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RUN-UP ON A RUBBLE MOUND BREAKWATER: COMPARISON OF EMPIRICAL FORMULAE AND PHYSICAL MODEL RESULTS

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ABSTRACT

The present study focuses on the comparison of measured wave run-up values obtained in physical model tests with those predicted by different empirical formulae. Tests were carried out in a flume for a cross-section of a rubble mound breakwater with an armour layer of tetrapods and rock, and a recurved wave return wall. The model represents the cross-section of the south breakwater of Praia da Vitória harbour (Azores, Portugal) that directly protects quay 12. Two water levels and several incident wave conditions were tested. Empirical run-up formulae from Van der Meer & Stam (1992), Pullen *et al*. (2007) and Bonakdar & Etemad-Shahidi (2011) were applied for the tested conditions and comparisons were made with experimental values of the two percent wave run-up, $R_{2\%}$. Generally, the agreement between predictions and measurements was better for mean low water than for mean high water springs, with all formulae over predicting measurements for the latter.

Keywords: Run-up / Physical modelling / Empirical formulae / South Breakwater of Praia da Vitória

1. INTRODUCTION

Most climate scenarios predict the sea-level rise, as well as increased intensity and frequency of storms (IPCC, 2013). In order to improve the efficiency of the armour slope of rubble mound breakwaters (the most common coastal structure in Portugal) in these conditions, it is needed to understand its response with regard to the phenomena of wave run-up and overtopping, and hydraulic stability.

In particular, wave run-up characteristics on coastal structures are crucial for predicting the occurrence of overtopping, for studying coastal flooding and/or for evaluating the impact of this phenomenon on people's safety, on the integrity of goods and infrastructure, and on the normal performance of economic activities at the areas protected by these structures.

Furthermore, the run-up of coastal structures is one of the major physical phenomena to be considered in the design of new structures and in the safety assessment of existing ones, especially for climate change scenarios. The main objectives of its determination are the definition of the crest level of the structure, the identification of eventual overtopping events and of transmission through the structure.

There are several tools for assessing wave run-up: empirical methods such as formulae, artificial neural networks and model trees (e.g. Losada & Gimenez-Curto, 1981; Allsop *et al*., 1985; Van der Meer & Stam, 1992; Pullen *et al*., 2007; Bonakdar & Etemad-Shahidi, 2011); physical and numerical modelling (e.g. Dodd, 1998; De Rouck *et al*., 2001; Shiach, 2008); and field campaigns (e.g. De Rouck *et al*., 2007; Wenneker *et al*., 2016). The empirical formulae are the most expedite tool but they have their own limitations, mainly related to the assumptions and parameters in which they are based, and so their applicability should be assessed for each case study.

The Praia da Vitória' s south breakwater, located on the east coast of the Terceira Island, in the Azores archipelago, is one of the structures where wave run-up and overtopping are quite common and some events have a very high intensity. To understand the wave propagation, run-up, overtopping and armour stability of the cross-section that protects quay 12 of the commercial harbour and to obtain data for the calibration and validation of empirical formulae and numerical models, a set of physical model tests of this cross-section was performed in the framework of the HIDRALERTA project (Fortes *et al*., 2015). This was a good opportunity to evaluate the applicability of some existing formulae to this breakwater and to the wave/water level conditions at the site.

The present work focuses on the comparison of measured two percent wave run-up values, $R_{2\%}$, obtained in physical model tests and predicted by different empirical formulae, namely Van der Meer & Stam (1992), Pullen *et al*. (2007) and Bonakdar & Etemad-Shahidi (2011). The paper describes the study case, physical model tests and empirical formulae. Comparison between physical model and empirical formulae results are also presented and discussed.

2. CASE STUDY: SOUTH BREAKWATER OF PRAIA DA VITÓRIA (AZORES)

The port and bay of Praia da Vitória are located on the east coast of the Terceira Island, in the Azores archipelago. Praia da Vitória is sheltered by the north and the south breakwaters, which define a roughly rectangular basin with about 1 km x 2 km (Fig. 1).

