CATANIA HARBOR BREAKWATER: PHYSICAL MODELLING OF THE UPGRADED STRUCTURE

Martina Stagnitti[1], Claudio Iuppa[2], Rosaria Ester Musumeci[3] and Enrico Foti[4]

Most of the worldwide historical coastal and harbor structures have been severely damaged by extreme sea storms during their lifetime and hence need to be upgraded, also considering the effects of climate change (Hughes, 2014). Physical modelling is identified as the only feasible approach for the optimization of the upgraded structures, because of the existence of few studies concerning such an issue and the lack of specific design formulae (Burcharth et al. 2014; Croeneveld et al. 1984; Lara et al. 2019; Foti et al. 2020). Therefore, a novel general methodology for the design of upgrading solutions for existing breakwaters based on physical modelling is presented, considering the case study of the Catania Harbor breakwater. The results of the systematic extensive experimental campaign on possible solutions for upgrading the Catania harbor breakwater led to some general practical findings, which can be useful for the design of restoration options for existing breakwater at end of their lifetime.

Keywords: rubble mound structures; restoration; damage progression; overtopping

INTRODUCTION

Aging of coastal and harbor defense structures is a worldwide problem, which nowadays goes along with the need of upgrading for protection against the effects of climate change on coastal areas (Hughes, 2014), such as mean sea level rise (Church et al. 2013; Galassi and Spada 2014; Lambeck et al. 2011), increase of extreme storm surge height and frequency of occurrence (Lowe and Gregory 2005; Vousdoukas et al. 2016), inter-annual variability of wave characteristics (Camus et al. 2017; Chini et al. 2010; Hemer et al. 2013; Morim et al. 2019) and reduction of extreme sea levels return period (Vousdoukas et al. 2018). Indeed, the above-mentioned effects directly influence the hydraulic performances of coastal and harbor defense structures in terms of wave run-up height and overtopping discharge (Arns et al. 2017; Chini and Stansby 2012; Isobe 2013).

Great part of the historical coastal and harbor defense structures is in shallow-waters and consists of non-conventional breakwaters, which have been repeatedly modified over the years and usually converted into rubble mound structures (Lara et al. 2019). Therefore, traditional empirical design formulae defined for new structures may be not able to properly characterize the hydraulic behavior of both damaged and restored breakwaters, whose actual composition in terms of material and layers geometry is often unknown, because of the lack of documents and reports regarding the modifications implemented during their lifetime. Despite the practical relevance of the matter concerning the upgrade of aging coastal and harbor breakwaters, few researches has been carried out, which mainly consists in design exercises using only desk study tools or suggested methods for the selection of the possible restoration options for damaged breakwaters (Croeneveld et al. 1984) and adaptation solutions to the effects of climate change for existing structures (Burcharth et al. 2014; Foti et al. 2020; Isobe 2013; Koftis et al. 2015). In addition, some investigations on implemented restoration projects are available, that however only describe the design procedures, without defining formulae or models suitable for the design of upgrading options or testing the applicability of the traditional state-of-art design equations (Jensen et al. 2018; Ligteringen et al. 1993; Main et al. 2018; Santos-Ferreira et al. 2015). As regards the available numerical models, they have not been sufficiently calibrated to simulate all the common restoration concepts, particularly concerning the stability of the armor layer (Burcharth et al. 2014; Lara et al. 2019). As a consequence, physical modelling represents the most reliable approach to describe the response of existing or upgraded rubble mound structures and their possible failure modes (Burcharth et al. 2014; Croeneveld et al. 1984; Lara et al. 2019; Foti et al. 2020).

The choice of the most suitable restoration solutions for damaged rubble mound breakwaters must be addressed by several factors (Croeneveld et al. 1984; Burcharth et al. 2014; Foti et al. 2020). First, a field survey is required to assess the magnitude of the damage and identify its possible causes, in order

[1] Department of Civil Engineering and Architecture, University of Catania, Via Santa Sofia, 64, 95123 Catania, Italy. E-mail: martina.stagnitti@unict.it
[2] Department of Civil Engineering and Architecture, University of Catania, Via Santa Sofia, 64, 95123 Catania, Italy. E-mail: ciuppa@dica.unict.it
[3] Department of Civil Engineering and Architecture, University of Catania, Via Santa Sofia, 64, 95123 Catania, Italy. E-mail: rosaria.musumeci@unict.it
[4] Department of Civil Engineering and Architecture, University of Catania, Via Santa Sofia, 64, 95123 Catania, Italy. E-mail: enrico.foti@unict.it

to avoid the failure of the restored breakwater for the same reasons. Then, the required structure performances and acceptable risk of failure must be fixed, on the basis of the structure function (e.g. coastal or harbor defense) and economic relevance. Furthermore, the geometric characteristics of the breakwater and the local topography must be considered, together with specific environmental restrictions which could influence the design. Finally, the available materials, equipment and financial resources play a fundamental role for the evaluation of the economic and technical feasibility of different upgrading options.

