Proceedings of the 13th European Wave and Tidal Energy Conference 1-6 Sept 2019, Naples, Italy

Performances of a breakwater for wave energy conversion

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Abstract—**A numerical model for the optimization of the performances of an innovative overtopping breakwater for wave energy conversion is proposed. The performances of the device have recently been tested during three small scale tests in 2012, 2014, and 2015. The last campaign was carried out at the hydraulic laboratory of the University of Catania with the aim of identifying the probability distribution of overtopping volumes into the reservoir. The numerical model proposed is based on this validated stochastic description of the overtopping phenomenon and it is able to simulate the behaviour of the device operating under any assigned sequence of sea states, thus allowing to easily obtain results that would otherwise require to carry out time and cost expensive physical model experiments. The model was applied to optimize a new device embedded in the new breakwater of the port of Bosa Marina (Sardinia, Italy) and the analysis carried out in such a location allowed to identify the most relevant geometrical parameters which could influence the device performances.**

Keywords—**Wave Energy Converters, Wave overtopping, integrated device.**

ID 1309

Conference track WWD

This work has been partly funded by the EU funded project HYDRALAB PLUS (proposal number 654110), by the project “NEWS - Nearshore hazard monitoring and EarlyWarning System" (code C1-3.2-60) in the framework of the programme INTERREG V-A Italia Malta 2014-2020, and by University of Catania funded project “Interazione ondecorrenti nella regione costiera (INOCS)".

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I. Introduction

THE growing energy consumption and the expected reduction of fossil fuels have long represented a threat to the industrialized countries’ way of living, thus involving the scientific community in the challenge of providing new environment-friendly sources of energy. The most common renewable energies, such as solar and wind energy, have indeed met a remarkable technological improvement, but there are still unexploited natural processes which could be efficiently used to power our cities and our homes; among these, ocean tides and waves represent an exceptionally vast and untapped source of energy [1].

Since 1973 the research on ocean energy has become more and more intense, leading to the patenting of more than 1000 Wave Energy Converters (WECs) [2], but so far, their costs have proven too high to allow their widespread commercialization [3]. To break this economical barrier several researchers are developing devices integrated in traditional breakwaters [4][5][6][7][8][9][10], this allows to exploit advantages [4] such as: i) easier access to the device, which simplifies its construction, maintenance and connection to electrical grid; ii) similarity with the traditional rubble mound breakwaters, which simplifies both design and construction activities; iii) reduced sea waves intensity compared to offshore sites, which diminishes the mechanical stress undergone by the device (as well as the amount of energy extractable).

Among such a class of devices, the Overtopping BReakwater for Energy Conversion (OBREC) (Fig. 1) has been recently conceived to extract energy from the overtopping discharges [4]. The device consists of a concrete frontal ramp that reduces the friction of the breakwater and facilitate the overtopping of the waves into a reservoir above the sea level; the water flows back from this reservoir to the sea through one or more low-head turbines.

The evaluation of the feasibility of an investment in a WEC requires a method to estimate the amount of energy extractable by the device in a given site, as well as a procedure to predict how the variation of the parameters of the device will affect its productivity, thus allowing to identify its optimal configuration. These demands can be satisfied by carrying out physical model experiments or

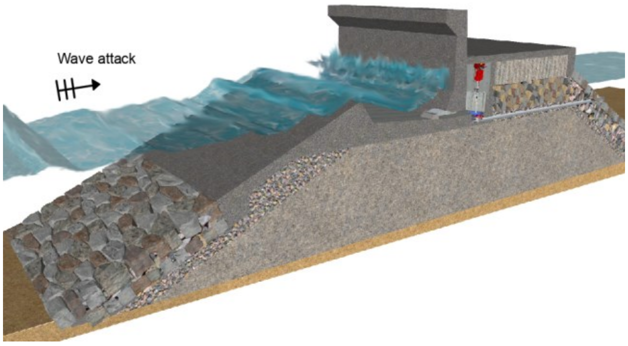

Fig. 1. 3D sketch of the OBREC.

time expensive numerical simulation based, for example, on the Reynolds Averaged Navier-Stokes Equation [11] [12].

To overcome the cost and time limits of the above-mentioned methods a numerical model based on a stochastic approach of the overtopping discharges is proposed. More in details, the performances of the OBREC was recently tested in the Hydraulic Laboratory of the University of Catania, with the purpose of identifying the probability distribution of the overtopping volumes for an OBREC device. The method to obtain the probability distribution is presented and discussed by Iuppa et al. [13] and has represented the basis for the implementation of a numerical model able to simulate the behaviour of the OBREC operating under any assigned sequence of sea states.

The model was applied to optimize a device to be embedded in the new breakwater of the port of Bosa Marina (Sardinia, Italy) and the analysis carried out in such a location allow to identify the most relevant parameters which could influence the device performances.

The paper is organized as follows: the second section describes the methodology used by the numerical model to predict the overtopping volumes which enter the OBREC reservoir and to estimate the device output electrical power, the third section describes the site of interest and the data relevant to its wave climate, the fourth section describes the main geometrical parameters of the device and their effect on its performances, the fifth section describes the results of the model applied to the site. The last section draws some conclusive remarks.

II. Methodology

The proposed model is based on the stochastic description of the overtopping phenomenon through the relations suggested by Iuppa et al. [13] for the OBREC device. Fig. 2 shows the main geometrical characteristics of an OBREC device. More in details: d is the water depth at the toe of the breakwater, d_w is the total height of the impermeable ramp; d_d is the height of the impermeable ramp below the still water level, R_r is the height of the ramp above the water level, h_s is the height of the reservoir bottom above the still water level, L is the cross-shore width of the reservoir, α is the ramp angle to the horizontal, and D is the exhaust pipe diameter.