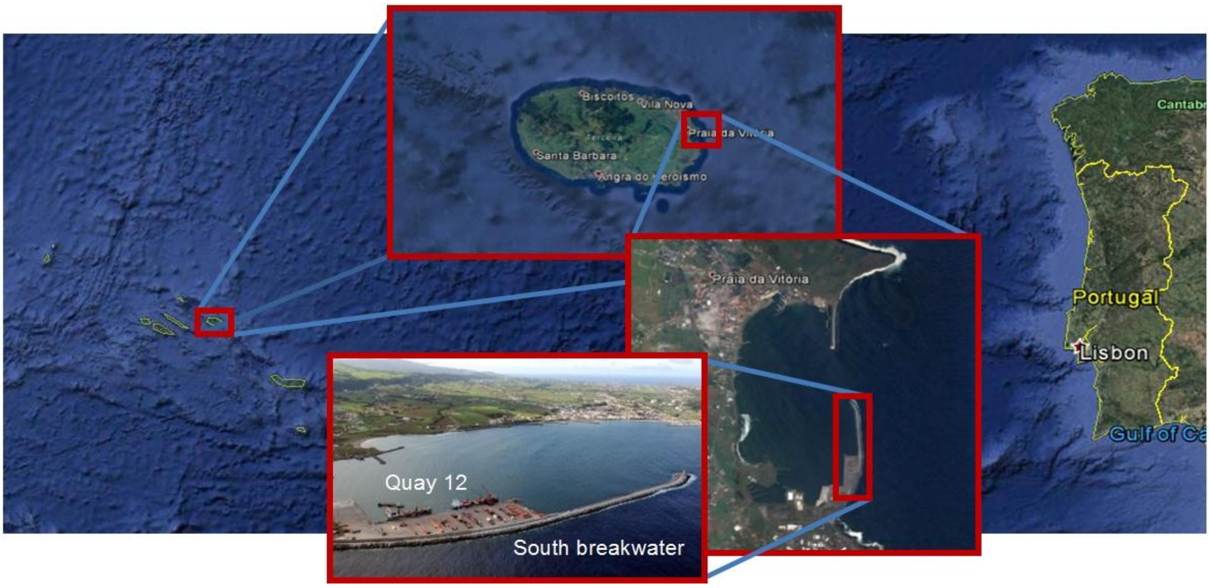


Fig. 1 – Location of the south breakwater of Praia da Vitória harbour and aerial view of the breakwater and quay 12 (Source: Google Earth)

The south breakwater was built in the 1980's and is rooted on the south end of the bay, near the Santa Catarina fort. This breakwater is approximately 1300 m long, aligned N-S, bending close to land. The main function of this structure is to protect the commercial (mainly quay 12) and fishing facilities. The breakwater cross-section that protects quay 12 has a recurved wave return wall to increase the protection from wave overtopping provided to this area.

Wave run-up and overtopping are very common, with an increase of these occurrences in the last years. For example, on 15 January 2016, hurricane Alex reached the Azores Archipelago and several events of wave run-up and overtopping occurred at the south breakwater with very high intensity (Fig. 2). The impact of these events may be on the stability of the structure, on the integrity of containers and equipment stored at quay 12, as well as on port activities.

Fig. 2 – Hurricane Alex hitting the south breakwater of Praia da Vitória harbour on 15 January 2016

3. METHODS

3.1 Physical model set-up and test conditions

Physical model tests were conducted in one of LNEC's irregular wave flumes (Fig. 3), which is approximately 50 m long and it has an operating width and an operating water depth of 80 cm. It is equipped with a piston-type wave-maker and an active wave absorption system, AWASYS (Troch, 2005), which allows the dynamic absorption of reflected waves.