On the basis of the above-mentioned addressing factors, four main repair methods are usually contemplated for the upgrade of damaged rubble mound breakwaters (Croeneveld et al. 1984): i) addition of units of the same type of the existing ones, eventually reinforced or slightly greater to increase their weight; ii) replacement of the entire armor layer, removing all the original units; iii) reconstruction of the rubble mound structure, replacing not only the original armor layer, but also the underlayers; iv) provision of a submerged toe berm or detached breakwater to reduce the wave impact on the existing structure. If the adaptation of existing rubble mound breakwaters to the effects of climate change is considered, the above-mentioned upgrading strategies can be applied with some tricks to face the increased external forcing. For instance, the overtopping rates can be reduced by heightening the existing wave wall or the construction of a new one. Furthermore, if the construction of an additional or totally new armor layer is considered, the rise of the structure crest level to reduce the overtopping discharges or the reduction of the seaside slope to limit wave run-up and increase the armor layer stability can be designed (Burcharth et al. 2014; Foti et al. 2020).

The selection of the blocks for the armor layer restoration is a difficult issue, because there is a lack of in-depth investigations on the interaction between the existing units and the additional ones. In this regard, Carver (1989) provided an inventory of existing US Army Corps of Engineers (USACE) projects that have used dissimilar armor blocks for repair and rehabilitation of rubble mound coastal breakwaters. The results of the survey showed that in 1989 only the 24% of the considered districts had experienced the use armor units different from the existing ones for the armor layer restoration, thus highlighting the necessity to perform further systematic experimental studies for the evaluation of the interfacing and stability response of the different armor blocks. Currently, the only guidance for the choice of shape and size of the additional armor units comes from traditional formulations for new constructions, prototype experience, engineering judgment, inferences from model tests of similar structures, or site-specific model tests. It can be observed that the use of additional armor units heavier than the existing ones could be considered if the present structure appears undersized with respect to the design wave action, in the absence of technical or practical limitations. Instead, the use of additional armor units smaller (i.e. lighter) than the existing ones could be contemplated to fill the voids of the damaged breakwater with limited movements of the present blocks. However, the structural response of the whole armor layer to the design wave load must be evaluated by means of physical model tests. The same applies to the use of units with different interlocking level than the existing ones, for which special attention on the regularization of the laying surface and on the transition zones must be paid.

The characteristics of the existing structure, together with the details about the selected adaptation options and the design hydrodynamic conditions should be the input data for models able to describe the behavior of the upgraded structure under wave attack, in terms of stability and hydraulic performances. However, the only available literature formulas and models for the evaluation of the response of rubble mound breakwaters to the wave action were empirically defined for the case of new structures, usually on the basis of experimental results (Hudson, 1959; van der Meer, 1988b, 1988a; van Gent et al., 2003). Therefore, further research for the definition of design formulae and models specific for the upgrade of existing rubble mound breakwater is needed, also considering the possibility to adapt the traditional ones on the basis of experimental results on restored structures.

To summarize, the design of upgrading solutions for rubble mound breakwaters is complicated by the following factors: i) uncertainties in the assessment of the design conditions considering the effects of climate change; ii) lack of knowledge about the actual composition of the existing structures, which is known only punctually at best; iii) absence of systematic investigations on the behavior of upgraded rubble mound breakwaters, and consequently lack of specific formulas and models. In this context, the present work aims to contribute to the development of a physical model-based approach for the design of upgrading solutions for rubble mound breakwaters. The emblematic case study of the Catania Harbor breakwater was selected for the application of the proposed methodology, which consists in the calibration of literature empirical formulas using experimental results on the upgraded structure. In this context, the procedure for a detailed experimental analysis of the damage progression of the upgraded structure under increasing wave load is presented, considering both traditional and novel techniques.

DESCRPTION OF THE CASE STUDY

The Harbor of Catania is located on the East coast of Sicily, in the middle of the Mediterranean Sea (see Fig. 1). The strategic position, which is equidistant between the Suez Channel and the Strait of Gibraltar and between the European and the North-African ports, made the Harbor of Catania one of the Italian commercial ports of national interest, with its operative piers of about 4.2 km total length. At present, the harbor basin is mainly protected by the 2.25 km long outer breakwater of Levante, which during its lifetime was subjected to several structural interventions that had modified its composition and length.

A detailed reconstruction of the history of existing breakwaters is often not possible. For the case of the Catania Harbor, after the extreme sea storm of 1601, which destroyed the first port of Catania commissioned by Alfonso of Aragon, a sequence of construction and destruction of a new harbor defense structure took place over the XVII century, exactly in the position occupied by the current Catania harbor breakwater. In the XVIII century, the Bourbon government funded the construction of a vertical type breakwater using 330 t cyclopean blocks. During the XIX century, the breakwater was reinforced through the conversion into a composite structure and lengthened up to 258 m. However, the composite breakwater failed during construction between 1930 to 1931, because of the lack of horizontal connectivity between layers. Since no measures for the upgrade of the monolithic behavior of the structure were implemented, in 1933 a second failure of the breakwater occurred (Allsop et al. 1996; Franco 1994; Oumeraci 1994; Takahashi 2002). The structure was subsequently rehabilitated as a rubble mound breakwater, which has been progressively extended until the present 2.25 km length and, from the late '70s, modified with additional layers of 62 t artificial parallelepiped blocks.

Currently, the Catania harbor breakwater appears severely damaged by extreme sea storms and the 62 t cubes armor layer shows evidence of degradation and structural failures both under and above the mean sea level. A strong volumetric reduction is observed, with visible sign of thinning and narrowing of the original cross sections. It is worth to point out that extreme marine events occurred during the Catania harbor breakwater lifetime not only caused structural damages to the armor layer, but also produced huge overtopping discharges that result in significant inconveniences for the port operability. Given the structural and hydraulic problems of the Catania harbor breakwater, the need to rehabilitate the seaside armor layer is clear. Therefore, the Port Authority proposed a restoration project for the harbor breakwater to ensure safety for the port activities.