The wave-by-wave overtopping volumes distribution are well fitted by a two-parameter Weibull distribution function [14][15] [16] which can be written as follows

$$P_{V_{in}} = \mathrm{P}[V_{in} > V] = \exp\left[-\left(\frac{V}{a}\right)^{b}\right] \qquad (1)$$

where $P_{V_{in}}$ is the exceedance probability, V_{in} is the generic individual overtopping volume, V is the reference value, a is the scale factor and b is the shape parameter. The

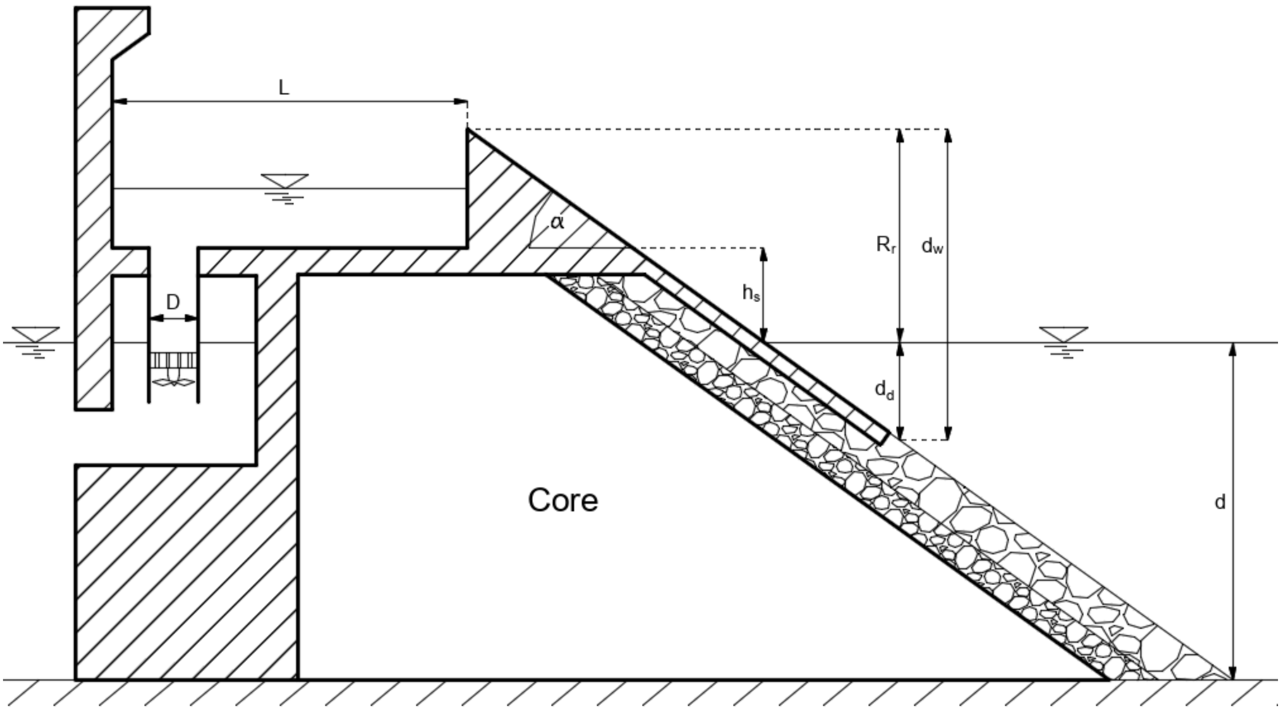

Fig. 2. Sketch of an OBREC cross-section.

parameter a can be estimated through the following relation

$$a = \frac{1}{\Gamma\left(1+\frac{1}{b}\right)} \frac{q_{in} T_m}{P_{ow}} \tag{2}$$

where Γ is the gamma function, q_{in} is the mean average overtopping discharge, T_m is the mean wave period and P_{ow} is the overtopping probability.

The non-dimensional mean overtopping discharge can be defined as

$$q_{in}^* = \frac{q_{in}}{\sqrt{g H_{m0}^3}} \tag{3}$$

where g is the gravity acceleration and H_{m0} is the incident significant wave height at the toe of the structure; q_{in}^* can be calculated as

$$q_{in}^* = \gamma_V \; c_1 \; \exp\left[-\left(c_2 \frac{R_r^*}{\gamma_f}\right)^{c_3}\right] \tag{4}$$

where γ_V is a calibration coefficient; R_r^* is the relative crest free-board of the front reservoir, equal to the ratio between the crest free-board of the front reservoir R_r and the incident significant wave height H_{m0}; c_1, c_2 and c_3 are coefficients calculated as suggested by van der Meer and Bruce [17], whereas γ_f is a coefficient which takes into account the effect of the friction of the frontal slope. The coefficients γ_V and γ_f can be calculated as proposed by Iuppa et al. [13] and Iuppa et al. [18], respectively,

$$\gamma_V = \begin{cases} 0.378\, R_r^{*2} - 0.88\, R_r^* + 1.65 & if\;\; R_r^* \geq 0.8 \\ 1 & if\;\; R_r^* < 0.8 \end{cases} \tag{5}$$

$$\gamma_f = \begin{cases} \tanh\left[7.47\left(\frac{d_d}{L_{m-1,0}}\right)^{0.42}\right] & if\;\; \frac{d_d}{L_{m-1,0}} \geq 0.006 \\ 1 & if\;\; \frac{d_d}{L_{m-1,0}} < 0.006 \end{cases} \tag{6}$$

where d_d is the length of the submerged ramp and $L_{m-1,0}$ the deep-water wavelength related to the spectral incident energy wave period $T_{m-1,0}$.

