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Integrated assessment of the hydraulic and structural performance of the OBREC device in the Gulf of Naples, Italy

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1 **Integrated assessment of the hydraulic and structural performance of the OBREC**
2 **device in the Gulf of Naples, Italy**

3
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13 **Abstract**

14 The aim of this work is to analyze the performance of the Overtopping Breakwater for Energy
15 Conversion (OBREC), developed by the team of the University of Campania and installed at
16 prototype scale in the port of Naples. It is a multifunctional coastal structure aimed to protect harbors
17 and produce energy, based on the overtopping principle. This device has been preliminary analyzed
18 by means of experimental and numerical investigations.

19 This contribution provides for the first time an integrated assessment of the OBREC hydraulic and
20 structural performance, by means of the measurements collected at the prototype installation and of
21 numerical modelling i.e. methodologies not affected by the scale effects. The numerical model,
22 developed in the openFOAM environment, is calibrated on the field data gathered during a storm
23 event and is then applied to extend the information related to the OBREC response under the typical
24 wave climate. The results obtained are proposed in terms of overtopping discharge rates and pressures
25 acting along the OBREC profile, under several sea states.

26 Based on these results some recommendations and indications regarding the optimal cross section
27 design are given to maximize the energy production, without compromising the structural stability,
28 and to promote the device exploitability.

29

30 **1. Introduction**

31 More than the 50% of the shoreline of Europe, US, Australia, and Asia has been modified by hard
32 engineering, including groins and breakwaters to protect against erosion and wave action in support
33 of recreational boating and other activities [14]. The shorelines are also facing more frequent and
34 severe extreme events, increasing the financial costs of maintaining these structures under future
35 climate change scenarios [23][45].

36 Recently, to counter this situation, a series of integrated approaches have been developed to preserve
37 the ecosystems, minimize the social and the environmental impacts. Some examples are represented
38 by eco-compatible sediment management for beach maintenance, a. o. “Sand Engine”, “Mud Motor”
39 and beach drainage [9][36]. Other interventions provide ecosystem enhancement through the
40 restoration and/or creation of habitats, such as sand dunes [22][29], saltmarshes restoration [17] or

41 intertidal buffer areas; actions to reduce the impact on society and economy including insurance
42 frameworks, risk communication and landscape planning. Furthermore, new designs of breakwaters
43 and dikes have been developed to combine coastal protection with leisure activities, such as artificial
44 reefs and promenades [34], or with the production of marine renewable energy [28][42]. Their
45 principal aim is to improve the use of coastal areas, while minimizing the installation/maintenance
46 costs [1].

47 Specifically, in the framework of the energy production by means of Wave Energy Converter (WEC)
48 devices, two promising technologies have been designed to be entirely embedded into traditional
49 coastal defence structures to share the costs, i.e. Oscillating Water Column (OWC hereafter) and the
50 Overtopping Device (OTD hereafter) [42]. The former is capable of harvesting energy from the
51 oscillation of the seawater inside a semi – submerged chamber, while the latter collects the seawater
52 of incident waves in a reservoir above the sea level, then released by means of turbines. Both of them
53 are concrete – made devices, which can be placed or integrated in new or existing breakwaters, aimed
54 to i) protect the harbour inshore area, and ii) produce energy.

55 Some prototype of OWCs exist, such as the REWEC3 [3], the PICO [33], the LIMPET [2] and the
56 MUTRIKU [37] power plants. The first onshore OTD device, i.e. the Slot Sea Cone Generator [43],
57 consisted of a concrete dike with multiple reservoirs, suited for a wide spectrum of different wave
58 conditions. However, this technology resulted to be economically unfeasible due to its complex
59 geometry. Starting from this experience, the team of the University of Campania developed a new
60 Overtopping Breakwater for Energy Conversion, OBREC hereafter [41]. It is a traditional rubble
61 mound breakwater including a top concrete structure, i.e. the device that consists of a ramp followed
62 by a reservoir closed by a crown wall. The reservoir is located above the still water level and it is
63 linked with the machine room by means of a pipe passing through the crown wall. The potential
64 energy of the overtopping waves collected inside the reservoir is converted by means of low – head
65 turbines in the machine room.

66 OBREC was preliminary investigated under ordinary and extreme wave conditions by means of both
67 small-scale experiments [41] and numerical modelling [31][32] to support the design of the prototype
68 installed in the port of Naples. During the autumn-winter 2016/17, a first rough monitoring campaign
69 was performed to observe the OBREC functioning under a real sea state and to check if the energy
70 conversion occurred in safe conditions [10]. To provide more detailed information about the wave-
71 structure interactions, a second monitoring campaign was carried-out between Autumn 2017 and
72 Spring 2018, in the context of the project BRIGRID (Bridging the Gap Innovations in Disaster
73 Resilience) (<https://brigaid.eu/>), funded by the EU Horizon2020 program.

74 This paper aims to support and extend the information obtained from of this second field monitoring
75 campaign by means of 2D numerical modelling. Section 2, presents the pilot plant, the monitoring
76 system, the storm events recorded at the beginning of the 2018 used to calibrate the numerical model
77 and the wave conditions characterizing the typical wave climate at the installation site, used to analyse
78 the integrated response of the device. Section 3 is dedicated to the numerical modelling set-up,
79 including the description of the model, the implemented wave conditions, the characteristics of the
80 structure and the numerical domain and the numerical measurements performed. Section 4 compares
81 the numerical results with the field data acquired during a real single storm event, focusing on the
82 structural response of the device. Furthermore, in Section 5, the same model has been adopted to
83 provide an integrated assessment of the structural and the hydraulic performance of the OBREC
84 device under the typical wave climate of Naples. Based on these results, Section 6 gives some

85 recommendations and indications about the optimal design of the OBREC geometry to maximize the
86 energy production.
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88 **2. OBREC pilot plant**

89 **2.1 Description of the prototype installation**

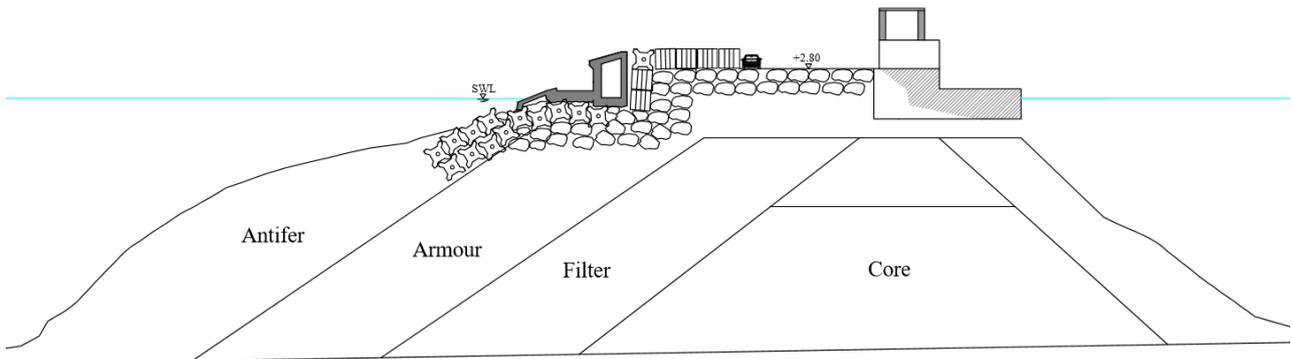
90 The first OBREC pilot plant was installed at the end of 2015 in the existing San Vincenzo breakwater,
91 which protects the Port of Naples (Italy) in the Middle Tyrrhenian Sea. The geographical coordinates
92 of the site are 40° 49' 58.68'' N and 14°16' 03.64'' E (Fig. 1), which resulted to be ideal for a first
93 installation, due the following reasons:

- 94 • the low-occurrence of extreme events, which allows a safety installation and maintenance
95 procedures;
- 96 • the principal direction of the waves is quasi orthogonal with respect to the device, leading to
97 perform a 2D numerical investigation;
- 98 • the non-breaking wave conditions, thanks to the 25 m water deep at the toe of breakwater;
99

100 The San Vincenzo breakwater is a traditional rubble mound breakwater, provided with an external
101 layer made of artificial concrete blocks (the so called “Antifers”, i.e. grooved concrete with holes), a
102 filter layer made of rocks (characterized by different diameters) and a quarry run core. The actual
103 offshore slope can be approximated at 1:2. However, the random placement of the Antifers created a
104 curved profile, which acts like a berm due to the combination of the relative water depth and typical
105 higher waves at the site. A road, located at the rear side of the structure, allows the access to the
106 breakwater (Fig. 1), simplifying the construction and maintenance operations and reducing the costs
107 with respect to offshore installations.

108 The general cross section of the device consists of a ramp, which improves the run-up process,
109 followed by a reservoir-crown wall system, collecting the overtopping waves. The latter is linked, by
110 means of a pipe, with the machine room hosting the turbines. The crown wall, aimed to protect the
111 inshore area, is provided with a parapet on top of it to reduce the overtopping discharge at the rear
112 side of the structure. The overall installation has a crest level of 4.50 m above the still water level
113 (SWL in Fig. 1).

114 The structure was cast in-situ, except for the prefabricated ramps, and placed directly on the external
115 layer. However, to enhance the overall stability, the foundation involved the installation of 12
116 concrete micropiles constituted by steel tubular reinforcement with an external diameter of 300 mm,
117 and thickness of 25 mm and a total length of 10 m.
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Fig. 1. Location of the pilot plant in the middle of the San Vincenzo breakwater (Port of Naples. Italy).