Figure 1. Location of the Catania Harbor (satellite view from Google Earth) and indication of the outer breakwater representative sections n. 10 ($H_{s,Tr=100}/h$=0.38; $k_{,Tr=100}h$=0.50) and n. 40 ($H_{s,Tr=100}/h$=0.36; $k_{Tr=100}h$=0.60).

EXPERIMENTAL CAMPAIGN

Upgrading solutions for the Catania harbor breakwater

Due to the lack of empirical formulas and numerical models for the description of the behavior of rehabilitated existing breakwaters under wave attack, the design of the optimal restoration project requires the experimental comparison between different restoration options. In the present work, six upgrading solutions for the Catania harbor breakwater were considered (see Table 1).

First, the simple raising up to +8.50 m and +9.50 m above MSL of the wave wall without armor layer restoration was considered (respectively configuration E and EM), in order to quantify the effects of the wave wall height on the reduction of the overtopping discharge and to verify the stability level of the existing armor layer. Then, four armor layer restoration options were studied, to identify the most suitable additional armor units and laying method to ensure stability and acceptable overtopping rates under the design hydrodynamic conditions. The armor layer restoration originally proposed by the Port Authority consists in filling the voids left by the 62 t cubic armor blocks dragged away by sea storms with 30 t Antifer blocks, thus recovering the original design section with a 1:2 slope. Furthermore, a quarry stone toe berm should provide additional protection whereas the wave wall crest raised up to +8.50 m above MSL should reduce the overtopping discharge (configuration AS). An alternative to the Port Authority proposal was considered, which differs only for the fact that a double layer of 30 t Antifer blocks is placed over the damaged structure (configuration AD). Finally, two different armor layer restorations using 62 t parallelepiped block equal to the existing ones were analyzed. The first solution consists in placing a single layer of 62 t cubic blocks over the damaged armor layer, providing a quarry stone toe berm to support the additional units and the wave wall crest raised up to +8.50 m above MSL (configuration CM). Instead, the second option involves the preliminary regularization of the existing armor layer, by moving the existing blocks where necessary. Then, the 62 t cubic blocks are placed over the regularized surface following the original design section with a 1:2 slope. As in the previous options, a quarry stone toe berm should provide additional protection whereas the wave wall crest raised up to +9.50 m above MSL should reduce the overtopping rate (configuration CS).

Experimental setup and measurement techniques

2D physical model tests were performed in the wave tank (18.0 x 3.6 x 1.2 m) of the Hydraulic Laboratory of the University of Catania (see Fig. 2). The horizontal base of the tank was covered with coarse sand, in order to reproduce the bathymetry at the toe of the structure and its possible variations due to the wave motion. The flap-type wavemaker produced irregular waves, by means of JONSWAP spectra.

The characteristics of the wave motion were measured by means of five resistive gauges, whose location in the wave tank is showed in Fig. 2. The properly spaced gauges n. 1, 2, 3 and 4 were located 1.5 m away from the toe of the model, in order to measure the characteristics of the incident and reflected wave motion using the four-gauge method of Faraci et al. (2015). Furthermore, the gauge n. 5 was placed 5.0 m in front of the wavemaker for a further monitoring of the hydrodynamic conditions.

The video camera R1 (Sony FDR-AX53) was located on a scaffolding at a distance of 1.90 m from the toe of the structure (see Fig. 2), so as to frontally frame the whole model and to allow the acquisition of records useful for the analysis of the movements of the armor units caused by the incident wave. Therefore, the calculation of the traditional damage parameters N_d (Hudson 1959) and N_{od} (Hedar 1960) could be performed, using the following equations:

$$N_d = \frac{N_{moved}}{N_{total}} * 100\% \qquad (1)$$

$$N_{od} = \frac{N_{moved}}{B/D_{n50}} \qquad (2)$$

where N_{moved} is the number of displaced units, N_{total} is the total number of armor units, B is the width of the tested section and D_{n50} is the median nominal diameter of the armor units.

Table 1. Summary of the proposed upgrading solutions for the Catania Harbor breakwater.

Configuration	Description
E	Existing structure with wave wall crest raised up to +8.50 m above MSL
EM	Existing structure with wave wall crest raised up to +9.50 m above MSL
AS	Originally proposed armor layer restoration with 30t Antifer, a quarry stone toe berm and wave wall crest raised up to +8.50 m above MSL
AD	Double layer 30t Antifer block armor layer restoration, a quarry stone toe berm and wave wall crest raised up to +8.50 m above MSL
CM	Single layer 62t parallelepiped block armor layer restoration, a quarry stone toe berm and wave wall crest raised up to +8.50 m above MSL
CS	Armor layer restoration with 62t parallelepiped blocks laid following the AS section, also moving the existent blocks if necessary, a quarry stone toe berm and wave wall crest raised up to +9.50 m above MSL