Fig. 3 - Overview of the irregular wave flume

The model was built and operated according to Froude's similarity law, with a geometric scale of 1:48. The impermeable bottom was composed by a 21 m horizontal stretch, followed by a 1:39 foreshore, 11.55 m long. Twelve wave gauges were distributed along the flume - B1, B2, S1, L1 to L8 and S2 (Fig. 4). Three resistive-type wave gauges (B1, B2 and S1) were installed close to the wave-maker to measure the free-surface elevation (Fig. 4). Seven other resistive-type wave gauges (L1 to L7) were deployed along the flume, also to record the free-surface elevation. One wave gauge was positioned on the armour slope (S2) for measuring wave run-up and a wave gauge (L8) was located at the top of the recurved wall to identify overtopping events. Every gauge recorded at a sampling frequency of 50 , except for gauges B1 and B2, which recorded at 40 Hz.

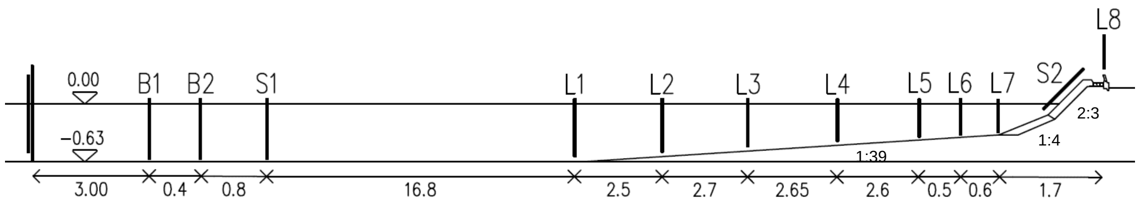


Fig. 4 - Gauge's location along the flume - B1, B2, S1, L1 to L8 and S2 (units: m)

The tested cross-section consisted of an upper armour slope of 2(V):3(H) with two layers of 200 g tetrapods and a lower 1:4 slope with two layers of 5 to 80 g rock (Fig. 5). The structure crest was composed of three distinct parts: a 16.78 cm wide armour crest, with a freeboard of 26.25 cm (relative to mean low water); a 16.78 cm wide concrete slab (6.25 cm permeable and 10.42 cm impermeable), with a freeboard of 23.13 cm; and a recurved wave return wall, with a freeboard of 31.25 cm.

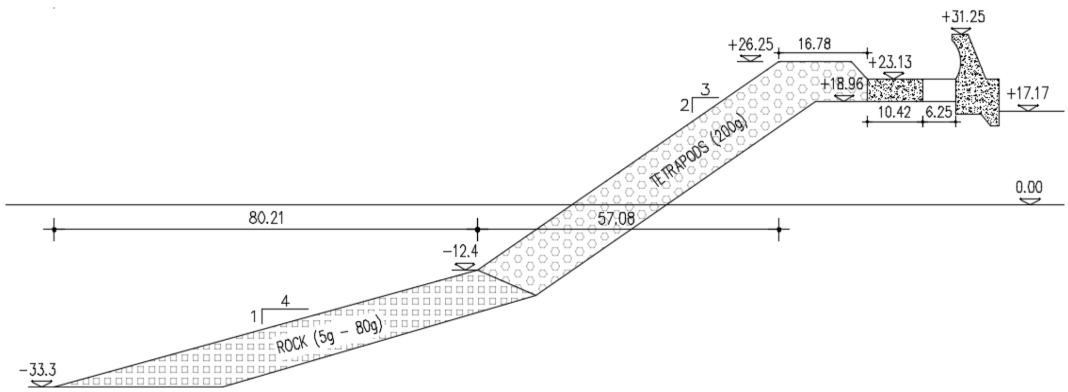


Fig. 5 - Tested cross-section (units: cm)

Tests were carried out for two water levels of 0.00 m(CD) and +2.00 m(CD) (prototype values) corresponding to Mean Low Water (MLW) and Mean High Water Springs (MHWS), respectively. A JONSWAP spectrum (with a peak enhancement factor of 3.3) was used, with five nominal spectral peak periods, T_p , of 8 s to 18 s, and several spectral significant wave heights, H_{m0} , of 5 m to 9 m. These prototype values corresponded to the measured T_p and H_{m0} values in front of the structure, based upon L5, L6 and L7 wave gauges. Each test was run for a single water level, a significant wave height and a spectral peak period, during a time period equivalent to 1000 waves (approximately 3 or 4 hours in the prototype). Table 1 summarises the nominal tested conditions in front of the structure and the number of repetitions for each test. A total of 130 tests were performed.