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The prototype is composed by 2 configurations, divided by a wall septum, which differ for the height of the ramp, i.e. R_c . Specifically, the RS-LAB (*Real Scale laboratory*) and the NW-LAB (*Natural Waves Laboratory*) cross sections are characterized by a ramp 2.0 m and 1.2 m high, respectively (values referred to the mean tide level, see Fig. 2). In both cases, this parameter was tuned on the typical wave climate characterizing the installation site. In particular, the NW-LAB is intended to capture the mean run-up of the most frequent waves (i.e. $H_s = 0.75$ m, see Table 1), while the RS-LAB refers to the higher energetic sea states and operative consideration about the nominal hydraulic head of commercial low-head Kaplan turbines, i.e. about 1.5-2 m [32]. The ramp includes a double inclination of 78° and 22° , according to the field observations and to the results obtained from the numerical investigations performed by Palma et al. [32]. The submerged quasi-vertical part of the ramp was introduced to i) improve its resistance to bending and fatigue, and to ii) enhance the stability of the device placed on the armour layer.

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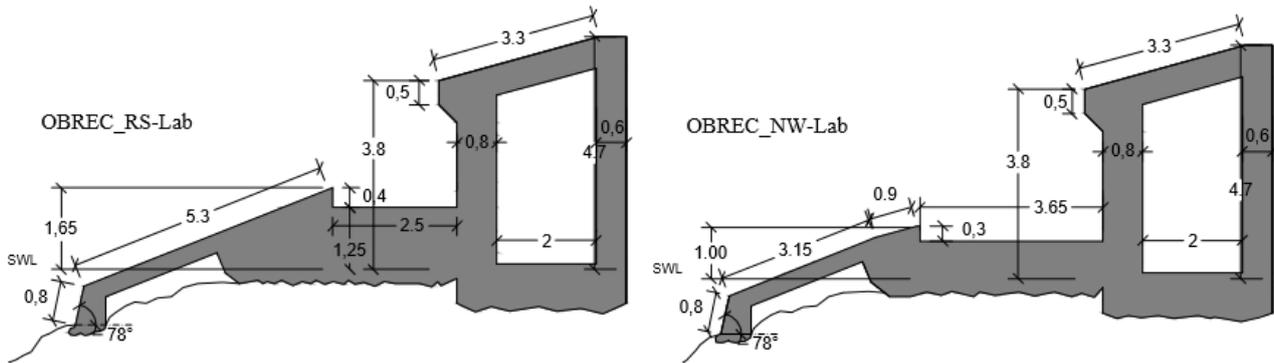
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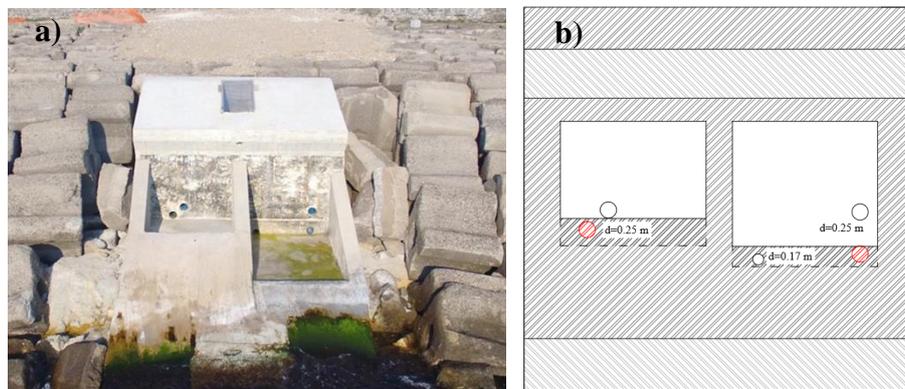
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The whole installation has the same longitudinal dimension, i.e. from the beginning of the ramp to the inshore edge of the machine room. The common parts for both the configurations are i) the rear crown wall, provided with a bullnose, and ii) the machine room (6.0 m x 3.4 m). According to this spatial constraint, the reservoirs have different dimensions, i.e. 2.5 m and 3.65 m for the RS-LAB and NW-LAB (see Fig. 2), respectively. The wider reservoir is associated to the lower ramp, which is the more frequently overtopped, and vice versa. Indeed, the experimental and numerical investigations performed by Iuppa [24] and Palma [32], respectively, demonstrated that the selection of the reservoir longitudinal width can be subordinated to the height of the ramp. This latter parameter has to be tuned on the specific wave climate and directly affects the overtopping volumes and, therefore, the hydraulic performance. The reservoir is linked with the machine room by means of pipes passing through the crown wall. The RS-LAB is provided with two pipes of 0.25 m of diameter, while the NW-LAB has also a third one of 0.17 m of diameter (see Fig. 3a and 3b). At this stage, the flow for both the configurations is passing through the biggest pipe only, which is located in the lowest position (Fig. 3b, 3c). The other pipes will give the chance to test the effects on both the power production and on the loadings on the crown wall induced by the use of more than one pipe contemporarily and by the different locations of the openings.

150



151
152 **Fig. 2. Cross-sections of the OBREC prototype configurations in the Naples harbour: a) RS-Lab**
153 **configuration; b) NW-Lab configuration [10].**
154



155
156 **Fig. 3. a) Frontal view of the OBREC prototype installed in the port of Naples, and b) position of the**
157 **pipes (in red the open ones).**
158

159 2.2 Field Monitoring Campaign: equipment and measurements

160 The pilot plant is provided with several instruments to gather information both on the wave conditions
161 and the hydraulic and structural response of the structure. A GPS wave buoy, the Directional Wave
162 Spectra Drifter (DWSD), is located 100 m far from the San Vincenzo breakwater, in front of the
163 OBREC device, i.e. latitude $40^{\circ}49'57.68''$ – longitude $14^{\circ}16'3.43''$. The DWSD buoy was provided
164 by the Scripps Institution of Oceanography (<https://scripps.ucsd.edu/>). It uses the GPS sensor package
165 in order to measure $w(t)$, $u(t)$ and $v(t)$, which represent respectively the vertical, horizontal E-W and
166 the horizontal S-N buoy velocity components [6]. These data are acquired from the changes in the
167 frequency of the GPS signal. The sampling of $u(t)$, $v(t)$ and $w(t)$, acquired at 1 Hz, is performed for
168 17 min every hour, and the main wave data parameters are transmitted in real-time through the Iridium
169 satellite system. The information related to the wave conditions are accessible on a dedicated website
170 together with the battery voltage and pressure, the temperature and humidity of the hull. The DWSD-
171 buoy located in front the pilot plant, is shown in Fig. 3.

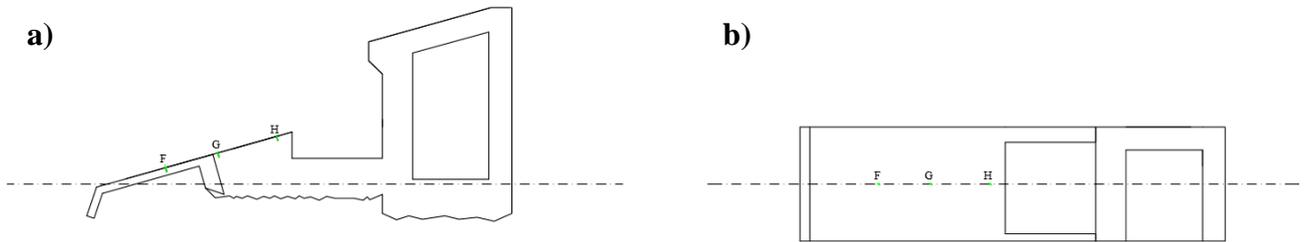
172 To assess the hydraulic performance, the water flowing through the pipes is collected in a shunt tank,
173 located in the machine room. Inside the tank, a resistive gauge measures the hydraulic head with an
174 acquisition frequency of 10 Hz, with the purpose of optimising i) the position/dimension of the pipe,
175 and ii) the design of the shunt tank, considering the typical wave climate. When an overflow occurs,
176 the excess water is driven back to the sea by means of a specific overflow discharge.

177 To measure the pressures acting on the device, 3 pressure transducers were installed on the RS-LAB
178 ramp, i.e. F, G and H in Fig. 5; and 4 along the NW-LAB crown wall and in correspondence of the

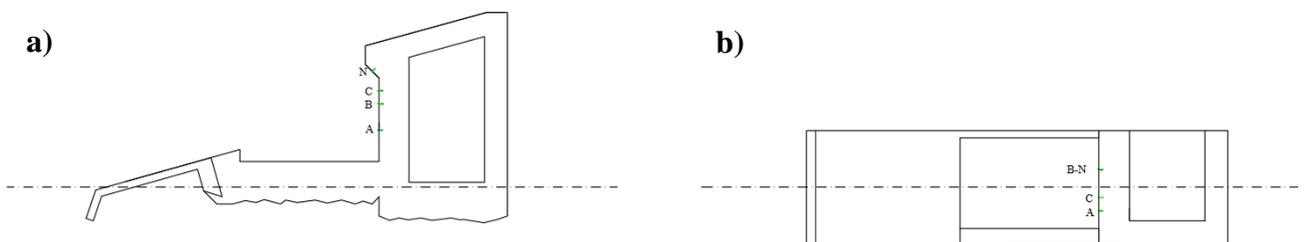
179 bullnose, i.e. A, B, C and N in Fig. 6. Their characteristics were selected according to the expected
 180 hydrodynamic loads, which were measured and analysed during the small-scale laboratory campaigns
 181 performed by Contestabile [11]. Specifically, these transducers can measure the pressures ranging
 182 from 0 to 100 bar, with a full scale F.S. precision at 25° of 0.4%, under a temperature range from
 183 - 40° to 125°. The output signals vary between 4 and 20 mA. Although the original acquisition
 184 frequency is 1 kHz, the signal was instantaneously low filtered to 100 Hz, by keeping the maximum
 185 value of 10 consecutive measures. This procedure reduces the amount of the gathered data, without
 186 losing information about the highest loads.
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 189 **Fig. 4. The DSWD located in front of the OBREC prototype.**
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 192 **Fig. 5. a) Longitudinal and b) flat sections of position of the pressure transducers for the RS-LAB**
 193 **configuration.**



194
 195
 196 **Fig. 6. a) Longitudinal and b) flat sections of position of the pressure transducers for the NW-LAB**
 197 **configuration.**



Fig. 7. Pressure transducers installed on the OBREC device.

