



Design of an overtopping wave energy converter: the role of the seaward outflow

Saeed Osouli^{a,b,*}, Ivan Sabbioni^b, Maurizio Brocchini^a, Gianluca Zitti^a,
Matteo Postacchini^a

^a Department of Ingegneria Civile, Edile e Architettura, Università Politecnica delle Marche, Ancona, Italy

^b Mentucci Aldo Costruzioni, Senigallia, Italy

ARTICLE INFO

Keywords:

OBREC
Numerical analysis
Hydraulic performance
Hydrokinetic turbine
Renewable energy
Conveyance system

ABSTRACT

Overtopping Breakwaters for Energy Conversion (OBREC) typically route overtopped water into a basin landward of the structure, protected from large sea-level fluctuations. Considering that numerous coastal structures suitable for OBREC integration are incompatible with the construction of a protected basin, we study an alternative configuration that releases the stored water seaward, introducing the influence of the open sea on the available hydraulic power along the conveyance system. The approach aims to expand OBREC applicability to natural shorelines by enabling installation on shore-connected breakwaters, structures that are widely deployed, for example, along the Italian coast of the Adriatic Sea. The study presents an integrated design workflow, including offshore and nearshore wave characterization, port-scale wave modelling, ramp slope, device orientation, and turbine assessment, to characterize the wave structure interaction and performance of OBRECs integrated into typical Adriatic coastal protections. This framework serves to optimize a full-scale prototype that will be deployed at the Port of Ancona, Italy. In particular, the effect of wave conditions and related sea-level fluctuation on the extractable power is examined, showing that for seaward discharge without waves and landward discharge, the highest mean energy is available next to the outlet discharge system, where flow velocities are maximal. On the other hand, for seaward discharge with waves and increased sea-level, the location of maximum available energy shifts progressively toward the basin, with a significant reduction in available power up to about 90%.

1. Introduction

Ocean waves represent a high-density renewable energy source, characterized by predictable patterns and minimal propagation losses. However, the exposure of wave energy converters (WECs) to large and variable environmental forces makes them still not economically competitive with conventional energy sources. Nevertheless, the co-location of overtopping WECs with necessary harbor or coastal defence infrastructure can offset initial investment costs, rendering wave energy a more economically viable and environmentally sustainable alternative (Vicinanza et al., 2014) and (Iuppa et al., 2016). However, such an integrated installation requires a comprehensive assessment of the hydrodynamic interaction between overtopping WECs and the local wave conditions they experience along breakwaters, under both calm and storm conditions. To this end, a key objective of this study is to investigate how outlet hydrodynamic conditions affect the

efficiency of overtopping breakwaters for energy conversion (OBREC). The conventional OBREC concept limits its application to detached breakwaters, where overtopped water can be discharged into the protected area. Its use on shore-connected breakwaters would significantly expand the potential installation over longer sections and reduce overall costs, but this requires seaward discharge configuration.

OBREC systems perform best under weak (nearshore wave power < 4 kW/m) and mild wave climates (4–10 kW/m) (Contestabile et al., 2017a; Cavallaro et al., 2020; Contestabile et al., 2020). This suggested the installation of OBRECs in naturally sheltered coastal areas, such as breakwaters designed to protect seaports. In this context, this paper focuses on the design of a real-world OBREC application in Ancona, Italy, which is exposed to an average wave energy of approximately 1.8 kW/m (Vicinanza et al., 2011). Fig. 1 shows the OBREC configuration object of this study. Unlike conventional systems, which discharge water only toward the protected area, the OBREC considered

* Corresponding author at: Department of Ingegneria Civile, Edile e Architettura, Università Politecnica delle Marche, Ancona, Italy.

E-mail address: s.osouli@pm.univpm.it (S. Osouli).

<https://doi.org/10.1016/j.apor.2026.105254>

Received 28 April 2026; Received in revised form 1 September 2026; Accepted 2 September 2026

Available online 7 September 2026

0141-1187/© 2026 The Author(s). Published by Elsevier Ltd. This is an open access article under the CC BY-NC-ND license (<http://creativecommons.org/licenses/by-nc-nd/4.0/>).

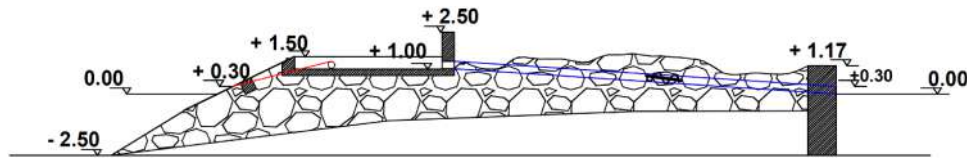


Fig. 1. OBREC's configuration.

here includes two discharge systems: a traditional conveyance system directed landward (blue line in the figure) and an additional pipe discharging seaward (red line). Since the monthly average power output of the OBRECs has been found to be highly irregular (Cavallaro et al., 2020), the analysis encompasses calm and storm conditions, reflecting the operational limits of the device. Special emphasis is placed on the challenges of seaward water discharge during extreme events, identified as a critical threshold for WEC operability.

Several numerical and experimental studies have investigated the performance of OBREC systems. For instance, the IH2VOF numerical model has been used to evaluate their efficiency, showing that long waves are minimally affected and that total forces are slightly reduced compared to vertical-faced breakwaters (Di Lauro et al., 2020). Optimization approaches, such as the Harmony Search Algorithm, have been applied to structural parameters including reservoir width and crest freeboard under different wave and wind conditions (Kralli et al., 2019), often combined with Kaplan turbines and EurOtop formulations. However, further validation is still required before these approaches can be reliably applied in practice (Darma et al., 2026). Even if different studies have addressed prediction methods, geometry optimization, and hydraulic and structural performance of overtopping-based energy converters (Musa et al., 2020; Cavallaro et al., 2020; Barbosa et al., 2019). To the best of our knowledge, this is the first study to investigate how downstream outlet boundary conditions redistribute the hydraulic power available along the conveyance system of a seaward-discharge OBREC and how this redistribution affects turbine placement. This study addresses this gap by evaluating OBREC configurations installed on shore-connected breakwaters, with particular focus on seaward discharge and its effect on the redistribution of available hydraulic power along the conveyance system.

To provide a comprehensive analysis of the system, the implementation of an Archimedes screw turbine within the OBREC system is also assessed, as this turbine type is highly compatible with adverse conditions—such as the presence of sediments—and several studies show relevant efficiency at low flow rates (Aref et al., 2023).

The paper is organized as follows. Section 2 presents the materials and methods, including the comparison between model and buoy data, the selection of representative wave conditions, and the transfer of offshore waves to the nearshore through analytical and numerical modelling. Section 3 describes the CFD simulations of ramp overtopping and the fluid–structure interaction of the Archimedes turbine. Section 4 presents the results, focusing on ramp geometry, turbine performance, and the available power within the two discharge pipes, which represents the main contribution of this work. Finally, Section 5 summarizes the main findings and discusses their implications for energy performance and site-specific deployment implications, particularly in the context of Italian coastal infrastructure.

2. Materials and methods

This study employs a model chain to design an OBREC, as illustrated in Fig. 2. The process begins with wave forcing characterization, based on observed and modelled dataset, which are propagated toward the breakwater. Analytical and numerical models for detailed hydrodynamic analysis of the OBREC system, concerning a) the overtopping evaluation; b) the flow and available power within the conveyance system for different outlet conditions; c) numerically predicted turbine

performance, comparing the constant head configuration with the semi-submerged conditions.

