2DV RANS-VOF NUMERICAL MODELING OF
A MULTI-FUNCTIONAL HARBOUR STRUCTURE

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This paper is focused on the analysis of a multifunctional structure developed by the Second University of Naples, named OBREC, which is an Overtopping Breakwater for Energy Conversion. The hydraulic and structural performance are evaluated by means of the 2DV numerical model IH-2VOF developed by the University of Cantabria, in terms of average discharge rate, wave reflection coefficient and pressures acting on the structure. The results are compared with the laboratory experiments carried out at Aalborg University (Denmark) and with recent formulae and a new Artificial Neural Network. Furthermore, the numerical model is used to obtain information related to the wave loadings where experimental data were not available. This numerical analysis is a useful support to the ongoing monitoring of the prototype installation in the port of Naples.

*Keywords: multi-functional structure; breakwater; wave energy; OBREC; wave overtopping; pressures*

# INTRODUCTION

The development of installations aimed to exploit marine energy has been subject of an increasing interest in the recent years. However, the so called “Blue Growth” is dependent on the reliability of the devices and on the economic feasibility of the installations. To overcome the existing technological barriers and minimize conflicts, the concept of multi-purpose platform was developed for offshore cases (Zanuttigh et al. 2016a; Azzellino et al., 2013). Offshore installations maximize the energy harvesting with respect to near-shore installations; however, their distance from shore affects the costs of energy transfer. Therefore, attractive solutions may be onshore wave energy converters integrated in coastal or harbor protection such the REWEC3 (Boccotti et al. 2007) and the Sea-wave Slot-cone Generator (Vicinanza and Frigaard, 2008; Buccino et al., 2015a; Buccino et al., 2015b).

Starting from this concept, an innovative solution has been developed by the Second University of Naples (Vicinanza et al., 2013a; Vicinanza et al. 2014; Contestabile et al., 2015; Contestabile et al. 2016; Contestabile et al, 2017a; Contestabile et al, 2017b) and consists of an Overtopping BReakwater for Energy Conversion, OBREC hereafter (Figure 1), which combines harbor protection and energy production. This structure represents a modification of a traditional rubble mound breakwater, where the crest is replaced with a reservoir designed to capture the overtopping waves and produce electricity. The energy is extracted via low head turbines, using the difference between the reservoir and the mean sea water levels. Besides the production of wave energy, the presence of the top basin reduces the wave overtopping discharge (Van Doorslaer et al. 2015) in extreme events, improving the harbor safety (Cappietti and Aminti, 2012).

The aim of this contribution is to assess the OBREC hydraulic and structural performance, by means of numerical modelling. Several analyses are performed according to different wave conditions, structural modifications and breakwater geometries with the purpose to increase the exportability of the OBREC installation.

The structure of the paper is as follows. The laboratory campaigns are first synthesised, in terms of tested structures/wave conditions, measurements and main outcomes. Then the numerical modelling with the IH-2VOF code is presented, including the numerical set-up and results, such as the overtopping discharge rate inside the reservoir *qreservoir*, the reflection coefficients *Kr* and pressures *p* acting on the OBREC device. The numerical results are compared with theoretical formulations and with the results derived by the application of new Artificial Neural Network (Zanuttigh et al. 2016b, Formentin et al. 2017). A final paragraph draws the main conclusions of the work.

# Laboratory investigations

Two laboratory campaigns were carried-out at Aalborg University (Denmark) in 1:30 scale, in 2012 and 2014 (Contestabile et al. 2016a), respectively. The tests were performed in the wave flume, which was 25 m long, 1.50 m wide and 1.20 m deep, and included ordinary and extreme wave conditions.

In both the 2012 and the 2014 campaigns, the wave series were irregular and generated based on the 3 parameters of the JONSWAP spectrum, by defining the wave height *Hm0*, the frequency *fp* and the so-called peak enhancement factor *γ* (*γ* = 3.3 in all tests). Each test contained at least 1000 waves. The tests are synthetized in Table 1, where the results of the 2012 campaign are reported according to the kind of wave condition, while the 2014 tests are divided based on the geometric configuration.

