Innovative rubble mound breakwaters for wave energy conversion

This paper presents a new Wave Energy Converter named Overtopping BReakwater for Energy Conversion (OBREC) which consists of a rubble mound breakwater with a front reservoir designed with the aim of capturing the wave overtopping in order to produce electricity. The energy is extracted via low head turbines, using the difference in water levels between the reservoir and the mean sea water level. The new design should be capable of adding a revenue generation function to a breakwater while adding cost sharing benefits due to integration. The design can be applied to harbour expansions, existing breakwater maintenance or upgrades due to climate change for a relatively low cost, considering the breakwater would be built regardless of the inclusion of a WEC.

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Introduction

The energy consumption is in constant growth and so is the impact on the environment determined by the systems used for the production of energy. Global energy demand is expected to increase around 35% from 2010 to 2040 [1]. Given that traditional energy sources are finite, Nations are mobilized by encouraging the implementation of systems based on renewable energy. For example, the European Community has established to achieve by 2020 a 20% share of energy produced from renewable sources (Renewable Energy Directive 2009/28/EC). In any case oil will remain the largest single source of energy until 2040 [2].
Nowadays new devices based on wave energy recourses are under development. However since wave energy is not going to be economically competitive, it will be very difficult to become a possible contender in the energy market. The main aim of the present study is to give a WEC design solution having a reliable technology and a positive payback period of the investment.
The only solution to reduce the device structure costs is to move from standalone device to hybrid systems embedded in other costal or offshore structure (offshore wind farms, offshore oil platforms, ports, costal defence). Integration and sharing costs will become a solution for WECs to be competitive with other renewable energy devices.
The ongoing research reported here gives information on an innovative coastal structure designed in terms of safe hydraulic performance and global stability but also able to produce electricity in a balanced cost-benefit frame. Overtopping BReakwater for Energy Conversion (OBREC) is a new rubble mound breakwaters with a front reservoir designed with the aim of capturing the incoming wave overtopping to produce electricity. The energy is extracted via low head turbines, using the difference in water levels between the reservoir and the average sea water level (Fig. 1).

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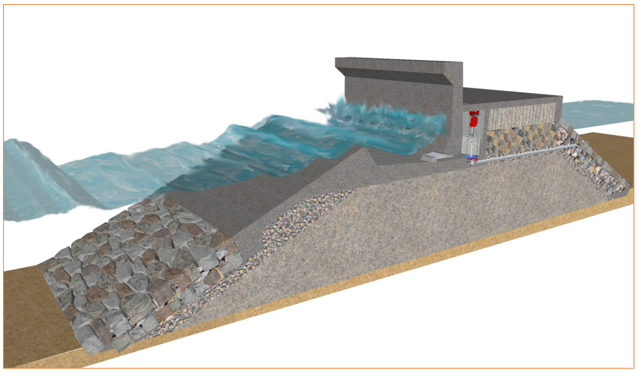

FIGURE 1 Innovative rubble mound breakwater with frontal reservoir for energy production

Laboratory experimental tests on OBREC have been carried out at Aalborg University (Denmark) in 2012 and 2014. The results of 2012 tests have been presented by [3, 4, 5]. The main aims were: (*i*) comparing OBREC with a traditional rubble mound breakwater in terms of hydraulic performances and loadings; (*ii*) validating the existing prediction methods to estimate structure reflection coefficient, overtopping and wave loadings; (*iii*) providing new formulae to design the first prototype.
The main aims of 2014 tests were to complete the analysis on OBREC geometric parameter variations. The paper is organized as follows: in the next paragraph a summary of 2012 tests is reported, then preliminary results of 2014 tests are presented. The paper ends with some conclusions and remarks for future works.

