EFFECT OF WAVE STORM REPRESENTATION ON DAMAGE MEASUREMENTS OF BREAKWATERS

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ABSTRACT

In this work, the effects of storm representation on damage measurements of a breakwater are studied in a small-scale physical wave flume. In particular, a two-layer breakwater with cubic armor units randomly placed is considered. A storm recorded in the Mediterranean Sea is the basis to analyze the armor layer damage. Storm sea states are represented in the laboratory using four different methodologies: i) Real storm methodology (RSM, used here as benchmark test) ii) Classical methodology (CM) iii) Equivalent Magnitude Storm with a triangular shape (EMSI) and iv) Equivalent Magnitude Storm with a trapezoidal shape (EMSA). For the four methodologies, the experiments are repeated five times. Considering laboratory occupation times, results suggest that a minimum of five repetitions of CM should be performed to obtain an average damage value that can represent the average damage given by RSM. Although this recommendation is linked to the tested conditions, it provides useful information to set-up experiments dealing with similar breakwater and wave conditions.

Keywords: Wave storm representation, Armor layer stability, Physical modeling.

1. INTRODUCTION

Providing safe harbourage has historically been a critical engineering challenge due to a continuous increase in the economic value of ships, transported goods, and harbor infrastructures. In ocean harbors, breakwaters work by reducing wave energy transmission from seaward to leeward. For that purpose, to ensure breakwater structural integrity and serviceability, a comprehensive structural design is required (USACE, 2012).

A cost-effective selection of a breakwater frequently takes into account the water depth at the toe of the structure and design wave conditions. For depths shallower than 15 m, a rubble-mound type breakwater statistically optimizes costs (Tutuarima and D'Angremond, 1988). Though, rubble-mound breakwaters have also been constructed at depths deeper than 20 m (Molinero Guillén, 2008).

In their essence, rubble-mound breakwaters are gravity structures, which derive their stability from armor units weight and interlocking. In general, natural rocks or concrete artificial units are used in the armor layer through one or more layers, although rarely more than two layers are used (Bakker et al., 2003). Armor units with a simple bulky shape, such as natural rocks, concrete cubes, Antifer cubes (Bakker et al., 2003) and modified cubes, are mainly stabilized by their weight. On the other hand, armor units developed more recently, such as Dolos

(Burcharth and Liu, 1992), XBlocks (Hakenberg et al., 2004) or Accropodes (Holtzhausen and Zwamborn, 1991), base their stability mostly on interlocking forces (Burcharth et al., 1993). Concrete units are necessary when available rock supply in diameters/weight is not enough to deal with the design wave conditions (Muttray and Reedijk, 2008).

Several semi-empirical formulations have been proposed to calculate the stability of rubble-mound breakwaters (e.g., Hudson, 1953; Iribarren, 1938; Van der Meer, 1988a, 1988b 1999; Losada et al., 2005). Nevertheless, Van der Meer's formula (Van der Meer, 1988a, 1999) is one of the most used nowadays for rock armored slopes in deep water. In general, rubble-mound breakwaters semi-empirical formulas are calibrated with data from small/medium-scale physical tests. These tests, in general, consider only a reduced number of geometric configurations and wave conditions. Moreover, some of these formulas do not take into consideration all the variables that can affect the stability of the rubble-mound armor layer (Van der Meer, 1995). Consequently, at the initial stage of the design process, ocean engineers might often use formulations out of its validity range regarding breakwater configuration or incident wave conditions. For these reasons, a verification procedure through physical model tests is recommended, in particular when safety margins are reduced (Kirkegaard et al., 2011).

Although small-scale models have been extensively used to study the hydraulic behavior of marine structures (Huges, 1993; Kirkegaard et al., 2011), there are several uncertainties related to their use. This includes uncertainties pertaining to scale effects (Heller, 2011; Lykke Andersen, 2006; Oumeraci, 1984), length of scale models (Reis et al., 2008; Romano et al., 2015), generation of extreme waves (Oliveira et al., 2017 and references cited therein) and test repeatability (Marzeddu et al., 2017), among others.

The hydraulic behavior of a rubble-mound breakwater depends on wave-structure interaction and block-block interaction processes, which are complex phenomena that can generate a turbulent environment close to the structure (Losada et al., 2005; Sierra et al., 2010). Then, unlike semi-empirical formulae which give deterministic outputs, results from physical experiments can present high variability associated to test repeatability (Marzeddu et al., 2017; Melby and Kobayashi, 1998; Stagonas et al., 2016; van Gent et al., 2018). To perform test repetitions, the reconstruction of the damaged breakwater is mandatory. To attempt to replicate the same breakwater porosity, particular attention must be put on the random placing of the armor units during the reconstruction process. This can be a very time-consuming and expensive procedure. In fact, it could take a time comparable with the realization of a complete set of tests. However, random placement is also essential to prevent armor units from being lifted by excess pore pressure that can develop inside the breakwater (Muttray and Reedijk, 2008).