Table 1 - Number of repetitions for all nominal wave and water level conditions in front of the structure (in prototype values).

Water level	MHWS (2 mCD)					MLW (0 mCD)			
T_p (s) / H_{m0} (m)	8	10	12	14	18	8	10	12	14
5.0	2	-	-	-	-	-	-	-	-
5.5	2	3	-	4	2	3	2	-	2
6.0	-	-	4	-	4	-	-	2	-
6.5	-	3	3	3	-	-	2	2	4
7.0	-	-	3	-	-	-	3	2	-
7.5	-	2	3	11	-	-	2	2	7
8.0	-	3	3	2	-	-	-	-	3
8.5	-	-	3	20	-	-	-	-	3
9.0	-	-	-	11	-	-	-	-	-

Fig. 6 illustrates a physical model test and the location of the run-up wave gauge (S2), which follows the slope of the tetrapod armour.

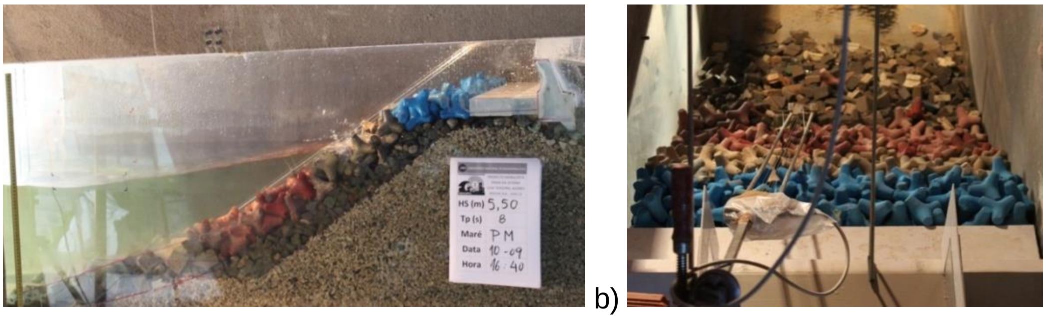
a)  b)

Fig. 6 - Physical model tests: a) cross-section tested; b) run-up wave gauge

3.2 Empirical formulae

In the present work, 2% run-up experimental values, $R_{2\%}$, obtained during the physical model tests are compared with the predicted values obtained through three different empirical formulae, namely Van der Meer & Stam (1992), Pullen *et al*. (2007) and Bonakdar & Etemad-Shahidi (2011). These formulations are briefly described in the next paragraphs.

<u>Van der Meer & Stam (1992) formulae</u>

Van der Meer & Stam (1992) proposed Eqs. (1) to Eq. (3) to calculate the average trend of experimental run-up values exceeded by *2%* of run-up events in a test, $R_{2\%}$, for permeable (*P*=0.5, with *P* the notional permeability factor) and homogenous (*P*=0.6) rock-armoured slopes:

$$R_{2\%} / H_s = 0.96\xi_m \quad \text{for } 1.0 < \xi_m \leq 1.5 \tag{1}$$

$$R_{2\%} / H_s = 1.17\xi_m^{0.46} \quad for\ 1.5 < \xi_m \leq 3.1 \tag{2}$$

$$R_{2\%} / H_s = 1.97 \quad for\ 3.1 < \xi_m \leq 7.5 \tag{3}$$

where H_s is the significant wave height defined as the highest one-third of wave heights; ξ_m is the breaker parameter given by $\xi_m = tan\,\alpha / (s_m)^{0.5}$, where α is the angle of the structure front slope measured from the horizontal, $s_m = H_s / L_m$ is the wave steepness, $L_m = gT_m^2 / 2\pi$ is the mean deep water wavelength, based on the mean wave period obtained from time-domain analysis, T_m, and the acceleration due to gravity, g.