## Tested configurations

The OBREC structure is a modification of a traditional rubble mound breakwater, provided with a concrete reservoir placed over the crest aimed to capture the overtopping waves. Figure 1 shows the cross section of the laboratory models, with the indications of all the main geometric characteristics, tested during the two laboratory campaigns. The common characteristics of the configurations are:

* the average size of the rocks (in terms of nominal diameter *Dn50*), e.g. *Dn50* = 50 mm for the armour layer, *Dn50* = 20 mm for the filter layer, *Dn50* = 2 mm for the core part;
* the OBREC offshore slope **equal to (armour and plate), with the exception of the 2014 curved configuration (see Fig. 1d), where the sloping plate is characterized by two slope angles, e.g. 52° and 17° in the upper part.

The first test campaign (AAU2012) was aimed to compare and evaluate the difference of the hydraulic performance (Vicinanza et al. 2014), between the OBREC and a traditional rubble mound breakwater with a crown wall on the top. This latter physical model was already tested by Nørgaard et al. (2013). A total of 48 tests (summarized in Tab. 1) were carried-out, considering two structures, which differ only for the height of the sloping plate, e.g. *dw,low* = 0.075 m and *dw,high* = 0.125 m, at model scale (Fig. 1a and b, respectively). The laboratory structure width at the bottom is 2.56 m, whereas the width of the reservoir is *Br* = 0.6 m. For the extreme conditions, a special configuration provided with a parapet (named *nose*), placed on top of the crown wall, was tested to reduce the overtopping discharge at the rear side of the crown wall, e.g. the *qrear* (Van Dooslaer and De Rouck, 2010). The 2014 configurations were then all designed with such a parapet (as shown in Fig. 1c and d), because of its effectiveness.

The laboratory tests of the second campaign (AAU2014) were focused on the influence of some geometrical parameters on the hydraulic performance, such as the horizontal reservoir width and the sloping plate shape and length. As already anticipated, two configurations were investigated:

* a flat profile with a slope angle equal to 34°, according to the research conducted by Kofoed (2006), aimed to maximize the overtopping discharge *qreservoir*;
* a curved sloping plate, where the slope angle varies linearly between 52° to 17°, which represents an adaptation from the convex profile tested by Kofoed (2002).

A submerged prolongation of the sloping plate was introduced with respect to the 2012 configuration, to improve the overtopping process. The reservoir width *Br*, e.g. the horizontal distance between the crown wall and the beginning of the sloping plate *Br* (see Fig. 1c and d), was set equal to: 0.10 m, e.g. small configuration; 0.20 m, e.g. large configuration; 0.30 m, e.g. extra-large configuration.

A total of 200 tests were carried-out, whose main characteristics are synthesised in Table 1. Preliminary results have been already presented by Iuppa et al. (2016).

 

**b)**

**a)**



**d)**

**c)**

Figure 1. OBREC configurations during the 2012 campaign (a, b) and the 2014 campaign (c, d).

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| Table 1: Main wave and geometrical characteristics of the laboratory campaigns, at model scale. |
|  | h [m] | Hm0 [m] | Tm-1,0 [s] | Rc [m] | Rr [m] | Br [m] |
| 2012 | (min–max) | (min–max) | (min–max) | (min–max) | (min–max) | (min–max) |
| Extreme conditions | 0.30-0.34 | 0.141-0.177 | 1.68-2.26 | 0.20-0.24 | 0.075-0.125 | 0.415-0.488 |
| Extreme conditions withnose | 0.34 | 0.145-0.161 | 1.66-2.28 | 0.20 | 0.035-0.085 | 0.415-0.488 |
| Production conditions | 0.27 | 0.037-0.138 | 1.05-2.14 | 0.27 | 0.105-0.155 | 0.415-0.488 |
| 2014 |  |  |  |  |  |  |
| Small structure | 0.27-0.35 | 0.02-0.12 | 0.76-2.2 | 0.147-0.227 | 0.045-0.129 | 0.219-0.460 |
| Large structure | 0.27-0.35 | 0.05-0.13 | 0.76-2.2 | 0.147-0.227 | 0.045-0.129 | 0.219-0.460 |
| Extra-Large structure | 0.27-0.35 | 0.05-0.118 | 0.76-2.2 | 0.147-0.227 | 0.045-0.129 | 0.219-0.460 |

## Measurements

The sensors along the wave flume and across the structure were used to obtain:

* the wave reflection from the structure;
* the pressures acting on the OBREC cross section; and
* both the average overtopping discharges at the front reservoir *qreservoir* and inshore the crown wall *qrear*.