2.1. Observed and model data

In the vicinity of the breakwater where the OBREC installation is planned, both in-situ buoy measurements and modelled data from the Copernicus Marine Service dataset are available. The spectral significant wave height (H_{m0}) and spectral wave period ($T_{m-1,0}$) from the Copernicus Mediterranean Sea Waves Reanalysis product were used for comparison with observed data. Observations from the Italian National Wave Buoy Network,¹ specifically the buoy significant wave height (H_s), mean wave period (T_m), and average wave direction, were analysed. The selected time series spans the period from 2021–07–01 00:00 to 2023–12–28 13:30. The buoy observations were collected at 30-minute intervals, whereas the Copernicus model outputs were provided hourly. To enable direct comparison, reported in Fig. 3, the Copernicus data were linearly interpolated to match the temporal resolution of the buoy records.

Wave roses are also presented in Fig. 4 to further compare the directional characteristics of the two datasets.

Figs. 3 and 4 show that, at the buoy location, the Copernicus data agree well with the buoy observations for significant wave height ($R^2=0.813$), show moderate agreement for mean wave period ($R^2=0.577$), and show poor agreement for wave direction (linear RMSE = 122.2° ; angular RMSE based on the shortest directional difference = 55.7°). This is confirmed by the comparison of wave roses. The buoy wave rose shows three dominant wave approach sectors: NNE (north-northeast), ESE (east-southeast), and NNW (north-northwest),² with the latter being underrepresented in the Copernicus wave rose. However, due to the local coastal geometry and port breakwaters, waves from ESE cannot propagate effectively into the port; therefore, only the NNE and NNW directions were considered in the subsequent analysis. (Fig. 7, top panel)

2.2. Data selection

To define the hydrodynamic simulation scenarios, two alternative orientations for the wavemaker boundary are established, which are consistent with the principal wave directions identified in the previous section. For each orientation, two wave conditions were investigated: a frequent scenario, with probability density $p = 0.05$, and a severe scenario, with probability density $p = 0.005$, corresponding to those adopted by Jäger et al. (2019). The first scenario is representative of an operational condition and could be used to evaluate the long-term operational performance of the OBREC, while the second will be used to simulate the OBREC seaward discharge in extreme wave condition. This choice reflects the objective of the analysis, which is to investigate conditions that maximize the influence of waves on the seaward discharge and, consequently, on the hydraulic power available along the conveyance system. The severe wave condition is expected to provide the maximum outlet backpressure on the internal hydraulics of the

¹ <https://www.mareografico.it/en/homepage.html>.

² These abbreviations denote the principal directions from which waves approach the study area.

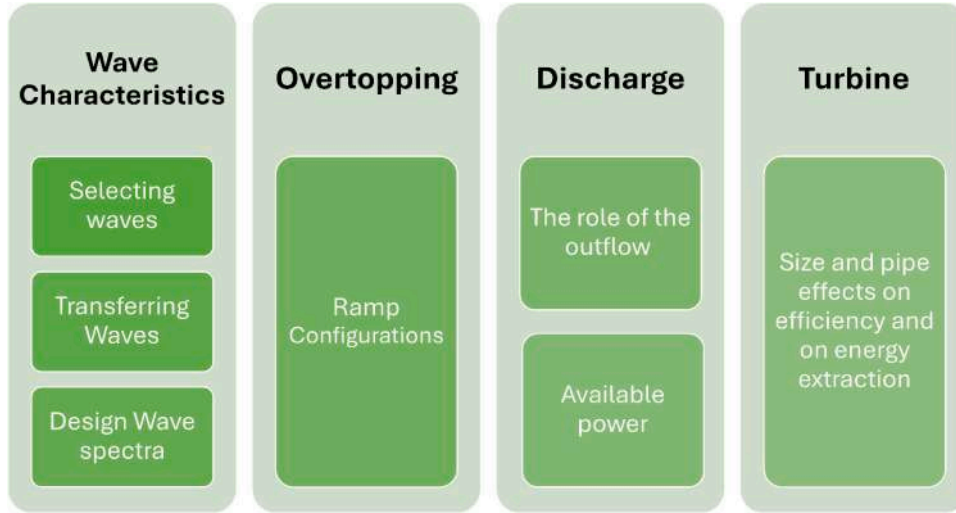


Fig. 2. Workflow of this research.

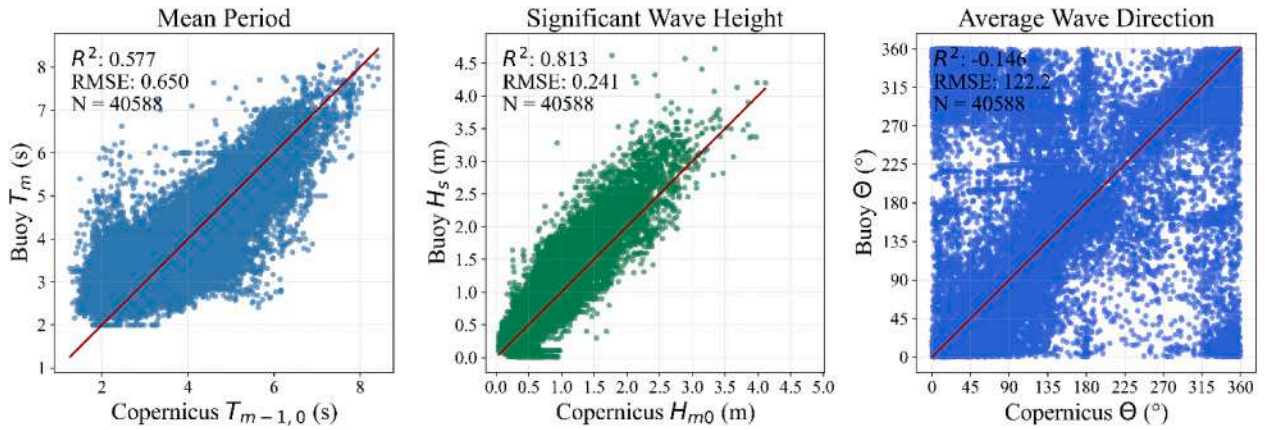


Fig. 3. Comparison of wave parameters from buoy observations and Copernicus wave reanalysis data.

conveyance system. From this perspective, the lower-probability scenario is more suitable for highlighting the maximum wave-induced effects on the system behaviour. Therefore, the conveyance-system results should be interpreted as a severe-condition assessment of power redistribution along the pipe, rather than as a probability-weighted estimate of long-term operational energy production.

The probability-weighted annual energy production will be investigated in future work once the prototype becomes available.

The bivariate distributions, shown in Fig. 5, illustrate the joint probability density function (PDF), estimated using a Gaussian kernel density estimation (Scott, 2015), for the comparison of wave data from both the buoy (blue) and Copernicus (red) (Serinaldi et al., 2022). The comparison demonstrates that while the model-derived data provide a reasonable representation of offshore wave conditions for all scenarios of interest, they consistently underestimate the observational data in terms of both mean and extreme wave conditions. Considering this comparison, buoy data were used to select the wave characteristics for wave events, which are summarized in Table 1. The wave direction relative to the North is given by the angle α and the parameter $(\alpha_p)_0$ denotes the angle of incidence of the deepwater waves relative to the wavemaker boundary (see Fig. 7, top panel).