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201 2.3 Wave climate at the pilot plant

202 Due to the short period of wave monitoring with the DWSD buoy, the typical wave climate in front
203 of the OBREC has been calculated by propagating the hind cast data from the European Centre for
204 Medium-Range Weather Forecasts (ECMWF) collected offshore the Gulf of Naples during the period
205 2006-2016.

206 For studying the long-term variation of the wave height, the use of the ERA-Interim dataset could be
207 considered adequate, i.e. reliable and generally slightly conservative for wave energy assessment [13].
208 The mean wave power was found to range between 1.6 and 2.5 kW/m. Table 1 shows the wave
209 climate corresponding to an average year (yearly mean wave energy density of 1.87 kW/m). In
210 particular, the typical wave climate is reconstructed by means of 8 sea states (Table 1), which are
211 characterized by a significant wave height H_s (i.e. the average height of the highest one-third of all
212 the measured waves), a mean period T_m (i.e. the mean of all the wave periods in a time-series
213 representing a certain sea state) and a certain frequency of occurrence Fr during the year. The wave
214 properties determine the energy flux P_{abs} per unit of wave – crest length, defined in Eq. (1):
215

$$P_{abs} = \frac{\rho g}{64} T_m H_s^2 \frac{1}{1000} \quad (1)$$

216
217
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where ρ is the water density and g is the gravitational acceleration. By multiplying P_{abs} by the values
of Fr , it is possible to obtain the annual contribution of the theoretical wave energy flux per unit
length P_{year} , given by each sea state as reported in Table 1, by neglecting all the losses.

Table 1. Characteristics of the wave conditions of the typical climate of Naples.

Sea states	H_s [m]	T_m [s]	P_{abs} [kW/m]	Fr [%]	P_{year} [kW/m]
1	0.25	2.25	0.06	0.1498	0.01
2	0.75	3.90	1.01	0.5912	0.60
3	1.25	5.03	3.61	0.2083	0.76
4	1.75	5.95	8.37	0.0409	0.34
5	2.25	6.75	15.68	0.0083	0.14

6	2.75	7.46	25.90	0.0012	0.02
7	3.25	8.11	39.33	0.0003	0.01
8	3.75	8.71	56.25	0.0001	0.01

2.4 Storm conditions in the monitoring period

During the first semester of 2018, 5 storm events with a quasi-orthogonal wave direction were recorded by the DWSD buoy. Table 2 reports the dates, the duration, the storm duration-averaged significant wave heights $H_{s,mean}$, the maximum significant wave heights $H_{s,max}$, and the mean period T_m . These storm events correspond to typical winter storms in the study site (i.e. expected to have from 1 to 2 years return period). As anticipated, the original acquisition frequency is 1 kHz, instantaneously low-filtered to 100 Hz. Although this procedure strongly reduced the amount of data, the elaboration of the raw signals required a huge effort. The procedure is tuned on the first available storm event S1, which is then selected to be reproduced in the numerical model.

The storm S1 (see Table 2) is characterized by a rising phase during the first 2 days, a peak phase of 6 h and a decay phase that lasted up to 2 days (see Fig. 8).

The field and numerical data were compared at the peak phase to focus the analysis on the most severe part of the storm event. Indeed, it is characterized by $H_{s,mean} = 2.13$ m and $T_p = 8.37$ s, used as input parameters to describe the wave attack in the numerical simulations. In both the cases, physical and numerical testing, the response of the structure is determined by measuring the pressures exerted by the wave attack along the OBREC profile. The results obtained have been synthetized by computing the maximum pressures p_{max} and the statistical representative values such as p_{250} and p_{100} , which represent the non-exceedance values of 99.7% and 90% respectively.

Table 2. Storm events occurred at the beginning of the 2018, characterized by a mean period T_m and a storm duration-averaged significant wave heights, $H_{s,mean}$, and a maximum significant wave heights $H_{s,max}$, respectively.

Storm event	Date	Duration [days]	$H_{s,mean}$ [m]	$H_{s,max}$ [m]	T_m [s]
S1	(6 – 10)/1/2018	5	1.99	2.47	5.58
S2	(15 – 18)/1/2018	4	1.72	2.02	5.20
S3	(1 – 4)/2/2018	4	1.98	3.07	5.85
S4	(20 – 22)/3/2018	3	1.93	2.21	5.52
S5	(31 – 2)/4/2018	3	2.47	2.81	6.36

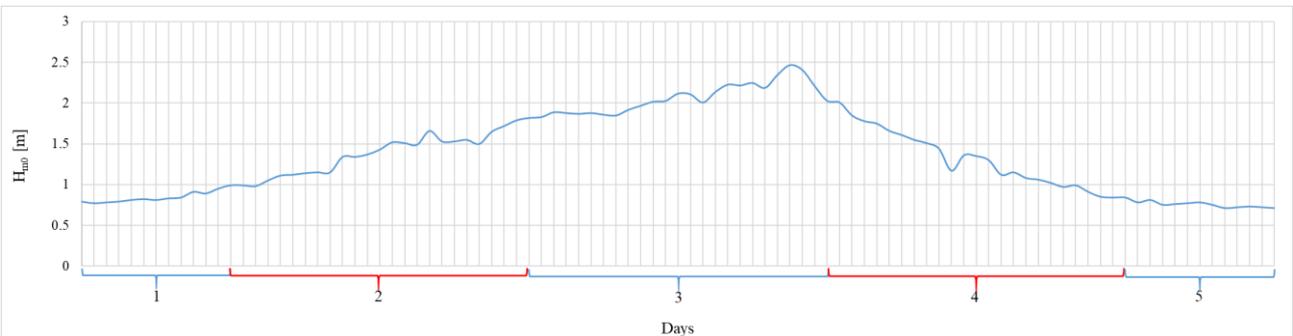


Fig. 8. Variation of the significant wave height H_s during the storm event S1, described in Table 2. Measurements derived from DSWD located in front of the OBREC prototype (see Fig. 4).

3. Set-up of the numerical model

3.1 Overview of the numerical model

The 2D numerical simulations have been performed with the free, open source CFD software openFoam (www.openfoam.org). Specifically, the toolbox *waves2Foam*, originally developed at the Technical University of Denmark by Jacobsen et al. [25] and capable of generating/absorbing the waves, has been used.

The principal direction of the waves in the installation site is mainly perpendicular to the OBREC device all over the year. Therefore, this reduces the error due to simulating the wave – structure interactions of the real site by means of a 2DV representation. The model is based on the Volume Average Reynolds Average Navier Stokes equations (VARANS), implemented using the (Volume of Fluid) VOF method. The library *waves2foam* is a modification of the native one, i.e. *interFoam*. It is capable of solving 2 incompressible, isothermal immiscible fluids, i.e. air and water, simultaneously tracked using the scalar field γ (equal to 0 for air and 1 for water, and any intermediate value is a mixture of the two fluids), considering the wave generation/absorption by means of the relaxation zone technique. In the momentum balance equation, an extra term is included to take into account of the surface tension between the two phases. The generation/absorption of free surface water waves is based on the application of the relaxation zone technique (active sponge layers), and includes a large range of wave theories for regular and irregular waves.

The turbulence Large Eddy Simulations (LES) model [7] has been introduced for this study case, since the Direct Numerical Simulation (DNS) approach, in which the full Navier–Stokes equations are numerically solved directly from the smallest to the largest eddies, would be computationally too expensive. On the other hand, the Reynolds-averaged Navier–Stokes (RANS) approach solves only the averaged quantities and it is therefore used when it is not necessary to simulate the detailed instantaneous flow. The LES approach computes directly the large-scale motions (large eddies) of turbulent flow, while models only the small-scale [46]. Therefore, it has been preferred because it enormously reduces the computational cost with respect to DNS, being more accurate than RANS. The geometries, the numerical domain and the mesh have been defined, at prototype scale, by means of Gmsh [21], a free 3D finite element mesh generator with a built – in CAD engine and post – processor.

The characteristics of the PC used to simulate the wave conditions are reported in Table 3, here below:

Table 3. Characteristics of the PC used to simulate the wave conditions.

Ubuntu version	14.04 LTS
Memory	31.2 GiB
Processor	Intel Xeon(R) CPU ES-2690 v3@2.6 GHz x 16
Graphics	Gallium 0.4 on NVE7
OS type	64 bit
Disk	1.9 TB

3.2 Wave attacks

The OBREC response is analyzed i) during a monitored storm event, and ii) under the typical wave climate of Naples. Therefore, the following wave attacks were selected:

- storm wave condition: a time window of 6 h is selected from the whole S1 data series. It is centered around the peak phase (i.e. 3 h before and 3 h after), characterized by significant wave height $H_s = 2.13$ m and a peak period $T_p = 8.37$ s;
- typical wave conditions: a sub-set of wave conditions is used to assess the hydraulic and structural performance of the OBREC under the typical wave climate (Table 1). Each wave attack lasted in this case for or about 600 waves.

The wave attacks were reproduced by implementing a Jonswap wave spectrum characterized by a peak enhancement factor $\gamma_P = 3.3$.