The water overtopping the reservoir was controlled by depth gauges, which activated the pumps to allow the water discharging from the reservoir above a fixed threshold water level. The wave volumes overtopping the crown wall were collected into a box inshore the structure, where a similar control of the water discharge was performed by means of depth gauges. In both cases, the values of *qreservoir* and *qrear* were reconstructed by the combination of the signals acquired from the depth gauges and the pumps.

The wave reflection coefficient *Kr*was derived from 4 wave gauges positioned in front of the structure, according to Klopman and Van der Meer (1999) recommendations.

In the 2012 campaign, 3 and 6 pressures transducers were installed in the *dw,low*and the *dw,high*configurations respectively; 5 transducers were placed across the reservoir outside bottom, to evaluate the uplift pressure and 17 on the upper/lower crown wall.

In the 2014 campaign, a total of 14 pressure transducers were used to estimate the pressures/forces induced by the waves on the structure. Specifically, 5 pressure transducers were located along the sloping plate, 2 across the reservoir outside bottom, 5 on the lower/upper crown wall and 1 on the parapet.

## Main experimental results from both the laboratory campaigns

The OBREC is characterized by similar or reduced values of *Kr* with respect to traditional rubble mound breakwaters. The inclusion of the submerged part of the sloping plate in the 2014 design improves the overtopping rates, while increasing *Kr*. This latter aspect can be also justified by the absence of the berm, according to the results obtained by Zanuttigh et al. (2009).

The *qreservoir* can be roughly approximated by the formula for dikes by EurOtop (2016), with a friction reduction factor *f* = 0.7.

The selection of the best profile of the sloping plate should be further investigated to balance the energy production and the safety level of structure, e.g. reducing the *qrear*.

To ensure similar safety level of traditional breakwaters, the OBREC has to be provided with a parapet capable to reduce the average rear overtopping discharge *qrear* up to the 80% with respect to the original cross section without parapet.

Based on these experimental results the OBREC device led to the integration of a new functionality into an existing or new breakwater, without compromising its primary function of harbor defense.

# NUMERICAL MODELLING

The IH-2VOF code, a 2DV RANS-VOF code developed by the University of Cantabria (Losada et al. 2008), was used to model the OBREC device. The numerical modelling focused on the 2012 campaign reproducing both configurations, e.g. *dw,low* and *dw,high* (see Fig. 2a). The *dw,high* was also examined without the berm (see Fig. 2b), similarly to the 2014 campaign. All the geometries were tested under ordinary and extreme wave conditions.

## Numerical model set up and tests

In the numerical model, some changes to the original OBREC cross section were needed to assure model stability and correct representation of the physical processes:

* to allow the emptying of the reservoir, a pipe was introduced between the reservoir and the area landward the structure, while in the physical model the overtopping discharge was pumped–out;
* to avoid numerical instabilities, the space between the plate and the reservoir was filled-in and the thickness of the upright section was increased.

The representation of the OBREC porous layers implied the definition of several parameters, which characterize the permeable layers, such as the porosity *n*, the linear friction coefficient **, the non-linear friction coefficient **, the added mass coefficient *cA* and the nominal diameter *Dn50*. The sensitivity to the change of these parameters, which were set from the literature (Van Gent 1995, Lynett 2000, Hsu 2002), is shown for *qreservoir* in the following Section.



**a)**



**b)**

Figure 2. Schematization of the OBREC in the numerical model: a) with berm, and b) without berm.

The tested wave conditions are reported in Table 2, together with thefreeboard crest of the sloping plate *Rc*. The numerical wave series were implemented by defining the wave height *Hm0*, the peak period *Tp*, the dispersion factor related to the JONSWAP spectrum *γ*, the water depth at generation *h*, the duration of the simulation *t* and the frequency *f*. The tests included at least 500 waves, which were found sufficient to perform statistical wave overtopping analysis (Romano et al. 2014). The tests 1.4.1 and 1.5.1 represent the extreme conditions, while the other tests correspond to the ordinary conditions to assess the structural and the hydraulic performance, respectively.