2.3. Wave transfer from deep to intermediate waters

Wave transfer from deep to intermediate waters was assessed using Goda's method (Goda, 2010; Go10 hereafter), and the results are shown

in Table 2. The parameter $\alpha_{p,h1}$ represents the angle of refracted waves to be used as a boundary condition for the wave-resolving numerical model FUNWAVE-TVD (Shi et al., 2011), as discussed below. This methodology requires T_p , the wave period corresponding to the frequency f_p at the spectral peak ($T_p = 1/f_p$), which was estimated from T_m as $T_p \cong 1.18T_m$, in agreement with Go10. Table 2 also shows the wave steepness, estimated as $H_s/L_{p,0} = 2\pi H_s/(gT_p^2)$, where $L_{p,0}$ is the deep-water peak wavelength based on T_p (van der Meer et al., 2018). For the wave scenarios considered, this parameter ranges between 0.025 and 0.03.

The wave height in shallow water can be calculated as:

$$H_s = K_r \{ \min[(\beta_0 H_{s,0} + \beta_1 h), \beta_{max} H_{s,0}, K_s H_{s,0}] \} \quad (1)$$

Where β_0 , β_1 and β_{max} are parameters depending on both deep-water wave steepness $H_{s,0}/L_{p,0}$ and seabed slope i . K_r and K_s are refraction and shoaling coefficients, respectively.

The wave incidence angle at the target location, θ , is also a function of the incident deep-water angle, θ_0 , and the relative water depth, $h/L_{p,0}$.

$$\theta = f(\theta_0, h/L_{p,0}) \quad (2)$$

Wave processes in shallower waters, including diffraction caused by the breakwaters in the Port of Ancona, were considered using a Boussinesq-type model, as discussed in the next section.

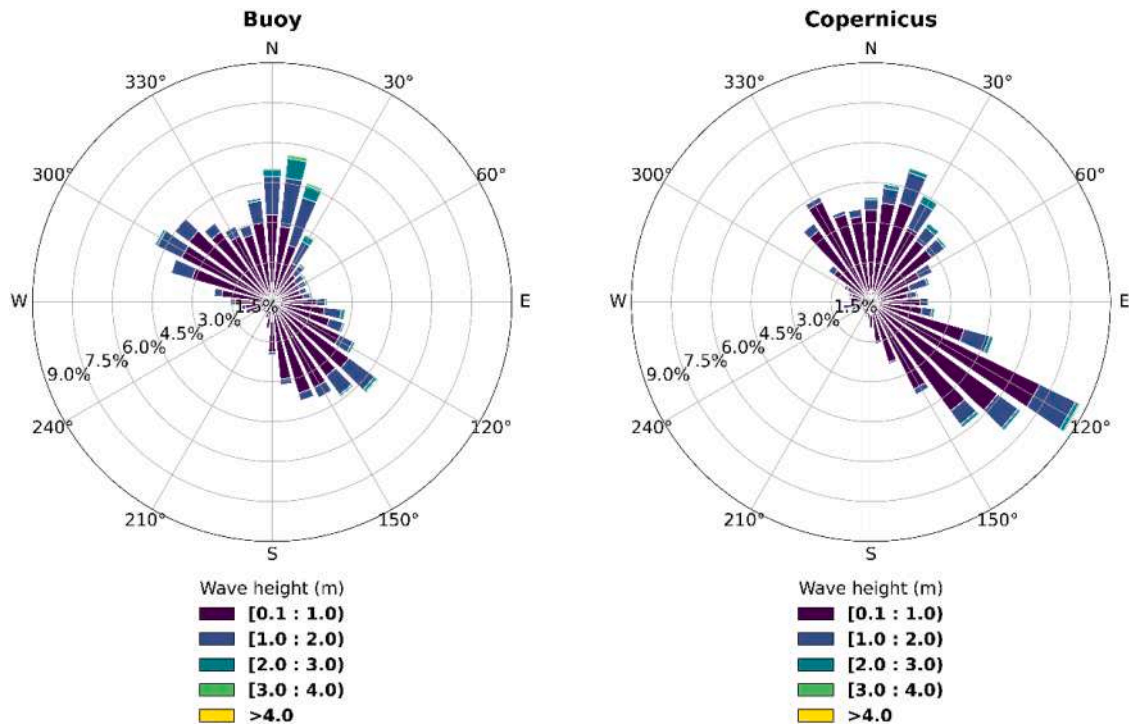


Fig. 4. Wave roses derived from buoy observations (left) and Copernicus wave reanalysis data (right), showing the directional distribution of wave occurrence and significant wave height at the buoy location. The comparison is used to identify the dominant wave approach sectors considered in the subsequent analysis.

2.4. Wave transfer to the port area

The wave circulation within the port area was described using a 2DH model, while more computationally expensive CFD-based models were used for the detailed analysis of the near-turbine flows.

2.4.1. FUNWAVE-TVD model

The wave-resolving FUNWAVE-TVD model was used to simulate waves in intermediate to shallow water. It is based on Boussinesq-type equations, resulting in a 2DH model; the vertical structure is therefore not directly resolved, but approximated (Shi et al., 2011). One of the essential inputs for the model is the bathymetry file, which was constructed by merging two data sources: a high-resolution bathymetric survey of the port area and data from the European Marine Observation and Data Network (EMODnet³). The resulting bathymetry and the potential deployment location of the device within the Port of Ancona are shown in Fig. 6 and Fig. 7, respectively. Green-to-yellow triangles denote bathymetric data outside the harbour obtained from EMODnet, while red-to-blue dots indicate depth measurements obtained from the port survey.

Fig. 7 (bottom panel) shows the numerical domain and the location of numerical gauges, while Table 3 provides the coordinates of the gauges within the domain. Gauge G7 is located near the potential site of the device. The coordinate system is defined such that the origin (0,0) is located at the lower-right corner of the numerical domain.

FUNWAVE-TVD simulations were performed on a system equipped with an AMD Ryzen 9 5950X processor (16 cores, 32 threads) and 64 GB of RAM. The outputs were saved every 0.067 s (15 Hz), and the first 3000 time steps, corresponding to 201 s, were excluded from the power spectral density (PSD) analysis to remove transient effects. The water depth at the wavemaker was set to 16 m, and the grid resolution was 2 m in both directions. Periodic boundary conditions with 60 m-wide sponge layers were applied at both lateral boundaries and at the seaward side of

the wavemaker.

2.4.2. Numerical wave propagation

The wave characteristics reported in Table 2 were used as wave-maker boundary conditions for FUNWAVE-TVD. The number of incident waves needed to achieve a statistically reliable overtopping analysis was chosen based on the available literature. Vicinanza et al. (2014) and Contestabile et al. (2017b) used approximately 1000 incident waves, and similarly, Di Lauro et al. (2019, 2020) reported test durations equivalent to 500–1000 waves to obtain representative time series and reliable spectra. On the other hand, Romano et al. (2015) and Palma et al. (2019, 2020) showed that around 500–600 waves are generally sufficient for statistical analyses of wave overtopping. Based on these references, 500 incident waves were used, providing a balance between computational efficiency and statistical reliability.

Fig. 8 (bottom panel) shows H_s for different wave scenarios simulated with FUNWAVE-TVD. An increase in wave height is observed after Gauge 4 ($X = 1200$ m), due to wave shoaling followed by breaking. The top panel of Fig. 8 shows the variation of the peak wave period T_p along the transect. Overall, in all simulated scenarios, T_p exhibits only minor changes, with a slight decrease as the waves propagate toward the OBREC location.

In Fig. 9, snapshots are presented for different scenarios after a simulation time of $t = 3000$ s, corresponding to several hundred peak wave periods depending on the wave condition (approximately 380–460 peak wave periods). At around $X = 1400$ m, areas of the detached breakwater that are less sheltered by the emerged breakwaters, exhibit flow patterns suitable for device placement. These results highlight the influence of the breakwaters on wave circulation within the port and in front of the OBREC.