The design and modeling of rubble-mound breakwaters require the understanding of the forcing agents. Historically, significant wave height, wave period, wave direction, and water level have been considered the primary design variables for coastal structures (Van der Meer, 1995). Based on these variables, physical model tests have been carried out for the design/optimization of breakwaters (Huges, 1993). Normally, the test methodology to study armor layer stability starts by selecting the return period (typically between 40 to 500 years) for the designed structure (Puertos del Estado, 1990). Based on the chosen return period, a significant wave height can be obtained from extreme wave datasets. For that, it is assumed that extreme events follow an extreme probability density distribution. Often a Fisher-Tippet-family extreme value distribution or a Weibull distribution is considered. Then, a series of tests

with 60, 80, 100 and 120% of the obtained wave height may be performed to monitor damage evolution (Owen and Allsop, 1984).

More recently, Martín-Hidalgo et al. (2014) and Martín Soldevilla et al. (2015) studied the damage evolution of a cubic armor layer analytically with real and synthetic storm models, which are schematic representations of a real storm. These authors, considering the maximum energy flux model proposed by Melby and Kobayashi (1998), showed that the damage evolution is affected by the sequence of events in a storm. Martín Soldevilla et al. (2015), based on an exhaustive comparison between synthetic storm models, suggested that the Equivalent Magnitude Storm (EMS) model gives best results for the most common type of storms (predominant sea, swell or both). According to Martín Soldevilla et al. (2015), a triangular storm shape is recommended for typical sea storms whereas a trapezoid shape is more appropriate for more developed storm conditions. Also, by appropriately using these storm patterns it would be possible to define the evolution of the synthetic storm for the required structure lifetime and return period. Although the synthetic storm models proposed by Martín-Hidalgo et al. (2014) and Martín Soldevilla et al. (2015) can be useful to analyze the stability of rubble-mound breakwaters, the performance of these models compared to classical methodologies (Owen and Allsop, 1984) were never verified through laboratory experiments.

In this study, we aim to understand the laboratory effects of storm representation on damage measurements in the armor layer of a breakwater. For that, we study in a small-scale wave flume the stability of a two-layer breakwater with cubic armor units. Firstly, a real storm measured in the Mediterranean Sea (Bolaños et al., 2009) is simulated using a real storm methodology (RSM). Then three different representation models of the real storm are simulated: the classical methodology (CM)(Owen and Allsop, 1984) and the Equivalent Magnitude Storm models with triangular (EMSI) and trapezoidal (EMSA) shapes (Martín-Hidalgo et al., 2014; Martín Soldevilla et al., 2015). The four methodologies are repeated five times. For the assessment of the damage in the armor layer of the breakwater, the relative damage parameter based on the moved units, N_{od} (see equation (7) for a definition), proposed by Van der Meer (1999) is used.

This paper is organized into four sections. In section 2 the set-up of the small-scale hydraulic tests is introduced. Section 2 includes the presentation of the experimental layout, methodologies used for storm representation, and the technique used to measure breakwater damage. In Section 3 and 4 we present and discuss the results, respectively. Finally, in section 5 we present the main conclusions of this work.

2. PHYSICAL MODEL TESTS

2.1. Experimental Layout

The experiments were carried out in the CIEMito wave flume at the Laboratori d'Enginyeria Marítima (LIM) of the Universitat Politècnica de Catalunya (LIM-UPC BarcelonaTech). The flume is 18 m long, 0.38 m wide, and 0.56 m deep. It is equipped with a piston type wave paddle, which can generate regular and irregular waves. Moreover, a pumping system allows the generation of currents in the flume. Since its inauguration in 2010, CIEMito has been successfully used for academic and scientific projects related to ocean engineering (e.g., Oliveira et al., 2012; Marzeddu et al., 2017).

The experimental layout used in this work is shown in Figure 1. A rubble mound breakwater model placed on a 1:30 smooth slope was tested. A 7.41 m long flat bottom was implemented between the paddle and the beginning of the slope. The toe of the breakwater was set at 12.41 m from the paddle and at 5.01 m from the start of the slope. The breakwater had a 1:1.5 slope on the seaward, a 0.383 m crest high, and a 0.07 m horizontal crest width. Water depth in front of the paddle was 0.3 m.

A permeable breakwater with two layers of cubic armor units, a filter layer, and a core layer was built. The core and filter layers consisted of rubble gravel. The armor layer consisted of resin cubes with a lead core to achieve, at the working scale, the same overall mass-density of prototype concrete blocks considering that fresh water is used in the laboratory. The material properties of the three layers, nominal weight (M_{50}), nominal diameter (Dn_{50}) and density (ρ) are summarized in Table 1. The toe of the breakwater was built with the same material as the filter.

The same breakwater configuration was tested under four storm representation methodologies, and for each methodology five repetitions were performed. After each repetition, the structure was rebuilt keeping the same breakwater configuration for the next tests. In this sense, the porosity of the armor layer was kept constant for all the different test with a value of around 38%. For that, during the rebuilding process, the same number of blocks was placed in the two layers and in the same area. Precisely, 23 rows of 17 blocks were deployed in an area of 0.385x0.345 m² (length x width in the slope plane) for each layer. Every block was numbered with a three-digit unique number (Figure 2b) to be recognized later during the visual counting of displaced blocks. Based on visual observations it was considered that during the laboratory tests no damage to the filter, toe, and core layers occurred. Therefore, these parts of the breakwater were not rebuilt.