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| Table 2. Characteristics of the tested wave conditions e relative freeboard crest *Rc*. |
| Test | Hm0 [m] | Tm-1.0 [s] | h [m] | Rc [m] |
| 1.4.1  | 0.188  | 1.811  | 0.34  | 0.035 |
| 1.5 1 | 0.193  | 2.23  | 0.34  | 0.085 |
| 2.1.4 | 0.069 | 1.529 | 0.27 | 0.105 |
| 2.1.5 | 0.069 | 1.327 | 0.27 | 0.105 |
| 2.1.6 | 0.064 | 1.092 | 0.27 | 0.105 |
| 4.1.5 | 0.068 | 1.327 | 0.27 | 0.155 |
| 4.1.10 | 0.132 | 2.090 | 0.27 | 0.155 |
| 4.1.11 | 0.132 | 1.796 | 0.27 | 0.155 |
| 4.1.12 | 0.132 | 1.554 | 0.27 | 0.155 |

As for the laboratory campaigns, several wave gauges are installed inside the numerical flume to evaluate *Kr*, *qreservoir* and the pressures acting on the OBREC device.

The wave reflection coefficient *Kr* is derived from the 4 wave gauges located in front of the structure.

The value of *qreservoir* is computed thanks to a gauge placed on top of the sloping plate (see Fig. 3b), by integrating (along the vertical) cell by cell the horizontal velocity component multiplied by the cell height.

The numerical pressure transducers are placed along the structure in the same position as in the laboratory (Fig. 3). However, in the numerical model, the pressure transducers 13, 12, 11 and 10 (Fig. 3) can be used to evaluate both the uplift and the downward pressures.

 

**b)**

**a)**

Figure 3. Water gauges across the structure: a) laboratory model, b) numerical model. The two cross sections have the same scale highlighting the necessary modifications to the numerical scheme.

## Wave reflection

The results of the wave reflection analysis are reported in Table 3, where the laboratory and the numerical results are compared with the theoretical predictions by Zanuttigh and Van der Meer (2008):

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|  |  | (1) |

Equation (1) can be applied to straight slope data in design conditions, e.g. *Rc*/*Hm0* ≥ 0.5, *Hm0*/*Dn50* ≥ 1.0, *s0* ≥ 0.01, under perpendicular wave attack. Equation (1) can be also applied to the structures with berm provided that the slope included in the calculation of the Irribarren and Battjes parameter is the average slope *incl* in the run-up/run-down area, e.g. ± 1.5 *Hm0* with respect to the still water level. The slope *incl* is here calculated based on the update approach proposed by EurOtop (2016):

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|  |  | (2) |

where in this case , *B* is the horizontal extension of the berm and *hb* its submergence with respect to the still water level. The berm falls below the run-down for the tests characterized by the smallest *Hm0*, e.g. 2.1.4, 2.1.5 2.1.6 and 4.1.5 (in Table 2), and therefore the average slope *incl* is indeed the slope of the sloping plate *off*. The Irribarren and Battjes surf similarity parameter *ξ0* is computed as:

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|  |  | (3) |
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The values of the coefficients *a* and *b* were set equal to 0.12 and 0.87, e.g. rubble mound breakwaters. The prediction formula overestimates the values of *Kr* for the tests where the structure is schematised as a straight slope breakwater, e.g. the first 4 in Table 3, while it tends to slightly underestimate those ones affected by the effectiveness of the berm, e.g. the last 3 tests in Table 3.

 The numerical model systematically overestimates the experimental results and the deviation is on average the 35%. This result can be partially justified by the increased length of the sloping plate in the numerical model, with respect to the experiments. It can also be partially explained by the need to optimize the model calibration for both *qreservoir* and *Kr*.

The absence of the berm produces an increase of *Kr* of the 26% on average, as it could have been expected based on the results by Zanuttigh et al. (2009). This previous numerical work showed that the structures with a submerged berm were characterized by a lower *Kr* than the corresponding straight slopes. A toe protection is therefore recommended especially if the OBREC is installed in a breakwater without berm.

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| Table 3. Laboratory and predicted Kr vs. numerical model results, for the configurations with and without berm. Results for all the ordinary tests in Table 1. |
|  | 2.1.4 | 2.1.5 | 2.1.6 | 4.1.5b | 4.1.10b | 4.1.11b | 4.1.12b |
| Laboratory | 0.24 | 0.21 | 0.18 | 0.23 | 0.43 | 0.36 | 0.27 |
| Zanuttigh et al. | 0.45 | 0.40 | 0.35 | 0.40 | 0.34 | 0.30 | 0.27 |
| Mod. With berm | 0.45 | 0.40 | 0.31 | 0.39 | 0.58 | 0.52 | 0.46 |
| Mod. Without berm |  / |  / |  / | 0.53 | 0.69 | 0.63 | 0.58 |

## Wave overtopping discharge

The model calibration was carried-out to optimize the representation of both *Kr* and *qreservoir*. Table 4 reports, for a specific test, the numerical results *qreservoir,n* obtained by changing the porosity values (Palma et al. 2016), and keeping constant the other material parameters (** = 1000, ** = 1.1, 1.0 and 0.8 for the armour layer, the filter layer and the core, respectively).

