

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 harbours 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 coastal 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.

Nomenclature

B	= cross-shore width of the OWC reservoir
h	= water depth at the structure toe
H_{m0}	= significant wave height at the structure toe
HD	= width of the SSG reservoir mouth
K_r	= reflection coefficient
L_o	= wave length based on spectral wave period T_{m-1}
L_p	= wave length based on peak wave period T_p
R	= reduction factor
R_c	= freeboard of the crest of the OWC device or of the SSG reservoirs
T_m	= average wave period
T_p	= peak wave period
α_d	= down slope
α_{inc}	= slope in the run-up/down
γ_f	= roughness factor in the overtopping discharge
ζ_o	= surf similarity parameter based on L_o

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 Wageningen 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).

The majority 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.

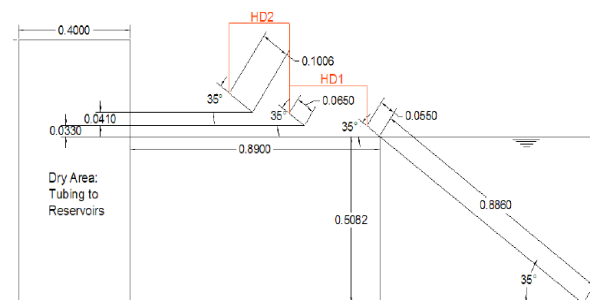


Figure 2: Sketch of the SSG, measures in m, side view.

Tested wave attacks (see Tab. 1) were 2D irregular waves with Jonswap 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].

Wave	H_{m0} [m]	T_p [s]
W1	0.03	1.02
W1a	0.03	2.07
W2	0.07	1.28
W2a	0.07	2.92
W2b	0.07	0.92
W3	0.10	1.53
W3b	0.10	1.13
W3c	0.10	2.53
W4	0.13	1.79
W4b	0.13	1.31
W4c	0.13	2.92

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 ξ only

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

being α 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 $T_{m-1,0}$. Wave length is computed according to Guo formulation [26].

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

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

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

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

where a and b are directly dependent on the roughness factor γ_f 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 ξ , 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 ξ ;
- 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 $\pm 1.5 \cdot H_{m0}$.

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:

$$\xi_o = \frac{[\tan \alpha_d \cdot (h - 1.5H_{m0}) + \tan \alpha_{incl} \cdot 1.5H_{m0}]/h}{\sqrt{H_{m0t}/L_0}} \quad (4)$$

$$\xi_o = \tan \alpha_{incl} / \sqrt{H_{m0}/L_0}$$

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

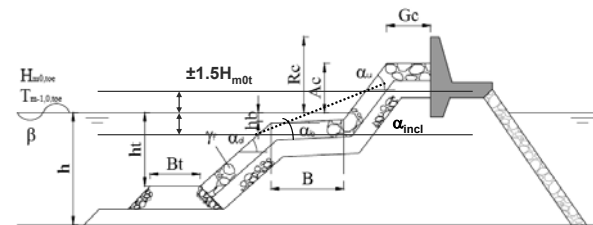


Figure 3. Structure parameters in the database, based on CLASH schematization, from [27].

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 α_{incl} .

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 ξ is calculated based on Eq. (4).

The improvement that is obtained by adopting the average slope in Eq. (4) instead of the downstream slope α_d can be seen by comparing the predictions by

Eq. (2) with α_d , Fig. 5, and by Eq. (2) with the slope provided by Eq. (4), Fig. 6.