The breakwater tested is a scaled model of the breakwater used by Van der Meer (Van der Meer, 1988a) with a scale factor of 3. Indeed, the stability formulae of Van der Meer (Van der Meer, 1988a, 1999) derived for this configuration has been worldwide used for the design of armored slopes in deep water (although it can be used for shallow water and concrete units too).

Five resistance wave gauges were used for measuring the free surface evolution at 2.00, 2.82, 3.53, 10.66 and 11.20 m from the wave-maker. A sampling rate of 64Hz was used. The degree of accuracy of these sensors is approximately 0.001m (Oliveira et al., 2012). To measure the damage caused by the wave attacks on the breakwater, an action camera with a resolution of 12Mpx (4032*3024) was used. Pictures of the breakwater were taken after each wave attack with one fixed camera parallel to the breakwater slope for the calculation of the relative damage.

2.2. Wave conditions

Several methods exist in literature to reproduce wave storms in physical model tests (Hughes, 1993; Frostick et al., 2011). A storm measured in the Mediterranean Sea was simulated using four different storm representation methodologies named here as i) Real Storm Methodology (RSM), ii) Classical Methodology (CM) (Owen and Allsop, 1984), iii) Synthetic Storm Methodology with triangular shape (EMSI) and iv) Synthetic Storm Methodology with trapezoidal shape (EMSA) (Martín Soldevilla et al., 2015).

The storm measured in the Mediterranean Sea on December 26th, 2008 at 65m depth in front of the village of Blanes in the Catalonia Coast was recorded by a Datawell Wave Rider buoy, which belongs to XIOM network placed at 41°39.003'N, 2°48.940'E (Bolaños et al., 2009). This buoy registers 20 minutes of raw data every hour and returns wave statistics and spectra. The measured storm was an extreme event that produced severe damage to beaches and structures along the Catalan coast (Figure 3).

The sea state curve exceeded the threshold of 1.5 m for the significant wave height (H_s) during 74 hours starting at 10:00 UTC +01:00. For the Catalan coast, it is often considered a storm event as a period of successive sea states in which H_s exceeds 1.5 m during a minimum period of 6 hours (Casas-Prat and Holthuijsen, 2010). At the peak, the December 26th storm showed a H_s of 4.65 m and a peak wave period (T_p) of 11.6s. The evolution of H_s , T_p , mean wave direction (θ_m) and wave steepness (s_{op}) during the storm is shown respectively in Figure 4a, b, c and d.

The storm was scaled to optimize the wave generation in the CIEMito wave flume regarding H_s and T_p considering a water depth in front of the paddle of 0.3 m. A working scale of 1/80 based on Froude law (1/80 length scale and $\sqrt{1/80}$ time scale) was chosen. This corresponds to a 24 m depth in front of the paddle at a prototype scale. Wave conditions were propagated from the depth where the storm was recorded (65 m) to 24 m depth using linear wave theory. At its peak, the scaled storm presents an H_s and T_p of 0.054 m and 1.3 s in front of the paddle, respectively.

Considering a standard number of waves in a sea state $N = 1000$, linear wave theory to obtain peak wave conditions at the toe of the scaled structure ($H_s=0.059\text{m}$), and the material properties of the breakwater model (see Table 1) it is possible to evaluate the expected damage, N_{od} , using Van der Meer's formulae:

$$\frac{H_s}{\Delta D_{n50}} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \right) S_{om}^{-0.1} \quad (1)$$

obtaining $N_{od} = 2.5$, where Δ is the relative buoyant density ($\Delta = 1.26$), D_{n50} the side dimension of the cubes ($D_{n50} = 14.5$ mm) and S_{om} is the wave steepness ($S_{om} = H_s/L_{0m} = 0.0314$ where L_{0m} is the wave length in deep waters calculated based on the mean wave period T_m). Considering that $N_{od} = 2$ represents the complete failure of the structure (Van der Meer, 1999), it could be expected an elevated level of damage during the tests.

2.2.1. Real Storm Methodology

RSM consists of representing in the wave flume the exact evolution of H_s and T_p from the beginning to the end of the storm. This is achieved by discretising the storm in a set of different sea states represented by a specific H_s and T_p . The discretization of the sea state is done taking into account that the buoy returns, at prototype scale, one measurement every hour. Then, each sea state is simulated as an independent wave time series in the laboratory. The duration of each time series in the lab is 402 s (1 hour at prototype scale).

In this work, only the part of the storm with H_s higher than 3m (as recorded at 65 m water depth) was reproduced in the lab. This threshold corresponds to H_s of the first sea state with $H_s > 0.6H_{s_{peak}}$ (being $H_{s_{peak}}$ the significant wave height at the peak of the storm). The selection of this value has been done in order to have comparable initial values between all the methodologies. Furthermore, according to the stability formulae of Van der Meer, no

significant damage should be expected in the breakwater for waves in the laboratory representing H_s recorded lower than 3 m. Using this new threshold, the duration of the storm is reduced at 21h at prototype scale leading to a significant reduction of laboratory time.

The December 26th storm was then discretized in 21 sea states. For a working scale of 1/80 this corresponds to target H_s and T_p in front of the paddle ranging from 0.035 to 0.054 m and 1.12 to 1.6 s, respectively (see Figure 5). Then, a total of 21 irregular wave conditions (generated following a Jonswap spectrum) were tested. The length of each time series was 402 s and, a Jonswap spectral parameter (γ) of 3.3 was used to represent the sea states.