Figure 4 shows the values of *qreservoir,n* compared with the corresponding experimental ones *qreservoir,e* and the design formulae related to sloping structures (*qreservoir,p*), developed by Van der Meer and Bruce (2013) and adopted by the EurOtop (2016), e.g. eq. 1 and 2:

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| --- | --- | --- |
|  |  | (4) |

with the maximum:

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| --- | --- | --- |
|  |  | (5) |

In the equations above, *off* represents the offshore slope angle, *b* is the berm influence factor, *f* is the roughness influence factor, *b* the oblique wave attack influence factor, ** is the influence factor for a vertical wall and *m-1,0* is the breaker parameter and *Rr* is the front wall freeboard crest.

The lower the values of *qreservoir*, the better the agreement among experiments, numerical and theoretical predictions (eq. (2) and (3)). With increasing *qreservoir*, the numerical results are closer to the experiments than the predictions, as it was expected considering that the formulae are essentially based on traditional structures and the calibration of *f* is questionable (Vicinanza et al. 2014). The values of *qreservoir* increases for the OBREC layout compared to a traditional breakwater with similar overall dimensions, since the offshore rock slope is partially replaced by a concrete sloping plate (Vicinanza et al. 2014).

Table 5 reports the comparison of *qreservoir* between the configurations with and without berm. The OBREC performance remains constant, at least for the tests characterized by the greater discharge rates. In presence of the berm the dissipation by breaking is higher, but the values of *Kr* are lower than without the berm, leading to similar *qreservoir*.

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| Table 4. Laboratory vs. model overtopping discharges obtained by varying the porosities assigned to the layers (Test 2.1.5). |
| Configuration | Armour | Filter | Core | qreservoir,e [l/s/m] | qreservoir,n [l/s/m] |
| 1 | 0.8 | 0.7 | 0.6 | 0.046 | 0.073 |
| 2 | 0.7 | 0.6 | 0.05 | 0.046 | 0.056 |
| 3 | 0.6 | 0.05 | 0.04 | 0.046 | 0.006 |
| 4 | 0.7 | 0.05 | 0.04 | 0.046 | 0.004 |



Figure 4. Computed (numerical and predicted by formulae) non-dimensionalised values of *qreservoir,n/p* vs. laboratory non-dimensionalised values of *qreservoir,e*, for the ordinary wave conditions in Table 2.

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| Table 5. Comparison of the *qreservoir* values [l/s/m] between the configurations with and without berm. |
| Test | With berm | Without berm |
| 4.1.5b | 0.0038 | 0.0020 |
| 4.1.10b | 0.96 | 0.89 |
| 4.1.11b | 0.62 | 0.59 |
| 4.1.12b | 0.41 | 0.40 |

## Pressures across the structure

The pressure analysis is focused on the OBREC performance in extreme conditions (Tests 1.4.1, e.g. *dw,low*configuration and 1.5.1 in Table 2, e.g. *dw,high*configuration). The experimental and numerical pressures are reported in terms of *p250*,which corresponds to the non-exceedance level of about 99.7%.

During the extreme tests, the lab equipment was insufficient to pump-out the water from the reservoir. Therefore, in the numerical modelling, two schematizations have been proposed to analyse the loads acting on the plate and across the reservoir for *dw,low*configuration (Tables 6 and 7):

* the same cross section modelled during the calibration (Model 1 in Tables 6 and 7). This scheme is closer to the real prototype configuration, which was tested in more recent experiments (Contestabile et al. 2016).);
* the structure with a closed reservoir as in the laboratory experiments (Model 1 in Tables 6 and 7). The two Models 1 and 2 were considered to schematise the best and the worst case respectively, e.g. the full or partial incapacity of the reservoir to let the water flowing out.