In order to ensure a suitable number of waves for a reliable statistical analysis, i.e. 600 waves [35], the length of the simulations vary according to the sea state and, specifically, according to the peak period. Therefore, the runtime of simulations is in the range of 1-2 weeks, for the shortest (sea state 3 in Table 1) and the longest (sea state 8 in Table 1) simulations, respectively.

3.3 Characteristics of the tested cross sections

The 2 cross sections composing the OBREC prototype, have been tested separately. Hence, two numerical model, representing half of the structure (see Fig. 9), have been defined. This allowed to drastically reduce the numerical domain, minimizing the computational effort without compromising the interactions between waves and porous media. Indeed, an open-flow condition and an absorption relaxation zone (Fig. 9), is implemented as the outlet boundary, to support the free outflow of the water. The porous layers, composing the San Vincenzo breakwater, are characterized by a cell size equivalent to the nominal diameter of the rocks. In *waves2foam*, the flow through the porous media is modelled by means of the extended Darcy-Forchheimer equation [19]. The term related to the turbulent flow follows the expression developed by Van Gent [40], representing the effect of the oscillatory flows through the Keulegan-Carpenter (KC) number.

$$b = \beta \left(1 + \frac{7.5}{KC} \right) \frac{1-n}{n^3} \frac{1}{gD_{50}} \quad (2)$$

The parameters needed to characterize a specific layer are: porosity n [18]; added mass coefficient c_A [40]; KC number, linear and non-linear friction coefficients α and β . These parameters have been set based on the sensitivity study performed by Jensen et al. [26] on a traditional rubble mound breakwater, similar to the structure involved in the present study case, see Table 4. Furthermore, they have been selected by calibrating them against the measured pressure data, to obtain an overall reliable estimation of the loads acting along the OBREC profile.

The offshore slope, composed by Antifers, is represented as a straight line (Fig. 9). In the reality, the random placement of the concrete blocks produced a curved edge, which acts like a berm. This simplification has to be taken into account during the analysis of the numerical results.

Table 4. Characteristics of the porous layers composing the San Vincenzo breakwater.

	n	KC	c_A	$D_{50}[m]$	α	β
Antifer	0.50	128	0.34	2	500	2.0
Armour	0.45	128	0.34	1.4	500	2.0
Filter	0.45	128	0.34	0.75	500	2.0

326

327 The OBREC configurations have been numerically tested by means of 2D simulations, which
 328 represent a good approximation of the real case since that the main wave direction is orthogonal to
 329 the prototype installation.

330 To analyze the OBREC behavior under the typical wave climate, the two configurations were
 331 provided with a pipe with the same nominal dimension of the prototype. However, due to the
 332 limitation of the 2D model, the pipe is modeled as a rectangular hole covering the whole width of the
 333 domain. Test cases with open pipe conditions are referred hereinafter as “open”.

334 For the analysis of the selected storm event, an additional case is proposed, by closing the pipe
 335 (“closed” in the following), to represent the most severe conditions, corresponding to the prototype
 336 observation that the reservoirs do usually work in saturated conditions during severe storms.

337

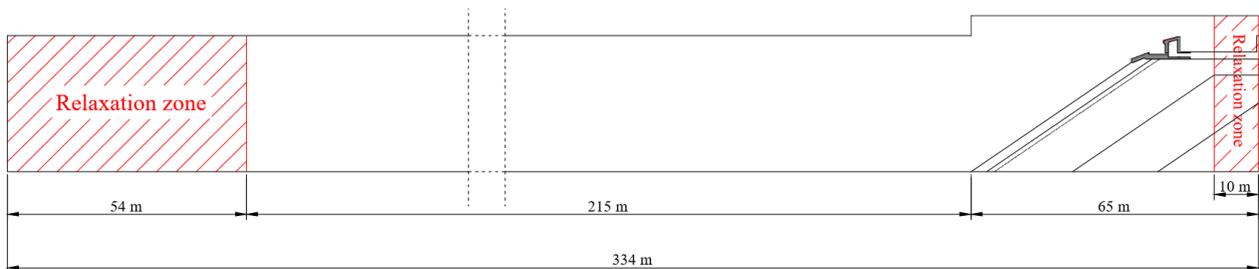
338 3.4 Numerical wave flume and mesh

339 The numerical domain is defined considering the extreme wave condition, i.e. Test 8 in Table 1, to
 340 correctly represent the wave development for all the sea states considered.

341 The length of the domain is set equal to 334 m, which corresponds to 3 times the wavelength
 342 $L = 107.5$ m. It includes an inlet relaxation zone for the wave generation, which has to be at least half
 343 of the wavelength (i.e. 54 m), followed by two wavelengths (i.e. 215 m) to let the wave to be
 344 completely developed before reaching the structure (Fig. 9). The breakwater footprint length is 65 m
 345 (see Fig. 11), including the outlet relaxation zone (10 m).

346 The wave height affects the extension of the vertical dimension of the domain. To minimize the
 347 computational effort, i.e. the number of the cells, the domain is divided in 2 parts, which differ for
 348 their vertical extension and for the mesh adopted. The first part is 31 m high, i.e. 26 m of water depth
 349 plus 5 m to allow the correct representation of the waveform. In the second part, where the wave –
 350 structure interaction occurs, the vertical dimension is extended up to 35 m.

351



352

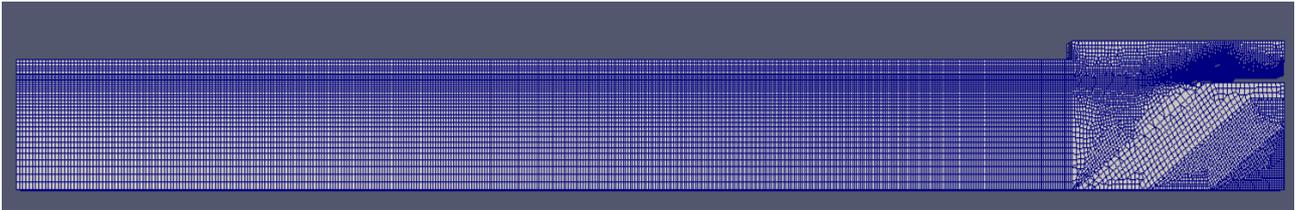
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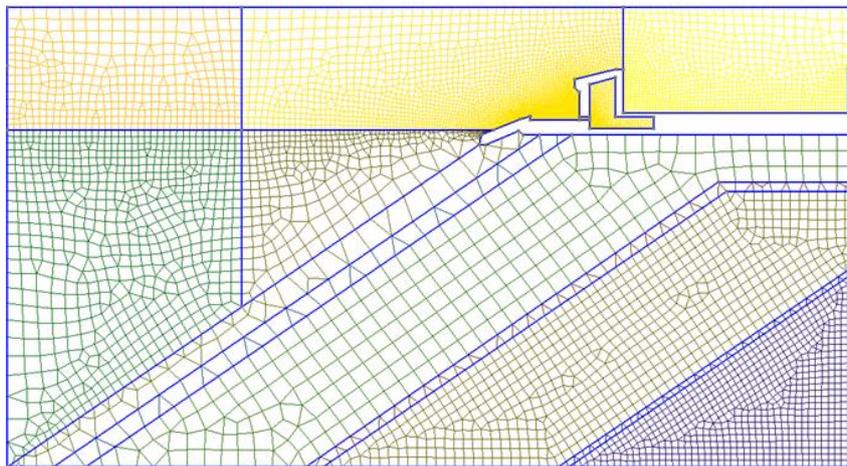
Fig. 9. Scheme of the numerical domain.

355 The mesh characterizing the first part of the domain is structured and graded, both in the horizontal
 356 and vertical directions (Fig. 10). In particular, the cell dimension decreases in correspondence of the
 357 still water level and going towards the structure. The second zone is characterized by an unstructured
 358 mesh, more flexible and suitable for peculiar geometries, i.e. the OBREC device installed in a
 359 traditional breakwater. The grid reaches its minimum dimension, i.e. 0.02 m, in correspondence of
 360 the reservoir and the pipe inside the OBREC, where a high accuracy of the results is required
 361 (Fig. 11). The mesh here described has been adopted for all the tested wave conditions. In the worst
 362 case, i.e. sea state 3 in Table 1, during the wave propagation the discretization is

363 $L/\Delta x \approx 100$ and $H_s/\Delta y \approx 10$. The latter ratio, due to the refinement performed in correspondence of
364 the structure becomes $H_s/\Delta y \approx 60$, to better represent the wave-structure interaction.
365 The bottom edge of the numerical domain has been set as impermeable. The left and the right edges
366 are combined with the inlet and outlet relaxation zones. The former is responsible of generating the
367 wave trains, while the latest allows the free outflow of incoming water, taking into account also of
368 the presence of the porous media. The atmosphere patch characterize the top edge and the end of the
369 pipe, to let the fluids to freely flow out of the domain.
370



371
372 **Fig. 10. Mesh characterizing the numerical domain.**
373



374
375 **Fig. 11. Unstructured mesh characterizing the second part of the domain, occupied by the breakwater.**
376