This methodology was repeated five times and the cumulative damage at the end of each repetition was recorded. After each entire storm repetition, the breakwater was rebuilt.

2.2.2. Classical Methodology

For the tests carried out following the CM proposed by Owen and Allsop (1984), the pair H_s, T_p at the peak of RSM were associated to the so-called 100% step (CM100). The idea behind CM is to have a synthetic storm with a right triangle shape. Then, the only and main design parameter is assumed to be H_s at CM100. A sequence of tests with 60% (CM60), 80% (CM80), 100% (CM100) and 120% (CM120) of the design H_s are performed without reconstructing. The damage of the structure is evaluated between steps. Owen and Allsop (1984) suggested to perform tests with constant wave steepness, considering values typically associated to storm conditions, e.g., from 0.02 to 0.06. Usually, 3 to 4 wave steepness ($S_{0p} = H_s/L_{0p}$, where L_{0p} is the wave length in deep waters calculated based on T_p) values can be selected.

Only one constant steepness $S_{0p}=0.02$ is considered here, which is given by H_s, T_p recorded at the peak of the storm. For the working scale of 1/80, this corresponds to target H_s and T_p in front of the paddle of 0.054 m and 1.3 s, respectively (see Figure 5). S_{0p} was maintained constant for all the test steps considering that, globally, the wave steepness remains fairly constant during all the recorded real storm (Figure 4d). The test steps with 60, 80, 100 and 120% of the design H_s with the associated T_p were then performed to monitor damage evolution under CM. This corresponds to a target H_s of 0.032, 0.043, 0.054 and 0.065 m, respectively. In Figure 6 each step in H_s, T_p corresponds to one of the four wave conditions.

The duration of each step corresponds to approximatively 3 hours at prototype scale and is considered to be composed of $N=1000$ waves. The 1000 waves were divided into 3 sub-steps of same H_s, T_p and represented each by $N=333$ waves. This was done to have a comparable number of waves for each time series for all the methodologies (see Table 2). In fact, having the same number of waves in the time series allows having a similar probability of non-exceedance of a determinate wave height. The random phase sequence used to build the irregular wave train was different in the 3 sub-steps. Then, each sub-step had the same H_s, T_p, N but was represented by a different irregular wave train.

Similar to RSM, CM was repeated five times. At the end of each repetition, the cumulative damage was recorded, and then the breakwater was rebuilt.

2.2.3. Synthetic Storm Methodology

The Synthetic Storm model used in the experiments was the Equivalent Magnitude Storm (EMS) proposed first by Martín-Hidalgo et al. (2014) and improved later by Martín Soldevilla et al. (2015). EMS intends to reproduce a real storm with a simple geometric form. Then, H_s at

the peak of the simple geometric form is represented by H_{speak} . The duration of EMS is established such that its magnitude equals the magnitude of the measured storm. According to De Michele et al. (2007), the magnitude of a storm is the area describing the storm history above a given threshold of H_s . Different simple geometric forms have been proposed in the literature for EMS (Martín Soldevilla et al., 2015). The triangle form is recommended for typical sea storms whereas the trapezoid shape is more appropriate for more developed storm conditions. Martín Soldevilla et al. (2015) studied other shapes such as the parabola, and the trapezium. However, the authors found that both shapes overestimate structure damage for all kind of the wave conditions analyzed. The equivalent triangle magnitude storm (EMSI) intends to reproduce a real storm with an isosceles triangle in which its peak is represented by H_{speak} of the recorded storm.

On the other hand, the equivalent trapezoid magnitude storm (EMSA) aims to reproduce a real storm with a trapezoid in which the minor base is at the same height as H_{speak} of the recorded storm. The area of EMSA is given by:

$$M = \frac{1}{2} \cdot (b_L + b_S) \cdot h \quad (2)$$

where M is the energy of the synthetic trapezoidal storm, h is the height of the trapezium, and the coefficients b_L (long base, which equals the duration of EMSA) and b_S (short base) are the unknown trapezium bases (representing time). M equals the area between H_s curve of the recorded storm and H_s storm threshold (3m).

In order to calculate b_L and b_S , a dimensionless recorded storm with axes xx and yy ranging from 0 to 1 is considered:

$$xx = \frac{D_i}{D_{total}} \quad (3)$$

$$yy = \frac{H_{s,equiv,i}}{H_{s,equiv,peak}} \quad (4)$$

where D_i is the time step (in hours), D_{total} is the duration of the recorded storm (in hours), $H_{s,equiv,i}$ is the significant wave height at each time step subtracted by H_s storm threshold (m) and, $H_{s,equiv,peak}$ is the significant wave height at the peak of the recorded storm subtracted by H_s storm threshold (m). Similar to M , the magnitude of the dimensionless recorded storm, M_{dim} , can then be calculated.