These equations are valid for relatively deep water in front of the structure, for a Rayleigh distributed wave height and are based on laboratory tests performed mostly with a standard Pierson-Moskowitz spectrum.

<u>Pullen *et al.* (2007) formulae</u>

Pullen *et al*. (2007) prediction for the 2% mean wave run-up value, $R_{2\%}$, for rock and rough slopes can be described by:

$$R_{2\%} / H_{m0} = 1.65\gamma_b \gamma_f \gamma_\beta \cdot \xi_{m-1,0} \tag{4}$$

with a maximum of

$$R_{2\%} / H_{m0} = 1.00\gamma_{f\,surging}\gamma_\beta \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}} \right) \tag{5}$$

where γ_b is the influence factor for a berm; γ_f is the influence factor for structure roughness; γ_β is the influence factor for oblique wave attack; $\xi_{m-1,0}$ is the breaker parameter calculated as $\xi_{m-1,0} = tan\,\alpha / (s_{m-1,0})^{0.5}$, with $s_{m-1,0} = H_{m0} / L_{m-1,0}$, $L_{m-1,0} = gT_{m-1,0}^2 / 2\pi$ and $T_{m-1,0}$ the spectral mean period, calculated with moments m_{-1} and m_0 of the spectrum.

From $\xi_{m-1,0} = 1.8$, the roughness factor $\gamma_{f\,surging}$ increases linearly up to 1 for $\xi_{m-1,0} = 10$, which can be described by:

$$\begin{aligned} &\gamma_{f\,surging} = \gamma_f + (\xi_{m-1,0} - 1.8)(1 - \gamma_f)/8.2 \quad for\ 1.8 < \xi_{m-1,0} \leq 10 \\ &\gamma_{f\,surging} = 1.0 \quad for\ \xi_{m-1,0} > 10 \end{aligned} \tag{6}$$

For a permeable core, $R_{2\%} / H_{m0}$ reaches a maximum, given by:

$$R_{2\%} / H_{m0} = 1.97 \tag{7}$$

Pullen *et al*. (2007) provide values for roughness factors, γ_f, for permeable rubble mound structures with different types of armour layer (Table 2).

Table 2 - Values for roughness factors, γ_f, for permeable rubble mound structures (slope of 1:1.5) with different types of armour layer. Values in italics are estimated/extrapolated (adapted from Pullen et al., 2007).

Type of armour layer	γ_f	Type of armour layer	γ_f
Smooth impermeable surface	1.00	Antifers	0.47
Rocks (1 layer, impermeable core)	0.60	HARO's	0.47
Rocks (1 layer, permeable core)	0.45	Accropode™	0.46
Rocks (2 layers, impermeable core)	0.55	Xblock	0.45
Rocks (2 layers, permeable core)	0.40	CORE-LOC	0.44
Cubes (1 layer, random positioning)	0.50	Tetrapods	0.38
Cubes (2 layers, random positioning)	0.47	*Dolosse*	*0.43*

<u>Bonakdar & Etemad-Shahidi (2011) formulae</u>

Bonakdar & Etemad-Shahidi (2011) investigated wave run-up on rubble mound structures using M5' model trees (Wang & Witten, 1997), trained and tested with the experimental data set of Van der Meer & Stam (1992) and validated with the prototype run-up measurements on the Zeebrugge breakwater, Belgium (De Rouck *et al*., 2007). Their prediction for the 2% wave run-up, $R_{2\%}$ for rock and rough slopes is as follows:

$$R_{2\%} / H_s = 0.86\xi_m^{0.69} \quad \text{for } \xi_m \le 2.1 \tag{8}$$

$$R_{2\%} / H_s = 1.16\xi_m^{0.31} \quad \text{for } 2.1 < \xi_m \le 3.9 \tag{9}$$