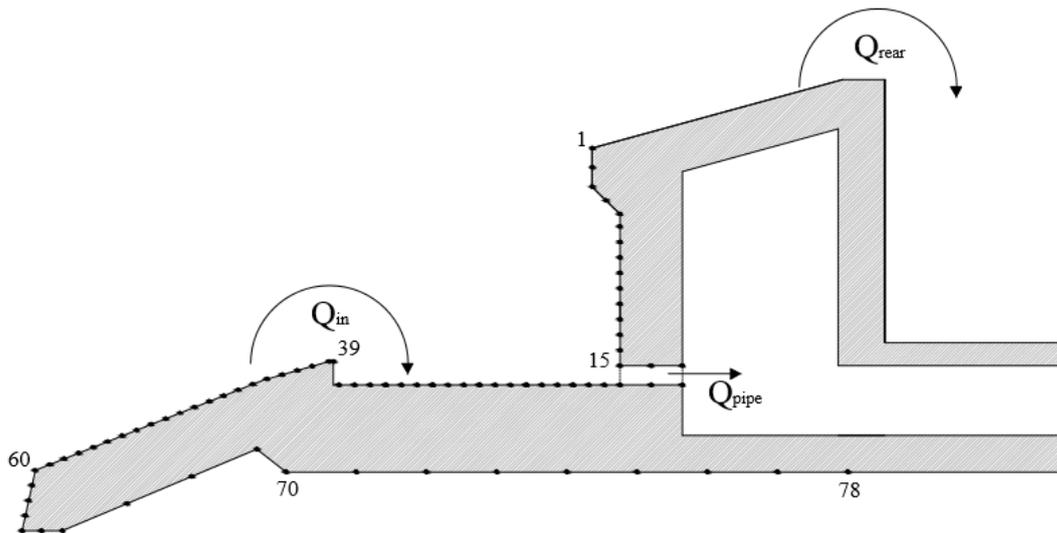
377 **3.5 Numerical Measurements**

378 Several wave gauges and probes are placed inside the numerical channel and along the structure to
379 analyze the surface elevations, the velocities profiles and the pressures acting on the device. The
380 general output Time Step and the Courant number are equal to 0.1 s and 1, respectively, for all the
381 simulations. However, the sampling frequency of the pressures, performed during the runtime, has
382 been forced to 100 Hz, to be consistent with the field data acquisition.

383 To track the free surface location, 35 wave gauges are set every 10 m from the beginning of the
384 domain until the structure toe. The wave reflection coefficient K_r is evaluated, applying a 3-point
385 method [27] to the 3 gauges positioned at 155, 156.1 and 157.8 m, about 2 wavelengths L far from
386 the structure.

387 The average overtopping discharge rates inside the reservoir Q_{in} , flowing through the pipe Q_{pipe} and
388 overtopping the rear side of the structure Q_{rear} (Fig. 12) have been computed by integrating the
389 horizontal water velocity components along the vertical direction. The velocity output is given every
390 0.005 m inside the pipe, while is discretized every 0.01 m at the rear side of the structure and inside
391 the reservoir. The different accuracy is to achieve the best representation of the different dynamics
392 related to these 3 parts of the structure.

393 In order to have a more comprehensive understanding of the OBREC functioning under the typical
 394 wave climate, an additional parameter has been computed, i.e. $T_{\%,full}$. It represents the average
 395 percentage of the pipe section filled by water, during the entire simulation. Fig. 12 shows the location
 396 of the pressure measurement points along the OBREC profile. A linear pressure distribution between
 397 two consecutive transducers has been assumed along the structure, considering the short distance
 398 between the probes, i.e. 0.02 m along the entire profile, except for the probes related to the uplift
 399 pressures.
 400



401
 402 **Fig. 12. Example of the measuring points for the NWLAB configuration, i.e. pressure probes and**
 403 **discharge rate inside the reservoir Q_{in} , through the pipe Q_{pipe} and at the rear side of the structure Q_{rear} .**
 404

405 4. Results

406 4.1 Numerical and observed wave loading under a storm

407 The results for the two numerical OBREC configurations are directly compared with the pressures
 408 measured during the peak phase of the storm event S1 (see Table 5), by the transducers installed on
 409 the OBREC prototype (see Fig. 6). The results are here proposed in terms of maximum and statistical
 410 values, p_{max} and p_{250} in Table 5. The latter one corresponds to the non-exceedance level of about
 411 99.7% (i.e. the average of the highest 4% of values).

412 Table 5 shows that the numerical model tends to generally overestimate the field data, except for the
 413 upper part of crown wall of the open configuration, i.e. pressure transducers C and N. This is due to
 414 the 2D representation of the pipe that covering the entire width of the numerical domain absorbs part
 415 of the overtopping waves. In a 3D representation, the continuity of the wall would enhance the wave
 416 run-up, increasing the maximum pressures acting on the upper part of the crown wall and against the
 417 bullnose (see the values of p_{max} of C and N in Table 5 related to the closed configuration). Therefore,
 418 the monitored p_{max} are expected to fall in the middle of these two cases, i.e. open and closed reservoirs.
 419 However, the numerical values of p_{250} , which are more representative than p_{max} for the purposed of a
 420 fatigue analysis, show a good agreement with the field measurements, while they result in a cautious
 421 estimation along the OBREC profile even if the solver does not take into account of the
 422 compressibility of the fluids.

423 The numerical values of p_{max} and p_{250} on the ramp are about 4 times higher than the monitored ones
 424 (Table 5). These differences could be related to the following main simplifications:

- 425 - the wave spectra were sampled by the DWSD buoy with a frequency of every 17 to 60 min
 426 and, therefore, it is impossible to perfectly replicate the wave profile by the numerical model;
 427 - the offshore slope of the external layer is represented as a straight line. In the reality, the
 428 random placement of the blocs created a curved edge (see Fig. 1), which acts like a berm,
 429 anticipating the local breaking process;
 430 - the ramp is numerically represented as completely in-built on the breakwater, while in reality
 431 it is placed on the Antifers blocks (Fig. 13), leading to significant change in the representation
 432 of the flow under the plate and of the resulting lift force on the device;
 433 - numerical simulations are observed to generate lower level of run-down and more noisy
 434 pressure signal, with higher spikes compared to the physical tests, according to a previous
 435 study on the Seawave Slot-cone Generator [5];
 436 - the effect of air, not modeled in the numerical model, have some influence on the correct
 437 representation of impact waves series, as also found by Di Lauro et al. [15]. In particular, the
 438 oscillatory nature of the pressure signals is influenced by the compression of the air-pocked
 439 trapped between the wave and the parapet, leading to a general overestimation in numerical
 440 model.

441

442 **Table 5. Maximum and statistical pressure values p_{max} and p_{250} in kPa, measured during the monitoring**
 443 **compared with the numerical results related to both the open and closed OBREC configurations.**

Probes	F	G	H	A	B	C	N
p_{max} [kPa]							
Prototype	9.0	8.8	8.8	31.8	37.9	45.6	38.4
Open Conf.	20.1	21.2	29.5	71.5	47.2	30.5	36.2
Closed Conf.	25.6	27.1	30.8	78.1	85.6	70.7	62.7
p_{250} [kPa]							
Prototype	5.0	4.8	4.8	17.9	21.3	25.6	21.7
Open Conf.	19.6	20.7	22.7	35.2	28.7	27.8	32.2
Closed Conf.	24.4	25.5	28.9	37.3	39.6	38.9	38.6

444



445

446

447

Fig. 13. Construction of the bottom slab and positioning of the prefabricated ramps.

448

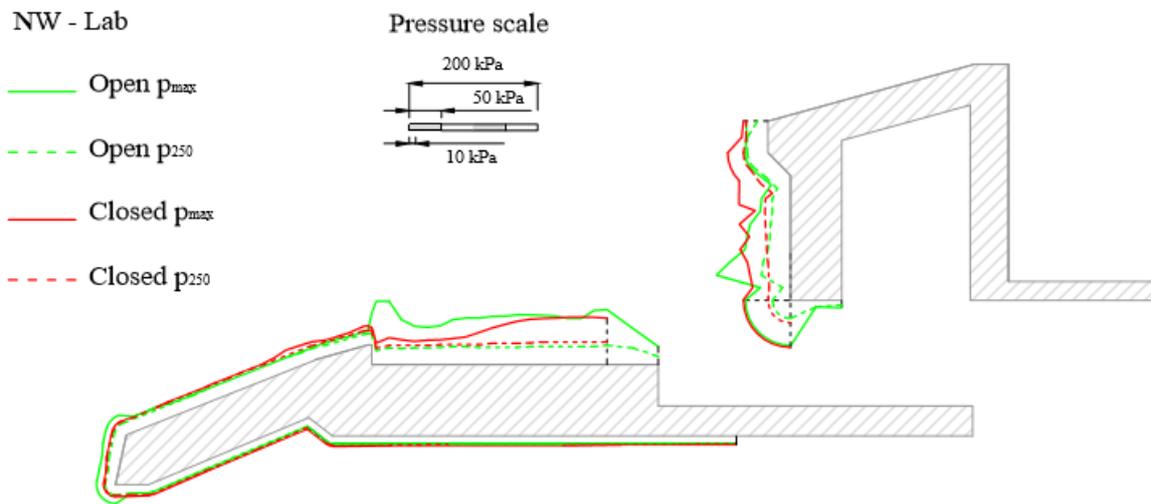
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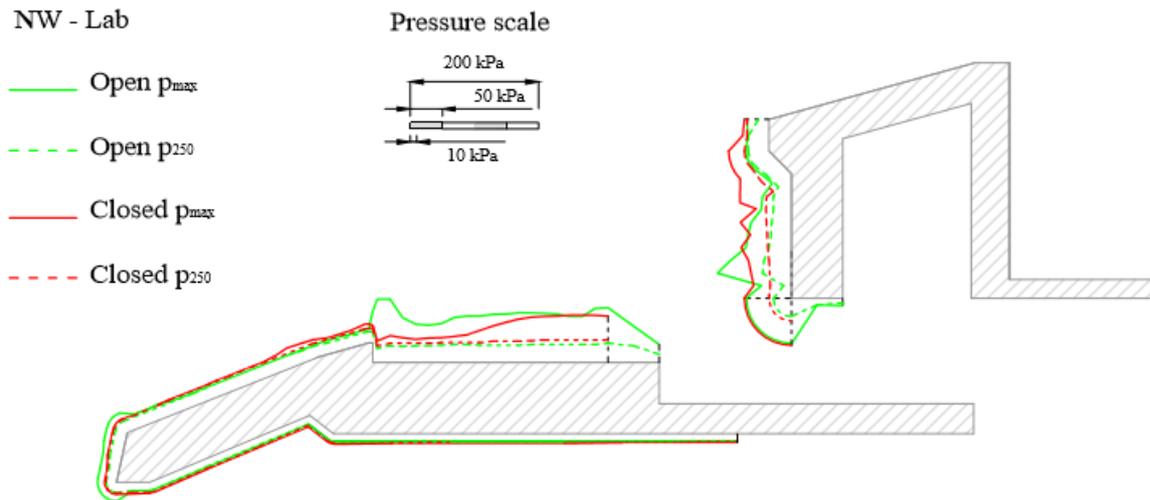
451