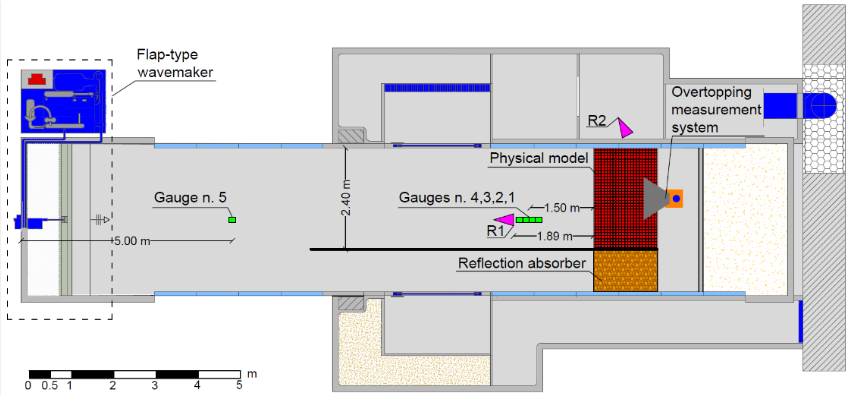

Figure 2. Wave tank (18.0 x 3.6 x 1.2 m) of Hydraulic Laboratory of the University of Catania and location of the measurement instruments used for the experimental campaign on the upgraded Catania harbor breakwater.

The overtopping phenomenon was monitored by the video camera R2 (Sony HDR-CX410VE) placed on a side scaffolding (see Fig. 2). In addition, the measurement of the mean overtopping discharge rate q was performed by using an overtopping tank (see Fig. 2). In particular, the collection tank (0.40x0.40x0.30 m) properly anchored to the ground and connected to three drainage pumps (NEWA JET 6000) was equipped with a ramp to channel the water that overflow behind the structure. An acoustic sensor (Pepperl Fuchs UC500-30GM70-IE2R2-V15) allowed the measurement of the water level inside the collection tank and the emptying of the latter until a minimum threshold when the fixed maximum level is reached. Then, the mean overtopping discharge for a certain time interval was calculated by means of a calibration law which relates levels and volumes inside the collection tank. It is worth to point out that the tank emptying process is sufficiently rapid to not significantly influence the measurement of the overtopping discharge.

Model geometry and test conditions

The physical model of the upgraded Catania harbor breakwater was a 1:70 geometrically undistorted model, according to the most current literature on the subject (Frostick et al. 2011). The structure was built into a channel wide 2.40 m created inside the wave tank to avoid three dimensional effects due to the excessive width of the model. The armor units and the berm material were designed so that the similarity in terms of stability number between model and prototype was ensured, together with the turbulent motion (i.e. Reynolds number of the armor units greater than $1 \div 4 * 10^4$, van der Meer 1988a). Instead, the size of quarry stones used for the construction of the filter layers and the core was increased, in order to minimize the viscous scale effects. In addition, painted armor blocks were employed to both reduce the friction scale effects and facilitate the analysis of the displacements of the armor units. It is important to point out that the structure was repaired only at the end of the set of experiments on a certain configuration, so as to carry out the evaluation of the cumulative damage of the armor layer.

Two representative sections of the breakwater were tested, i.e. section n. 10 and n. 40 (see Fig. 1). Section n. 10 was selected because of its strong geometric irregularities and it is characterized by a ratio between the 100-year return period significant wave height and the water depth ($H_{s,Tr=100}/h$) equal to 0.38 and a product between the 100-year return period wave number and the water depth ($k_{,Tr=100}h$) equal to 0.50. Section n. 40 was chosen because it is representative of the most offshore (i.e. exposed to the wave load) part of the breakwater and it is characterized by a $H_{s,Tr=100}/h$ equal to 0.35 and $k_{,Tr=100}h$ equal to 0.60. For each of the two sections, the six upgrading solutions summarized in Table 1 were tested.

Several preliminary tests were carried out to properly calibrate the hydrodynamic parameters that govern the wave motion inside the wave tank. Subsequently, the experimental procedure for testing the structural and hydraulic performances of all the considered configurations was defined, in order to obtain reliable and comparable results.

The characteristics of the sea states in terms of significant wave height H_s, peak wave period T_p and number of waves N_w and the values of mean sea level h simulated during the tests are presented in Table 2. For each tested configuration, first, a shakedown test was carried out for the settling of the structure with three consecutive sea state of 1500 waves and significant wave height corresponding to 5 years-return period. Then, traditional tests considering sea states of 4500 waves divided into three intervals of 1500 waves and significant wave height corresponding to 10, 50 and 100 years-return period (the latter is the design return period) were performed. If no damage was observed, a further sea state of 4500 waves divided into three intervals of 1500 waves and a significant wave height equal to 120% of the 100 years-return period one was reproduced. Finally, an investigation on the effects of mean sea level rise was carried out for the existing structure with wave wall crest raised up to +8.50 m above MSL (i.e. configuration E), by means of the simulation of sea states of 4500 waves divided into three intervals of 1500 waves and significant wave height corresponding to 50 and 100 years-return period in the presence of a 0.02 m (i.e. 1.40 m in the prototype scale) increase in mean sea level (Lambeck et al. 2011).

PRELIMINARY EXPERIMENTAL RESULTS

Damage dynamics

The preliminary results of the damage analysis show that the already damaged existing armor layer (i.e. configurations E and EM) presents a certain stability, likely attributable to the strong level of settlement of the actual shape due to the action of past storms. The maximum values of the cumulative N_d and N_{od} reached by the existing armor layer at the end of the series of tests are respectively 0.36% and 0.09 for section n. 10 and 2.22% and 0.53 for section n. 40. The different structural response of the two sections demonstrates that the deterioration processes suffered by the present armor layer of the Catania harbor breakwater led to the creation of voids of different shape and size along the structure, causing a geometric and thus structural non-uniformity among the cross sections. However, for the two tested sections, the intermediate damage level proposed by The Rock Manual (CIRIA and CETMEF 2007) is not exceeded. As regards the effects of climate change, the rise of the mean sea level causes a negligible increase (less than 12%) of the damage level reached by the existing armor layer.