As suggested by van der Meer and Bruce [17], the coefficient c_1 can be calculated as

$$c_1 = \begin{cases} 0.09 - 0.01\;\; (2 - cot\alpha)^{2.1} & if\; cot\alpha < 2 \\ 0.09 & if\; cot\alpha \geq 2 \end{cases} \tag{7}$$

where α is the ramp angle to the horizontal plane.

The coefficient c_2 can be calculated as

$$c_2 = \begin{cases} 1.5 + 0.42\;\; (2 - cot\alpha)^{1.5} & if\; cot\alpha < 2 \\ 1.50 & if\; cot\alpha \geq 2 \end{cases} \tag{8}$$

with a maximum of $c_2 = 2.35$.

The coefficient c_3 is always 1.3.

The probability of overtopping can be evaluated as

$$P_{ow} = \exp[-(c_{obrec} \times R_r^*)^2] \tag{9}$$

where

$$c_{obrec} = \frac{1}{0.36\, d_w^* + 0.92} \tag{10}$$

with d_w^* being the dimensionless vertical height of the sloping plate, calculated as the ratio between the vertical height of the sloping plate d_w and the sum of the crest free-board of the front reservoir R_r and the water depth at the toe of the structure h_t, that is

$$d_w^* = \frac{d_w}{h_t + R_r} \tag{11}$$

The shape factor of the Weibull distribution is calculated as

$$b = C_b[\exp(-2.0 R_r^*) + (0.56 + 0.15\;\; cot\alpha)] \tag{12}$$

where C_b is a coefficient calculated as

$$C_b = 1.16 - 0.15\;\; d_w^* \tag{13}$$

The individual volume duration T_v, necessary to obtain the individual overtopping flow rates from the individual overtopping volumes, can be calculated as

$$T_v = T_m \;\; C_{t1} \; \left(\frac{V_{in}}{V_m}\right)^{C_{t2}} \tag{14}$$

where V_m is the mean overtopping volume and C_{t1} and C_{t2} are two coefficients. The mean overtopping volume is the expected value of the Weibull distribution, hence in can be calculated as

$$V_m = a \;\; \Gamma\left(1 + \frac{1}{b}\right) \tag{15}$$

whereas the coefficients C_{t1} and C_{t2} are calculated as

$$C_{t1} = 0.3 \;\; R_r^{*\,-0.193} \tag{16}$$

and

$$C_{t2} = 0.431\, \exp(-1.17\;\; R_r^*) \tag{17}$$

For every sea state, a series of overtopping volumes into the reservoir is generated by considering a random series of probability and by using (2)-(13). Using (14), the individual volume duration associated to each overtopped volume and the associated mean discharge can be evaluated.

Once the inflow discharges for each assigned sea state are generated, the initial condition of the reservoir is set as empty, and a numerical integration of the continuity equation is carried out.

The water volume variation inside the reservoir is evaluated as:

$$\Delta V = [Q_{in}(i) - Q_{out}(i)]dt \tag{18}$$

where $Q_{in}(i)$ and $Q_{out}(i)$ are respectively the inflow and the outflow rate during the time interval i, and dt is the time step of the numerical integration.

The new volume of water inside the reservoir is calculated as:

$$V(i+1) = V(i) + \Delta V \tag{19}$$

where $V(i+1)$ and $V(i)$ are, respectively, the new and the previous volumes inside the reservoir. If the new volume exceeds the maximum volume of the reservoir, the new volume is set equal to such an upper limit:

$$V(i+1) = V_{max} \tag{20}$$

The hydraulic head on the turbines (E) is calculated differently whether the volume of water inside the reservoir is greater or lower than the volume of the pipes above the s.w.l. (V_{pipes}):

$$\begin{cases} E(i+1) = \dfrac{V(i+1)}{N\pi\left(\dfrac{D}{2}\right)^2} & if\ V(i+1) \le V_{pipes} \\ E(i+1) = \dfrac{V(i+1) - V_{pipes}}{Ll} + h_s & if\ V(i+1) > V_{pipes} \end{cases} \tag{21}$$

where N is the number of pipes, D is the pipes' diameter, L is the cross-shore width of the reservoir, l is the longitudinal length of the reservoir and h_s is the height of the bottom of the reservoir on the sea water level.

The outflow rate and the output power (P) of the turbines are calculated according to the performances curves of the turbine.

The outflow rate calculated can be used in (18) to determine the new variation of the volume of water inside the reservoir, and hence all the quantities related to the next time interval.

After the entire period of operation of the device has been scanned, the overall performances of the OBREC is described by many indices, such as the yearly average output power, the yearly average outflow rate, the average hydraulic head and the percentage of time when the reservoir is full.