State of the art

OBREC is the outcome of composite seawalls evolution consisting of a frontal obstacle that dissipates the energy of the incoming wave reducing the wave loadings and related damage. These kinds of coastal projects have been constructed for many years in Japan and one example is the port of Mori [6], in which the Authors estimated a 30% reduction of the total cost. The contribute of wave overtopping reduction has been confirmed in [7]. The Authors demonstrated that a front reservoir solution is much more cost-effective than conventional cross section, such as beamed structures and mild slope structures. Another example is the composite seawalls proposed by [8], which represent an evolution with respect to the previous structures, for it is able to accumulate the wave. This new structure is composed by a reservoir realized in front of the seawall. The water is accumulated in the reservoir through the wave overtopping and realize a head difference between the water inside the reservoir and the mean sea water level. The head difference generates a flow which can be used to run a turbine and to produce electricity.
In order to maximize the energy production, the WAVEnergy AS (Stavanger, Norway) have developed a device called Seawave Slot-cone Generator (or SSG) [9]. The SSG is an overtopping device that uses a number of reservoirs placed one on top of each other. The energy of incoming waves is stored as potential energy. Then, the captured water runs through turbines for electricity production. The peculiarities of SSG are: (*i*) its flexibility to work with a wide spectrum of different incident wave condition; (*ii*) availability of grid connection; (*iii*) the recirculation of water inside the harbour. This kind of structure should be constructed where wave energy is high because it has very high construction costs that may be compensated only by large electricity production.
OBREC is a solution to reduce installation and maintenance costs. In fact, it is a simpler structure than SSG and tends to be more economically viable than offshore floating as Wave Dragon [10] and WaveCat [11]. The device is still under development and physical model tests have been conducted to evaluate the hydraulic and structural performance.

Physical model AAU 2012

Prototypes in scale 1:1 are the most important development phase for WECs. However, a preliminary analysis in a smaller scale is necessary to understand the physical phenomena that characterise the device performance and its specific limits. For these reason, physical model tests on OBREC have been carried out at Aalborg University (Denmark) in 2012 in 1:30 length scale (Froude

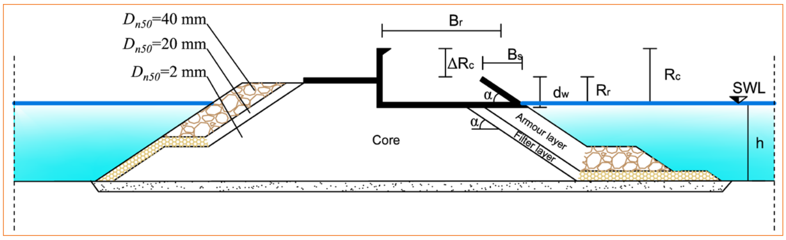

FIGURE 2 Model cross section in 2012 experimental campaign: definition of the principal geometrical parameters

scaling) [4]. The principal aim was to understand the different behaviour between OBREC and traditional rubble mound breakwaters. The main studied parameters were: reflection coefficient, overtopping at the rear side of the structure, overtopping in the front reservoir and the loading on the structure.
The wave flume has a length of 25 m and a width of 1.5 m. Moving from the paddle a horizontal bottom characterized the initial 6.5 m, followed by a 1:98 slope that continues until just before the model. The rubble mound material characteristics were: D_{n50} 40 mm for the armour layer; D_{n50} 20 mm for the filter layer; D_{n50} 2 mm for core.
Figure 2 shows the section of the physical model. The principal geometrical parameters are: d_w height of sloping plate; R_r crest freeboard of front reservoir; R_c crest freeboard of crown wall; B_r reservoir width; α slope angle of the structure; h water depth at the toe of the structure.
A total of 48 tests were carried out under different wave condition. The parameter ranges for the OBREC structure are reported in Table 1.
The instrumental apparatus consists of: 28 pressure transducers for the estimation of the pressures/forces induced by the waves on the structure; 3 wave gauges for the estimation of the incident and reflected wave; 4 wave gauges installed in the boxes used to measure overtopping discharge at both the rear side of the model and in the frontreservoir. The incident and reflected spectra were determined using the approach of [12] and the positioning of the wave gauges was based on suggestions by [13].
The results below are expressed in terms of dimensionless parameters which are: break parameter $\xi_{m-1,0}$, relative reservoir crest freeboard R^*_r, relative crest freeboard of crown wall R^*_c, wave-structure steepness s^*_{Rr}, the parameter s^*_{Rc} and the non-dimensional average overtopping q^*. The parameters are defined as follow:

$$\xi_{m-1,0} = \tan(\alpha) \Big/ \sqrt{\frac{H_{m0}}{L_{m-1,0}}} \qquad (1)$$

$$R^*_r = \frac{R_r}{H_{m0}}; \; R^*_c = \frac{R_c}{H_{m0}} \qquad (2)$$

$$s^*_{Rr} = \frac{R_r}{H_{m0}} \frac{R_r}{L_{m-1,0}}; \; s^*_{Rc} = \frac{R_c}{H_{m0}} \frac{R_c}{L_{m-1,0}} \qquad (3)$$

$$q^* = \frac{q}{\sqrt{gH^3_{m0}}} \qquad (4)$$

Number of test	h [m]	H_{mo} [m]	$T_{m-1,0}$ [s]	R_c [m]	d_w [m]	R_r [m]	B_r [m]
48							
Extreme	0.30	0.141	1.68	0.20	0.075	0.075	0.415
(min-max)	0.34	0.177	2.26	0.24	0.125	0.125	0.488
Extreme with nose		0.145	1.66		0.075	0.035	0.415
(min-max)	0.34	0.161	2.28	0.20	0.125	0.085	0.488
Production		0.037	1.05		0.075	0.105	0.415
(min-max)	0.27	0.138	2.14	0.27	0.125	0.155	0.488

TABLE 1 Test 2012: Wave characteristic and reservoir geometrical parameter for OBREC model (model scale)

Hydraulic performance

The principal results from the hydraulic point of view can be summarized from [5] as follow:

- the OBREC shows a similar or reduced average reflection coefficient with respect to traditional rubble mound breakwater;
- the overtopping at the rear side of the structure is greater than traditional rubble mound breakwater, but the use of a parapet wall with a protuberance reduce by 50-60% the average overtopping;
- the overtopping at the rear side of the crown wall is well fitted by Eq. 5 (range of application: $0.014 < \Delta Rc/L_{m-1,0} < 0.038$; $0.035 < s_{0m} < 0.058$; $1.24 < R_c^* < 1.38$);
- the overtopping in the front reservoir is well fitted by Eq. 6 (range of application: $0.45 < d_w/\Delta R_c < 1.08$; $0.0123 < s_{Rr} < 0.202$).

$$q^*_{rear} = 6.47 \cdot e^{-112 \cdot (s_{Rc})} \tag{5}$$

$$q^*_{reservoir} = \left(35.1 + 2.38\frac{d_w}{R_c}\right) \cdot e^{\left(-58.99+17.7\frac{d_w}{R_c}\right)s_{Rr}} \tag{6}$$

Reflection

The results showed that the presence of the reservoir does not determine increments of the reflection coefficient (K_r). Moreover, in some cases K_r is reduced. The comparison with various prediction methods has shown that the method of [14] may be used to estimate the values of K_r. In [14] the reflection coefficient can be expressed with the equation below:

$$K_r = \tanh\left(a\xi_0^b\right) \tag{7}$$

where a and b are two coefficients the value of which only depends on the roughness factor γ_f. The OBREC behaviour can be assumed as an impermeable rock with γ_f equal to 0.40 (a = 0.12 and b = 0.87). This method overestimates the values of K_r with apposite safety margin.

Overtopping discharge at the rear side of the structure

OBREC replaces the typical frontal rock area of traditional rubble mound breakwater with a concrete slope, significantly reducing the roughness of the structure. For this reason, with respect to a traditional rubble mound breakwater, OBREC slightly increases the overall wave overtopping. In order to reduce the overtopping without significantly raising the height of the wall, a parapet may be placed on the top of the crown wall. Indeed, test results have shown that the configuration with nose can reduce the overtopping of about 50-60%. In [5] a new prediction formula has been proposed to estimate the overtopping (see Eq. 5). This formula is a function of the significant wave height at the toe of structure, the wave length in deep water, the crest freeboard of the crown wall and the crest freeboard of the front reservoir.