$$R_{2\%} / H_s = 1.56\xi_m^{0.15} \quad \text{for } \xi_m > 3.9 \tag{10}$$

3.3 Wave characteristics and wave run-up analysis

With the time series of the free surface elevation, measured at the toe of the structure (wave gauge 7), a time domain analysis and a spectral analysis were applied. The time analysis provided the values of H_s and T_m for each incident time series of surface elevation, while the spectral analysis gave H_{m0}, $T_{m-1,0}$ and T_p values.

A time analysis of the values recorded by the run-up wave gauge (S2) supplied the $R_{2\%}$ experimental values. Note that despite the definition of wave run-up, as the maximum vertical extent of wave uprush on a structure above the still-water-level (SWL), the run-up measurement accuracy in these experimental tests is limited by the breakwater crest. So, only run-ups below the armour crest are considered accurate. On the other hand, if the run-up exceeds the crest of the wave return wall, there will be wave overtopping, which means that there will be less backrush to affect uprush.

Taking into account that the studied section is composed by two different slopes of 1:4 (rock, γ_f =0.40, Table 2) and 2:3 (tetrapods, γ_f =0.38), for all formulations, the breaker parameter, ξ , was determined for an equivalent slope and an equivalent roughness factor, both calculated iteratively according to the recommendations of Pullen *et al*. (2007).

4. RESULTS AND DISCUSSION

In this section, run-up results obtained both from the physical model data and from the analysed empirical formulae are illustrated for the two water levels of MHWS and MLW and the peak wave period of 14 s. Fig. 7 presents the $R_{2\%}$ values as a function of H_{m0} or H_s (depending on the formulae analysed), whereas Fig. 8 presents $R_{2\%} / H_{m0}$ or $R_{2\%} / H_s$ values as a function of $\xi_{m-1,0}$ or ξ_m . It can be concluded that, in general:

- For both MHWS and MLW, measured and predicted values of $R_{2\%}$ increase as H_{m0} or H_s increase;
- For the tested values of $\xi_{m-1,0}$ or ξ_m , measured and predicted values of $R_{2\%} / H_{m0}$ or $R_{2\%} / H_s$ increase with the breaker parameter, both for MHWS and MLW;
- For MHWS, results from Van der Meer & Stam (1992) formulae present the greatest deviation from measurements but the predicted values of run-up are generally bigger than the measured ones, regardless of the formula used. This may be due to the fact that once the run-up wave reaches the horizontal crest of the armour, measurements are not so reliable, because run-up water flows tend to either follow the horizontal armour crest or overtop the structure (see Fig. 6) In these cases, water loses contact with run-up gauge, which cannot detect it. This explanation may justify why most of the measured two percent run-ups are lower than the armour layer freeboard, whereas two percent run-up predictions are higher.
- For MLW, predictions by Pullen *et al*. (2007) and Bonakdar & Etemad-Shahidi (2011) are of the same order of magnitude as the measurements, and Van der Meer & Stam (1992) results are greater than the measured values.

