed as a straight line whilst in reality there will be irregularities along with the slope profile. The actual slope, obtained from a laser scanner, could be implemented in the model but only

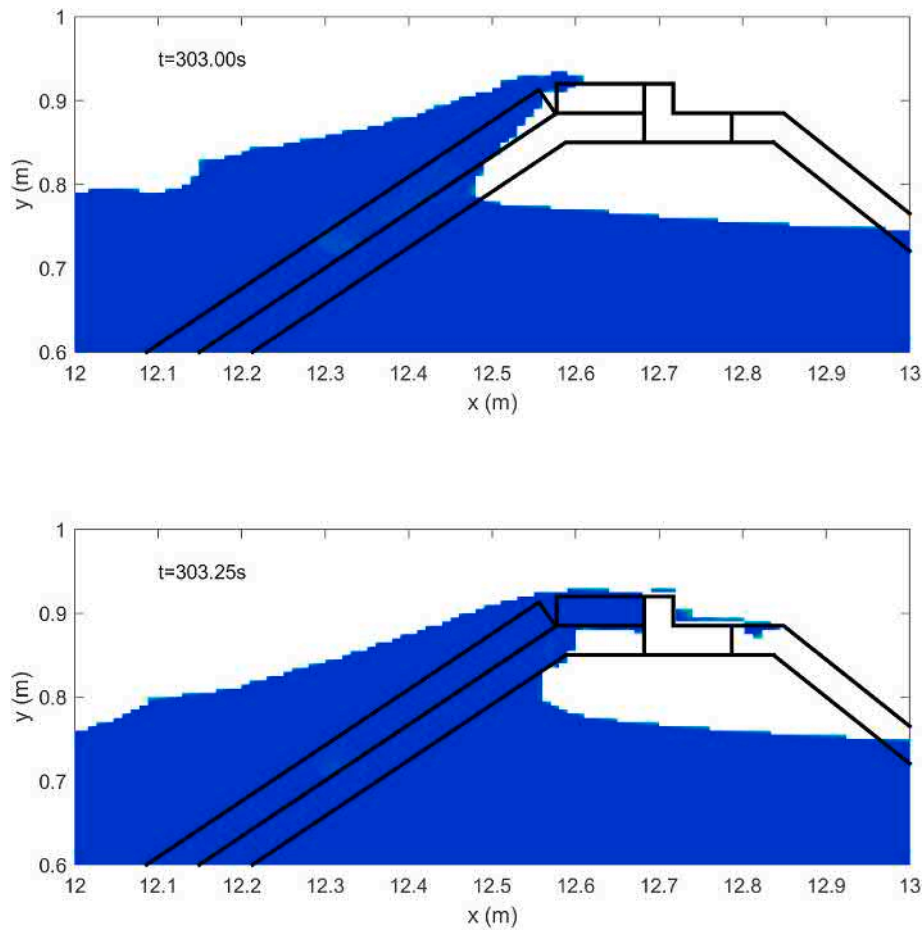


Fig. 12. Snapshots of an overtopping event for two consecutive time steps.

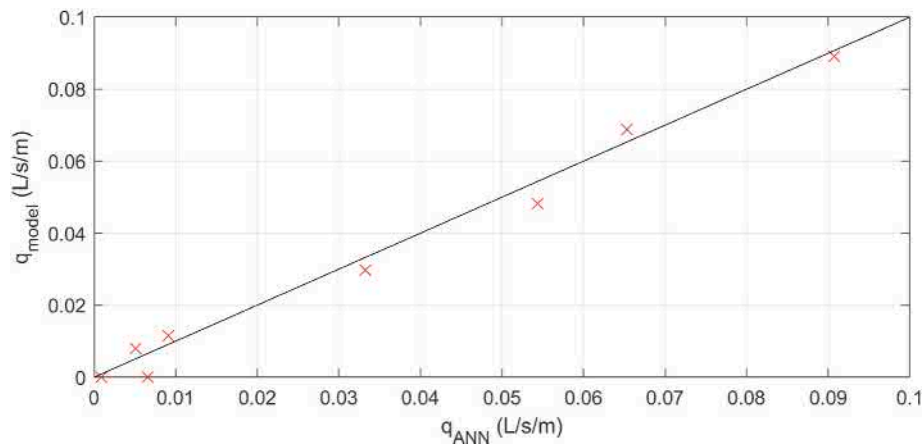


Fig. 13. Comparison of mean overtopping discharges between ANN estimate and model prediction.