Overtopping discharge in the front reservoir

Overtopping into the front reservoir has a high importance to understand the potential energy production of the device. In this phase it is important to understand how the overtopping is influenced by the geometrical characteristics of the reservoir. For this reason, different configurations have been tested under "production" wave conditions. The comparison with existing prediction methods showed a relatively good estimation from Van der Meer formula [15]. However, Eq. 6 has been proposed, which is a function of: height of sloping plate, crest freeboard of the crown wall, crest freeboard of the front reservoir, significant wave height at the toe of the structure and the wave length in deep water. The new method has shown a good agreement with the observed data.

Wave loading on the structure

The estimation of the wave loading and the structural response are important aspects to take into account for a consistent assessment on innovative devices. The analysis has been carried out by analysing the individual parts of the structure (see Fig. 3). The forms and magnitude of the wave pressures/forces acting upon the structures can be divided into impulsive, when they are rapidly varying and

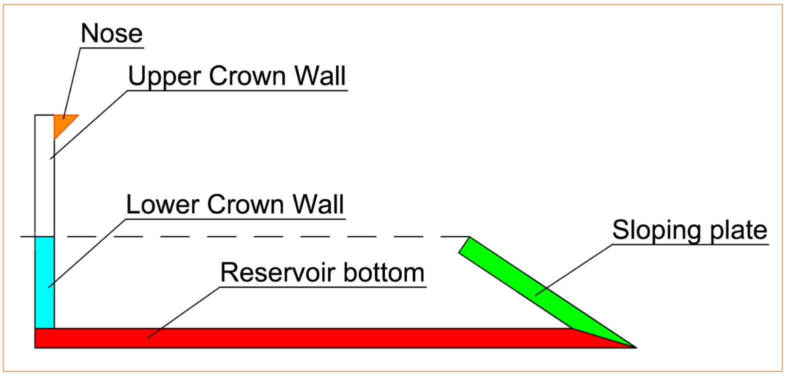

FIGURE 3 Overtopping device cross section

the pressure spatial gradient is extremely high, and non-impulsive, when they are slowly-varying in time and the pressure spatial gradient is relatively mild. The principal results of the tests can be summarized as follow:

- the wave loading on ramp may be evaluated averaging the non-impulsive and impulsive pressure distribution estimate with [16] using Goda's formula and [17] using Goda's formula modified by [18], respectively;
- the pressure distribution at the reservoir bottom may be assumed as triangular;
- the horizontal force on OBREC upper crown wall is well fitted by the modified formula proposed by [19], introducing a new empirical coefficient;
- the horizontal force on OBREC lower crown wall may be estimated with [20], introducing a correction parameter.

The measures of the wave loading on the ramp have been compared with the [16] using Goda's formula for non-impulsive conditions and with the [16] using Goda's formula modified by [18] for impulsive conditions. The comparison showed an over-prediction for the impulsive force and an under-prediction for the non-impulsive force. This behaviour is determined by the dynamics that occurs in the reservoir. Indeed, the backwash interacts with the uprush and generates a quasi-breaking wave conditions. Averaging the results for impulsive and non-impulsive conditions a good agreement is obtained.
The analysis of the wave loading in the reservoir bottom has shown that a triangular pressure distribution can be assumed based on [16]. Indeed, an error less than 20% has resulted from the comparison between observed and calculated data.
The behaviour of the wave loading on the crown wall is quite similar to the classical configuration of a berm in front of a crown wall. Two separate analyses have been performed for both the upper and the lower crown walls. For the upper crown wall the results have been compared with the formula proposed by [19]. The comparison has allowed the modification of the run-up formula by [5] in Eq. 5:

$$R_{u,0.1\%} = \gamma_{runup} \cdot c \cdot H_{0.1\%} \cdot \xi_m^{0.55} \qquad \xi_m > 3.5 \tag{8}$$

where $R_{u,0.1\%}$ is the wave run-up height exceeded 0.1% of the coming waves; γ_{runup} is a correction factor; c is the an empirical coefficient to take into account the change of the roughness factor from a traditional rouble mound breakwater; $H_{0.1\%}$ is the wave height exceeded 0.1% of the coming waves; ξ_m is breaker parameter. The empirical coefficient c is described as follow:

$$c = \frac{0.722 \cdot \gamma_{f,OBREC}}{\gamma_{f,trad}} \tag{9}$$

where $\gamma_{f,OBREC}$ is equal to 0.45 and $\gamma_{f,trad}$ is equal to 0.7.
For the lower crown wall the comparison with the formula of [20] has shown that the formula does not correctly interpret the measured loading. Even in this case, a modification factor has been introduced, which amplifies the force $F_{Takahashi}$ deriving from [20] and gives a better representation of the force $F_{H,Lcw}$ on the crown wall:

$$F_{H,Lcw} = \gamma_{falling} \cdot F_{Takahashi} \tag{10}$$

where $\gamma_{falling}$ is the amplification factor estimate with Eq. 11.

$$\gamma_{falling} = 2 \cdot \left(\frac{\gamma_{runup}}{R_r^*} \right)^{-1.3} \qquad (11)$$

The analysis of the results has allowed to both understand the behaviour of the structure quite well and design prediction methods. However, new questions emerge about:

- the wave loading acting on the nose: the presence of the nose causes a reduction of the overtopping, but also an increase in the wave loading on the structure;
- how to increase the wave overtopping in front reservoir, without increasing, for example, the reflection coefficient and the overtopping rear the structure;
- how the geometrical parameters, e.g. the reservoir width and the length of the ramp, may be influenced by the performance of the device;
- how the uplift forces on the reservoir bottom can be reduced.

In such a framework, a second series of physical model tests were carried out in 2014. Different geometric configurations have been investigated by varying the width of the reservoir and the slope profile of the ramp. Moreover, the behaviour of the device subject to still water level variations has been simulated. Pressure transducers have been placed on the nose.

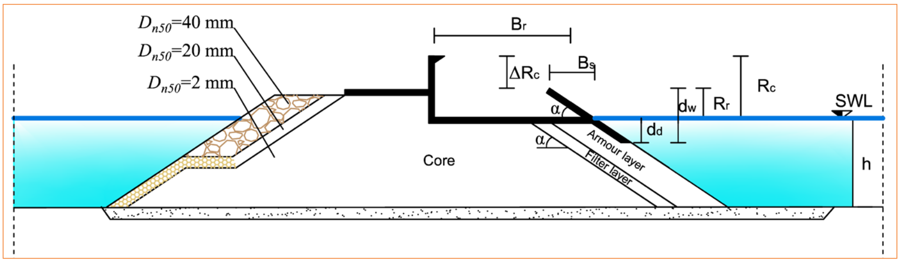

FIGURE 4 Model cross section in 2014 experimental campaign: definition of the principal geometrical parameters

Physical model AAU 2014

A second series of physical model tests on OBREC were carried out at Aalborg University (Denmark) in 2014 in 1:30 length scale (Froude scaling), compared to the prototype. Figure 4 shows the section of the physical model where d_d is the draft length.

Table 2 shows the wave characteristic and reservoir geometrical parameter for OBREC at model scale.

The principal parameters studied are: reflection coefficient, overtopping rear the crown wall of the structures, overtopping in the front reservoir and the loading of the wave motion.

Figure 5 shows the definition sketch for the overtopping Q_{in} and its three components (see Eq. 12), $Q_{reservoir}$ is the flow through section S_2, Q_{rear} is the flow through section S_3 and Q_{over} is the overflow. The water collected in the reservoir generates the $Q_{turbine}$.

$$Q_{in} = Q_{reservoir} + Q_{rear} + Q_{over} \qquad (12)$$

The overtopping discharge in the front reservoir has gone into a box. A depth wave gauge was installed in the box to measure the $Q_{turbine}$ and to control the pump emptying the box when reaching a certain

Number of test	h [m]	H_{mo} [m]	$T_{m-1,0}$ [s]	R_c [m]	d_w [m]	R_r [m]	B_r [m]
200							
min	0.27	0.021	0.77	0.167	0.192	0.065	0.219
max	0.35	0.122	2.27	0.227		0.125	0.419

TABLE 2 Test 2014: Wave characteristic and reservoir geometrical parameter for OBREC model (model scale)