Fig. 10 shows the results of the four wave scenarios, corresponding to the two main wave directions defined in Table 2 at Gauge 4. For the frequent scenarios ($p = 0.05$), the incident wave characteristics of the NNW direction are significantly less energetic than those from the NNE direction (see Figs. 8 and 10). However, both cases exhibit significant wave height smaller than 0.5 m near to the shoreline, which is

³ <https://emodnet.ec.europa.eu/geoviewer/>.

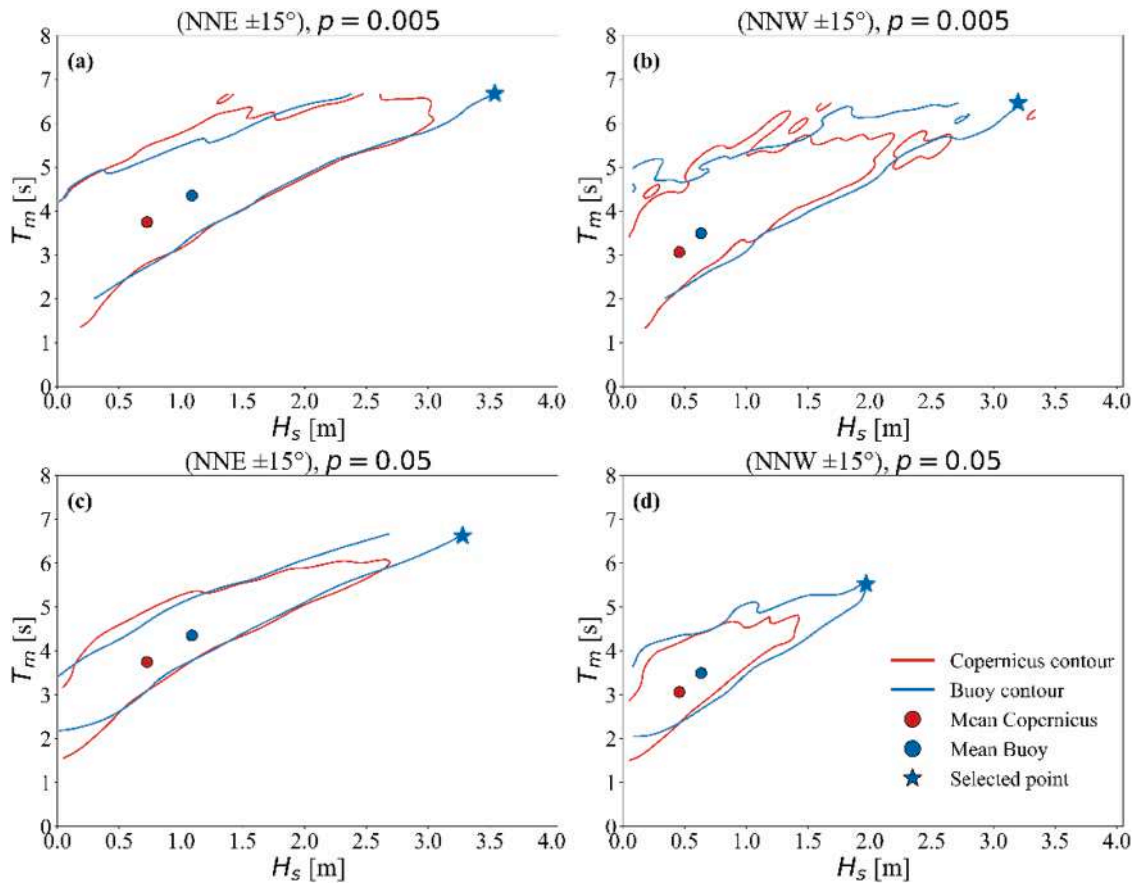


Fig. 5. Bivariate density contour lines of T_m and H_s corresponding to $p = 0.005$ (a and b) and $p = 0.05$ (c and d), illustrating the joint probability distribution of mean wave period and significant wave height at the buoy site (blue) and the corresponding Copernicus location (red). Mean values from both datasets (circles) are indicated, and the buoy data selected for subsequent analyses is marked with a star. (For interpretation of the references to color in this figure legend, the reader is referred to the web version of this article.)

Table 1

Selected wave characteristics at the buoy location for the dominant NNE and NNW wave directions. The table reports the significant wave height, mean wave period, wave direction relative to north, and incident angle relative to the numerical-domain boundary for two probability-density levels, $p = 0.05$ and $p = 0.005$.

		Density $p = 0.05$	Density $p = 0.005$
NNE	H_s (m)	3.28	3.54
	T_m (s)	6.62	6.67
	α (to North) [°]	22.5	22.5
	$(\alpha_p)_0$ (to numerical domain boundary orthogonal) [°]	81.5	81.5
NNW	H_s (m)	1.97	3.2
	T_m (s)	5.52	6.46
	α (to North) [°]	337.5	337.5
	$(\alpha_p)_0$ (to numerical domain boundary orthogonal) [°]	36.5	36.5

incompatible with the construction technology of the OBREC within the reference breakwater. Further, comparing the $p = 0.005$ scenarios, the incident wave characteristics of the NNE and NNW directions are similar. However, simulation results show that the NNW waves ($H_s = 0.79$ m and $T_p = 7.80$ s) exhibit a higher energy level in the spectrum at Gauge 4. This difference may be attributed to the geometry of the breakwaters at the Port of Ancona, which provide greater protection for waves approaching from the NNE direction ($H_s = 0.73$ m and $T_p = 7.80$ s). This finding suggests that the device should be oriented toward the NNW to harness more of the available wave energy under storm

Table 2

Wave characteristics estimated using Go10.

		Density $p = 0.05$	Density $p = 0.005$
NNE	H_s (m)	2.53	2.74
	T_m (s)	6.62	6.67
	T_p (s)	7.81	7.87
	$S(H_{s,0}/L_{0p})$	0.027	0.028
	$\alpha_{p,h1}$ [°]	35.00	35.00
NNW	H_s (m)	1.67	2.68
	T_m (s)	5.52	6.46
	T_p (s)	6.51	7.62
	$S(H_{s,0}/L_{0p})$	0.025	0.03
	$\alpha_{p,h1}$ [°]	27.00	23.00

condition. Hence, the NNW waves with $p = 0.005$ (the severe condition) at Gauge 7, near the OBREC and characterized by $H_s = 0.65$ m and $T_p = 7.02$ s, were used as the boundary condition to evaluate the influence of the OBREC outlet.

3. Wave-structure interaction

The FLOW-3D model was used to analyze both the near-field dynamics and the wave-structure interaction required for the design of the OBREC. This commercial software employs the Volume of Fluid (VOF) method to capture fluid interfaces and free surfaces (Hirt and Nichols, 1981) and has been applied to investigate the overtopping over the ramp, turbine efficiency, and power performance of the system. Details of the governing equations and numerical methodologies implemented

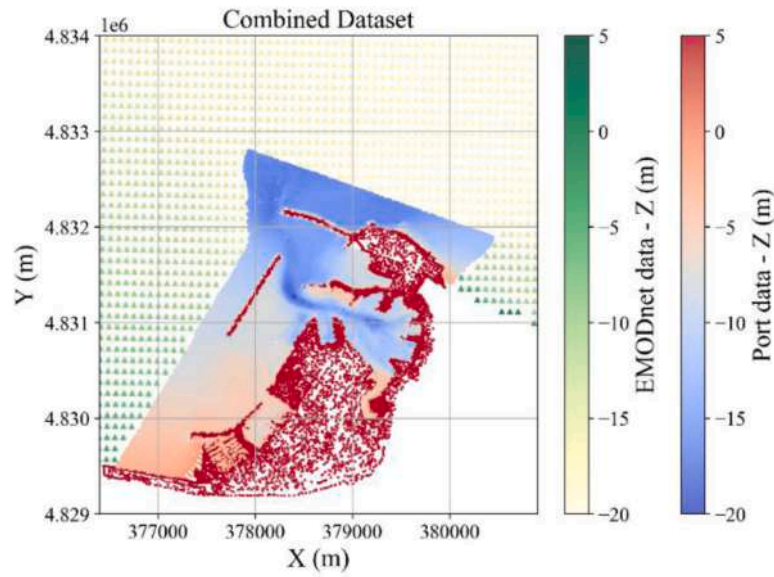


Fig. 6. Combined dataset for bathymetry of the Port of Ancona.