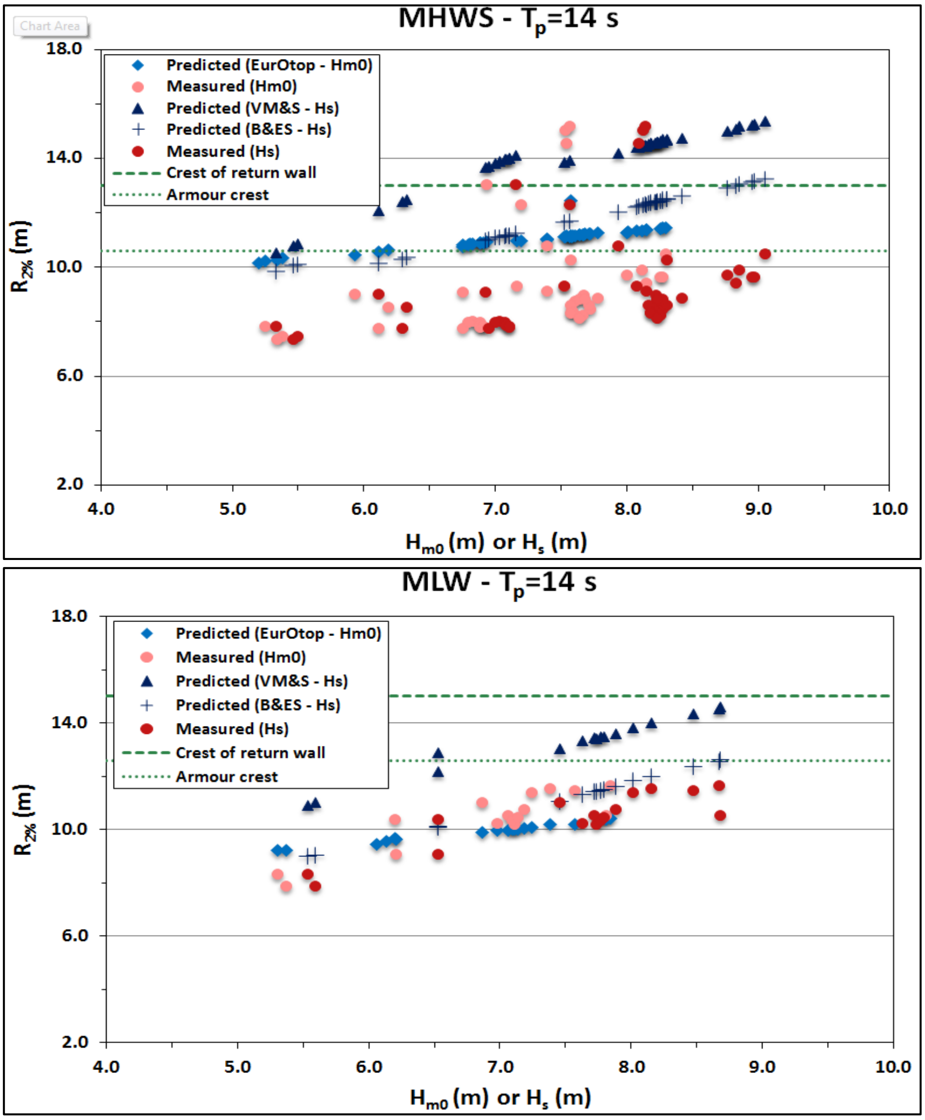


Fig. 7 – Measured and predicted $R_{2\%}$ versus H_{m0} or H_s, for MHWS and MLW and T_p of 14 s

5. CONCLUSIONS

This work presented the physical model tests carried out in a flume for measuring wave run-up on a cross-section of the south breakwater of Praia da Vitória harbour that directly protects quay 12. The cross-section is characterised by a lower rock armour slope of 1:4, an upper tetrapod armour slope of 2:3 and a recurved wave return wall. Two water levels and several incident wave conditions were tested.

The study focused on the comparison of measured two percent wave run-up ($R_{2\%}$) values with those predicted by the empirical formulae of Van der Meer & Stam (1992), Pullen *et al.* (2007) and Bonakdar & Etemad-Shahidi (2011).