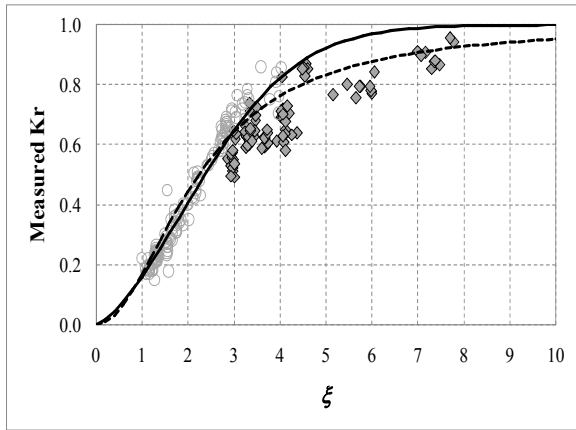


Figure 4. Measured values of the reflection coefficient at the SSG structure (diamonds) and measured values for smooth straight slopes (circles) from the reflection database by [21]. Dashed line is Eq. (2), solid line is Eq. (3).

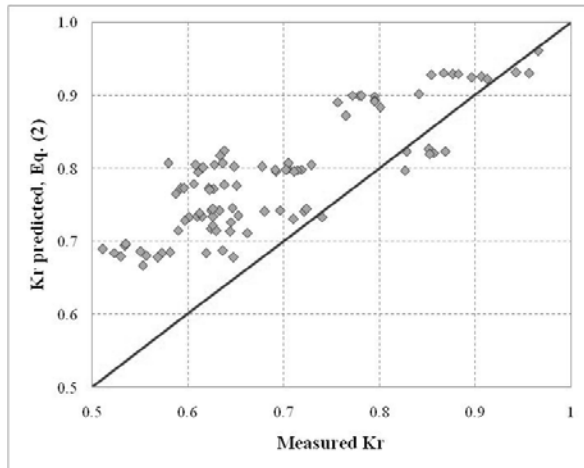


Figure 5. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2), being in ξ the slope $\alpha = \alpha_d$.

Figure 7 shows the predictions that can be achieved by coupling Eq. (3), with the weighted average slope, Eq. (4). The prediction capacity of Eq. (3) in Fig. 7 is thus directly comparable with Eq. (2) in Fig. 6. The overall performance of the formulae, Eqs (2) and (3), is synthetically described below in terms of the percentage rms errors

- 11.6% rms-error given by Eq. (2) where ξ is evaluated based on $\alpha = \alpha_d$
- 8.6% rms-error given by Eq. (2) where ξ is evaluated based on Eq. (4)
- 16.9% rms-error given by Eq. (3) where ξ is evaluated based on $\alpha = \alpha_d$
- 12.6% rms-error given by Eq. (3) where ξ is evaluated based on Eq. (4)

Both Eqs (2) and (3), but especially Eq. (3) tend to overestimate the measured values whatever is the expression adopted for the surf similarity parameter. The overall performance of both formulae is

significantly improved by using Eq. (4). So the importance for reflection process of what is happening in the run-up/down area is confirmed.

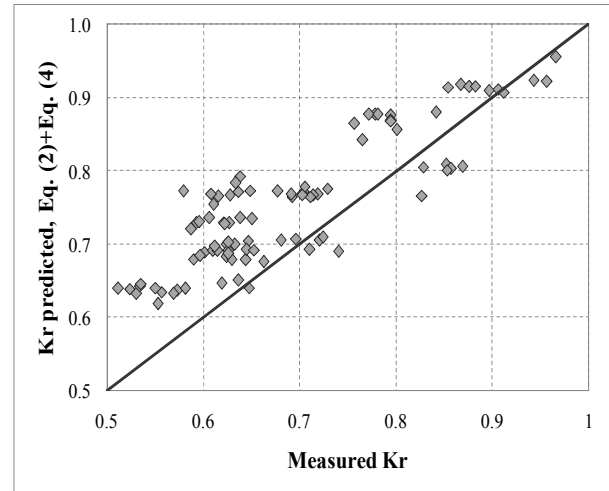


Figure 6. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2) and Eq. (4).