III. WAVE CLIMATE DATA

The wave energy resource assessment shows that the West coast of Sardinia is one of the most energetic area in

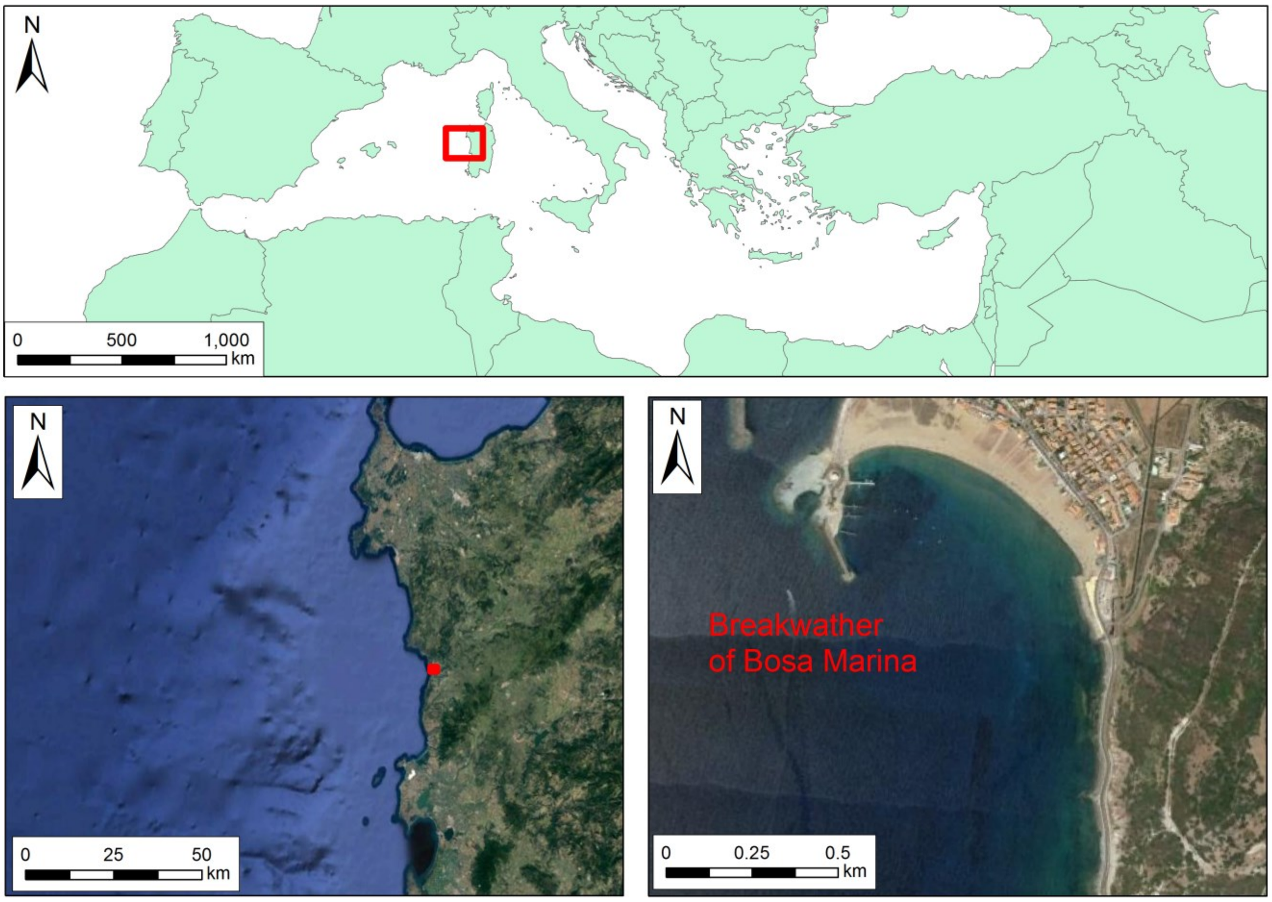

Fig. 3. Location of Bosa Marina (Sardinia, Italy).

the Mediterranean Sea [19]. The necessity to maximize OBREC's productivity, combined with the expected future extension of the breakwater of Bosa Marina (see Fig. 3), has led to choose the latter location for the application of the numerical model.

The analysis of the offshore and nearshore wave climate has been performed by means of a high-resolution coastal propagation model [20], forced with wave data from the European Centre for Medium-Range Weather Forecasts (ECMWF). The result of the analysis consists of a sequence of sea states which describes the yearly average wave climate affecting the location of the future breakwater.

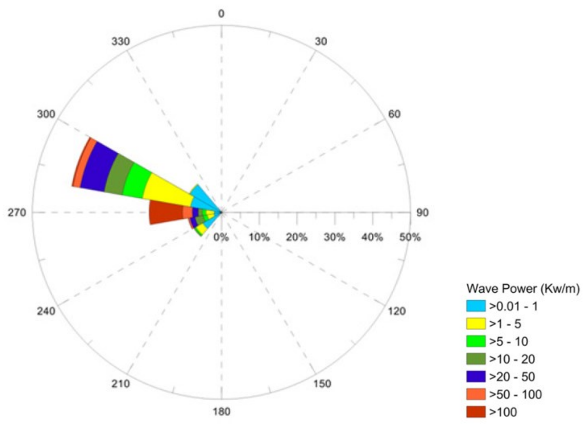

Fig. 4. Wave Power climate at Bosa Marina (Sardinia, Italy).

Fig. 4 and 5 show, respectively, a graphic representation of the characterization of the yearly average wave energy in terms of significant wave height and energy period. It can be noticed that over the year a considerable amount of energy would reach the device in the form of waves with a wave height greater than 2.5 m.

The yearly average power of the incident waves in front of the new breakwater is 5.45 kW per meter of wave width.

IV. Main Investigated Parameters

The identification of the optimal configuration of the OBREC device with regards to the electrical power production was carried out by analysing the influence of the following four geometrical parameters: crest free-board of the front reservoir (R_r); height of the bottom of the reservoir on the swl (h_s); length of the submerged ramp (d_d); number of turbines installed (N).

1) *Crest free-board of the front reservoir (R_r)*

The crest free-board of the front reservoir represents the main obstacle to the overtopping of the waves.