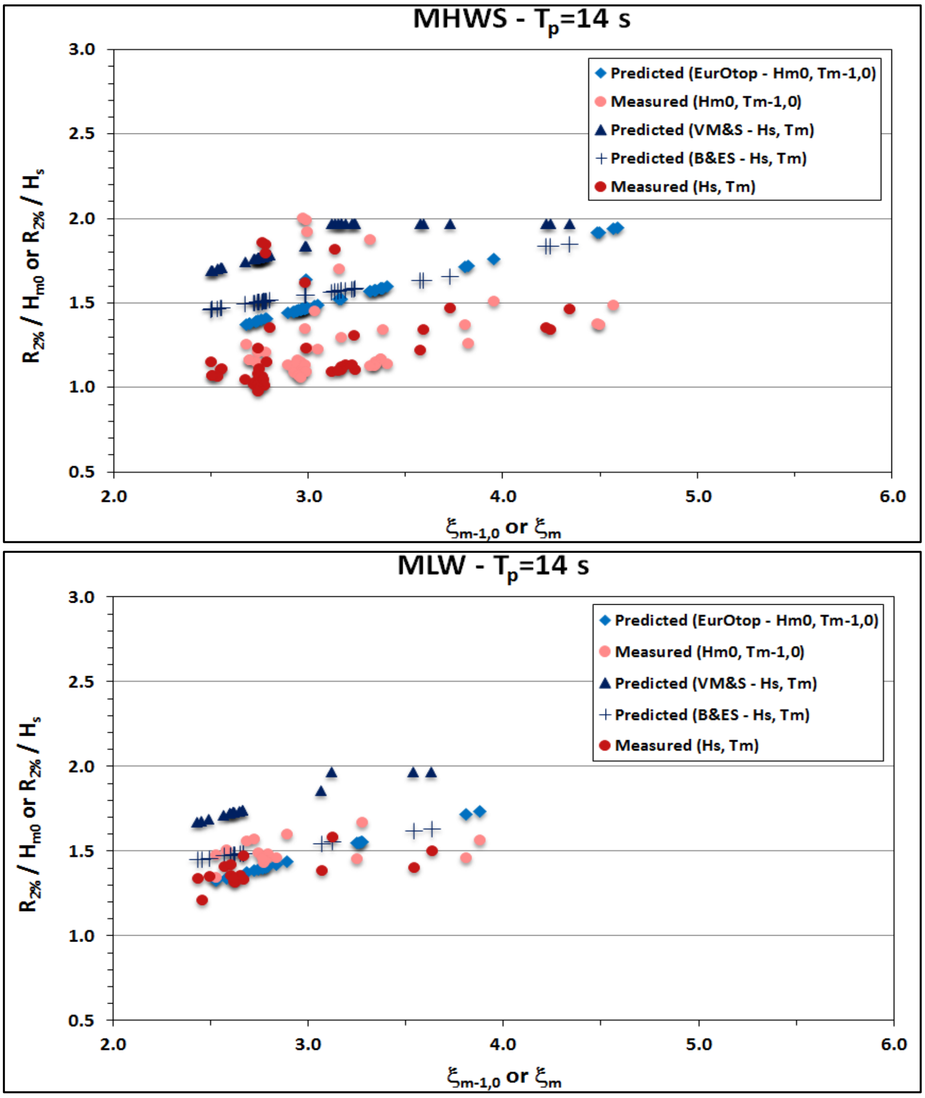


Fig. 8 - Measured and predicted $R_{2\%} / H_{m0}$ or $R_{2\%} / H_s$ versus $\xi_{m-1,0}$ or ξ_m, for MHWS and MLW and T_p of 14 s

The agreement between predictions and measurements was better for mean low water (MLW) than for mean high water springs (MHWS), with all formulae over predicting measurements for the MHWS, whereas for MLW, Pullen *et al*. (2007) and Bonakdar & Etemad-Shahidi (2011) predictions were of the same order of magnitude as the measurements.

The divergences between predictions and physical model data may be because wave run-up measurements are not so reliable once run-up waves reach the horizontal armour crest of the rubble mound breakwater. In this case, water may lose contact with run-up gauge, which cannot detect it, especially when run-up water follows the horizontal armour crest, has considerable amount of air bubbles, overtops the run-up gauge or hits the gauge strongly.

Consequently, current and future work include the comparison of presented run-up measurements and predictions with another mean of estimating run-up in physical models, by using a video monitoring technique (Andriolo *et al*., 2016).

6. ACKNOWLEDGMENTS

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