Then, a dimensionless trapezoidal storm with $b_{Ldim}=1$ (long base of dimensionless trapezoidal storm) and $h_{dim}=1$ (height of the dimensionless trapezium storm) can be used to calculate b_{Sdim} (short base of the escalated storm):

$$M_{dim} = \frac{(b_{Sdim} + b_{Ldim}) \cdot h_{dim}}{2} \quad (5)$$

Finally, considering $h=H_{s,equiv.peak}$ and $b_s/b_L = b_{sdim}/b_{Ldim}$, the duration of EMSA can be obtained by:

$$b_L = \frac{2 \cdot M}{(1 + b_{sdim}) \cdot H_{s,equiv.peak}} \quad (6)$$

At prototype scale, the magnitude of December 26th, 2008 storm (measuring the energetic content), EMSI and EMSA is $M=21.7$ m·h. The duration of the EMSI is 26.2 h instead of the 21 h of the recorded storm, while the duration of EMSA is 21 h.

At the working lab scale of 1/80, for EMSI $D_i = 402$ s, $D_{total}=10452$ s (26 steps), $H_{s,peak} = 0.054$ m and $T_{p,peak} = 1.32$ s (Figure 7). Similarly, for EMSA $D_i = 402$ s, $D_{total}=8442$ s (21 steps), $H_{s,peak} = 0.054$ m and $T_{p,peak} = 1.32$ s (Figure 8).

Similar to CM, for both EMSI and EMSA only one S_{0p} was considered. Then, S_{0p} was calculated based on H_s , T_p at the peak of the recorded storm assuming a value of approximately 0.02.

However, after several tests the primary results showed that for EMSI after step 17-18 no further damage was observable and, for this reason, the authors decided to stop the test sequence after 21 steps.

To study repeatability effects, EMSI and EMSA were both repeated five times. Similar to RSM and CM, the cumulative damage at the end of each repetition was recorded. After each repetition the breakwater was rebuilt.

2.3. Relative damage measurement

From the pictures taken perpendicular to the breakwater at the end of each step, it is possible to calculate the relative damage N_{od} . As every single block is identified with a unique three-digit number (Figure 2b), their position before and after each test can be detected. Van der Meer (1999) defines the relative damage N_{od} as the actual number of units displaced related to a width (along the longitudinal axis of the structure) of one nominal diameter. Furthermore, the author defines damage level associated to the relative damage, starting from $N_{od} < 0.5$ (acceptable damage) until $N_{od} = 2$ that represents the complete failure of the structure. According to Van der Meer (1988a), N_{od} can be obtained by:

$$N_{od} = \frac{N_{du} D_n}{w} \quad (7)$$

where, N_{du} is the number of units displaced more than one nominal diameter (D_n) and w is the width of a cross-section. In the case of a wave flume (bi-dimensional), we consider that the cross-section equals the width of the flume not affected by the flume walls. Then, in our tests, only 0.30 m of the flume cross-section is considered for the relative damage measurements. An example of the methodology used in the experiments to measure N_{od} (treated pictures), is presented in Figure 9a.

To minimize the variability of N_{od} measurements, the following protocol was applied for the execution of all tests:

1. Construction of the filter layer;

2. Construction of the armor layer placing 17 blocks in each of the 23 rows for the two layers;
3. Perpendicular (view) picture of the breakwater for the visual counting (*Reference picture*);
4. Wave attack sequences;
5. Perpendicular (view) picture of the breakwater to compare with the reference picture for the visual counting of the number of blocks that are displaced during the wave attack;
6. Phase 4 and 5 repeated until the end of the test program (end of the storm or no damage increase);
7. Deconstruction of the damaged breakwater.

After phase 7 is finished, a next storm test can be executed starting again by phase 1. N_{od} was analyzed for all the repetitions of RSM, CM, EMSI, and EMSA at critical spots in time:

1. RSM-I, the main peak of the storm after 1981 waves and 0.67 h of the test;
2. RSM-II, the secondary peak after 4069 waves and 1.45 h of the test;
3. RSM-III, the end of the test program after 6680 waves and 2.35h of the test;
4. CM100, corresponding to the design wave height reached after 3000 (1000 waves of CM60 + 1000 waves of CM80 + 1000 waves of CM100) waves and 1h of the test;
5. CM120, end of the test program after 4000 waves and 1.342h of the test;
6. EMSI-I, the peak of the synthetic storm after 4604 waves and 1.45h of the test ;
7. EMSI-II, the end of the test program after 7361 waves and 2.35h of the test;
8. EMSA-I, the start of the peak of the storm after 3084 waves and 1.006h of the test;
9. EMSA-II, the end of the peak of the storm after 4305 waves and 1.453h of the test;
10. EMSA-III, the end of the test program after 7084 waves and 2.35h of the test.

3. RESULTS

3.1. Waves

H_s and T_p were calculated for all the tests at 3.53 and 11.20 m from the paddle (WG3 and WG5) to control wave generation repeatability. Figure 10 and Figure 11 show the evolution of H_s and H_{max} (without incident and reflected waves separation), respectively, for each repetition of the four storm representation methodologies. It is possible to observe that low variability of H_s and H_{max} occurs.

General speaking, generated H_s was lower than the target H_s by less than 5%. These differences can be due in part to the leakage around the paddle, which can decrease the amplitude of generated waves (e.g., Madsen, 1971; Oliveira et al., 2017).