The numerical model allows also a direct comparison between i) the NW-LAB and the RS-LAB configurations, and ii) the closed and the open cases. Fig. 14 and Fig. 15 show qualitatively the pressures, i.e. p_{max} and p_{250} , acting along the open and closed cross sections during the storm S1, for the NW-LAB and RS-LAB cases, respectively. Main results can be summarised as follow:

- 452 - p_{max} and p_{250} shown similar qualitative distribution, especially for uplift and loading on the
- 453 sloping ramp;
- 454 - NW-LAB results to be more stressed with respect to the RS-LAB configuration, because of
- 455 the lower ramp that leads to a greater overtopping and less wave protection;
- 456 - the closed configuration provide higher pressures in the upper part of the crown wall;
- 457 - in the open case, the presence of the pipe, that limits the run-up along the crown wall, make
- 458 the lower part and the reservoir more stressed;
- 459 - the pressure distribution along the ramp and the bottom edge of the device does not show a
- 460 strong impulsive component as for the crown wall and the reservoir. This observation is
- 461 supported also by the similarity between the maximum and the statistical values along these
- 462 two parts of the structure;
- 463 - for the uplift pressures, the presence of contiguous porous layers, characterized by smaller
- 464 diameters, going towards the inshore edge of the breakwater, damps the loads.
- 465



466 **Fig. 14. Qualitative representation of the maximum and statistical pressures acting along the OBREC**
 467 **profile, during the numerically simulated storm event A, for NW-LAB configuration (open and closed**
 468 **cases).**
 469
 470



471
 472 **Fig. 15. Qualitative representation of the maximum and statistical pressures acting along the OBREC**
 473 **profile, during the numerically simulated storm event A, for RS-LAB configuration (open and closed**
 474 **cases).**
 475

476 **4.2 Hydraulic and structural response under the typical wave climate**

477 The hydraulic and structural performance of the OBREC under the typical wave climate are analysed
 478 to improve the OBREC design. These 2 technical aspects are fundamental in terms of energy
 479 production, but also of structural reliability. Indeed, the tested wave conditions were selected
 480 considering that the ratio H_s/R_c had to be bigger than 1, i.e. sea state 3-8 for the NW-LAB and 4-8
 481 for the RS-LAB configurations (Table 1). The purpose is to obtain interesting information in terms
 482 of overtopping and loads.

483 Table 5 reports the discharge rates flowing inside the reservoir Q_{in} , through the Q_{pipe} and at the rear
 484 side of the structure Q_{rear} . Furthermore, the values of K_r and $T_{\%full}$, which represent the wave
 485 reflection coefficient and the percentage of time in which the pipe works in saturated condition, are
 486 presented. The values of Q_{in} , Q_{pipe} and $T_{\%full}$ are strictly related to each other. By analysing the general
 487 trend, all these values increase with the increasing of H_s (see Table 6 and Fig. 16). Indeed, for the
 488 highest waves the pipe lose its capacity of absorbing all the water that goes inside the reservoir.
 489 However, the analysis of the values of $T_{\%full}$ shows that the average percentage of filling of the pipe
 490 section, for the most severe wave condition here analysed, is the 62% and 48% for the NW-LAB and
 491 the RS-LAB case, respectively.

492 By comparing the 2 configurations the NW-LAB case shows, for all the sea states, higher values of
 493 Q_{in} , Q_{pipe} and $T_{\%full}$ due to then lower ramp, which allows more overtopping with respect to the RS-
 494 LAB. However, further considerations are needed to define which is the most promising
 495 configuration. Indeed, the characteristics of the NW-LAB cross section (see Fig. 2) minimize the
 496 distance between the still water level and the theoretical position of the rotor of the turbine the
 497 theoretical position of the rotor of the turbine, at the expense of the theoretical energy production.

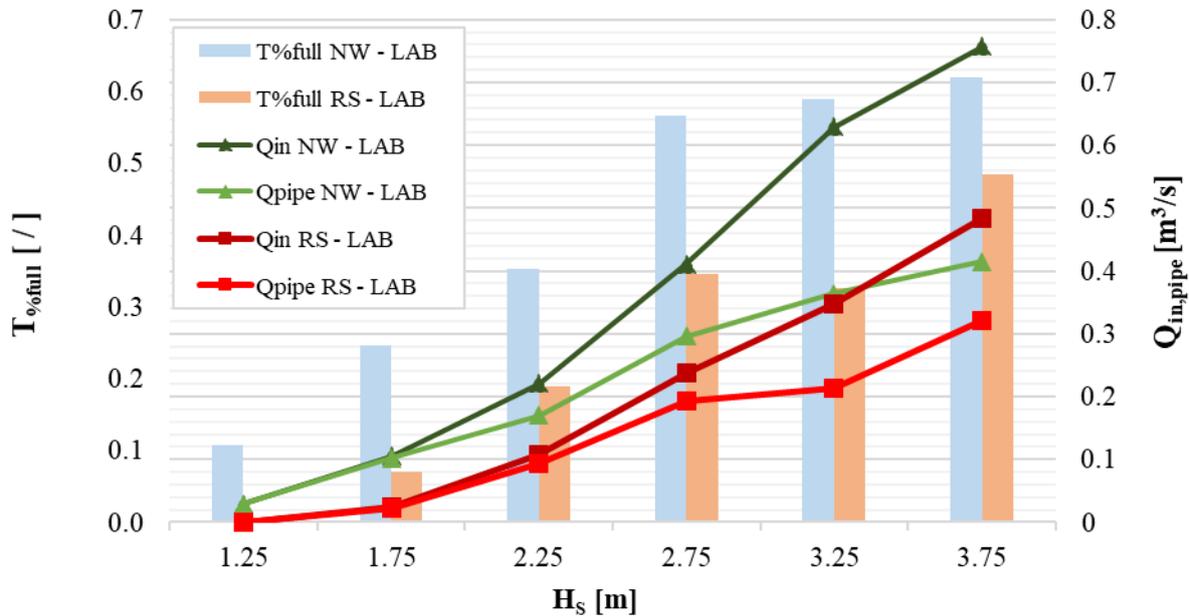
498 The values obtained of Q_{rear} in Table 6 respect the limits for wave overtopping for structural design
 499 of breakwaters, seawalls, dikes and dams suggested by the EurOtop [18]. Indeed, the presence of the
 500 parapet, on top of the crown wall, allows the minimization of the overtopping rate at the rear side of
 501 the structure [38[39], as already demonstrated during the preliminary experimental and numerical
 502 investigations [32[41].

503 The basic principle of the wave energy absorption, on which the OBREC has been designed, avoids
 504 the increasing of the values of K_r , with respect to traditional rubble mound breakwater, as already
 505 experience during the laboratory experiments performed by Vicinanza et al. [41].
 506

507 **Table 6. Discharge rate inside the pipe Q_{pipe} and at the rear side of the structure Q_{rear} , percentage of the**
 508 **time in which the pipe works as a full section and the reflection coefficient K_r for both the configurations.**

H_s [m]	NW_LAB					RS_LAB				
	Q_{in} [m ³ /s]	Q_{pipe} [m ³ /s]	$T_{\%full}$ [/]	Q_{rear} [m ³ /s]	K_r [/]	Q_{in} [m ³ /s]	Q_{pipe} [m ³ /s]	$T_{\%full}$ [/]	Q_{rear} [m ³ /s]	K_r [/]
1.25	0.029	0.029	0.11	0	0.46	/	/	/	/	/
1.75	0.105	0.102	0.25	0	0.50	0.024	0.0226	0.07	0	0.51
2.25	0.221	0.170	0.35	0	0.46	0.107	0.09	0.19	0	0.46
2.75	0.411	0.296	0.57	0	0.47	0.238	0.193	0.35	0	0.50
3.25	0.629	0.364	0.59	0.0047	0.45	0.346	0.214	0.33	0.02	0.47
3.75	0.757	0.415	0.62	0.0053	0.48	0.484	0.324	0.48	0.0188	0.54

509



510

511

512 **Fig. 16. Comparison between the discharge rate inside the reservoir Q_{in} and inside Q_{pipe} for both the**
 513 **configuration.**

514

515 Fig. 17 shows the pressures acting on the OBREC cross sections. For both the configurations, the
 516 loads do increase with increasing H_s . The crown walls and the reservoirs are affected by impulsive
 517 impacts, while the ramp and the bottom edge of the device to the quasi static ones. For both the
 518 NW-LAB and the RS-LAB configurations, the most stressed part of the structure is the crown wall.
 519 Specifically, for the former the impacts are concentrated just above the pipe, while for the latter just
 520 below the bullnose. The higher ramp and the smaller reservoir generally increase the magnitude of
 521 the pressures against the crown wall, leading to a more stressed bullnose with respect to the
 522 NW-LAB cross section.