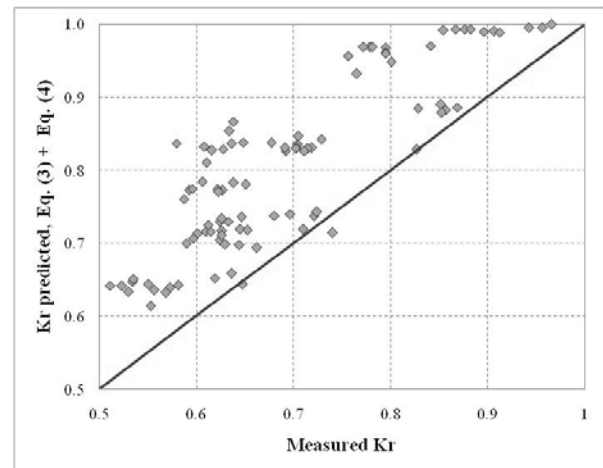


Figure 7. Comparison among measured values of the reflection coefficient and predictions obtained by [21], Eq. (3) and Eq. (4).

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 γ_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

$$R = \frac{R_{c1} - R_{c2} + HD_1 + h}{R_{c1} + HD_1 + h} \quad (5)$$

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]:

$$I_w = 1 - \frac{\sum_{k=1}^N (Xc_k - Xm_k)^2}{\sum_{k=1}^N \left[|Xc_k - \overline{Xm}| + |Xm_k - \overline{Xm}| \right]^p} \quad (6)$$

where Xc and Xm 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 ξ 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 ξ 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.

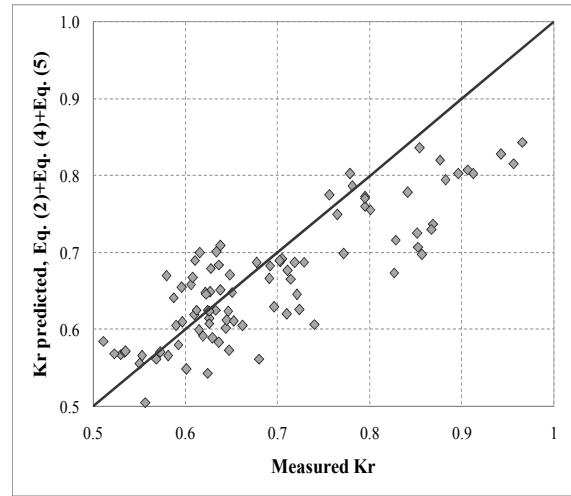


Figure 8. Comparison among measured values of the reflection coefficient and predictions obtained by [20], Eq. (2) and Eq. (4) corrected by the reduction factor R expressed by Eq. (5).

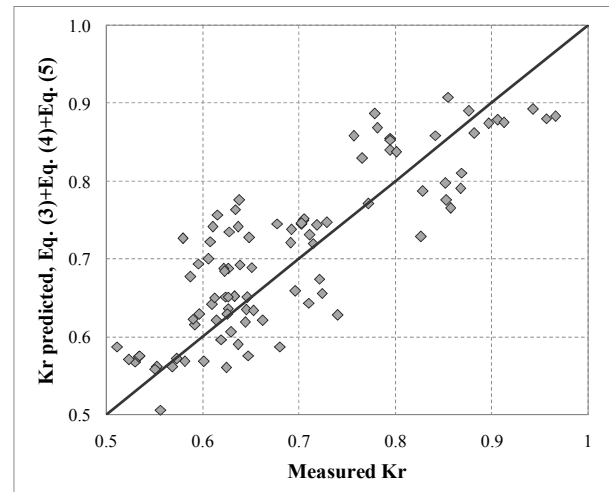


Figure 9. Comparison among measured values of the reflection coefficient and predictions obtained by [21], Eq. (3) and Eq. (4) corrected by the reduction factor R expressed by Eq. (5).

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_{m0}/h .

4 The OWC device and tests

The tests were carried out in scale 1:40 in the wave tank at Wavengen 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].