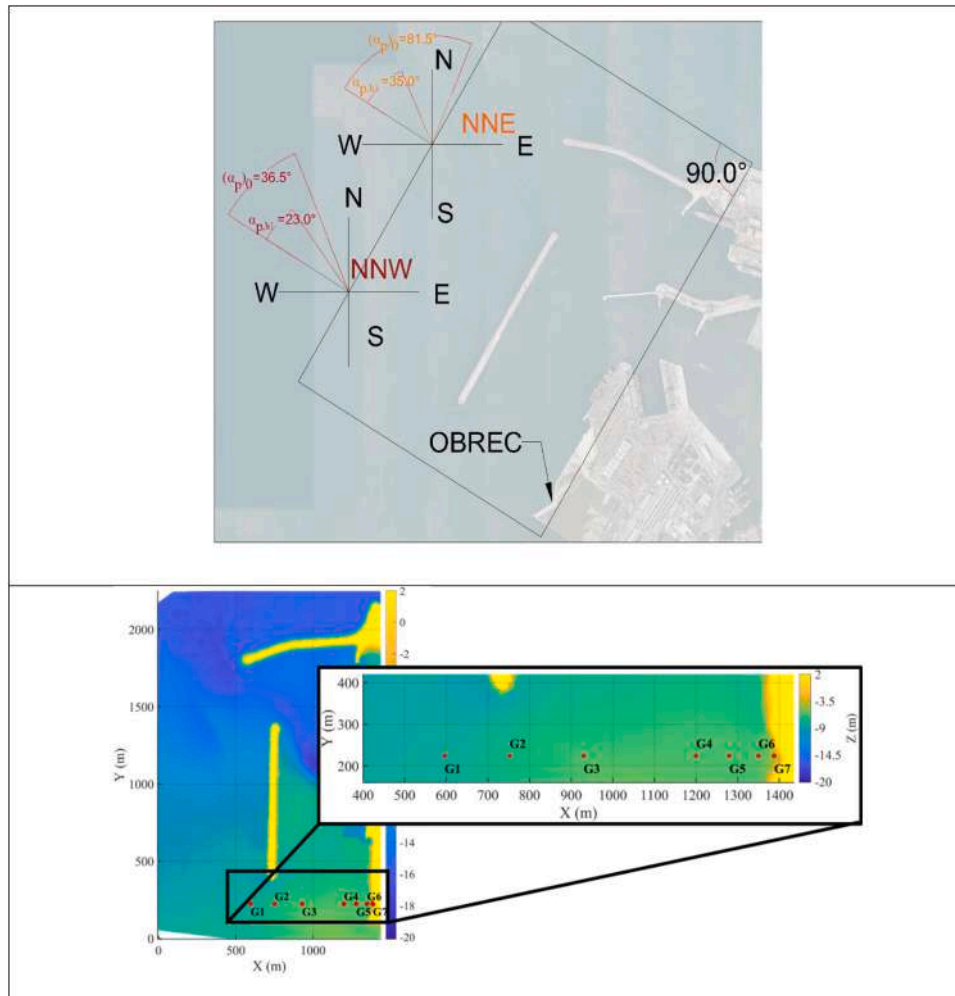


Fig. 7. Top: potential deployment location of the device in the Port of Ancona; the black rectangle indicates the computational domain. Bottom: computational domain and locations of the numerical gauges.

Table 3
Locations of the numerical gauges.

	G1	G2	G3	G4	G5	G6	G7
X (m)	596	752	930	1200	1280	1350	1388
Y (m)	224	224	224	224	224	224	224

in the software are provided in [Appendix B](#).

3.1. FLOW-3D model validation

To validate the 2D numerical model, a configuration similar to that reported by [Iuppa et al. \(2019\)](#) (see the top panel of [Fig. 11](#)) was reproduced. One of their laboratory experiments was simulated using several grid resolutions in order to identify an appropriate cell size. The selected experiment had the following characteristics: $H_{m0} = 0.0446$ m, $T_p = 0.95$ s in a JONSWAP spectrum, with a water depth of 0.257 m and a freeboard (R_f) of 0.017 m. Reynolds stresses were modeled using the $k-\varepsilon$ (RNG) turbulence model, hereafter referred to as RNG. In [Fig. 11](#) (bottom panel), the measured free-surface elevations are compared with the numerical predictions obtained using different cell sizes. All cell sizes captured the free-surface evolution with a similar pattern; however, finer grid resolutions produced higher wave crests and deeper wave troughs, resulting in larger wave heights.

The formulation of the dimensionless mean overtopping discharge (q^*) is presented below:

$$q^* = \frac{q_{in}}{\sqrt{gH_{m0}^3}} \quad (3.1)$$

For reference, the EurOtop (2007) Manual ([Pullen, 2007](#)) provides the

following empirical formulation for the maximum dimensionless overtopping discharge:

$$q_{EurOtop}^* = 0.2 \times \exp\left(\frac{-2.6 \times R^*}{\gamma_f \gamma_\beta \gamma_\nu}\right) \quad (3.2)$$

where $q_{(in)}$ is the mean overtopping discharge obtained from the numerical simulation, H_{m0} is the incident significant wave height in the frequency domain at the toe of the structure estimated using the wave separation method at gauge locations, and g is the gravitational acceleration. γ_f , γ_β , and γ_ν represent the influence factors for the permeability and roughness of the slope, oblique wave attack, and the presence of a vertical wall on the slope, respectively. A smooth ramp with waves incident perpendicularly to the slope was assumed, and no vertical wall was installed on top of the structure; therefore, all three coefficients were set equal to one. Dimensionless freeboards were also estimated as $R^* = R_f / H_{m0}$.

[Table 4](#) reports the values of H_{m0} and q^* , together with the corresponding errors relative to the experimental measurement. The numerical predictions exhibit errors on the order of 20% for H_{m0} and 35% for q^* . Differences of similar magnitude (approximately 35%) between numerical and experimental results have been reported in several studies by [Di Lauro et al. \(2019\)](#), supporting the reliability of the present results within the typical uncertainty range of overtopping simulations. Both turbulence models yield similar error levels; however, the $k-\omega$ model shows better agreement with the experimental data. Based on these results, a cell size of 0.005 m was selected for further simulations, corresponding to grid-resolution ratios of approximately 1/200 and 1/8 with respect to the wavelength and wave height, respectively.