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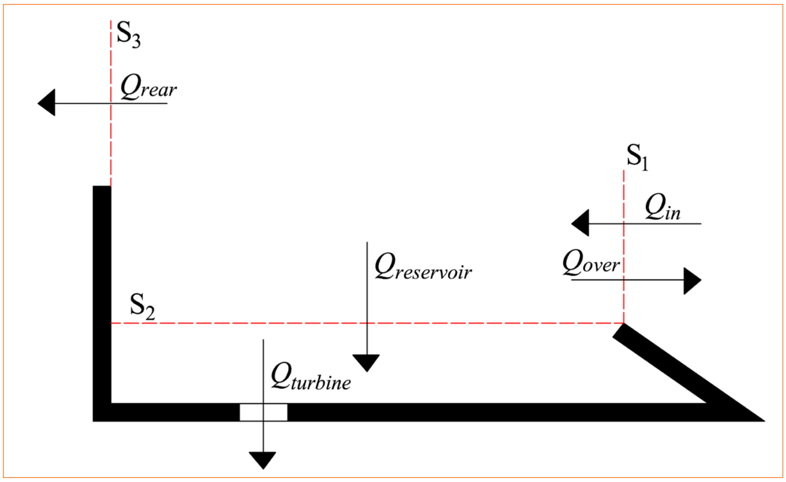

FIGURE 5 Definition of overtopping input, overtopping in front of the reservoir, in the rear side and the overflow

impermeable (γ_f = 0.4) may be used. In 2014 tests the effect of the increases in the draft length is evident in the reflection coefficient. The reduction of the roughness causes an increase in the value of K_r. This could represent problems for navigation and for the structure stability.

Figure 8 shows the comparison of the non-dimensional average wave overtopping rate of the rear crown wall model, measured in the tests performed in 2012 and 2014, respectively. The analysis shows that there is no change due to the increase in d_d, and the wave overtopping has similar trend and Eq. 5 may be also used for the some configurations analysed in 2014.

level. The hydraulic head has been measured with a pressure transducer installed in bottom of the reservoir. For all tests, overtopping discharge at the rear side of the model was determined using a ramp to guide the overtopping wave volumes into a box.

Figure 7 shows a comparison of the reflection coefficient measured in 2012 and 2014 tests, the isolines refer to the prediction formula of Zanutting and Van der Meer [14] with different value of the roughness factor γ_f. As indicated in [5]: the test results of 2012 show a similar behaviour of the traditional rubble mound breakwater, the reduction of the d_w causes a reduction of K_r and in a safe design the Zanutting and Van der Meer formula [14] for rock

The non-dimensional average wave overtopping rate in the front reservoir from the experiments 2014 are compared in Figure 9 with both the measures of 2012 tests and the prediction formula of Kofoed [21], where the value of the parameter λ_{dr} has been estimated with the regression analysis. It is noted that in range 1.5<R^*_r<2 the experimental curves show a different behaviour: this is due to the different length of the draft. In 2012 tests the d_d is equal to 0 m, whereas in 2014 tests it is equal to 0.067 m. It is apparent that the overtopping increases as the draft increases too. The prediction formula of Kofoed [21] has been designed for a structure with smooth impermeable slopes analogue to the configuration