The increase of R_r causes both a reduction of the non-dimensional mean overtopping discharge and of the probability of overtopping. Furthermore, the value of R_r is the maximum hydraulic head on the turbines, thus implying that an increase in its value, despite causing a reduction of the overtopped volumes, could allow to extract enough energy to compensate the decrease due to the lost volumes of water.

2) *Height of the bottom of the reservoir on the sea water level (h_s)*

The overtopping volume into the reservoir starts accumulating at the height of the bottom of the reservoir,

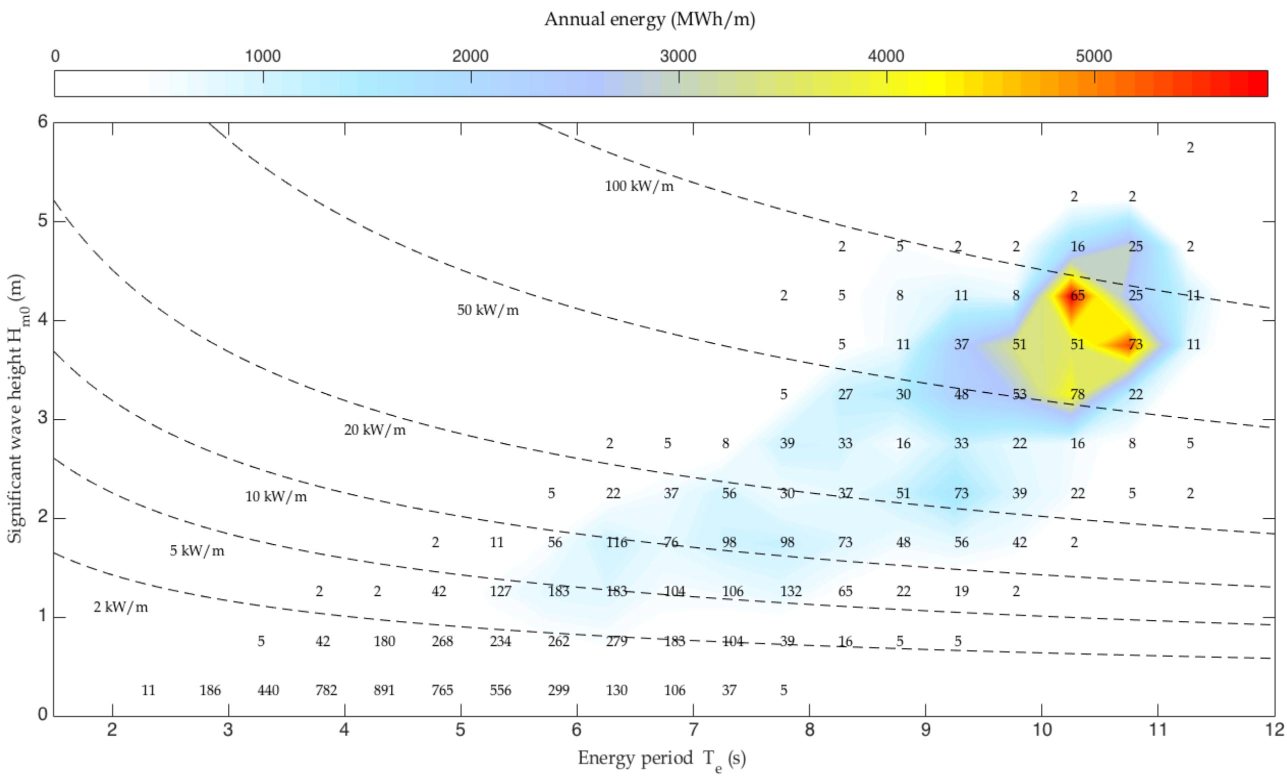

Fig. 5. Characterization of the yearly average wave energy in terms of significant wave height H_{m0} and energy period T_e offshore to Bosa Marina. The numbers within the graph indicate the occurrence of sea states (in number of hours per year).

thus implying that an increase in h_s causes an increase in the mean hydraulic head on the turbines. However, for a given value of R_r, the increase of the height of the bottom of the reservoir on the sea water level produces also a reduction of its maximum volume and consequently the increase of the probability of rejecting part of the overtopping volumes due to the already reached maximum volume.

3) Length of the submerged ramp (d_d)

The submerged ramp is included in the device with the purpose of lowering the friction of the breakwater, thus it is expected its extension to affect positively the energy production. However, a longer ramp produces the increase of the cost of the structure.

4) Number of turbines installed (N)

The number of turbines (and related exhaust pipes) installed in the device affects the value of the outflow rate. A greater number of turbines implies a fast emptying of the reservoir, thus reducing both the mean hydraulic head and the probability of rejecting part of the overtopping volumes due the already reached maximum capacity.

Three further geometrical parameters which could be relevant to evaluate the output power of the OBREC are the cross-shore width of the reservoir (L), the slope angle of the structure (α) and the longitudinal width of the OBREC (l). The first of these parameters affects the space occupied by the breakwater and may therefore be determined by priorities other than the output power. In the present case a value of L = 5 m has been adopted.

For the slope angle of the structure a value of $\alpha = 34°$, usual for the traditional breakwaters, has been adopted.

The longitudinal length of the OBREC, that is the length of the breakwater where the OBREC is installed, does not affect the performances of the device per unit length if the wave climate is approximately constant with the length, hence it is not relevant to the proper dimensioning of the device. In the present analysis a longitudinal length equals to l = 100 m has been adopted.