Table 6 reports the pressures acting on the sloping plate and the uplift ones related to the reservoir. Model 2 gives a better estimation of the experiments than Model 1, both for the plate and the reservoir. In case of the uplift pressures, Model 1 overestimates the statistical values due to the presence of the pipe that decreases the porous part of the structure. Inside the reservoir, Model 2 gives higher statistical values of the downward pressures than Model 1, as it is expected since the overtopping waves cannot be partially discharged through the pipe. As aforementioned, no direct comparison of numerical versus experimental data inside the reservoir is possible, however the dynamics reproduced by Model 2, e.g. the overtopping waves hitting the full reservoir, should be more similar to the prototype operating in extreme conditions and therefore represents cautious conditions.

According to the results for *dw,low*configuration, Model 2 was selected to test also the case of *dw,high*, with and without a berm. Tables 8 reports the experimental and numerical pressures acting along the sloping plate, on the reservoir (e.g. uplift pressures) and at the crown wall. Model 1 gives a good estimation of the experimental values, mainly in the lower part of the plate. The absence of the berm does not change the general trend, leading to slightly higher statistical values, as for the downward pressures inside the reservoir (Table 9). In both Models, the uplift *p250* are well estimated. The discrepancy among the numerical and experimental pressure at the crown wall increases from the bottom to the top.

As already discussed in Contestabile et al. (2016), the modelling of complex structures, such as the OBREC device, is not always sufficient to obtain the complete and accurate description of its structural response. In this study, the dynamics related to violent wave impacts with very short duration may be strongly affected by the compressibility of the air pocket. The air entrainment process was not examined in details in the lab and is not reproduced by this version of the IH-2VOF. Therefore, the actual values of the pressures at prototype conditions may substantially differ from the measurements and from the computations. The dynamics inside the reservoir and on the wall will be further investigated thanks to the monitoring of the pilot installed in 2016 in the port of Naples.

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| Table 6. Laboratory (Lab) vs. numerical p250, values in kPa. The numbers correspond to the gauges in Figure 3. Results for Test 1.4.1. |
| With berm 2012 | 10 | 11 | 12 | 6 | 7 | 8 | 9 |
| Lab | 1.12 | 1.54 | 1.40 | 1.86 | 1.47 | 1.33  | 1.13 |
| Model 1 | 1.43 | 1.20  | 0.82  | 2.32 | 2.23 | 2.13 | 2.02 |
| Model 2 | 1.54 | 1.32 | 1.00 | 2.24 | 2.08 | 1.91 | 1.72 |

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| Table 7. Numerical downward p250, values in kPa. The numbers correspond to the gauges in Figure 3. Results for Test 1.5.1. |
| With berm 2012 | 6 | 7 | 8 | 9 |
| Model 1 | 1.23 | 1.22 | 1.17 | 1.86 |
| Model 2 | 1.36 | 1.46 | 1.60 | 1.92 |

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| Table 8. Laboratory (Lab) vs. numerical uplift pressures p250, values in kPa. The numbers correspond to the gauges in Figure 3. Results for Test 1.5.1. |
| Model 2 | 10 | 11 | 12 | 13 | 14 | 15 | 6 | 7 | 8 | 9 | 1 | 2 | 3 | 4 |
| Lab | 1.66 | 1.54 | 1.44 | 1.45 | 1.82 | 1.96 | 2.09 | 1.89 | 1.84 | 1.52 | 2.75 | 2.67 | 2.70 | 1.67 |
| With berm | 1.68 | 1.50 | 1.30 | 1.09 | 1.01 | 0.58 | 2.30 | 2.13 | 1.95 | 1.75 | 2.18 | 1.77 | 1.52 | 0.97 |
| Without berm  | 1.69 | 1.53 | 1.35 | 1.12 | 1.07 | 0.62 | 2.28 | 2.11 | 1.94 | 1.74 | 2.11 | 1.72 | 1.32 | 0.91 |

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| Table 9 Numerical downward p250, values in kPa. The numbers correspond to the gauges in Figure 3. Results for Test 1.5.1. |
| Model 2 | 6 | 7 | 8 | 9 |
| With berm | 1.71 | 1.60 | 1.80 | 2.06 |
| Without berm | 1.73 | 1.60 | 1.83 | 2.07 |

# Comparison with other numerical approaches

The laboratory and model values of *qreservoir* and *Kr* are here compared with the results obtained with the Artificial Neural Network (ANN) adopted by EurOtop (2016) and developed by Zanuttigh et al. (2016) and Formentin et al. (2017). This tool is able to deal with a variety of wave attacks and complicated structure geometries through the use of 15 physical parameters describing the structure cross-sections and the hydraulic conditions. Figure 5a reports the general schematization of the structures - that was partially derived from the CLASH project (Van der Meer et al., 2009) - and the main parameters adopted by the ANN tool. The full list of the 15 parameters of the ANN is given in both the above-mentioned works of the authors. The ANN is here used to derive Kr and qreservoir.