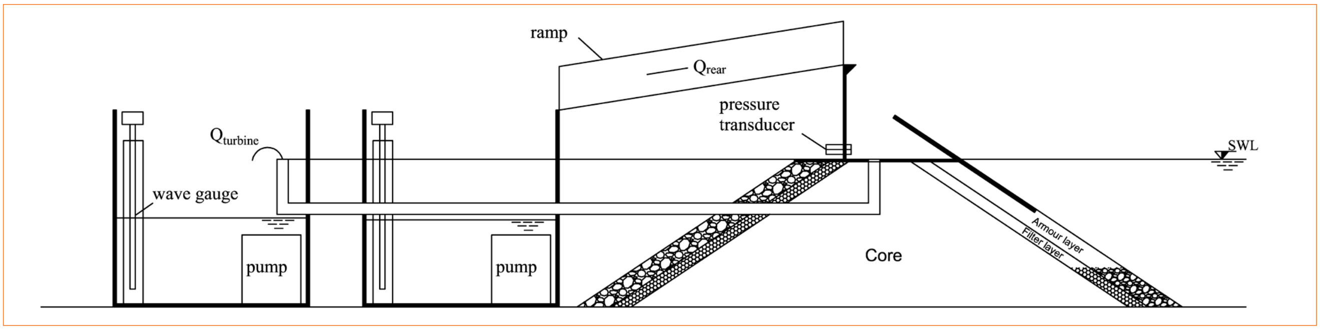

FIGURE 6 Wave-by-wave system for flow discharge measurement

of the model tested in 2014. Moreover, Kofoed [21] allows water to pass below the structure. Nevertheless, a good adaptation is noted. In the range $0.5<R^*_r<1.5$, the average wave overtopping rate in 2014 tests is constant: this is due the saturation of the reservoir. When the saturation

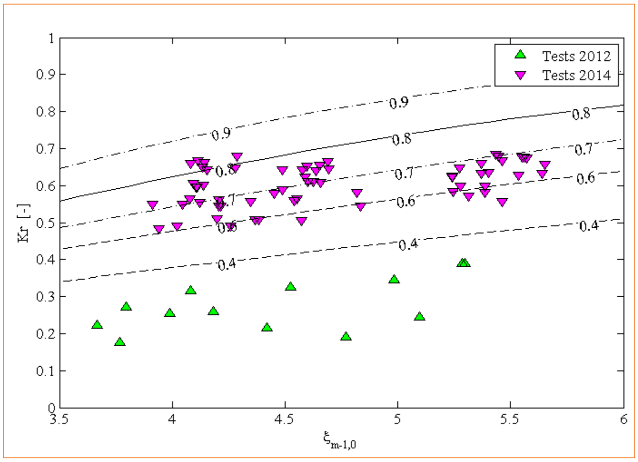

FIGURE 7 Comparison between the reflection coefficient of 2012 and 2014 tests results. The isolines refer to the prediction formula of Zanutting and Van der Meer [14] with different value of the roughness factor γf

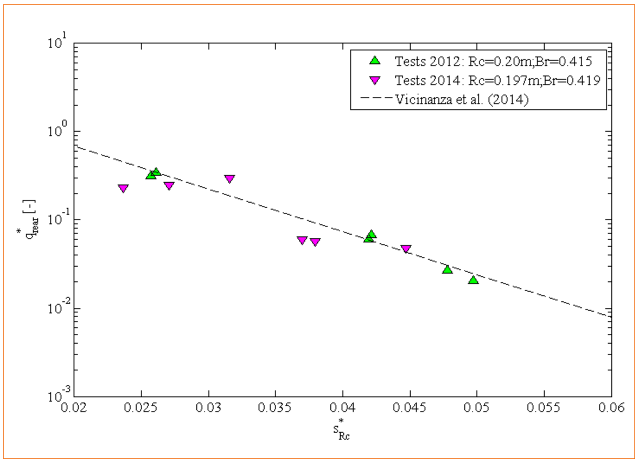

FIGURE 8 Comparison between non-dimensional average wave overtopping rate of the rear crown wall model resulting from 2012 and 2014 tests, and the prediction formula of [5]

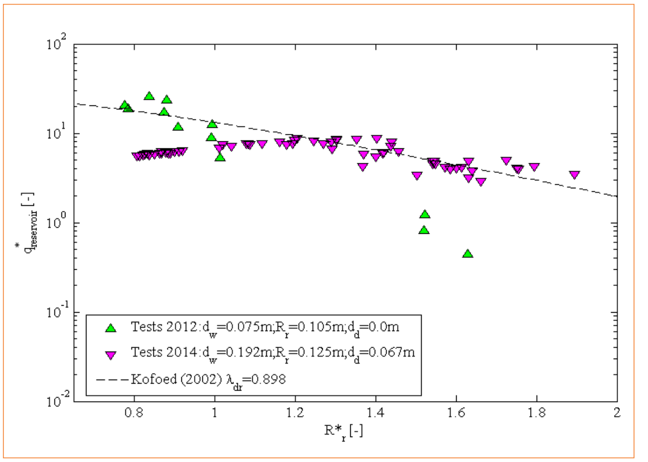

FIGURE 9 Comparison between non-dimensional average wave overtopping rate in the front reservoir resulting from 2012 and 2014 tests, and the prediction formula of Kofoed [21]