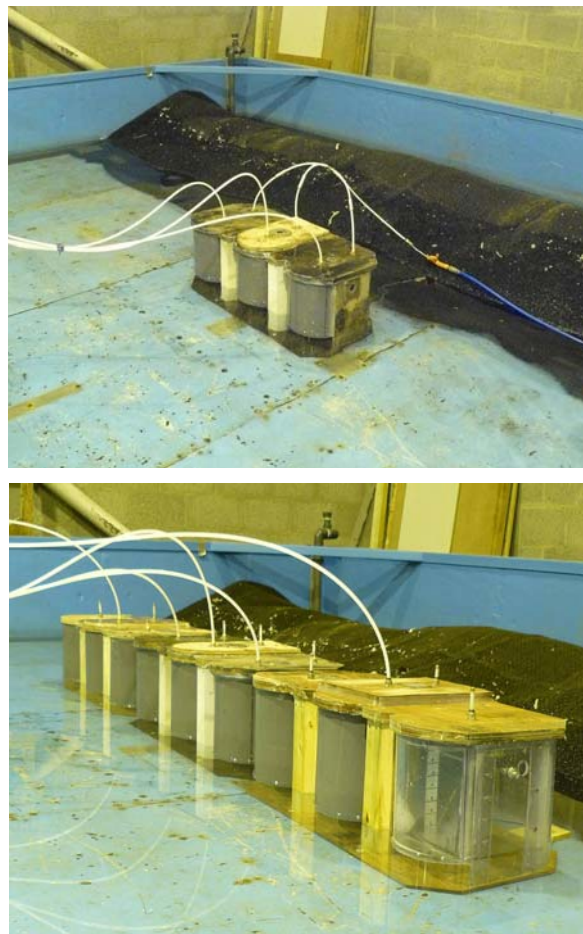


Figure 10: Pictures of the OWC devices in the wave tank.

Wave	H_{m0} [m]	T_m [s]
1	1.48	6.5
2	1.57	7.5
3	1.79	8.5
4	1.52	9.5
5	1.84	10.5
6	2.18	11.5
7	2.43	12.5
8	2.74	13.5
9	2.96	14.5
10	4.81	15.5
11	3.38	9.5
12	4.08	10.5
13	4.80	11.5
14	5.34	12.5
15	5.96	13.5
16	6.04	14.5

Table 2: Target wave attacks in the Wavengen 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.

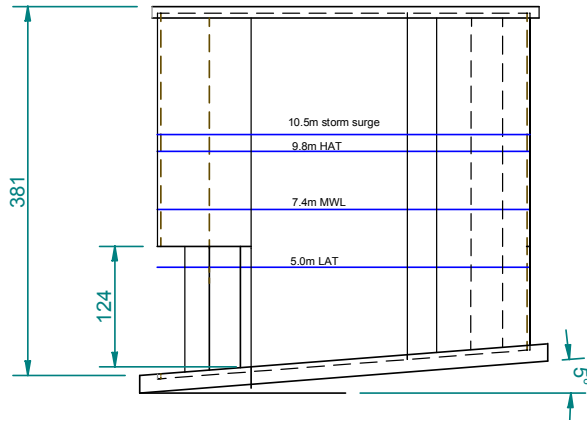


Figure 11: Cross-shore scheme of the OWC device. Measures in mm at laboratory scale.

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).

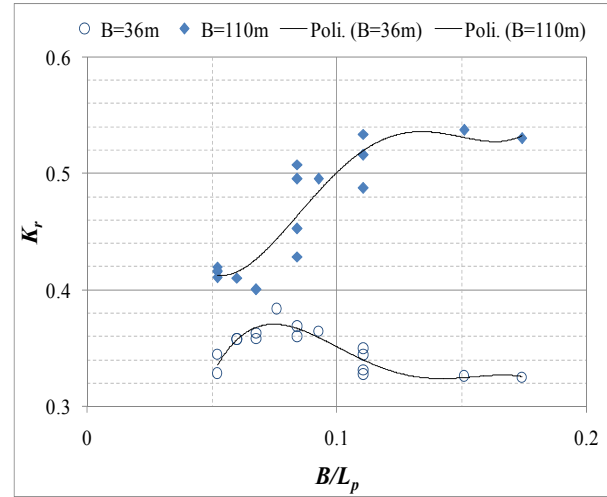


Figure 12: Dependence of the measured reflection coefficient on the chamber width to wave length ratio.

