Analysis of wave reflection from wave energy converters installed as breakwaters in harbour

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Abstract

Amplification and renovation of harbours, none the last for the need of straitening existing structures because of the increased storminess due to climate change, is a practice that is repeating itself all around the world.

To this purpose, integration of breakwaters and Wave Energy Converters (WECs) based on two different technologies, one based on the overtopping principle and the other of Oscillating Water Column (OWC) type, revealed to be suitable with different advantages compared to offshore installations, among the others: sharing of costs, cheaper accessibility and maintenance, lower loads on the structure, i.e. better survivability.

Nevertheless these devices must comply with the requirements of harbour protection structures and thus cope with problems due to reflection of incoming waves, i.e. dangerous sea states close to harbors entrances and intensified sediment scour, which can lead to structure destabilization.

The present paper aims to analyse wave reflection from OWC and Sea Slot-cone Generator (SSG) converters, based on experimental results obtained in 2D and 3D facilities.

The applicability of formulae available in the literature and derived from costal structures experience are checked.

Consideration on induced scour and structure stability are also carried out, and solution for design improvements are finally drawn.

Keywords: wave reflection, sea slot cone generator, oscillating water column, scour, experiments, formulae.

1 Introduction

For all the countries around the world, harbours represent a significant economic hub, due to their capability of attracting foreign direct investments, of sustaining tourism activities and of creating industrial and transportation employment. The increase of sea level and storminess induced by occurring and expected climate change pushes for the reinforcement of existing sea banks and breakwaters.

In the meantime, within the crisis weakening the economies of the developed world, an urgent need arises for stimulating economic growth by investing in the clean energy economy and in a sustainable environment.

In this frame, it is particularly relevant the investigation of a proper design of WECs that can be used in the amplification and/or renovation of harbours. In this paper, two types of WECs, one based on the overtopping principle and one of the OWC type will be
examined, with focus on wave reflection and induced scour at the WEC toe that can progressively decrease their stability as in case of traditional breakwaters.

Breakwater failures due to scour is reported by various authors, a.o. by [1], [2] and [3].

WECs based on the overtopping principle are Wave Plane [4], Wave Dragon [5], and SSG [6], [7], [8]. Looking in particular at SSG, studies were carried out so far on the design of these devices with respect to the loads and to the storing energy capacity that can be obtained by using multi-level reservoirs. No specific analysis was performed regarding wave reflection from these kind of structures, even if many experimental data exist.

For the purpose of wave reflection analysis, OWC converters might be roughly assimilated in principle to perforated wall breakwaters [9]. Many works exist regarding wave reflection of normally incident waves from single perforated wall structures [10], [11] and from structures with two or multiple perforated front walls [12], [13]. Effects of wave obliquity were also investigated from single perforated wall breakwaters [14], [15] and from breakwaters with two or multiple perforated front walls [16], [17]. As to irregular waves, experimental tests were conducted by [18], to examine the reflection coefficient of a perforated caisson sitting on a rubble mound, and by [19], to analyse the effect of irregular head-on waves on perforated caissons and single screens with different porosity.

Aims of this paper are to investigate for the first time the magnitude of wave reflection from nearshore WECs and to examine how and how far formulae for predicting the reflection coefficient available in the coastal engineering literature can represent wave reflection from near-shore WECs.

The paper presents the two experimental datasets adopted for the analysis, one related to 2D tests carried out at Aalborg University on a multi-level SSG device and the other one related to 3D tests carried out at Wavegen laboratory on a OWC device. Tested geometries and wave conditions are summarised.

The formulae available for smooth slopes [20], [21] are compared with the results obtained from the tests on SSG and proper indications and corrections are provided for design purposes.

2 The SSG device and tests

Tests were carried out in the shallow water wave flume at the Hydraulics and Coastal Engineering laboratory of the Department of Civil Engineering of Aalborg University.

The flume is 25 m long, 1.5 m wide and 1 m deep. The flume is equipped with a piston type wave generator with a stroke length of approximately 0.7 m. The software used for controlling the paddle system to generate regular and irregular waves is AwaSys developed by the same laboratory [22].

The multi-level SSG, 0.514 m wide, consists of 3 horizontal metal plates inclined of 35° with respect to the horizontal. In front of the SSG, a wooden run-up ramp 0.89 m, long inclined of 35°, leads the waves to the model. This slope of 35° was proven to be the optimal one for maximizing wave overtopping [23].