After selecting a grid size of 0.005 m, additional simulations from [Iuppa et al. \(2019\)](#) were carried out to evaluate the model under

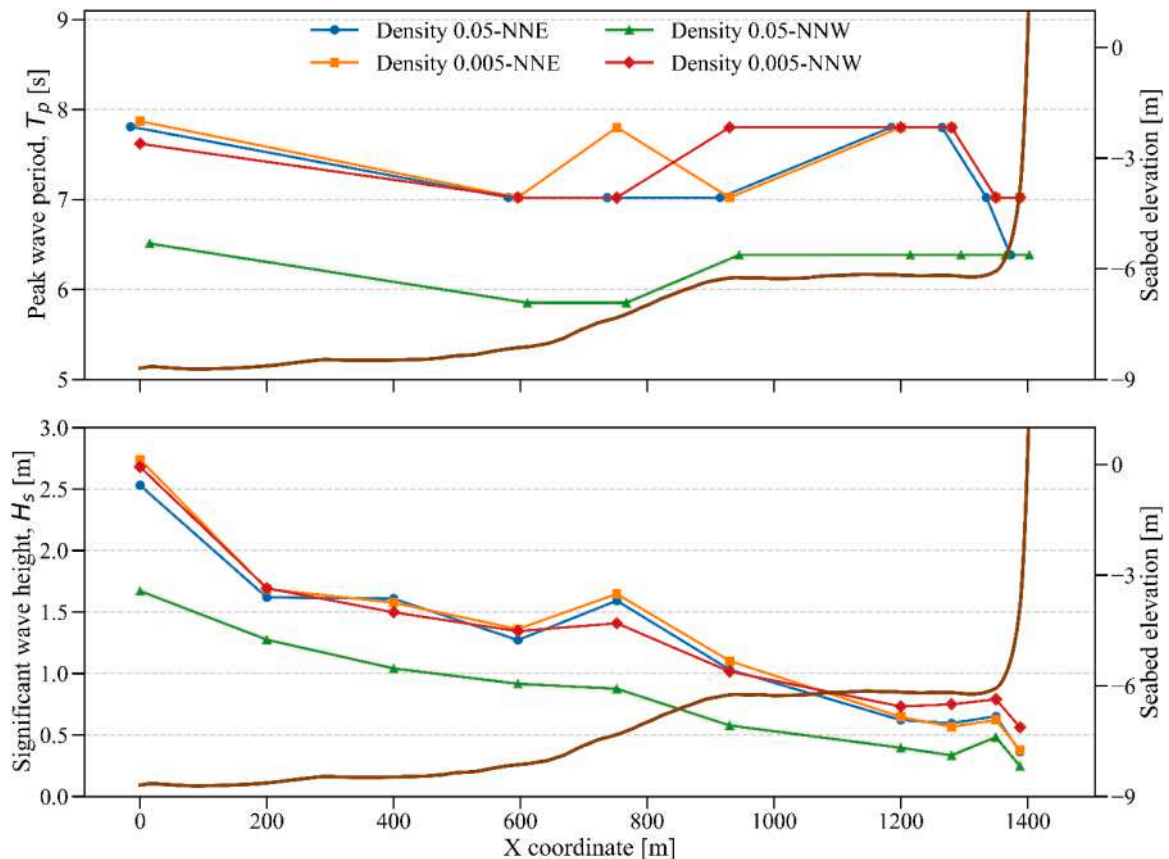


Fig. 8. Top panel: variation of the peak wave period along the gauge locations. Bottom panel: significant wave height across the gauges under different wave scenarios.

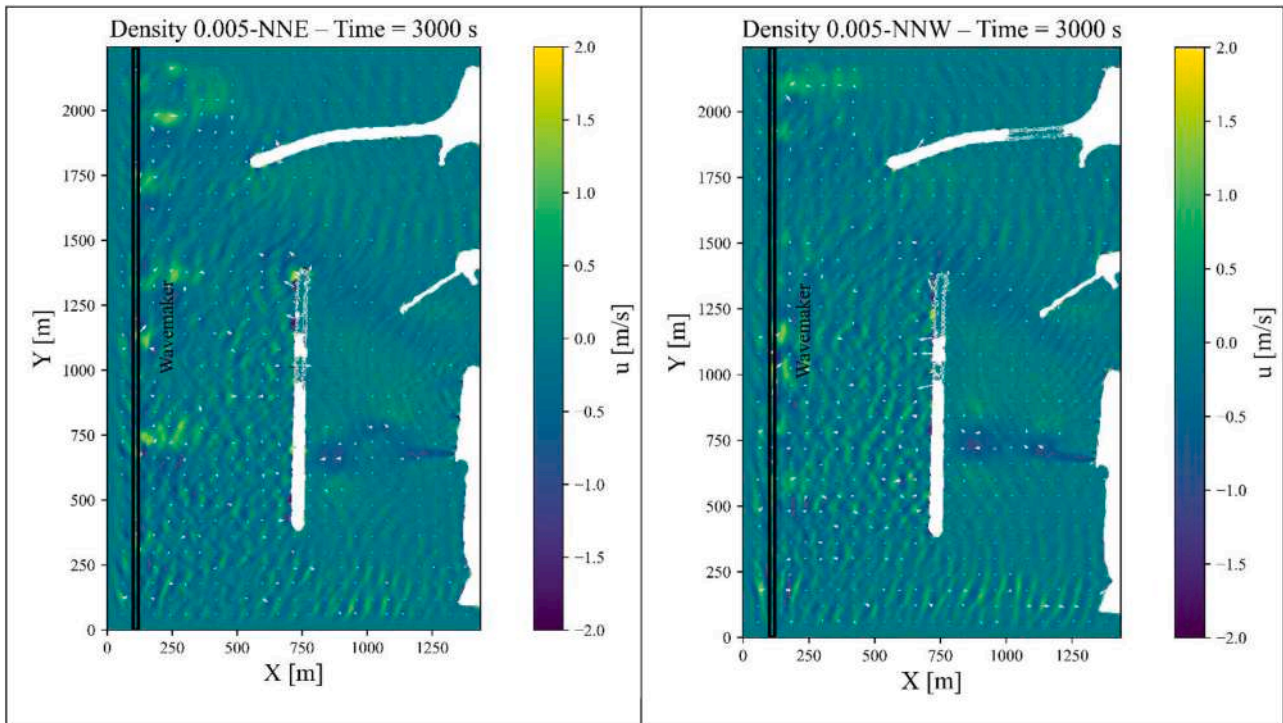


Fig. 9. Snapshots of the horizontal velocity component (u) at $t = 3000$ s.