The present analysis was performed assuming the use of the Kaplan turbines ZD760-LM-80 produced by the Electway Electric. Such turbines have a diameter equal to 80 cm and an operative head in the range 2 - 6 m. Its performances are shown in Table 1.

TABLE 1
PERFORMANCES OF THE ZD760-LM-80 KAPLAN TURBINE PRODUCED BY THE ELECTWAY ELECTRIC

Head (m)	Flow (m³/s)	Output (kW)
2	2.097	34.3
2.6	2.370	50.2
3.2	2.650	69.1
3.9	2.920	92.8
4.8	3.060	120.1
5.4	3.130	141.2

V. ANALYSIS OF RESULTS

The main results of the model concerning the four geometrical parameters described in section IV are presented and discussed in the following paragraphs.

A. R_r and h_s

The crest free-board of the front reservoir and the height of the bottom of the reservoir are analyzed together since their difference is directly responsible of the capacity of the reservoir itself.

Figures 6 to 9 show the results given by the numerical model applied to an OBREC with a length of the submerged ramp of 1 m, 20 turbines installed, h_s ranging between 2 m and 2.5 m, and R_r ranging between 2.25 m and 3.5 m.

Fig. 6 shows the yearly average output power of the OBREC P_{m0}.

Fig. 7 shows the yearly average outflow rate of the OBREC Q_{m0}. Such a parameter represents the ratio between the volume effectively collected by the reservoir and the time in which it has been collected, thus it is not equal to the average overtopping inflow rate (since part of the volumes can be rejected) and can hence increase even if R_r increases.

Fig. 8 shows the average hydraulic head E_m. This parameter, differently from the previous, is not averaged over all the period of functioning of the device, but over the period during which there is water inside the reservoir of the OBREC, hence it can increase even if P_{m0} decreases (because the time of functioning of the device change with R_r and h_s).

Fig. 9 shows the percentage of time when the reservoir is full t_p. Such a parameter is obtained as the ratio between the amount of time when the reservoir of the OBREC is full due to the already reached maximum volume and the overall time of functioning of the device.

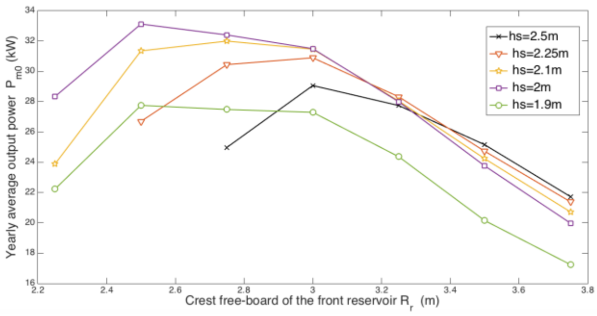

Fig. 6. Yearly average output power versus the variation of R_r and h_s (N=20 and d_d=1m).

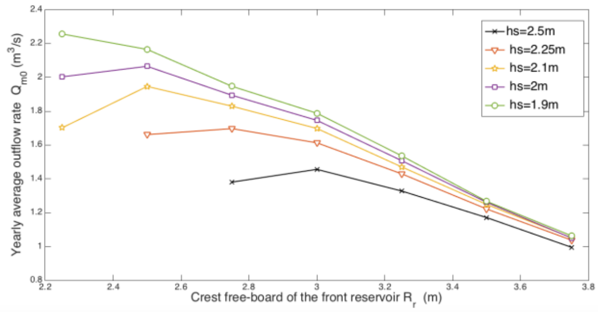

Fig. 7. Yearly average outflow rate versus the variation of R_r and h_s (N=20 and d_d=1m).

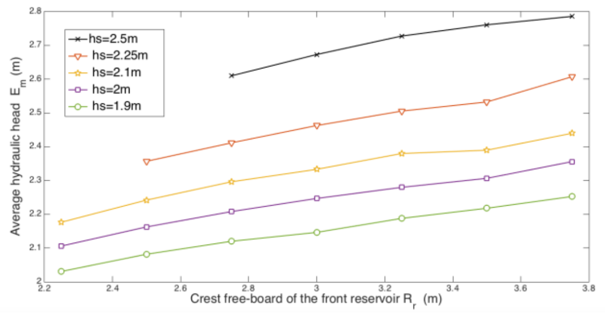

Fig. 8. Average hydraulic head versus the variation of R_r and h_s (N=20 and d_d=1m).

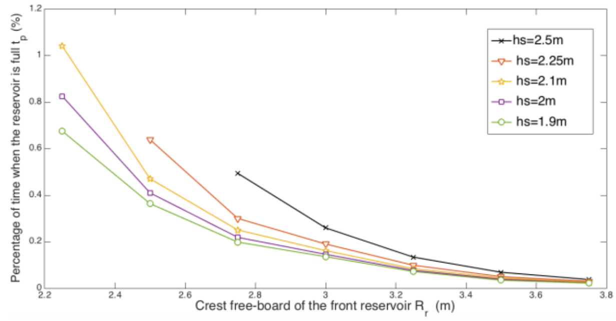

Fig. 9. Percentage of time when the reservoir is full versus the variation of R_r and h_s (N=20 and d_d=1m).

Fig. 6 show that, for a given h_s, the average power increases with the early increment of R_r. This result is mainly due to the increase in the volume of the reservoir and the consequent reduction of volumes rejected, as it is confirmed by the reducing values of t_p (Fig. 9) and the rising values of Q_{m0} (Fig. 7), which occur despite the reduction of the overtopping caused by the increment in R_r.