With respect to the upgrading options that consist in adding an extra armor layer over the existing one (i.e. configurations AS, AD, CM and CS), the preliminary results highlight the existence of two different behaviors of the structure in the case of sufficient or no sufficient support at the toe of the additional units. Indeed, as showed in Fig. 3, for the same stability number (i.e. the ratio between the incident significant wave height H_s and the product between the relative buoyant density Δ and the median nominal diameter D_{n50} of the armor stones), the configurations with insufficient support at the toe of the extra armor layer (Fig. 3b) reach damage levels up to 3÷7 times higher than the options which provide a proper one (Fig. 3a). In particular, configurations AS, AD and CM present a sufficient support at the toe of the extra armor units only for section n. 40, which is characterized by a more regular existing armor layer (i.e. laying surface for the additional blocks). Instead, configuration CS is the only one able to guarantee similar stability performances for both section n. 10 and n. 40, despite of the extent of irregularity of the existing armor layer. Indeed, thanks to the preliminary regularization of the laying surface, the support of the additional block is provided not only by the toe berm, but also by the existing cubes properly replaced.

The state-of-art formula proposed by van der Meer (1988b) for double layers of cubes laid on a slope of 1:1.5 with a notional permeability equal to 0.4 was here adapted to the experimental data on the upgraded Catania harbor breakwater, by means of the evaluation of the multiplicative correction factor *f*:

$$\frac{H_s}{\Delta D_{n50}} = f * a * \left(6.7\frac{N_{od}^{0.4}}{N_w^{0.3}} + 1.0\right) s^{-0.1} = f * a * \left(6.7\frac{N_{od}^{0.4}}{N_w^{0.3}} + 1.0\right)\left(\frac{2\pi H_s}{gT_m^2}\right)^{-0.1} \quad (3)$$

where a is an empirical factor equal to 1.00 with standard deviation equal to 0.10, N_{od} is the damage parameter defined by equation 2, N_w is the number of incident waves, Δ and D_{n50} are respectively the relative buoyant density and the median nominal diameter of the armor blocks, H_s is the incident significant wave height, T_m is the mean wave period and g is the gravity acceleration. It is worth to point out that equation 3 can be applied for Antifer units, whose geometry is very similar to the cubic one. Table 3 presents the estimation of the correction factor f and its standard deviation σ_f for the two cases of sufficient and not sufficient support at the toe of the additional armor layer.

Table 2. Characteristics of the sea states corresponding to a certain return period (*Tr*) in terms of significant wave height (H_s), peak wave period (T_p) and number of waves (N_w) and values of mean sea level (*h*) simulated during the tests on the physical model of the upgraded Catania harbor breakwater.

Tr [years]	Section n. 10				Section n. 40			
	H_s [m]	T_p [s]	N_w [n. of waves]	*h* [m]	H_s [m]	T_p [s]	N_w [n. of waves]	*h* [m]
5	0.06	1.15	3*1500	0.23	0.07	1.15	3*1500	0.27
10	0.07	1.20	3*1500	0.23	0.07	1.20	3*1500	0.27
50	0.08	1.31	3*1500	0.23÷0.25	0.09	1.31	3*1500	0.27÷0.29
100	0.09	1.36	3*1500	0.23÷0.25	0.10	1.36	3*1500	0.27÷0.29
120% $H_{s,Tr}$=100	0.11	1.36	3*1500	0.23	0.11	1.36	3*1500	0.27

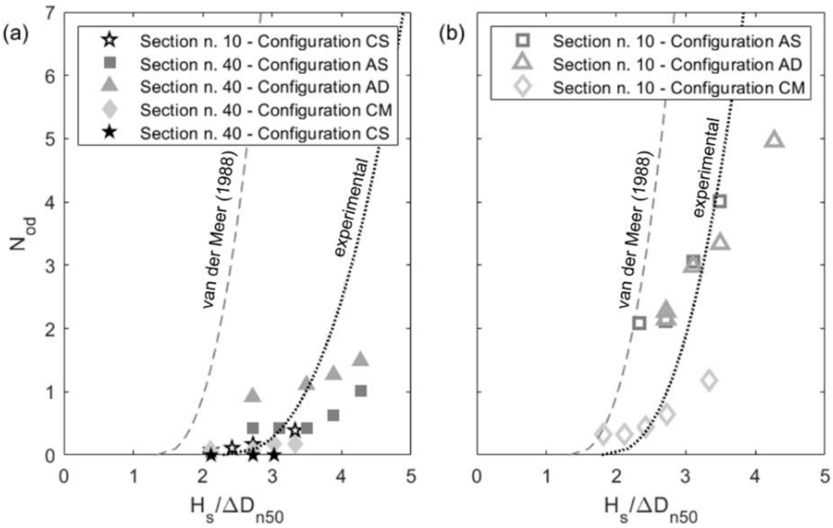

Figure 3. Experimental data on damage level, expressed by N_{od} as a function of the stability number (i.e. the ratio between the incident significant wave height H_s and the product between the relative buoyant density Δ and the median nominal diameter of the armor units D_{n50}): (a) upgrading options with a sufficient support at the toe of the additional armor layer; (b) upgrading options without a sufficient support at the toe of the additional armor layer.