The OBREC structural features have been schematized according to the parameters of Fig. 5a, resulting in the final layout shown in Figure 5b. By comparing Fig. 5b with the original configurations of the lab and the model cross-sections (Fig. 1a-b and 2a-b, respectively), it can be observed that the scheme provided to the ANN (Fig. 5b) includes some simplifications of the shape of the sloping plate and of the reservoir. These elements are indeed too peculiar to be represented by means of the 15 parameters of the ANN tool. Moreover, in order to be consistent with the lab measure of *qreservoir*, the presence of the crown wall and of the reservoir have been neglected, leading to the following adaptations:

* the maximum structure emergence comprehensive of the crown wall (*Rc*) results equal to the value of the crest emergence (*Ac*), see Fig. 5a;
* the crest width *Gc* is set equal to 0;
* the values of roughness factor (*γfd*) and the mean size of the structure elements (*Dd*) in the run-down area (i.e. within +1.5*Hm0,t* below the still water level) are respectively set to 0.4 and 0.05 m to represent the two layers of rocks;
* the values of roughness factor (*γfu*) and the mean size of the structure elements (*Du*) in the run-up/down area have been computed on the basis of a weighted average of the *γf* and *D* values characterizing rock slope (*γf* = 0.4 and *D =* 0.05 m) and the impermeable sloping plate (*γf* = 1 and *D =* 0 m), similarly to the approach for incl in eq. 1. The resulting values of the *γf* and *D* for the 7 tests are resumed in Table 10. Note that for the tests 4.1.10b, 4.1.11b and 4.1.12b, that are characterized by a higher sloping plate (see Fig. 1b) involving the whole run-up area, the average *γf,u* and *D* are respectively equal to 1 and 0.

The other structural elements such as the foreshore, the toe and the berm are included in the general schematization shown in Figure 5a.

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| Table 10. Values of roughness factors (*γf*) and of the mean sizes of the structure elements (*D*) characterizing the OBREC slope in the run-up (subscripts ‘u’) and run-down (subscripts ‘d’) area.  |
| Test | *γf,d* [-] | *γf,u* [-] | *Dd* [m] | *Du* [m] |
|  2.1.4  | 0.40 | 0.63 | 0.05 | 0.03 |
|  2.1.5 | 0.40 | 0.63 | 0.05 | 0.03 |
|  2.1.6 | 0.40 | 0.63 | 0.05 | 0.03 |
|  4.1.5b | 0.40 | 0.63 | 0.05 | 0.03 |
|  4.1.10b | 0.40 | 1 | 0.05 | 0 |
|  4.1.11b | 0.40 | 1 | 0.05 | 0 |
|  4.1.12b | 0.40 | 1 | 0.05 | 0 |



**a)**

**b)**



Figure 5. a) General schematisation of the structure based on CLASH. b) OBREC schematisation to evaluate qreservoir.

The results of the application of the ANN tool to the OBREC device are numerically reported in Table 11 in terms of *qreservoir*, and graphically shown in Figures 4 and 6, in terms of *qreservoir* and *Kr*, respectively.

For each test, Table 11 compares the lab values of *qreservoir* with the corresponding predictions obtained with the ANN, the numerical model and the EurOtop (2016) formulae. In order to ease the quantitative analysis of the performance of the three methods, the Table includes, for each method, the average values of the standard deviation *σ* and of the coefficient of determination *R2* computed between measurements and predictions. The numerical values of both the error indices reveal that the ANN predictions are accurate and on average the most precise with respect to the other methods.