The standard deviations at each storm step was calculated for the wave parameters H_s , H_{max} and T_p . The mean of the standard deviations ($\bar{\sigma} = (\sum_{i=1}^s \sigma_i) / s$, where s is the number of storm steps and σ_i is the standard deviation at storm step i) for each of the wave parameters at WG3 is summarized in Table 3. It is possible to note a very small variability in H_s , H_{max} and T_p due, most probably, to the intrinsic error in the measurement system. As explained in section 2.1 the measurement error of the resistive wave gauges is in the order of 1 mm.

3.2. Relative damage

Results for all the tested methodologies and all the repetitions are shown in Figure 12 and summarized in Table 4. Regarding damage progression, different behaviors can be observed for the four storm representation methodologies. In particular, RSM presents a relatively steady N_{od} growth from Time=0 to RSM-II. In average, 62% of the damage occurs from Time=0 to RSM-I ($\mu_{N_{od}}=2.07$) and 30% from RSM-I to RSM-II ($\mu_{N_{od}}=3.11$). Then, a small increase in $\mu_{N_{od}}$ is observed from RSM-II until the end of the test program (8% of the total). For CM instead, there is a clear increase of N_{od} until the end of the tests with a steeper growth from CM100 to CM120. In part, this is an expected result because CM is the only storm representation methodology in which H_s increases steadily from the start until the end of the test program. In average, 55% of the damage occurs from Time=0 to CM100 ($\mu_{N_{od}}=1.96$) and 45% from CM100 to CM120 ($\mu_{N_{od}}=3.59$).

For EMSI N_{od} presents a steep growth until EMSI-I ($\mu_{N_{od}}=1.52$, which corresponds to 90% of final $\mu_{N_{od}}$) to then smoothly grow until EMSI-II. On the other hand, for EMSA on average N_{od} occurs 66% between Time=0 to EMSA-I ($\mu_{N_{od}}=1.01$) and 33% between EMSA-I to EMSA-II ($\mu_{N_{od}}=1.52$). Almost no damage evolution is noticeable after EMSA-II (only 1% of total damage).

The final $\mu_{N_{od}}$ for RSM, CM, EMSI, and EMSA are 3.33, 3.59, 1.72 and 1.54, respectively. At RSM-I, CM100, EMSI-I, and EMSA-II (peak of the storm), $\mu_{N_{od}}$ are 2.07, 1.96, 1.52 and 1.52, respectively.

For RSM, $\mu_{N_{od}} \pm \sigma_{N_{od}}$ is 3.33 ± 0.63 . According to this data, it is clear that the phenomenon is far from deterministic. Similar observations can be made for the other three storm representation methodologies.

3.3. Spatial distribution of damage

If damage happens and the initial and final positions of the single armor unit are known, it is possible to determine where the extraction and deposition for a specific zone have taken place. The results of the spatial distribution of the damage for the four storm representation methodologies are presented in Figure 13. In this Figure, the x axis is the horizontal distance to the center cross-section of the flume and the y axis is the vertical distance to SWL (Sea Water Level). The zone with higher extraction of blocks is located between $SWL \pm H_{s_{peak}}$ ($y = \pm 5.4$ cm), though for RSM (Figure 13a) and CM120 (Figure 13b) small extraction zones are noticeable up to $SWL \pm 1.5H_{s_{peak}}$ ($y = \pm 8.1$ cm).

4. DISCUSSION

Overall, the comparison between storm representation methodologies shows a reasonably good agreement until the peak of the storm. Considering $\mu_{N_{od}}$, CM100 underestimates RSM-I results by 7% and, EMSI-I and EMSA-II underestimate RSM-I by 27%.

RSM gave a final $\mu_{N_{od}} \pm \sigma_{N_{od}}$ of 3.33 ± 0.63 . On the other hand, CM presented a final $\mu_{N_{od}} \pm \sigma_{N_{od}}$ of 3.59 ± 0.98 , which overestimate the results of RSM by only 7%.

For EMSI the final $\mu_{N_{od}} \pm \sigma_{N_{od}}$ was 1.72 ± 0.94 , showing a 48% lower $\mu_{N_{od}}$ in comparison with RSM. Similar behavior was observed for EMSA. In this case $\mu_{N_{od}} \pm \sigma_{N_{od}}$ was 1.54 ± 0.68 , being $\mu_{N_{od}}$ 54% lower than in RSM.

EMSI and EMSA are synthetic storms with the same total energy of RSM. The shapes of EMSI and EMSA are based on the assumption of a single peak storm. However, the December 26th, 2008 Mediterranean storm presents a second peak with $H_s = 0.98H_{s_{peak}}$. This fact could explain the increase in $\mu_{N_{od}}$ differences after the end of the first peak of the storm (EMSI-I and EMSA-II) between the two synthetic storms and RSM. After the end of the first storm peak, $\mu_{N_{od}}$ increases 13% and 1% for EMSI and EMSA, respectively, until the end of the storm. However, for RSM, $\mu_{N_{od}}$ increases 61% after the end of the first peak.

Large variability of N_{od} was found for all four storm representation methodologies tested. These results are in line with results reported by other authors (van Gent et al., 2018; Melby and Kobayashi, 1998) for dispersion of damage measurements on rubble mound breakwaters and highlight the high variability of the wave-structure interaction phenomenon simulated. Consequently, from the structural design point of view, results suggest that it can be unsafe to use a deterministic value of N_{od} obtained from a single small scale test.