A further increment in R_r causes a decrease in output power due to the ever-reducing values of Q_{m0} (Fig. 7) which are not balanced by the small increases of the average hydraulic head (Fig. 8). Consequently, the variation in capacity of the reservoir become less and less relevant, as it can be noticed by the reduction of the differences between the curves in Fig. 7 and in Fig. 9.

As regards the effect of the variation of h_s, it can be noticed that for high values of R_r (that is when the low capacity of the reservoir doesn't affect the performances of the device) it is convenient to adopt a configuration with higher values of h_s since it allows the average hydraulic head to rise (Fig. 8) with a minimal reduction of the collected volumes (Fig. 7). For low values of R_r (that is when the low capacity of the reservoir affects considerably the performances of the device) it is instead preferable to reduce h_s (and therefore E_m) in order to reduce the amount of water rejected. Of course, h_s lower than the minimum hydraulic head required by the turbine (i.e. 2 m) produces lower energy because the volume collected in the lower part of the reservoir are discharged without producing any power.

For the analysed site, the optimal configuration of the above described parameter is obtained for h_s =2 m and R_r = 2.5 m, with a corresponding yearly average output power of 33 kW.

Other tests carried out with the numerical model have indicated that such an optimal configuration is unchanged by the variation of the length of the submerged ramp and of the number of turbines installed. The optimal value of the height of bottom of the reservoir being identical to the minimum head required by the turbines cannot be considered a general rule, since other tests carried out with different turbines have shown different results.

B. d_d

The effect of the length of the submerged ramp, that is the portion of the traditional rubble mound replaced with a low-friction smooth concrete ramp, has been analysed in the configuration with the optimal values of R_r and h_s and with 20 turbines installed. The results concerning the yearly average output power are shown in Fig. 10.

The early increments in d_d strongly influence the performances of the device. Indeed, for d_d=1 m the output mean power increase more than 10 kW with respect to the case d_d=0 m. A further lengthening of the ramp causes diminishing increments in output mean power and it might as well imply an excessive structural stress undergone by the ramp itself.

In the present case a length of the concrete ramp equal to 2 m, corresponding to a yearly average output power of 37 kW, is evaluated as the optimal choice.

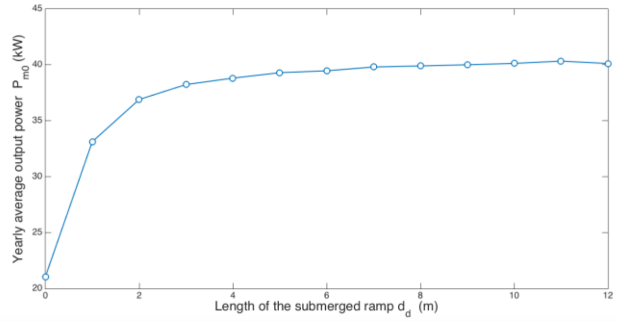

Fig. 10. Yearly average output power versus the variation of d_d (N=20, R_r =2.5 m and h_s=2 m).

C. N

The effect of the number of turbines has been investigated for the optimal configuration (i.e. R_r =2.5 m, h_s =2 m and d_d=2 m) and the results obtained concerning the device performances are shown in Fig. 11-12.

Fig. 11 shows the yearly average output power P_{m0}.

Fig. 12 shows the percentage of time when the reservoir is full t_p.

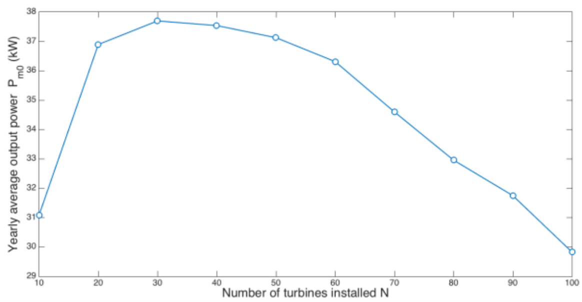

Fig. 11. Yearly average output power versus the number of turbines installed (R_r=2.5 m, h_s=2 m and d_d=2 m).

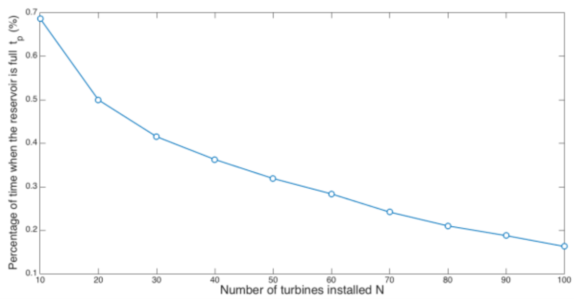

Fig. 12. Percentage of time when the reservoir is full the number of turbines installed (R_r=2.5 m, h_s=2 m and d_d=2 m).

The first increments in N cause an increase in the output power due to the reduction in the rejected overtopping volumes (Fig. 12) being more significant than the reduction in the average hydraulic head. The subsequent increments in N affect negatively P_{m0} because of the ever-reducing decrease of t_p and because of the emptying of the reservoir being too fast to allow the hydraulic head to raise up to the minimum required by the turbines.

The maximum value of the yearly average output power corresponds to N=30 (37.7 kW), but the optimal number of turbines installed is considered to be N=20 (37 kW), since the slight reduction in output power would be compensated for by the reduced cost of the turbines.

VI. CONCLUSIONS

A numerical model based on a stochastic approach for the optimization of the performances of the new Overtopping BReakwater for Energy Conversion (OBREC) is proposed.