one profile would be considered since a 2D model was used. Also, the mesh resolution, as shown in section 2.2.3, will result in some differences in results depending on the selected mesh size. The mesh size selection is always a compromise between accuracy and computational time constraints. In this study, different initial seed numbers for the generation of the wave series in the numerical model and experiments were used. In Vieira et al. (2021) the influence of a different initial seed number for the wave generation signal was carried out based on some of the tests presented in this study. The difference for prototype mean overtopping discharges above 1 L/s/m is within the range of 0–8%. For

the cases with smaller mean overtopping discharges ($q_{prototype} < 1$ L/s/m) the maximum differences are between 80 and 130%. Romano et al. (2015) carried out physical model tests for a rock armoured breakwater with different initial seed numbers. They found that, when the relative freeboard is larger than 1.4, the variability of the overtopping discharge may reach one order of magnitude for the same wave condition. Based on all of the above aspects, it is considered that the validation is adequate and the numerical model can be used for evaluating mean wave overtopping discharges for other incident wave conditions (within the range of validated values) and investigate the performance for slight

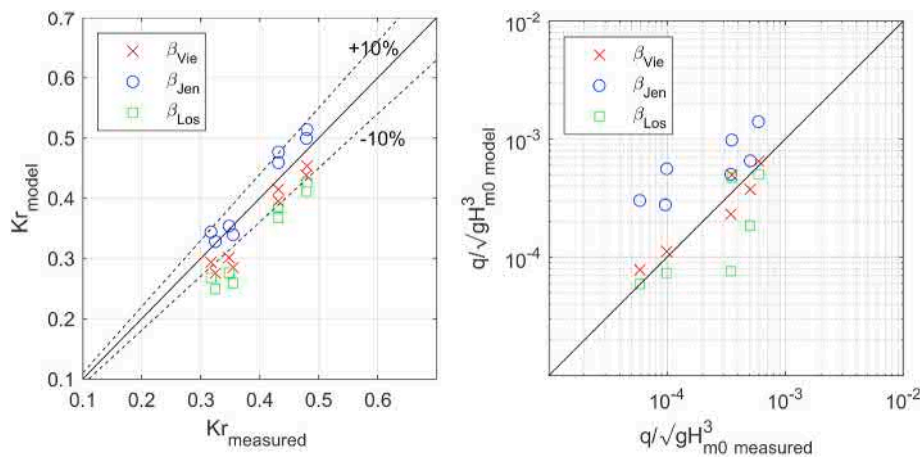


Fig. 14. Comparison between model and measurements of Kr (left) and q^* (right).

Table 7

nMSE (%) for Kr and q for the various β values.

β values	Kr	q^*
Losada et al. (2008)	99.5	47.4
Jensen et al. (2014)	14.7	301.8
This study	38.6	13.9

geometry modifications.

4. Conclusions

A correct calibration and validation of numerical models against physical model tests are essential to properly represent the wave-structure interaction processes. A precise model validation provides confidence to study properties not easily measured in the laboratory, to assess different structure geometries and/or incident wave and water level conditions. In the traditional “trial-and-error” approach adopted in previous studies, it is difficult to ensure the best combination of coefficients is achieved. To this end, this study presents a novel methodology for the calibration of the porous media coefficients in a VARANS-VOF model to obtain the optimum values for the simulation of wave overtopping discharges. The new method consists of running a limited number of simulations using different combinations of coefficient values (one for each layer) to be used in the development of the ANN. The ANN is subsequently used to evaluate a large range of different combinations of coefficient values to select the best performing in terms of mean wave overtopping discharge. The selected coefficient values are then applied to the numerical model and the results are validated against 2D physical model test results for single-layer cube (SLC) armoured breakwaters. This method is shown to perform well and results in a reduced number of simulations (48) compared to the required number to assess all possible combinations of values (1,000) considered in this study. A $nMSE$ of 13.9% was achieved for non-dimensional mean overtopping discharges between the model and laboratory test results, which is better than if values from existing literature for similar cases are applied.

The ANN developed in this study aimed at reducing the error in mean overtopping discharge estimates for the entire range of wave conditions and armour layer porosities with a unique combination of coefficient values. A better combination of values could be achieved if the objective was to focus on a single incident wave condition and armour layer porosity. For example, during the design stages, it is common to investigate the influence of structure geometry modifications on overtopping performance for the design condition (for example, the 100-year return period event). Depending on the focus of the model application, an ANN to simultaneously reduce the error in both mean wave overtopping and

reflection coefficient could be developed in the future.

CRediT authorship contribution statement

Filipe Vieira: Funding acquisition, Conceptualization, Methodology, Investigation, Visualization, Formal analysis, Writing – original draft. **Francisco Taveira-Pinto:** Funding acquisition, Writing – review & editing. **Paulo Rosa-Santos:** Resources, Writing – review & editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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