Table 3. Estimation of the correction factor f and its standard deviation σ_f based on the experimental results on the upgraded Catania harbor breakwater for the formula for armor layers made of cubic units proposed by van der Meer (1988b).

Support at the toe of the additional armor layer	f [-]	σ_f [-]
Sufficient	1.72	0.29
Not sufficient	1.35	0.20

Fig. 3 shows the overlay of the measured N_{od} as a function of the stability number (i.e. the ratio between the incident significant wave height H_s and the product between the relative buoyant density Δ and the median nominal diameter of the armor units D_{n50}), the original van der Meer (1988b) formula and its adaptation to the experimental data on the upgraded Catania harbor breakwater. It can be observed that the van der Meer (1988b) formula returns N_{od} greater than the adapted one for the same stability number. In other terms, the van der Meer (1988b) formula is more conservative and could led to excessive oversizing of the armor blocks if applied for the design of restoration solutions. However, it is worth to point out that the deviation of the experimental data from the van der Meer (1988b) formula may be partly due to scale effects caused by the small dimension of the physical model. Therefore, additional investigations are needed for the quantification of the above-mentioned scale effects in order to verify the possible need for further corrections to equation 3.

Even though configurations E and EM do not show significant stability problems, the addition of an extra armor layer should be provided to homogenize the geometry of the structure, ensuring the same hydraulic response along the entire breakwater. Configuration CS appears as the only options among the studied ones able to ensure the same optimum structural performances regardless of the characteristic irregularities of the existing armor layer, because of the preliminary regularization of the blocks laying surface before the placement of the 62 t artificial cubes. Indeed, the only quarry stone toe berm could be not able to provide a sufficient toe support to the additional armor layer for the most irregular sections since it is usually designed guaranteeing a certain geometric consistency along the entire breakwater. In addition, configuration CM seems able to provide sufficient stability under wave attack also for the most irregular cross sections, but only thanks to the strong gravitational resistance of the cubic armor units of 62 t. Finally, configurations AS and AD could withstand the wave load only if a proper support at the toe of the additional Antifer blocks of 30 t is constructed.

Overtopping discharge

The overtopping phenomena was studied with reference to the mean overtopping discharge measured during the tests. The following empirical relationships between the mean overtopping rate, the incident significant wave height and the structure crest level (EurOtop 2018) was considered to validate the experimental results:

$$q^* = \frac{q}{\sqrt{gH_S^3}} = 0.09 * exp\left[-\left(1.5 * \frac{R_C}{H_S * \gamma_f}\right)^{1.3}\right] \qquad (4)$$

where q^* is mean overtopping discharge q normalized with respect to the square root of the product between the gravitational acceleration g and the cube of the incident significant wave height H_s, R_c is the maximum value between the crest level and the wave wall height referred to MSL and γ_f is the roughness factor (equal to 0.50 for Antifer blocks and 0.47 for double layer of artificial cubes). Equation 4 is valid for seaside slopes between 1:2 and 1:4/3 and wave attack orthogonal to the breakwater. Fig. 4 shows that the experimental data agree well with equation 4.

The preliminary outcomes of the analysis on configuration E (i.e. existing armor layer and wave wall raised up to +8.50 m above MSL) for both section n. 10 and n. 40 highlight that the sea level rise cause a 3÷4 times increase of the mean overtopping discharge. Therefore, the experimental results demonstrate that the possible effects of climate change in coastal areas make the necessity for upgrading the Catania harbor breakwater even more evident.

The solutions that involve the addition of an extra armor layer (i.e. configurations AS, AD, CM and CS) have similar hydraulic performances and ensure mean overtopping discharge smaller than the options which consist in the simple heightening of the wave wall (i.e. configurations E and EM). In addition, the geometry of the existing armor layer seems not to influence the hydraulic response of the restoration options, which offers a similar behavior for both section n. 10 and section n. 40.

As a consequence, the choice of the best upgrading solutions should not be based only on the analysis of the overtopping phenomena. Indeed, the structural behavior of the possible options should be considered in combination with the hydraulic performances of the structure.

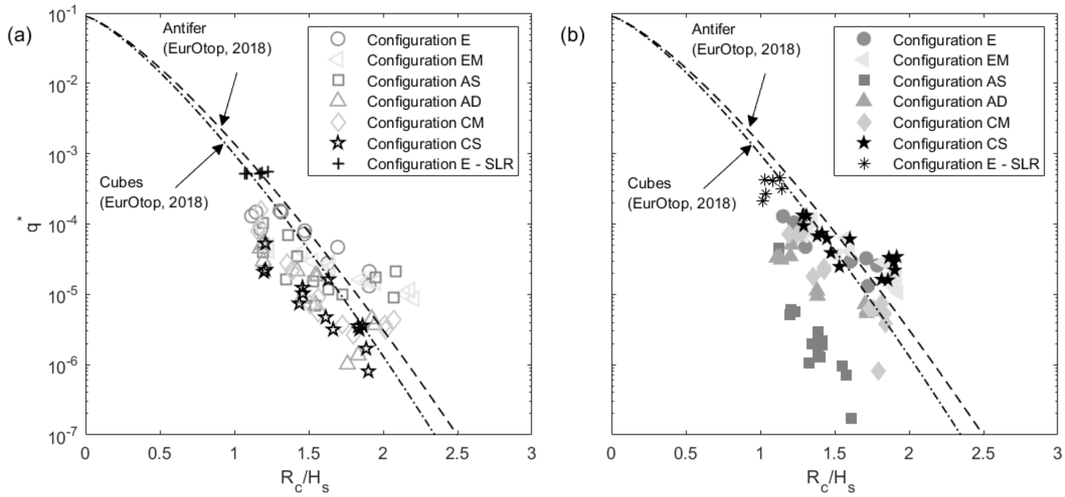

Figure 4. Experimental data on dimensionless mean overtopping discharge expressed as a function of the ratio between the crest level R_c and the significant wave height H_s, compared to the empirical formula suggested by EurOtop (2018): a) section n. 10; b) section n. 40.