The structure was confined in the flume by two wooden walls, approximately 2 m long, to guide the waves avoiding spurious reflection at the structure side. An artificial dissipating beach was realized outside these wall.

A frontal picture of the set-up is shown in Fig. 1.

The plates in the SSG can be removed to vary the number of reservoirs from 1 to 3 and can slide one respect to the others in order to change the , i.e. the distances $HD_1$ and $HD_2$ (see the sketch in Fig. 2).

Aiming of the tests were carried out with 2 reservoirs and the results will be presented only for this configuration.

Thirteen different geometries were tested with $0.30 \text{ m} < HD_1 < 0.053 \text{ m}$, keeping fixed the crest levels $R_{c1}$ and $R_{c2}$ respectively at 0.033 m and 0.072 m.

![Figure 1: Frontal view of the SSG device with 3 reservoirs.](image)

![Figure 2: Sketch of the SSG, measures in m, side view.](image)
Tested wave attacks (see Tab. 1) were 2D irregular waves with Jansswap spectrum (3.3 peak enhancement factor). Wave conditions (W1, W2, W3, W4) were selected among the most common in the North Sea (probability of occurrence greater than 5%).

Wave heights $H_{m0}$ were in the range 0.03-0.13 m, water depth was kept constant $h=0.51$ m (at the structure toe) and additional wave peak periods $T_p$ were reproduced (W1a; W2a,b; W3b,c; W4b,c) to investigate the effect of wave steepness.

Generated waves were measured with 3 resistance type wave gauges in front of the structure, the closest one placed at 1.96 m from the model. The data acquisition was performed at 50 Hz. For the wave analysis the software WaveLab 2.94 [24] developed at Aalborg University was used by adopting Mansard and Funke method [25].

<table>
<thead>
<tr>
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<th>$H_{m0}$ [m]</th>
<th>$T_p$ [s]</th>
</tr>
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</tr>
<tr>
<td>W1a</td>
<td>0.03</td>
<td>2.07</td>
</tr>
<tr>
<td>W2</td>
<td>0.07</td>
<td>1.28</td>
</tr>
<tr>
<td>W2a</td>
<td>0.07</td>
<td>2.92</td>
</tr>
<tr>
<td>W2b</td>
<td>0.07</td>
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</tr>
<tr>
<td>W3</td>
<td>0.10</td>
<td>1.53</td>
</tr>
<tr>
<td>W3b</td>
<td>0.10</td>
<td>1.13</td>
</tr>
<tr>
<td>W3c</td>
<td>0.10</td>
<td>2.53</td>
</tr>
<tr>
<td>W4</td>
<td>0.13</td>
<td>1.79</td>
</tr>
<tr>
<td>W4b</td>
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<tr>
<td>W4c</td>
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<td>2.92</td>
</tr>
</tbody>
</table>

Table 1: Target wave attacks in the Aalborg lab.

### 3 Wave reflection at SSG device

Most of the existing literature on wave reflection from coastal structures relates the reflection coefficient $K_r$ to the surf similarity parameter $\xi$ only

$$\xi = \tan \alpha / \sqrt{H_{m0}/L_0} \quad (1)$$

being $\alpha$ the structure off-shore slope, $H_{m0}$ the significant wave height at the structure toe and $L_0$ the wave length at the toe based on the spectral wave period $T_{m0}$. Wave length is computed according to Guo formulation [26].

Among the available formulae, we recall for smooth slopes the work by [20]

$$K_r = (a_1 \xi^2)/(b_1 + \xi^2) \quad \text{with } a_1=1, b_1=5 \quad (2)$$

and the recent analysis performed by [21] on an extensive homogeneous database

$$K_r = \tanh(a \xi^b) \quad (3)$$

where $a$ and $b$ are directly dependent on the roughness factor $\gamma_l$ in the overtopping discharge formula and for impermeable slopes -as in this case- assume the values $a=0.16, b=1.43$.

In all the above equations, it is supposed to have an off-shore straight structure slope. The problem we have thus to focus one is which can be the correct way to account for a non-straight structure to define the parameter $\xi$ in Eq. (1). This problem is crucial to achieve an adequate representation of the reflection process.