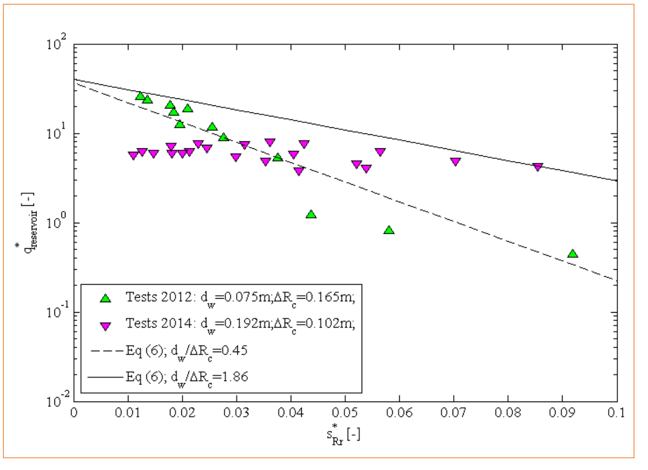

FIGURE 10 Comparison between non-dimensional average wave overtopping rate in the front reservoir resulting from the 2012 and 2014 tests and Eq. (6)

level is reached, large quantities of water are lost, but in this case the $Q_{reservoir}$ is almost constant and there is a reduction of the variability of the energy production.

Figure 10 shows the non-dimensional average wave overtopping rate in the front reservoir with respect to the parameter s^*_{Rr}. Here the average wave overtopping is compared with Eq (6). The comparison shows an over-prediction of Eq. (6) for low value of s^*_{Rr}. The reason for such behavior is that, for the wave with low energy (greater values of s^*_{Rr}), the average Q_{in} is similar to the average $Q_{reservoir}$. When the wave energy grows, the reservoir is under saturated conditions for a larger percentage of the time. This causes an increase both in the average Q_{rear} and the average Q_{over}. In this case Q_{in} becomes greater than $Q_{reservoir}$.

Conclusions

WECs can be an opportunity to reduce the impact on the environment determined by the traditional systems used for the production of non-renewable energy. A new device called OBREC is presented. In particular, a summary of 2012 small-scale laboratory tests is reported, and preliminary results of 2014 tests are presented.
In 2012 model tests OBREC shows similar or reduced average reflection coefficient with respect to the traditional rubble mound breakwater. However, the ramp causes an increase in the overtopping rear the structure, which may be reduced by placing a parapet on the top of the crown wall. Indeed, the "nose" reduces the average overtopping by 50-60%. The analysis of the results has allowed the definition of formulas for the prediction of the overtopping rear the crown wall, in the front reservoir and the wave loading on the structure.
However, new questions come out about: (*i*) the wave loading at the nose: the presence of the nose causes a reduction of the overtopping, but also an increase in the wave loading on the structure; (*ii*) how to increase the wave overtopping in the front reservoir, without increasing, for example, the reflection coefficient and the overtopping rear the structure; (*iii*) how the geometrical parameters, e.g. the reservoir width and the length of the ramp, may be influenced the performance of the device; how the uplift forces on the reservoir bottom can be reduced.
For these reasons, a second series of physical model tests were carried out in 2014. Here only preliminary results on hydraulic aspects were reported. The principal difference between the 2012 and 2014 model tests are the length of the reservoir and the measurements of the wave overtopping in the front reservoir. In particular, the comparison has shown that: an increase in d_d determines an increase in the reflection coefficient; wave overtopping at the rear side of the crown wall does not increase compared to the configurations tested in 2012. The prediction formula provided in [5] shows a good adaptation with the test results of 2014. The prediction method of Kofoed [21] and Eq. 6 seem unable to predict the wave overtopping in the front reservoir under sutured conditions. For this reason it will be necessary to develop a new method that takes into account the saturation of the reservoir.

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