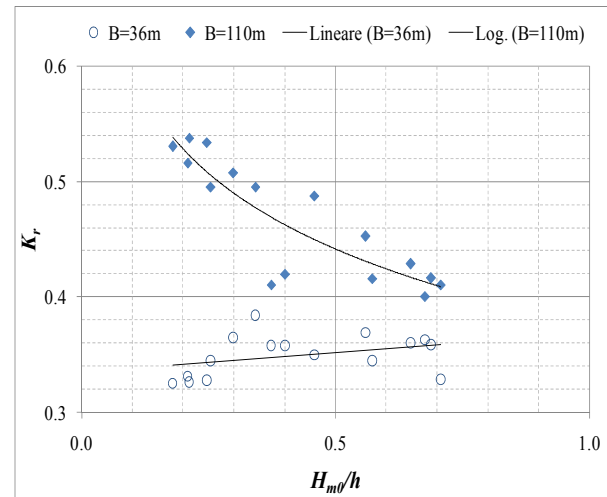


Figure 13: Dependence of the measured reflection coefficient on the incident wave height to water depth ratio.

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 than 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

$$K_r = 5204 \cdot \left(\frac{B}{L_p}\right)^4 - 2434 \cdot \left(\frac{B}{L_p}\right)^3 + 393 \cdot \left(\frac{B}{L_p}\right)^2 - 24 \cdot \left(\frac{B}{L_p}\right) + 0.9$$

This simplified fitting (cut at the first figure after the dot) provides a representation of K_r with rms-errors of 1.8% and 2.6% respectively, but obviously do not lead to any general prediction capacity.

The values of the reflection coefficient provided by the OWC device are much lower (never exceeding 55%) than the ones obtained from the tests on the SSG device previously presented (always exceeding 50%). It is worthy to note that the results from the tests are not directly comparable, since the facilities are different and in particular one is a wave tank and the other one is a wave flume, i.e. 3D effects can take place.

Indeed also for OWC devices it should be properly planned the placement of a toe protection. The toe protection has to cope with the design of the OWC and thus with the position of the chamber inlet that is very close to the sea bottom.

In order not to oppose to waves, the protection should be placed within the bottom in front of the structure. It can be prepared as follows: the sandy bottom below the structure should be replaced with suitable material to reduce settlement. The excavation at the structure toe, running parallel to structure length L_b should extend in the off-shore direction for a distance of around $L_0/4$ from the toe [32]. Note that the crest of the toe protection has to be aligned with the sea bottom level.

Geotextile can be lied down, to separate the sand from two layers of gravels/rocks, properly sized for stability of the toe protection layers [30]. Further, hydraulic stability may be investigated being relevant to avoid the possible movement of the rocks.

6 Conclusions

This paper analysed wave reflection from a SSG and an OWC device, based on 95 and 32 tests in a wave flume and in a wave tank respectively.

In case of the SSG device, available existing formulae from coastal experience [20], [21] provide sufficiently accurate predictions of K_r , provided that a proper representation is given to the structure slope and that a proper reduction factor is introduced.

The best agreement between measurements and predictions can be obtained by using Eq. (3) with the inclusion of an average structure slope, Eq. (4), that accounts of the slope in the run-up/down area and with the inclusion of an original reduction factor developed specifically for the SSG, Eq. (5).

In case of the OWC device, the available experimental data allowed to highlight the dependence of K_r on wave length L_p : with increasing L_p , K_r decreases, being constant the chamber cross-shore width B . More tests are needed to check the effects of the chamber width B to wave length ratio (resonance problems) and of the water depth to chamber height ratio (change of structure porosity).

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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