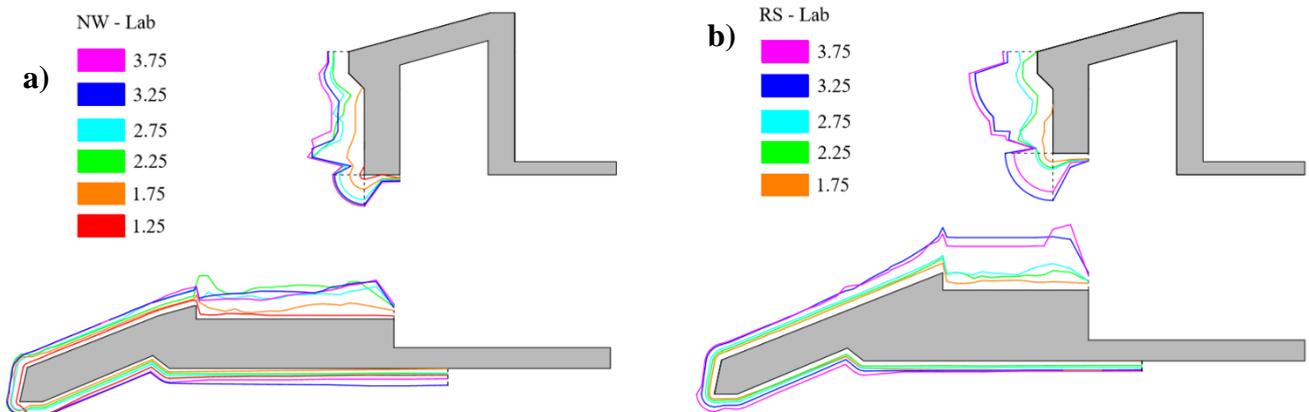
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523

524

An integrated perspective shows a strong connection between the hydraulic and the structural
 performance. The highest waves overtop the ramp with more energy, which is partially dissipated in
 the direct impact against the crown wall. This dynamic does not allow a significant increase of Q_{pipe}

525 for the most severe sea states. The smallest waves, characterized by less energy, tend to follow the
 526 OBREC geometry flowing through the pipe without any loss.
 527



528
 529 **Fig. 17. Maximum values of pressures along the OBREC device: a) NW_LAB and b) RS_LAB**
 530 **configurations.**
 531

532 4.3 Scale effects on wave loads

533 During the small-scale OBREC experiments [12], the ramp was characterized by a quasi-static
 534 loading time history, while the crown wall was affected by signals showing evident rapid variations
 535 in time, with a high force peaks typically described as impact wave loads [12]. For the full-scale
 536 prototype, this is not completely true, at least for the ramp. In this latter case, the monitored pressure
 537 signals can be classified as slightly breaking and broken wave loads, due to the presence of the berm
 538 that anticipates the breaking process. The differences between the small and full-scale dynamics point
 539 out the strong non-linear interaction between the fluid and the ramp, in contrast with the quasi-static
 540 conditions measured during the laboratory campaigns [12,[41]. In particular, in correspondence of
 541 drastic changes of roughness and permeability, the air-water interface give some numerical
 542 disturbance leading to misleading results.

543 Furthermore, the qualitative analysis of the field measurements highlights that all the recorded signals
 544 are characterized by a strong noise including a significant number of negative values. The latter ones
 545 are due to the high-aeration occurring during the wave-structure interactions, which produces sub-
 546 atmospheric pressures after the impact takes place, as already observed in large-scale experiments
 547 [30].

548 However, the general hydrodynamic at prototype during extreme wave conditions was already well
 549 represented during the small-scale experiments [41]. In the laboratory, the creation of a “water bag”
 550 that attenuates the loads both on the reservoir and on the lower part of the crown wall was observed.
 551 At prototype scale, this process is confirmed by the pipe saturation occurred during the extreme
 552 events. Furthermore, the laboratory, the field and the numerical results highlight that the main
 553 difference between the RS-LAB and NW-LAB configurations is the point where the wave impact
 554 occurs. In case of the higher ramp the highest waves are conveyed directly towards the upper part of
 555 the crown wall, while in the lower one, the wave falls inside the reservoir and then run-up along the
 556 crown wall.
 557

558 **5. Theoretical power production**

559 The purpose of this Section is to provide a preliminary assessment of the theoretical power flux
560 available at prototype scale. The optimization of the cross section according to the results is also
561 discussed.

562

563 **5.1 Methodology**

564 The assessment is based on hydraulic considerations disregarding the energy conversion method and
565 the power losses. The theoretical power production is obtained by combining the overtopping rates
566 flowing inside the shunt tank Q_{pipe} and the total hydraulic head h_{tot} , for each tested wave condition.

567 The hydrodynamics observed at the OBREC prototype, supported by the numerical modelling,
568 showed that the overtopping waves are firstly collected inside the reservoir and then conveyed, by
569 means of the pipe, in the machine room. Here, a shunt tank stabilizes the hydraulic head available for
570 the hypothetical PTO system (Fig. 18a). The size of the tank installed in the OBREC prototype, i.e.
571 0.45 m (width) x 0.45 m (height) x 1 m (longitudinal extension), is showed in Fig. 18b. The most
572 important parameter is the diameter of the hole on the bottom of the shunt tank, acting as intake for
573 the turbine. As aforementioned, due to the limitation of the 2D model, the pipe toward the crown wall
574 has been modeled as a rectangular hole covering the whole width of the domain. Therefore, the
575 hydrodynamics inside the reservoir and the power efficiency parameters are computed by means of
576 another specifically-designed numerical model, called OBRECsim [8]. OBRECsim solves the full,
577 dynamic, 1-D Saint Venant equation using an implicit, finite difference method. The code is based
578 on the continuity equation (2):

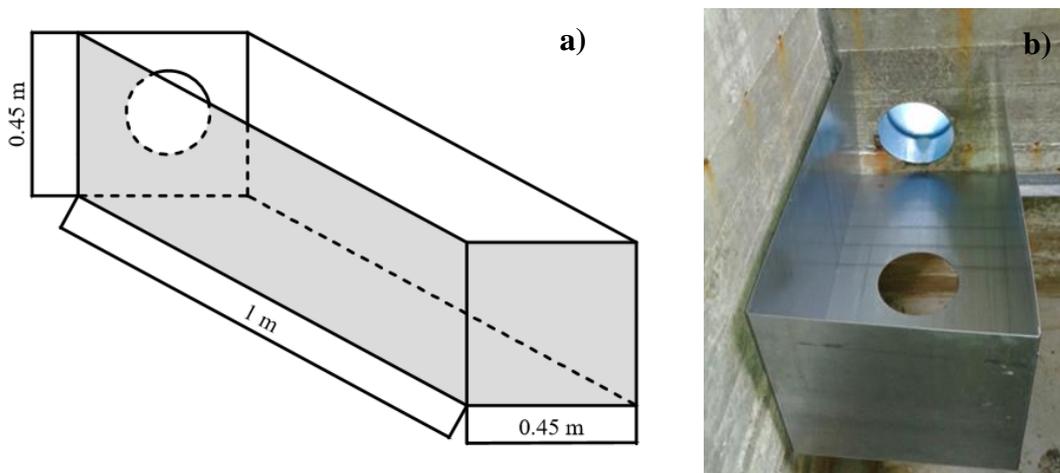
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$$Q^*_{reservoir} = Q^*_{in} - Q^*_{rear} - Q^*_{overflow} \quad (2)$$

580

581 where: $Q^*_{reservoir}$ is the flow through the turbines; Q^*_{in} is the total overtopping flow rate, derived from
582 the overtopping discharge formula fitted by Iuppa et al. [24] on the OBREC small-scale laboratory
583 tests; Q^*_{rear} is the overtopping flow rate at the rear side of the structure; $Q^*_{overflow}$ is the outgoing
584 reflected flow when the reservoir is saturated.

585



586

587

588

Fig. 18. Shunt tank: a) CAD scheme; b) installed in the prototype of the port of Naples.

589 The total hydraulic head h_{tot} is composed by 2 contributions (Eq. (3)), i.e. the difference between the
 590 still water level and the bottom edge of the reservoir h and the average hydraulic head dh established
 591 in the shunt tank (Fig. 18), dependent on Q_{pipe} :
 592

$$h_{tot} = h + dh \quad (3)$$

593
 594 where h is equal to 0.7 m and 1.25 m for the NW-LAB and RS-LAB sections, respectively. It is
 595 worthy to highlight that the value of dh represents only the static contribution, even if the observations
 596 pointed out the importance to consider also the kinetic one. Therefore, it represents a cautious
 597 assumption for the evaluation here performed.

598 The theoretical input power P_{in} to a generic Power Take Off (PTO) system, by neglecting the power
 599 losses, is given by:
 600

$$P_{in,i} = \rho \cdot g \cdot Q_{pipe,i} \cdot h_{tot,i} \quad (4)$$

601
 602 where ρ is the fluid density, g is the gravitational acceleration, h_{tot} is the total hydraulic head, Q_{pipe} is
 603 the overtopping discharge rate flowing through the pipe, and the subscript i is the sea state. The
 604 contribution given by each wave condition $E_{in,ws}$ (Table 7) is derived by multiplying the value
 605 obtained for a specific i for the frequency of occurrence Fr associated to i (in terms of hours/year).
 606 The sum of the values of $E_{in,ws}$ gives the theoretical available input power for the PTO system of each
 607 OBREC configuration during a year $E_{in,year}$.
 608

609 5.2 Results

610 Table 6 shows the values of dh , P_{in} , $E_{in,ws}$ and the resultant $E_{in,year}$ for each OBREC configuration,
 611 according to the results obtained by the numerical modelling. Most of the sea states give values of dh
 612 (Table 7) greater than the height of the shunt tank, i.e. 0.45 m, and therefore the values of P_{in} is
 613 computed accounting for this upper limitation of dh . The shunt tank should be resized to allow a
 614 higher available hydraulic head and thus energy production.