The model was applied to optimize a device to embed in the new breakwater of the port of Bosa Marina (Sardinia, Italy). The analysis carried out in such a location allowed to identify the four main geometrical parameters affecting the output power of the OBREC: the crest free-board of the front reservoir (R_r), the height of the bottom of the reservoir on the swl (h_s), the length of the submerged ramp (d_d) and the number of turbines installed (N) per 100m of longitudinal length of the device.

For the analysed site, where the yearly average power of the incident waves is 5.45 kW per meter of wave width, the results of the model identified the optimal configuration with R_r=2.5 m, h_s=2 m, d_d=2 m and N=20 with a corresponding yearly average output power of 0.37 kW per meter of longitudinal length of the OBREC.

This study analysed some parameters that may affect the device performance. Future work will focus on the influence of other parameters such as the width of the tank, the variable number of active turbines for a given sea state, etc.

REFERENCES

[1] "Implementing agreement on ocean energy systems: Annual report 2007," International Energy Agency and Ocean Energy Systems (IEA-OES). [Online]. Available: https://www.ocean-energy-systems.org/publications/annual-reports/document/oes-annual-report-2007/

[2] Iuppa, C., Cavallaro, L., Vicinanza, D., & Foti, E. "Investigation of suitable sites for Wave Energy Converters around Sicily (Italy)". Ocean Science Discussions, 12(1) (2015).

[3] Sannino, G.; Pisacane, G. "Ocean energy exploitation in Italy: ongoing R&D activities. Position paper/September 2017" (2017).

[4] Vicinanza, D.; Contestabile, P.; Nørgaard, J.Q.H.; Andersen, T.L. "Innovative rubble mound breakwaters for overtopping wave energy conversion". Coast. Eng. 2014, 88, 154–170

[5] Vicinanza, D., & Frigaard, P. (2008). “Wave pressure acting on a seawave slot-cone generator”. *Coastal Engineering*, *55*(6), 553-568.

[6] Viviano, A., Naty, S., Foti, E., Bruce, T., Allsop, W., & Vicinanza, D. (2016).” Large-scale experiments on the behaviour of a generalised Oscillating Water Column under random waves”. *Renewable Energy*, *99*, 875-887.

[7] Arena, F., Romolo, A., Malara, G., Fiamma, V., & Laface, V. (2016, November). "The first worldwide application at full-scale of the REWEC3 device in the Port of Civitavecchia: Initial energetic performances.” In *Progress in Renewable Energies Offshore: Proceedings of the 2nd International Conference on Renewable Energies Offshore (RENEW2016), Lisbon, Portugal, 24-26 October 2016* (p. 303). CRC Press.

[8] Contestabile, P., Iuppa, C., Di Lauro, E., Cavallaro, L., Andersen, T. L., & Vicinanza, D. (2017). Wave loadings acting on innovative rubble mound breakwater for overtopping wave energy conversion. *Coastal Engineering, 122*, 60-74. doi:10.1016/j.coastaleng.2017.02.001

[9] Boccotti, P., Filianoti, P., Fiamma, V., & Arena, F. (2007). “Caisson breakwaters embodying an OWC with a small opening-part II: A small-scale field experiment”. *Ocean Engineering, 34*(5-6), 820-841. doi:10.1016/j.oceaneng.2006.04.016

[10] Malara, G., Romolo, A., Fiamma, V., & Arena, F. (2017). “On the modelling of water column oscillations in U-OWC energy harvesters”. *Renewable Energy, 101*, 964-972. doi:10.1016/j.renene.2016.09.051

[11] Kofoed, J. P., Frigaard, P., Friis-Madsen, E., & Sørensen, H. C. (2006). Prototype testing of the wave energy converter wave dragon. Renewable energy, 31(2), 181-189.

[12] Kofoed, Jens Peter. *Wave Overtopping of Marine Structures: utilization of wave energy*. Diss. Hydraulics & Coastal Engineering Laboratory, Department of Civil Engineering, Aalborg University, 2002.

[13] Iuppa C., Cavallaro L., Musumeci E.R., Vicinanza D., Foti E. “Empirical method for estimating wave energy extracted by innovative Rubble Mound Breakwater” submitted to Coastal Engineering (2019).

[14] Van der Meer, J.W., Stam, C.J.M.. “Wave runup on smooth and rock slopes of coastal structures”. Journal of Waterway, Port, Coastal, and Ocean Engineering 1992;118(5):534–550.

[15] van der Meer, J.W., Janssen, J.P.. “Wave run-up and wave overtopping at dikes and revetments”. Delft hydraulics 1994;.

[16] Tedd, James, and Jens Peter Kofoed. "Measurements of overtopping flow time series on the Wave Dragon, wave energy converter." Renewable Energy 34.3 (2009): 711-717.

[17] van der Meer, J., Bruce, T.. “New physical insights and design formulas on wave overtopping at sloping and vertical structures”. Journal of Waterway, Port, Coastal, and Ocean Engineering 2013;140(6):04014025.

[18] Iuppa, C., Contestabile, P., Cavallaro, L., Foti, E., & Vicinanza, D. (2016). “Hydraulic performance of an innovative breakwater for overtopping wave energy conversion”. Sustainability, *8*(12), 1226.

[19] Liberti, L., Carillo, A., & Sannino, G. (2013). “Wave energy resource assessment in the Mediterranean, the Italian perspective”. *Renewable Energy*, *50*, 938-949.

[20] Vicinanza, D., Contestabile, P., & Ferrante, V. (2013). “Wave energy potential in the north-west of Sardinia” (Italy). Renewable Energy, 50, 506-521. doi:10.1016/j.renene.2012.07.015