CONCLUSIONS

Most of the worldwide historical coastal and Harbor breakwaters have been severely damaged by extreme sea storms during their lifetime and hence need to be upgraded, also taking into account the effects of climate change (Hughes, 2014). Physical modelling is the only feasible approach for the optimization of the upgraded structures, because of the existence of few studies concerning such an issue and the absence of sufficiently calibrated formulae or numerical models for the design of upgrading solutions for existing damaged breakwaters (Burcharth et al. 2014; Croeneveld et al. 1984; Lara et al. 2019; Foti et al. 2020).

Therefore, a novel methodology for the design of restoration solutions for existing breakwaters based on physical modelling is presented, through the case study of the Catania harbor breakwater. A systematic extensive experimental campaign on possible solutions for upgrading the Catania harbor breakwater was carried out, considering the existing structure and four upgrading options, which were tested under increasing significant wave heights, also considering the effects of sea level rise. The combined analysis of damage parameters and mean overtopping discharges led to the following general conclusions:

- Mean sea level rise makes more evident the need for upgrading the existing structure because it causes a 3÷4 times increase of the mean overtopping discharge.
- The choice of the best upgrading solutions should be addressed mainly by the structural performances of the possible options. Indeed, the differences in the hydraulic performances between the tested configurations is minor with respect to the variability of their structural responses.
- State-of-art formulae usually applied for new designs should be re-calibrated on the basis of experimental data on upgraded structures.
- The regularization of the laying surface before adding the extra armor layer ensures a sufficient support at the toe and hence the same performances regardless of the geometry of the existing breakwater cross sections.
- If enough support at the toe of the additional armor layer is guaranteed, in some cases even blocks smaller than the existing ones can be employed for armor layer restoration.

The above-mentioned findings lead to the identification of the option which involves the preliminary regularization of the existing armor layer before the addition of extra 62 t cubic blocks and the rise of the wave wall up to +9.50 m above MSL (i.e. configuration CS) as the optimal upgrading solution of the Catania Harbor breakwater.

To conclude, it is worth to point out that further investigations on different type of solutions (i.e. block types, interlocking degree, etc.) and structure geometries (i.e. offshore slope, blocks laying surface, toe protection, etc.) should be carried out to define a physical model-based methodology valid for the design of a wider range of upgrading solutions for different kind of existing breakwaters at the end of their service lifetime. In addition, the development of novel approaches (e.g. based on the Structure from Motion technique) to investigate the damage dynamics as a function of the geometry of the armor layer should provide a deeper insight into the interaction processes between waves and armor blocks.

ACKNOWLEDGMENTS

The Authors would like to thank the Port Authority of Eastern Sicily for partially supporting the work. This work has been also funded by the project “NEWS -Nearshore hazard monitoring and Early Warning System” (code C1-3.2-60) in the framework of the EU program INTERREG V-A Italia Malta 2014–2020 and by the project VARIO, funded by the program PIACERI of the University of Catania.

REFERENCES

Allsop, W., McKenna, J. E., Vicinanza, D., and Whittaker, T. T. J. 1996. New design methods for wave impact loadings on vertical breakwaters and seawalls. *Coastal Engineering 1996*, 2508–2521.

Arns, A., Dangendorf, S., Jensen, J., Talke, S., and Bender, J. 2017. Sea-level rise induced amplification of coastal protection design heights. *Scientific Reports*, 7(40171), 1–9.

Burcharth, H. F., Lykke Andersen, T., and Lara, J. L. 2014. Upgrade of coastal defence structures against increased loadings caused by climate change: A first methodological approach. *Coastal Engineering*, 87, 112–121.

Camus, P., Losada, I. j, Izaguire, C., Espejo, A., Menéndez, M., and Pérez, J. 2017. Statistical wave climate projections for coastal impact assessments. *Earth’s Future*, *5*(9), 918–933.

Carver, R. D. 1989. Prototype Experience with the Use of Dissimilar Armor for Repair and Rehabilitation of Rubble-mound Coastal Structures. *US Army Engineer Waterways Experiment Station*.

Chini, N., and Stansby, P. K. 2012. Extreme values of coastal wave overtopping accounting for climate change and sea level rise. *Coastal Engineering*, 65, 27–37.

Chini, Nicolas, Stansby, P., Leake, J., Wolf, J., Roberts-Jones, J., and Lowe, J. 2010. The impact of sea level rise and climate change on inshore wave climate: A case study for East Anglia (UK). *Coastal Engineering*, 57(11–12), 973–984.

Church, J. A., Clark, P. U., Cazenave, A., Gregory, J. M., Jevrejeva, S., Levermann, A., ... and Payne, A. J. 2013. Sea level change. PM Cambridge University Press.