The problem of identifying the correct representation of the slope in the Iribarren parameter was first analysed by [21], [27] for composite slopes and structures with berm. Main results can be summarized as follows:

- what reflects is the slope below sea water level (SWL);
- for combined slopes an average slope has to be included in $\xi$;
- wave reflection is influenced by wave breaking and run-up. The lower the run-up the greater the reflection, and the greater the energy dissipation by breaking on a berm, the lower the reflection. The presence of a toe and/or a berm should thus be accounted for whenever it may affect these processes, more specifically when the berm is placed in the run-up/down area +/-1.5 $H_{m0}$.

In the attempt to consider the presence of the berm even when it is at SWL or above it, the Authors thus suggested to use the following average structure slope:

$$\bar{\xi} = \left[ \tan \alpha_1 (h - 1.5 H_{m0}) + \tan \alpha_{inf} \cdot 1.5 H_{m0} \right] / h \quad (4)$$

$$\bar{\xi} = \tan \alpha_{inf} / \sqrt{H_{m0}/L_0}$$

where the second expression is used only when the water depth $h$ is such that $h \leq 1.5 H_{m0}$. Representation of the geometrical parameters reported in Eq. (4) is shown in the scheme in Fig. 3.

The weighted average slope in Eq. (4)

- is performed over the water depth at the structure toe $h$;
- makes use of the average slope in the whole run-up/down $\alpha_{inf}$.

Figure 4 shows the wave reflection coefficients derived from measurements at the SSG together with the database for smooth slopes by [21]. In this figure, the value of $\bar{\xi}$ is calculated based on Eq. (4).

The improvement that is obtained by adopting the average slope in Eq. (4) instead of the downstream slope $\alpha_d$ can be seen by comparing the predictions by
Eq. (2) with $\alpha_d$, Fig. 5, and by Eq. (2) with the slope provided by Eq. (4), Fig. 6.

Figure 4. Measured values of the reflection coefficient at the SSG structure (diamonds) and measured values for smooth straight slopes (circles) from the reflection database by [21]. Dashed line is Eq. (2), solid line is Eq. (3).

Figure 5. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2), being in $\xi$ the slope $\alpha = \alpha_d$.

Figure 6. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2) and Eq. (4).

Figure 7. Comparison among measured values of the reflection coefficient and predictions obtained by [21], Eq. (3) and Eq. (4).

Eq. (2) provides a much greater accuracy in the predictions. Anyway, it is worthy to note that in these tests no measurements of the roughness factor $\gamma_f$ is available, so that in Eq. (3) the standard values of $a$ and $b$ for the impermeable concrete slopes were selected.

In the case of the SSG device, wave run-up and run-down appear to be particularly relevant to the reflection processes because differently from the case of a berm, the “step” in the structure slope does not provide any dissipation. On a berm, waves usually break and dissipate so that the wider the berm the lower the reflection offered by the upper slope – and the greater the phase delay between the waves reflected from the down and upper part of the structure.
In the SSG, the ‘step’ in the slope is the mouth of the reservoir, so that part of the waves disappear into the reservoir and these waves obviously cannot produce any reflection from the upper part of the SSG slope. Moreover, the width of the reservoir mouth is not comparable, even at prototype scale, with the traditional berm width so that waves reflecting from the slope of the second reservoir are essentially in-phase with the waves reflecting from the first reservoir.

Further analysis is thus required to correctly represent the contribution to wave reflection from the SSG. Based on the observations just drawn above, it cannot be disregarded what happens in the whole run-up/down area. To this aim two contemporary measures can be adopted:

- the average structure slope including the run-up/down, i.e. Eq. (4); this slope is still adopted because includes the effect of the run-up/down but weights more what happens below SWL;
- a reduction factor for the reflection coefficient, to account for the water volume ‘lost’ inside the first reservoir, which is always placed in the run-up area

\[
R = \frac{R_1 - R_2 + HD_1 + h}{R_2 + HD_1 + h}
\]  

(5)

The performance of the formulae by [20] and [21] results very similar with this correction \( R \) in terms of rms-error, but the latter gives a much better accuracy in terms of Wilmot index \( I_w \) [28]:

\[
I_w = 1 - \frac{\sum_{i=1}^{n} (X_c - X_m)^2}{\sum_{i=1}^{n} \left| X_c - \bar{X} + X_m - \bar{X} \right|}
\]  

(6)

where \( X_c \) and \( X_m \) are the computed/estimated and measured values respectively, and the overbar denotes the average. If \( I_w \) equals 1 there is a perfect agreement among computations/estimations and measurements whereas if \( I_w \) equals 0 there is no match.