Considering RSM as the benchmark for the December 26th, 2008 Mediterranean storm, the methodology that returned the best results was CM. This methodology overestimated the final $\mu_{N_{od}}$ of RSM by only 7%.

Regarding the spatial distribution of the damage, results for the four storm representation methodologies are in agreement with previous studies showing erosion in the zone SWL $\pm 1.5H_{s_{peak}}$ (USACE, 2012).

The methodologies to represent wave storm evolution tested in this work are based on H_s evolution over time. In other words, these methodologies only provide synthetic evolution for the wave parameter H_s . Therefore, when using the synthetic storm representations tested here, some assumptions should be made regarding the evolution of T_p over time. In this work, T_p evolution for the synthetic storm methodologies is calculated based on the time averaged S_{0p} of the real storm. However, we should highlight that damage results could vary if other approach is used to calculate T_p evolution.

The Reynolds number for the armor layer in our model scale is 1×10^4 (calculated considering $H_s = 0.054$ m and $L = 0.0145$ m). A review of previous studies about scale effects on the hydraulic model of coastal structures reveals that researchers differ by almost two orders of magnitude on the critical Reynolds number (value below which scale effects can be expected). For instance, Jensen and Klinting (1983) found a critical Reynolds number of 6×10^3 . On the other hand, Shimada et al. (1986) stated that there would be negligible scale effects for Reynolds numbers larger than 4×10^5 . The Reynolds number for our tests fall in between these two limits. Clearly, in the filter and the core, at a small scale, the presence of scale effects have more chance to influence the results. To the authors' knowledge, there are no hydraulic lab experiments reported in the literature that combined the same experimental set-up and small model scale. However, a typical breakwater model scales, as a compromise between reasonable size (economics) and moderate scale effects, could range between 1/20 to 1/80 (e.g. Oumeraci, 1984; Wolters et al, 2010; Heller, 2011). Nevertheless, scale effects may therefore not necessarily be negligible for a 1/80 model scale. Therefore, a comprehensive

study with larger-scale tests should be performed to quantify possible scale effect (see, eg, Cappiети et al., 2007).

According to wave storm characteristics in the Catalan coast (Marcos, 2008; Bolaños et al., 2009), the December 26th, 2008 storm simulated in this study can be considered as a standard storm in terms of H_s (ten years return period), T_p and, direction. According to Marcos (2008), in the Catalan coast, storms longer than 48 hours have a 10% probability of exceedance. Then, the December 26th, 2008 storm can be considered a long storm with a double peak and a very fast H_s growth from its beginning to the peak of the storm for regional standards (Marcos, 2008). Nevertheless, we should notice that in other ocean environments, for instance the Portuguese continental coast (Oliveira et al., 2018) or Norwegian Sea (Feng et al., 2014), standard local storms are more energetic, can last longer and present a very different H_s evolution during the storm. For such energetic environments, the performance of the methodologies analyzed in this study could differ significantly from the results obtained here.

Time occupation of a wave flume should be considered as a cost in the design process of a breakwater. Consequently, the number and duration of tests should be optimized. Executing the type of analysis performed in this work can be extremely time consuming and consequently very expensive. For this series of tests, a run of CM lasts 1.25h. On the other hand, EMSI and EMSA take 2.35h to complete one run of tests. Moreover, we should also consider the effects of test repeatability on damage results. For the four methodologies tested here, we analyzed the sensitivity of the damage to the different number of repetitions (considering different permutations of the different number of repetitions). All the methodologies presented similar variability in the results. This fact suggests that the complex wave-structure interaction phenomena is responsible for repeatability effect. Overall, three main factors could be responsible for inducing the variability found in the test repeatability: i) the complex interaction between breaking waves and the breakwater, ii) the interaction between the blocks of the armor layer and iii) the armor layer porosity variability induced by the rebuilding of the armor layer.

5. CONCLUSIONS

In this study, we aimed to understand the effects of storm representation on the damage measurements of a breakwater armor layer. Small-scale physical model tests were performed, and a double layer of randomly placed cube armor units was used in the breakwater armor layer. For this experiments, four different test methodologies representing a storm recorded in the Mediterranean Sea were investigated: i) Real storm Methodology (RSM, used here as benchmark test) ii) Classical methodology (CM) iii) Equivalent Magnitude Storm with a triangular shape (EMSI) and iv) Equivalent Magnitude Storm with a trapezoidal shape (EMSA). For all the different storm representation methodologies experiments were repeated five times.

Regarding the effects of the December 26th, 2008 Mediterranean storm representation on damage measurements the main conclusions are as follows:

- Regarding N_{od} at the peak of the storm, in average CM gives results similar to RSM, and both EMSI and EMSA underestimate RSM by 27%;
- CM represents well RSM overestimating on average the final $\mu_{N_{od}}$ by less than 7%;
- EMSI and EMSA underestimate the final $\mu_{N_{od}}$ of RSM by 48 and 54%, respectively;

- A large variability of N_{od} was found for all four storm representation methodologies tested;
- The four methodologies present similar spatial distribution of damage.

These considerations suggest that a minimum of five repetitions of CM should be performed to obtain an average damage value that can represent the average damage given by RSM.

Although this recommendation is linked to the tested conditions, it provides useful information to set-up experiments dealing with similar breakwater and wave conditions.