As for Kr, Figure 6 indicates that the ANN tool gives a general overestimation of a factor 2 of the experimental values when *Kr,e* < 0.3, while it provides accurate predictions for the larger *Kr,e* (tests 4.1.10b, 4.1.11b). The ANN tends therefore to overestimate *Kr* for the configurations with the submerged berm as well as the numerical model, see Tab. 10.

|  |
| --- |
| Table 11. Experimental (Lab), numerical (IH-2VOF) and predicted (EurOtop) values of *qreservoir* vs. ANN results. The values are in [m3/s/m]. |
| Test | Lab  | **ANN** | IH-2VOF | EurOtop (2016) |
|  2.1.4  | 1.22E-03 | 8.98E-04 | 1.34E-03 | 4.82E-03 |
|  2.1.5 | 8.02E-04 | 5.54E-04 | 1.12E-03 | 4.85E-03 |
|  2.1.6 | 4.36E-04 | 2.26E-04 | 0.00E+00 | 3.68E-03 |
|  4.1.5b | 6.90E-05 | 1.04E-04 | 4.64E-05 | 6.51E-04 |
|  4.1.10b | 1.47E-02 | 1.05E-02 | 1.03E-02 | 1.11E-02 |
|  4.1.11b | 1.05E-02 | 7.34E-03 | 6.78E-03 | 1.12E-02 |
|  4.1.12b | 5.90E-03 | 4.91E-03 | 4.81E-03 | 1.12E-02 |
| R2 |  | 0.86 | 0.83 | - |
| σ |  | 0.0020 | 0.0022 | 0.0052 |



Figure 6. Computed values of *Kr* (numerical, predicted formulae and by the ANN, respectively *Kr,n, Kr,ANN and Kr,p*)vs. laboratory values of *Kr (Kr,e)* for the ordinary wave conditions in Table 2.

# CONCLUSIONS

This paper presents an overview of the two laboratory campaigns, carried-out at Aalborg University, and a numerical analysis, performed with the 2DV RANS-VOF code IH-2VOF, of the multifunctional harbour structure OBREC. The hydraulic and the structural performance of the OBREC with respect to a traditional rubble mound breakwater is evaluated by means of ordinary and extreme tests.

The IH-2VOF model can be used to extend the experimental database and to provide indications for design optimization. Particular attention should be paid to the representation of the flow motion through thin adjacent permeable and impermeable layers.

The model is calibrated to represent two OBREC configurations with berm, which differ only for the height of the sloping plate (*dw,low* and *dw,high*). The calibration (Palma et al. 2016) is carried-out by comparing the laboratory and the numerical values of the average wave overtopping discharge inside the reservoir (*qreservoir*) and of the wave reflection coefficients (*Kr*), under ordinary wave conditions. The *dw,high*configuration is then tested without the berm to analyse the change of the OBREC performance.

The numerical values of *qreservoir* are compared with the EurOtop (2016) formulae and a new artificial neural network (ANN) recently developed by Zanuttigh et al. (2016). The numerical model and the ANN give a better estimation of the experimental results, especially for the greater discharges, than the theoretical formulae developed for traditional breakwaters. The ANN tool provides good results even if the schematisation of the peculiar OBREC cross section, according to the reference CLASH structure, needed some adaptations.

The laboratory values of *Kr* are compared with the numerical results, the predictions of the ANN and the theoretical results derived from the formula by Zanuttigh and Van der Meer (2008). The numerical model systematically overestimates the experiments of the 35% on average. Both the formula and the ANN predictions overestimate the values of *Kr* when the berm falls below the run-down area.

The OBREC structural performance is evaluated in extreme conditions, by comparing the numerical and the experimental results in terms of statistical values of pressures, e.g. *p250*. Numerical simulations provide information on loads acting on different part of the structure also where no experimental data are available (a.o. downward pressures in the reservoir). The uplift values of *p250* in the reservoir and in the lower part of the sloping plate are well represented, if the structure is modified by closing the reservoir to reproduce the full-section operating condition of the reservoir during extremes. The laboratory pressures at the crown wall are underestimated and the discrepancy increases from the wall bottom to the top, may be due to violent wave impacts with very short duration, which may be strongly affected by the compressibility of the air pocket. The air-entrainment phenomenon is not reproduced by this version of the IH-2VOF and will be further investigated thanks to the monitoring of the pilot installed in 2016 in the port of Naples.

The performance of the *dw,high*configuration without berm is compared the corresponding case with the berm, leading to similar results with the exception of the wave reflection coefficients. The absence of the berm causes an increase of *Kr* of the 26% on average. This result is coherent with the study performed by Zanuttigh et al. (2009), in which the structures with a submerged berm were characterized by a lower *Kr* than the corresponding straight slopes. A toe protection is therefore recommended especially if the OBREC is installed in breakwaters without berm.

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