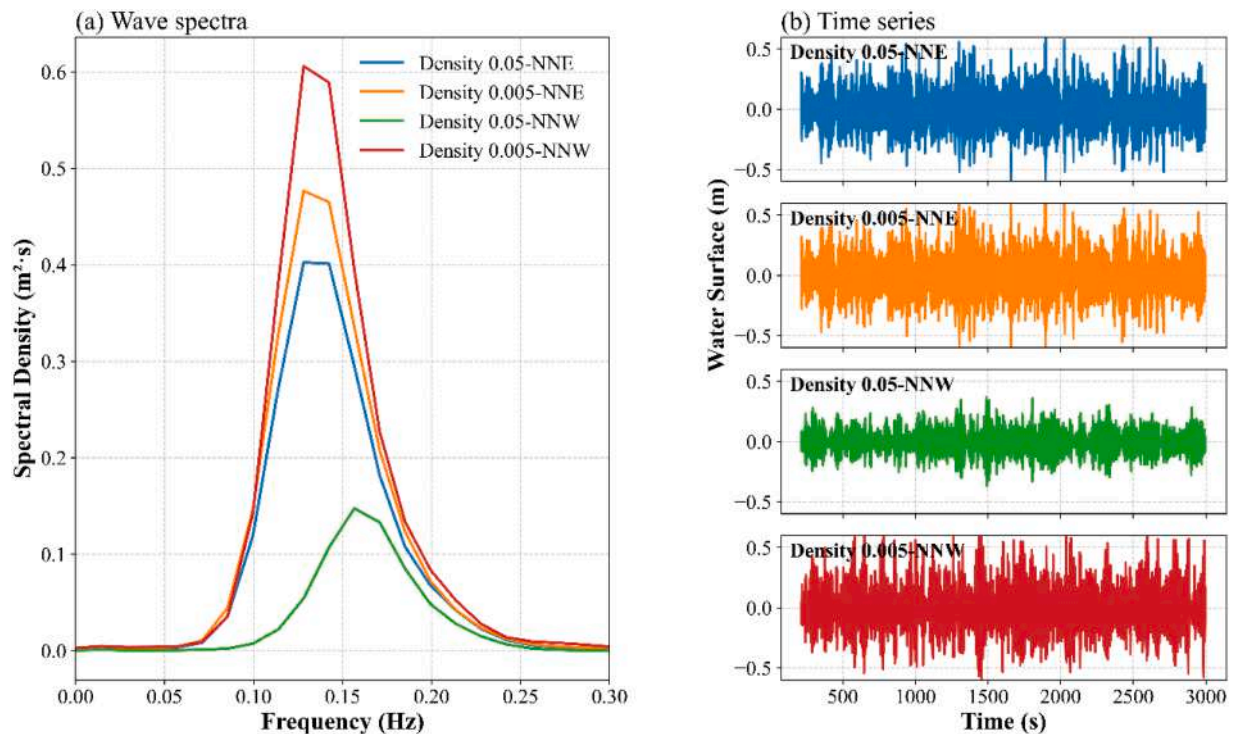


Fig. 10. FUNWAVE-TVD outputs at Gauge 4 for different wave scenarios Left: wave spectra Right: Time series.

different conditions. A total of 40 simulations, each corresponding to approximately 26–55 wave cycles, was performed following the configuration in which the sloping plate fully covered the OBREC ramp. The incident significant wave heights (H_{m0}) ranged from 0.03 m to 0.11 m, and the peak periods (T_p) varied between 0.73 s and 1.54 s. The water depth at the toe of the structure ranged from 0.179 m to 0.257 m, and the dimensionless freeboard ranged from 0.3 to 2.71.

The results for the two turbulence models are shown in Fig. 12. The mean absolute percentage error (MAPE), root mean square error (RMSE), and coefficient of determination (R^2) were calculated for the numerical predictions. For the $k-\omega$ model, the corresponding values are 87.86, 0.00836, and 0.769, respectively, while for the RNG model they are 55.10, 0.00894, and 0.736. The mean computational effort for the 40 simulations was 78 and 31 min for the $k-\omega$ and RNG models,

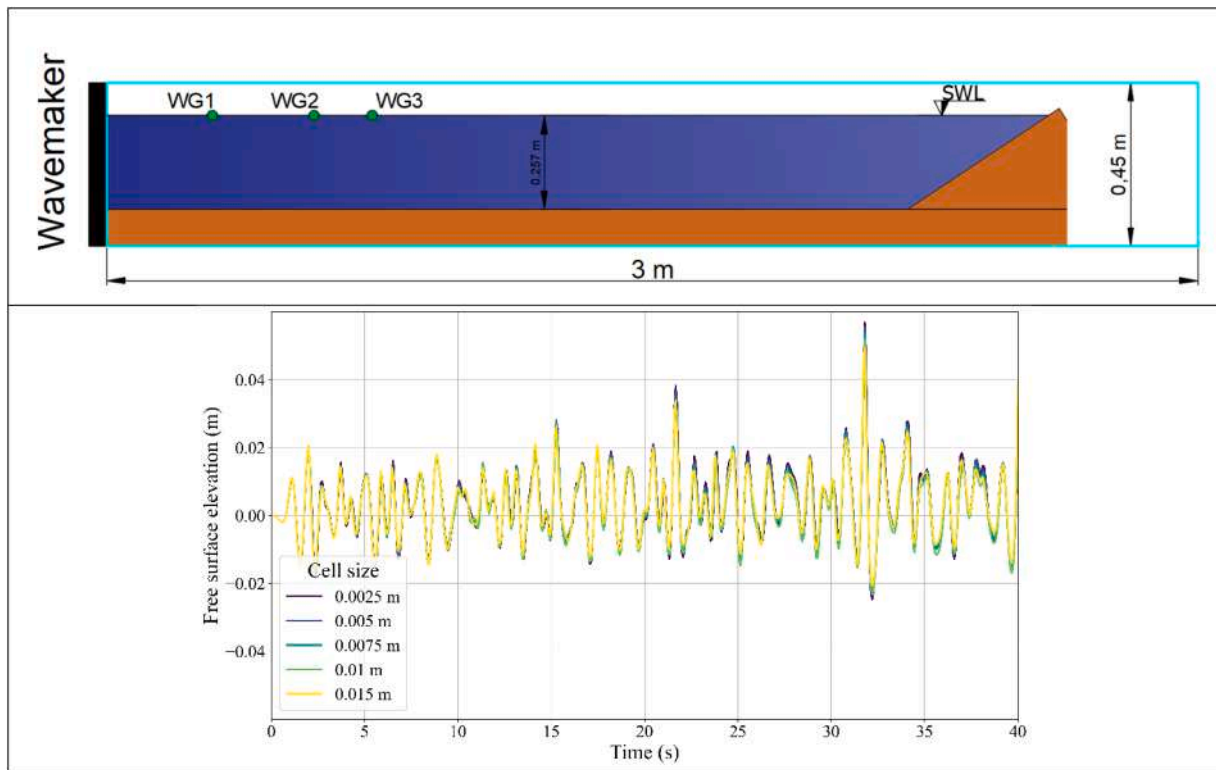


Fig. 11. Top panel: Numerical setup similar to that reported by Iuppa et al. (2019). Bottom panel: Comparison of numerically predicted free-surface elevation time series at 1.5 m from the toe of the structure (WG3) for different grid sizes using the RNG model.

Table 4

Results of the mesh-independence study.

Iuppa et al. (2019)		Metric / Model	0.0025	0.005	0.0075	0.01	0.015
H _{m0} (m)	0.0446	H _{m0} RNG (m)	0.0375	0.0370	0.0364	0.0362	0.0356
q*	0.0564	error H _{m0} RNG (%)	15.85	16.97	18.32	18.77	20.11
		q* RNG	0.0349	0.0347	0.0366	0.0421	0.0358
		error q* RNG (%)	38.04	38.36	35.05	25.38	36.42
		H _{m0} k- ω (m)	0.0378	0.0372	0.0367	0.0364	0.0356
		error H _{m0} k- ω (%)	15.18	16.52	17.65	18.32	20.11
		q* k- ω	0.0364	0.0361	0.0377	0.0430	0.0371
		error q* k- ω (%)	35.37	36.04	33.13	23.70	34.12