More in details:

- a rms-error of 6.6% and a \( I_w \) of 87.8% are given by Eq.s (2) and (5), with \( \xi \) estimated from Eq. (4),
- a rms-error of 6.3% and a \( I_w \) of 91.4% are given by Eq.s (3) and (5), with \( \xi \) estimated from Eq. (4).

The predictions are compared in Figures 8 and 9 for Eq.s (2) and (3) respectively. Eq. (2) with the inclusion of the reduction factor, Eq. (6), tends to provide non-cautious estimations of \( K_r \) when \( K_r \) is greater than 0.65.

So far we have focused on the prediction capacity of existing formulae only. A second but not less important aspect of this analysis consists in the consideration that wave reflection induced by the SSG is always high, not less than 50% and on average equal to 68%.

Traditional rubble mound breakwaters or breakwaters with armour units such as tetrapods, cubes, etc., usually give a 30-40% wave reflection. Wave reflection from caisson breakwaters indeed is around 45% and up to 90% so that these values are close to the ones obtained from SSG devices. In both cases of rubble mound and caisson breakwaters however proper rocky toe protections or perforated screens are designed in order to reduce wave reflection and the induced scour at the structure toe. Indeed for SSG devices it should be properly planned the placement of a toe protection. This protection has to cope with two opposing aspects. On one side, it has not to be too high, in order not to induce wave breaking and thus dissipate incident wave energy that can be transferred to run-up and then potential energy into the SSG. On the other side, it has to assure the stability of the structure by avoiding mechanisms of failure induced by the scour hole that may occur at its toe. To achieve both purposes in presence of a soft bottom may be rather difficult.

The design is indeed strongly influenced by the degree of wave reflection. The stability formulae proposed by [29], for a degree of reflection similar to that of a rubble mound breakwater, are much less...
severe than those proposed by [30] for the case of a vertical breakwater, where a toe protection block may be needed. In extreme cases, a proper excavation and reinforcement of the original bed may be necessary. A block may be required, placed over at least two layers of rubbles, with weight and perforation computed according to Tanimoto formula [31].

An ideal kind of protection may consist of geosynthetic bags placed on the bottom and filled in by sand extracted from submarine borrow areas. Such protection can be composed by few rows of bags in the cross-structure direction -ideally covering a distance of around $L_0/4$ from the toe [32]- and 2 layers of bags along the water depth, taking care of the ratio $H_m/h$.

4 The OWC device and tests

The tests were carried out in scale 1:40 in the wave tank at Wavegen laboratory. The tank is 20m long x 6m wide x 1.5m deep. It is equipped with a piston type wave generator with a stroke length of approximately 0.7 m. The wavemaker is composed by 8 independent paddles, to generate directional waves and spread/short-crested seas. However for the purposes of these tests only uni-directional waves were performed.

Bottom slope consists of ramps with different inclinations, being the average slope 1:70.

Picture and cross section of the tested OWC devices are shown in Figures 10 and 11 respectively. Two cases were tested: a device with three OWC chambers, covering a long-shore width $L_b=36$ m, and a similar device for a total $L_b=110$ m (measures at prototype scale). The structure is 15.24 m high and the chamber inlet extends up to 4.96 m from the bottom.

The OWC was fixed directly to the tank floor which has a slope of 5°. The structure, mounted on the tank centre line was not confined by any leading walls. However an artificial dissipating beach was fitted directly behind the device.

Sixteen wave attacks, consisting of 2D irregular seas with Bretschneider spectrum, were performed for both structures (see Tab. 2) for a total of 32 tests. Wave heights $H_{m0}$ were in the range 1.48-6.04 m, mean wave periods $T_m$ were in the range 6.5-15.5 s and water depth was kept constant $h=7.4$ m (at the structure toe).

Generated waves were measured with 3 resistance type wave gauges along the middle axis of the tank, the closest one placed at 3.14 m from the model (model scale measure). The data acquisition was performed at 20 Hz. Wave analysis was carried out both in time domain and in frequency domain by adopting Mansard and Funke method [25].