CIRIA and CETMEF 2007. The rock manual: the use of rock in hydraulic engineering (Vol. 683). CIRIA.

Croeneveld, R. L., Mol, A., and Nieuwenhuys, E. H. 1984. Rehabilitation Methods for Damaged Breakwaters. *Coastal Engineering* 1984, 2467–2486.

EurOtop, 2018. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B.

Faraci, C., Scandura, P., and Foti, E. 2015. Reflection of sea waves by combined caissons. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 141(2), 04014036.

Foti, E., Musumeci, R. E., and Stagnitti, M. (2020). Coastal defence techniques and climate change: a review. *Rendiconti Lincei. Scienze Fisiche e Naturali*, 31(1), 123–138.

Franco, L. 1994. Vertical breakwaters: the Italian experience. *Coastal Engineering*, *22*, 31–55.

Frostick, L. E., McLelland, S. J., and Mercer, T. G. 2011. Users guide to physical modelling and experimentation: Experience of the HYDRALAB network. *CRC Press*.

Galassi, G., and Spada, G. 2014. Sea-level rise in the Mediterranean Sea by 2050: Roles of terrestrial ice melt, steric effects and glacial isostatic adjustment. *Global and Planetary Change*, *123*, 55–66.

Hedar, P. A. 1960. Stability of rock-fill breakwaters (No. 26). *Akademiförlaget-Gumperts*.

Hemer, M. A., Fan, Y., Mori, N., Semedo, A., and Wang, X. L. 2013. Projected changes in wave climate from a multi-model ensemble. *Nature Climate Change*, 3(1), 1–6.

Hudson, R. 1959. Laboratory Investigation of Rubblemound Breakwater. *Journal of Waterways and Harbors Division*, *1*, 610–659.

Hughes, S. A. 2014. Coastal engineering challenges in a changing world. *Journal of Applied Water Engineering and Research*, *2*(2), 72–80.

Isobe, M. 2013. Impact of global warming on coastal structures in shallow water. *Ocean Engineering*, 71, 51–57.

Jensen, O. J., Bisgaard, A., Wood, H., and Genovese, N. 2018. Alderney Breakwater, a developed rehabilitation solution. *Coasts, Marine Structures and Breakwaters 2017: Realising the Potential*, 575-585. ICE Publishing.

Koftis, T., Prinos, P., Galiatsatou, P., and Karambas, T. 2015. An integrated methodological approach for the upgrading of coastal structures due to climate change effects. *Proceedings of the 36th IAHR World Congress, The Hague, The Netherlands* (Vol. 28).

Lambeck, K., Antonioli, F., Anzidei, M., Ferranti, L., Leoni, G., Scicchitano, G., and Silenzi, S. 2011. Sea level change along the Italian coast during the Holocene and projections for the future. *Quaternary International*, 232(1-2), 250-257.

Lara, J. L., Lucio, D., Tomas, A., Di Paolo, B., and Losada, I. J. 2019. High-resolution time-dependent probabilistic assessment of the hydraulic performance for historic coastal structures: application to Luarca Breakwater. *Philosophical Transactions of the Royal Society A*, 377(2155), 20190016.

Ligteringen, H., Van der Lem, J. C., and Ramos, F. S. 1993. Ponta Delgada Breakwater Rehabilitation Risk Assessment with respect to Breakage of Armour Units. *Coastal Engineering 1992*, 1341-1353.

Lowe, J. A., and Gregory, J. M. 2005. The effects of climate change on storm surges around the United Kingdom. *Philosophical Transactions of the Royal Society A: Mathematical, Physical and Engineering Sciences*, 363(1831), 1313–1328.

Main, R., Hartley, A., Dengate, C., Rowe, E., and Blacka, M. 2016. Coffs Harbour Northen Breakwater – Not Your Average Upgrade. *25th NSW Coastal Conference*.

Morim, J., Hemer, M., Wang, X. L., Cartwright, N., Trenham, C., Semedo, A., ... and Erikson, L. 2019. Robustness and uncertainties in global multivariate wind-wave climate projections. *Nature Climate Change*, 9(9), 711-718.

Oumeraci, H. 1994. Review and analysis of vertical breakwater failures—lessons learned. *Coastal engineering*, 22(1-2), 3-29.

Santos-Ferreira, A., Cabral, M., and Santos, C. 2015. The rehabilitation of north breakwater of Nazaré harbor, Portugal. *Procedia Engineering*, 116, 755-762.

Takahashi, S. 2002. Design of vertical breakwaters. *PHRI Reference Document Nr. 34*.

van der Meer, J. W. 1988a. Rock Slopes and Gravel Beaches under Wave Attack. *PhD Thesis*, 214.

van der Meer, J. W. 1988b. Stability of cubes, tetrapods and accropode. *Conference Breakwaters88*.

van Gent, M. R., Smale, A. J., and Kuiper, C. 2004. Stability of rock slopes with shallow foreshores. *Coastal Structures 2003*, 100-112.

Vousdoukas, M. I., Mentaschi, L., Voukouvalas, E., Verlaan, M., Jevrejeva, S., Jackson, L. P., and Feyen, L. 2018. Global probabilistic projections of extreme sea levels show intensification of coastal flood hazard. *Nature Communications*, 9(1), 1–12.

Vousdoukas, M. I., Voukouvalas, E., Annunziato, A., Giardino, A., and Feyen, L. 2016. Projections of extreme storm surge levels along Europe. *Climate Dynamics*, 47(9–10), 3171–3190.