The duration, in terms of a number of waves N , of each step of CM is a crucial parameter and should be a function of the average duration of the most energetic storms at the site of construction. In this regard, a N of approximately 1000 waves have been chosen for each step of CM.

The performed work and subsequent analysis demonstrate the importance of test repeatability in order to achieve statistically robust data. The practical constraints of test duration and cost require the clever combination of small and large-scale tests plus a good control of how the various geometrical and dynamical features (from flume dimensions to resulting porosity) affect wave-structure interactions. This is the way to: i) reduce uncertainties and experimental times in future hydraulic experiments and ii) improve the reliability of breakwater functional and resistant design.

Future work should focus on understanding the effects of storm representation for more energetic wave storms, which can occur in other ocean environments. In this regard, the performance of the storm representation methodologies studied here could differ significantly conditional on the input storm event. Moreover, future research should also evaluate the performance of other storm representation methodologies, such as the parabolic shape representation. In addition, future work is needed to assess the influence of T_p evolution representation on damage measurements of breakwaters.

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Layer	M_{50} [g]	Dn_{50} [mm]	ρ [g/cm ³]
Core	0.13	3.7	2.65
Filter	0.9	7	2.65
Armour	6.9	14.5	2.26

Table 1: Properties of the Material used to build the breakwater.

		mean H_{speak} [m]	T_{ppeak} [s]	n. steps	Steps duration [s]	duration [s]
RSM		0.050	1.32	21	402	8442
CM	CM100	0.054	1.32	3x3	3x300-3x400	3600
	CM120	0.063	1.4	4x3	3x300-3x425	4831
EMSI		0.052	1.32	21	402	8442
EMSA		0.054	1.32	21	402	8442

Table 2: Number of repetitions performed and storm discretization characteristics for each of methodology tested.

	H_s (mm)	H_{max} (mm)	T_p (s)
RSM	0.3	1	0.0003
CM	0.4	2.7	0.0004
EMSI	0.3	1.2	0.0003
EMSA	0.7	1.9	0.0007

Table 3: Mean of the standard deviation of significant wave height H_s , maximum wave height H_{max} and peak period T_p computed for each test step.

	$\mu_{N_{od}} \pm \sigma_{N_{od}}$ end	RSD (%)	$minN_{od}$	$maxN_{od}$	max/min (%)	$\mu_{N_{od}} \pm \sigma_{N_{od}}$ storm peak
RSM	3.33±0.63	19	2.31	3.83	166	2.07±0.56
CM	3.59±0.98	27	2.52	5.09	202	1.96±0.88
EMSI	1.72±0.94	54	0.67	2.65	396	1.52±0.86
EMSA	1.54±0.68	48	0.88	2.61	297	1.52±0.76

Table 4: Statistics of the recorded relative damage for all the tested methodologies. $RSD = \mu_{N_{od}} / \sigma_{N_{od}} \cdot 100$ is the Relative Standard Deviation.

Figure 1: Experimental layout – a) CIEMito wave flume configuration – b) breakwater cross-section – All dimensions in meters.

Figure 2: a) Picture of the tested breakwater – b) Numbered resin block with the lead core (dimensions in mm).

Figure 3: Damages on the Catalan coast during (a) and after (b) the 26/12/2008 Storm (from: Directe.cat, 2009; El Pais, 2008).

Figure 4: Storm measured on December 26th 2008 at 41°39.003'N, 2°48.940'E and 65 m water depth. – a) Significant wave height (H_s) – b) Peak wave period (T_p) – c) Mean wave direction (θ_m) – d) Wave steepness (S_{op}). Storm recorded by LIM-UPC BarcelonaTech (Bolaños et al., 2009).

Figure 5: Theoretical wave conditions for the Real storm Methodology (red line) in front of the paddle. The entire storm is discretized in steps of 402 s – a) Significant wave height H_s – b) Peak wave period T_p . Black line represents the evolution of measured storm.

Figure 6: Theoretical wave conditions for Classical storm methodology (red line) in front of the paddle – a) Significant wave height H_s – b) Peak wave period T_p . Black line represents the evolution of measured storm.

Figure 7: Theoretical wave conditions for Synthetic triangular storm methodology (red line) in front of the paddle - a) Significant wave height H_s – b) Peak wave period T_p . Black line represents the evolution of measured storm.

Figure 8: Theoretical wave conditions for Synthetic Trapezoidal storm methodology (red line) in front of the paddle - a) Significant wave height H_s - b) Peak wave period T_p . Black line represents the evolution of measured storm.

Figure 9: Analysed images. On top of the reference image (taken before the tests (a)) have been drawn arrows with the displacement to the final “destination” after a complete storm (image taken after the tests (b)). In this case $N_{od} = 2.7$.

674

675 Figure 10: Wave generation repeatability – Significant wave height H_s – a, c, e, g measured at
 676 3.53m from the wave paddle (WG3) – b, d, f, h measured at 1.21m from the toe of the
 677 breakwater (WG5).

678

679 Figure 11: Same as Figure 10 but for maximum wave height H_{max} .

680

681 Figure 12: Damage progression

Figure 13: Damage spatial distribution of extraction/deposition – x axis: horizontal distance to the center cross section of the flume – y axis: vertical distance to the SWL