615 For waves with $H_s < 1.25$ m, the NW-LAB configuration produces higher values of Q_{pipe} than the
 616 RS-LAB configuration (Table 6). The values of P_{in} for $H_s = 1.25$ m are similar between the two
 617 configurations. For higher wave heights, P_{in} grows more and more for the RS-LAB cross section,
 618 highlighting that the smaller values of Q_{pipe} are effectively compensated by the higher values of h_{tot} ,
 619 essentially due to the higher value of the constant contribution h .

620 It is worthy to remark that these results were derived in case of one hole and one corresponding 0.25
 621 diameter pipe. The optimization of the design and location of the holes for the pipes requires a fully
 622 3D modelling that is out of the scope of this contribution.
 623

624 **Table 7. Hydraulic head established in the shunt tank dh and power produced by the turbine P_{in} , for**
 625 **each sea state characterized by a certain frequency Fr and available theoretical power P_{year} . Values**
 626 **derived in case of one 0.25 m diameter hole.**

Sea states			NW-LAB			RS-LAB		
H_s	Fr	P_{year}	dh	P_{in}	$E_{in,ws}$	dh	P_{in}	$E_{in,ws}$
[m]	[/]	[kW/m]	[m]	[kW]	[kWh]	[m]	[kW]	[kWh]
0.25	0.1498	0.01	0.00	0.00	0.02	0.00	0.00	0.04

0.75	0.5912	0.60	0.01	0.16	842.02	0.00	0.08	427.74
1.25	0.2083	0.75	0.10	1.03	1886.72	0.05	1.00	1845.69
1.75	0.0409	0.34	0.20	2.09	751.62	0.16	2.74	983.72
2.25	0.0083	0.13	0.27	2.66	203.56	0.30	4.26	325.79
2.75	0.0012	0.03	0.31	2.93	25.63	0.40	5.13	44.96
3.25	0.0003	0.01	0.36	3.16	5.53	0.51	5.96	10.44
3.75	0.0001	0.00	0.37	3.12	2.19	0.54	5.82	4.08
Total	1.00	1.87			$E_{in,year}$			$E_{in,year}$
					3717.27			3642.46

627

628

629 A coherent comparison between the two configurations can be performed by considering the values
630 of $E_{in,year}$, which results to be 3717 kWh and 3642 kWh for the NW-LAB and RS-LAB cases,
631 respectively. Being the total available wave energy of 4620 kWh/m, the Capture Width Ratio (CWR)
632 is 12.9% and 12.6% for the NW-LAB and the RS-LAB, respectively. CWR is the percentage of the
633 available wave power that is captured by the device per metre of device width. The lower ramp of
634 NW-LAB captures a wider set of wave conditions, maximizing the OBREC operational window $T_{o.w.}$
635 with respect to the typical wave climate and P_{year} given the higher frequency of occurrence Fr of the
636 less energetic sea states. The NW-LAB is characterised by an operational window $T_{o.w.} (\sum Fr) > 32\%$

637

638 The parameters $E_{in,year}$ and $T_{o.w.}$ can be assumed as the reference parameters for the design optimisation
639 of the ramp. However, appropriate site-specific threshold values for these parameters should be
640 selected to derive the optimal value of R_c , while considering the design procedure and the technical
641 considerations here reported of general validity.

642

643 The theoretical results of the power production have finally to cope with the technical characteristics
644 of the selected PTO. The very low head turbines tested for OBREC can exploit minimum head
645 differences $\geq 1.5\text{m}$ [44], and therefore the NW-LAB configuration would not produce energy, while
646 the RS-LAB configuration would produce energy only for the highest waves (see Table 8). Therefore,
647 the final design of the device has to account for the limitations of the PTO system, which in this
648 specific case does not allow the selection of the NW-LAB configuration. These considerations point
649 out, once more, that the very low head turbine is still a challenge to be overcome (Contestabile et al.,
650 2020), and that improved technological solutions have to be developed to make feasible the
651 systematic production of marine renewable energy.

652

651 **Table 8. Yearly operating time, in terms of percentage per year, and total average hydraulic heads**
652 **$dh_{tot,real}$ computed for each sea state.**

#	H_{m0} [m]	NW-LAB		RS-LAB	
		$T_{o.w.}$ [%]	$h_{tot,real}$ [m]	$T_{o.w.}$ [%]	$h_{tot,real}$ [m]
1	0.25	0.00	0.70	0.00	1.25
2	0.75	12.92	0.71	1.71	1.25
3	1.25	14.63	0.80	5.50	1.30
4	1.75	3.79	0.90	2.33	1.41
5	2.25	0.85	0.97	0.67	1.55
6	2.75	0.10	1.01	0.09	1.65
7	3.25	0.02	1.06	0.02	1.76
8	3.75	0.01	1.07	0.01	1.79

Total	32.32	/	10.34	/
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653

654 **5.3 Cross section optimization for energy production**

655 The results obtained from the analysis reported in Section 0 could be useful to give some indications
656 about the optimal design of the OBREC device, in order to maximize the energy production.

657 According to its principle of operation, the main design parameter is the height of the ramp R_c , which
658 determines the run-up process, and therefore the overtopping rate. Its definition can be independent
659 from the reservoir width, which has to primarily satisfy the site-specific constraints related to i) the
660 available space for the overall structure, and consequently ii) the construction and the installation
661 costs. An undersized reservoir, with respect to the highest overtopping volumes, allows the
662 dissipation of part of the wave energy during the wave-crown wall impact, protecting the machine
663 room that contains sensitive instruments. Therefore, the selection of the reservoir width has to be
664 subordinated to the value of R_c and to the related estimate of the overtopping volumes.

665 For the specific case of the prototype installation, the lower ramp maximizes the theoretical power
666 available to the system, being capable of exploiting a wider range of wave conditions with respect to
667 the higher ramp. However, the selection of the proper value of R_c , during the design procedure, cannot
668 be independent from the minimum hydraulic head needed for the selected PTO system to produce
669 energy.

670

671 **6. Conclusions**

672 The general aim of this contribution is to carry-out an integrated assessment of the OBREC prototype
673 installed in the port of Naples, in terms of its hydraulic and structural performance. The OBREC
674 consists of a ramp followed by a unique reservoir closed by a crown wall, provided with a bullnose
675 on top of it. The reservoir is located above the still water level and it is linked with the machine room
676 by means of a pipe passing through the crown wall. Specifically, the prototype is composed by two
677 cross sections, which differ mainly for the height of the ramp. The lower one (NW-LAB) is aimed to
678 capture the most frequent waves, while the higher (RS-LAB) the highest ones.

679 The OBREC performance is analysed by combining the field measurements, gathered at the
680 installation site, and the numerical modelling performed in the openFOAM environment. The
681 calibration of the numerical model is based on the pressures monitored during a real storm event. The
682 OBREC cross sections were tested in 2D conditions, considering that the principal direction of the
683 wave propagation, in the Gulf of Naples, is orthogonal to the device. Under extreme events, two
684 cross section profiles were modelled for both NW-LAB and RS-LAB: an “open” and a “closed”
685 profile, which include (real case) or not the presence of the pipe, respectively, to represent the case
686 of pipe saturation observed.

687 The numerical model tends to generally overestimate the field data, except for the upper part of the
688 crown wall of the open configuration, probably due to the presence of the rectangular section covering
689 the width. Therefore, the monitored maximum pressures fall in the middle of these two cases, i.e.
690 open and closed reservoirs. The numerical values of p_{250} (non-exceedance level of 99.7%) show a
691 good agreement with the field measurements at the wall, while strongly overestimate the stresses
692 along the ramp. This could be due to the curved offshore profile of the real breakwater that slightly
693 anticipates the breaking process, which is not reproduced in the virtual domain. Furthermore, some
694 numerical mesh limitations lead to a different air-water interface under the ramp element.

695 The numerical model is then adopted to test the prototype under the typical wave climate of Naples.
696 The results obtained show that OBREC is capable of minimizing the overtopping discharge at the
697 rear side of the structure, without increasing the wave reflection.

698 The theoretical power available to the PTO system in a year, i.e. $E_{in,year}$, is derived by combining the
699 overtopping rates flowing inside the shunt tank Q_{pipe} and the total hydraulic head h_{tot} , for each tested
700 wave condition, accounting for the frequency of occurrence Fr associated to each sea state. The values
701 of $E_{in,year}$, in case of only one 0.25 m diameter hole, are equal to 3717.27 kWh and 3642.46 kWh for
702 the NW-LAB and RS-LAB, corresponding to CWR of 12.9% and 12.6%, respectively.

703 Indeed, the lower ramp captures a wider set of wave conditions characterized by an operational
704 window $T_{o.w.} > 32\%$, while for the RS-LAB the values of $T_{o.w.}$ resulted to be $> 10\%$. These two
705 parameters are crucial for the design optimisation of the ramp, even if site-specific threshold values
706 should be selected. The hydraulic optimization has finally to be combined with the technical
707 characteristics, and sometimes the limitations, of the available PTO systems. For the case of OBREC,
708 the selection of commercial low-head turbines implies that the NW-LAB configuration cannot be
709 selected because of the insufficient hydraulic head for turbine operation.
710

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718 **References**

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