5 Wave reflection at OWC device

Wave reflection from the OWC device is analysed based on the most relevant parameters highlighted by previous works for vertical breakwaters, perforated caisson breakwaters and perforated screens.

For vertical breakwaters, these parameters can be the crest freeboard to incident wave height ratio $R_c/H_{m0}$ [19] and the incident wave height to water depth ratio $H_{m0}/h$ [18].

![Figure 10: Pictures of the OWC devices in the wave tank.](image)

<table>
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<tr>
<th>Wave</th>
<th>$H_{m0}$ [m]</th>
<th>$T_m$ [s]</th>
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</tr>
<tr>
<td>16</td>
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</tr>
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</table>

Table 2: Target wave attacks in the Wavegen laboratory.

For perforated caissons and screens, these parameters are respectively the chamber cross-shore width to wave length ratio $B/L_p$ and the water depth to wave length ratio $h/L_p$ [19].
Wave length is evaluated from the formulation provided by Guo [26].

It is worthy to note that in these tests $R_c/H_{m0}$ is always greater than 1.0 so that no significant effect related to overtopping can be expected (for sure when $R_c/H_{m0}\geq 2$). Moreover, both the water depth $h$ and the chamber cross-shore width $B$ are constant, so that no real effect of these parameters can be observed.

If we look at Figures 12 and 13, wave reflection from the breakwaters of different long-shore width $L_b$ show a completely different behavior.

As a general consideration, in the case of $L_b=36$ m, values of $K_r$ are much lower (range of $K_r$ between 32-39%) than in case of $L_b=110$ m (range of $K_r$ between 40-54%).

In case of the breakwater with $L_b=36$ m, $K_r$ substantially decreases with $B/L_p$ and does not show any dependence on $H_{m0}/h$ being the slight tendency of $K_r$ to increase with $H_{m0}/h$ in the order of measurement errors.

In case of the breakwater with $L_b=110$m, $K_r$ shows a mirror-like tendency, since it increases with $B/L_p$ and clearly decreases with $H_{m0}/h$, i.e. with wave breaking.

It can be also appreciated that under similar conditions the values of $K_r$ for $L_b=110$m are characterized by a much greater scatter than in case of $L_b=36$ m.

The difference in the behavior for different values of $L_b$ can be explained by two facts. The OWCs were not confined by leading walls, so that reflection from the beach can contribute to the reflection measured from the wave gauges especially in the case of $L_b=36$ m. Moreover, the wave gauges for measurements are placed quite far from the devices when $L_b=36$ m (the closer gauge is at a distance around 3 times the device width), so that the reflected wave will for sure not be a plane one and this phenomenon will be more marked the narrower the device width with respect to the wave tank width.

As a consequence, the results obtained for $L_b=36$ m can be misleading whereas the more reliable results are for $L_b=110$ m. It is indeed necessary to remark also in this case that the absence of leading walls may lead to a partial contribution from the dissipating beach that should reduce the global value of $K_r$ (this consideration is based on the available results in presence of the beach only).

The dependence of $K_r$ on $B/L_p$ for the OWC device in Fig. 12 cannot be compared to the analysis carried out for perforated caissons with a single porous screen found by [19] under irregular head-on waves. In fact, the OWC porosity is much greater then the maximum screen porosity considered by the Authors (25%).

If one decides to relate $K_r$ essentially to wave length, i.e. to $B/L_p$, polynomial functions of the fourth order (solid lines in Fig. 12) can be obtained merely by data approximation. It is given in the following the expression for $L_b=110$ m only due to the restrictions applied to the dataset and discussed above.

![Figure 11: Cross-shore scheme of the OWC device. Measures in mm at laboratory scale.](image1)

![Figure 12: Dependence of the measured reflection coefficient on the chamber width to wave length ratio.](image2)

![Figure 13: Dependence of the measured reflection coefficient on the incident wave height to water depth ratio.](image3)
The reflection coefficient $K_r$ for both OWC and SSG devices is never lower than 40% and can rise up to 90%. It is consequently a significant design issue to construct a proper toe protection layer avoiding scour holes at the structure toe and consequent possible structure failure by sliding. It is generally recommended an in depth analysis of the sea bottom, the excavation and coverage with geotextile in case of very fine sand and clay, and the construction of a stable protection with rocks or geosynthetic bags.

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References