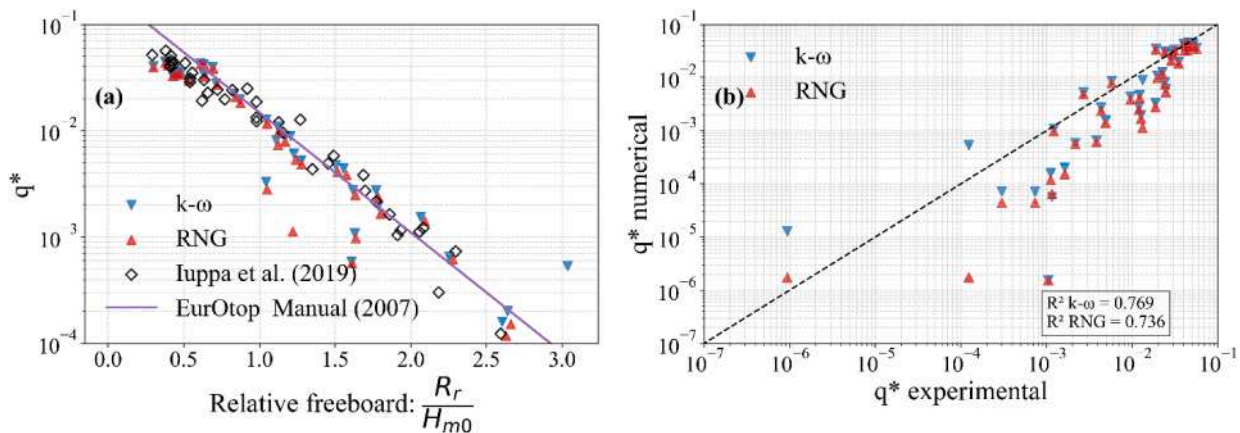


Fig. 12. Comparison between numerical simulations and experimental measurements. (a) Dimensionless mean overtopping discharge q^* as a function of relative freeboard R^* . (b) Comparison between numerical and experimental dimensionless overtopping discharge q^* (1:1 line indicates perfect agreement).

respectively. Considering both the error range and the computational effort, the RNG model represents a reasonable choice for practical overtopping applications, providing a lower computational cost despite

a slightly higher error for RMSE and R^2 .

Based on the comparison above, good agreement between numerical and experimental results is observed for small values of R^* (~ 1) and

high values of q^* (~ 0.04); however, discrepancies increase as R^* becomes larger. This trend is consistent with the findings of Iuppa et al. (2019), who attributed such behavior to the stochastic nature of overtopping; as the freeboard rises, the frequency of overtopping events drops sharply, leading to a smaller sample size that is more sensitive to statistical fluctuations.

Beyond the reduced sample size, several numerical and physical factors contribute to these deviations, including the smaller number of waves and shorter simulation time in the numerical approach (40 s compared to 700 s), differences in wave generation, limitations of the turbulence models, and numerical dissipation. In particular, the number of waves can significantly affect the results, as reported in the literature (e.g., Romano et al., 2015). However, the numerical results, even with a limited number of waves, provide acceptable predictions for overtopping applications.

3.2. Prototype scenarios

The prototype breakwater dimensions and FUNWAVE-TVD outputs used as wave boundary conditions for FLOW-3D are described here. Energy spectra (Fig. 10) expressed in terms of angular frequency and energy density, were applied to each configuration (Table 5), resulting in a total of 40 numerical tests. The geometric parameters, frontal slope above the still water level (α), ramp width (B_s), and crest freeboard of the front reservoir (R_r) are illustrated in Fig. 13 and summarized in Table 5. For each model, the freeboard and frontal slope above the still water level were varied, with the ramp width and mean weighted slope determined accordingly. Comparing overtopping for different geometries allows an assessment of the influence of these parameters on the hydraulic performance of the OBREC.

In the context of Archimedes screw turbine design, the literature recommends a minimum water head of 1 m to ensure proper operation (e.g., Shahverdi et al., 2021; Rohmer et al., 2016). To meet this requirement, a 1.5 m freeboard was incorporated into the basin design.

Prototype-scale mesh tests showed convergence for H_{m0} and reflection coefficients (C_r), while q^* remained more sensitive to grid resolution, with differences of approximately 35% between successive fine meshes (Appendix B, Table 11). This sensitivity is attributed to the intermittent and low-volume nature of overtopping events, which require a larger number of waves to achieve stable statistical averaging.

3.3. Archimedes screw turbine

Energy extraction from overtopped water collected in the basin is considered using an Archimedes screw turbine (AST). Two different ASTs, with diameters of 0.28 m and 0.26 m (see Fig. 14 and Table 6 - Models 1 and 2) were investigated numerically under full-pipe operation within a 0.30 m diameter pipe, with an imposed pressure difference of 13,000 Pa. For the numerical simulations, the $k-\omega$ turbulence model was applied using two mesh blocks. A grid resolution of 0.004 m was used in the vicinity of the turbines and 0.01 m in the remainder of the domain, resulting in a total of 1,335,600 cells. To investigate turbine efficiency,

the AST rotation was varied, considering both free rotation and prescribed motion with angular velocities ranging from 7 to 45 rad/s.

Accordingly, Models 1 and 2 are presented as numerical predictions without direct experimental validation at this stage. As a reference, we can consider the experimental and numerical results of Zitti et al. (2020), wherein a highly similar turbine was experimentally investigated in a flume featuring a cross-section three times the turbine diameter. Further, the objective of this work is not a precise quantitative estimation of the generated power, but rather an analysis of the hydrodynamic phenomena and their effects on the performance of a benchmark turbine.

The performance coefficient was then evaluated as $C_{p_{num}} = P_t / P_f$ where P_t is the mechanical power produced by the turbine and P_f represents the available hydraulic power of the incoming flow.

The mechanical power produced by the screw turbine (P_t) was obtained from the product of the turbine torque (T) and the angular velocity (ω). The corresponding dimensionless velocity parameter was defined through the tip speed ratio (TSR): $TSR = \omega R / v_{in}$, where v_{in} is the incoming flow velocity. Surface roughness effects were neglected in the present model, and mean values of angular velocity and flow velocity were used in the calculations. Two formulations were adopted for the available flow power P_f , depending on the hydraulic configuration.

In the first condition, the flow is driven by an imposed pressure head within a fully submerged in-pipe system (pressure-driven). Hence, the available hydraulic power is evaluated from the mechanical energy equation as:

$$P_f = \rho g Q \Delta H \quad (4a)$$

where ρ is the fluid density, g is gravitational acceleration, Q is the volumetric flow rate, and ΔH is the imposed hydraulic head difference.

Since $Q = Av_{in}$, this can also be written as: $P_f = \rho g Av_{in} \Delta H$

If the pressure boundary corresponds to a known static pressure difference Δp , then: $P_f = \Delta p Q$

This formulation accounts for the total hydraulic energy supplied by the imposed head.

In the second condition, in configurations where the turbine extracts energy directly from the flow velocity field without an imposed head difference (free stream), the available power is evaluated from the kinetic energy flux:

$$P_f = \frac{1}{2} \rho A v_{in}^3 \quad (4b)$$

This formulation represents the instantaneous kinetic energy transport across the turbine cross-section and was used to estimate the available power in the conveyance system and in the semi-submerged turbine configuration, where kinetic energy is the dominant available component.

The local available power within different sections inside a pipe can be estimated using Eq. (4b) and the wetted cross-sectional area of a partially filled circular pipe (A), calculated as:

$$A = R^2 \arccos\left(1 - \frac{h}{R}\right) - (R - h) \sqrt{R^2 - (R - h)^2} \quad (5)$$

where h and R represent the flow depth, and the pipe radius, respectively.

4. Results

Results for the full-scale device are presented in the following sections, addressing wave overtopping and reflection (Section 4.1), the available hydraulic power within the conveyance system under different outlet conditions (Section 4.2), and the performance of the installed turbine (Section 4.3).

Table 5

Geometric characteristics of the different models.

Model name	α (°)	Mean weighted slope (°)	B_s (m)	R_r (m)
1	34	34	2.25	1.5
2	30	33.04	2.60	1.5
3	26	31.83	3.08	1.5
4	22	30.28	3.71	1.5
5	18	28.26	4.62	1.5
6	34	34	1.5	1
7	30	33.31	1.73	1
8	26	32.41	2.05	1
9	22	31.23	2.48	1
10	18	29.65	3.08	1

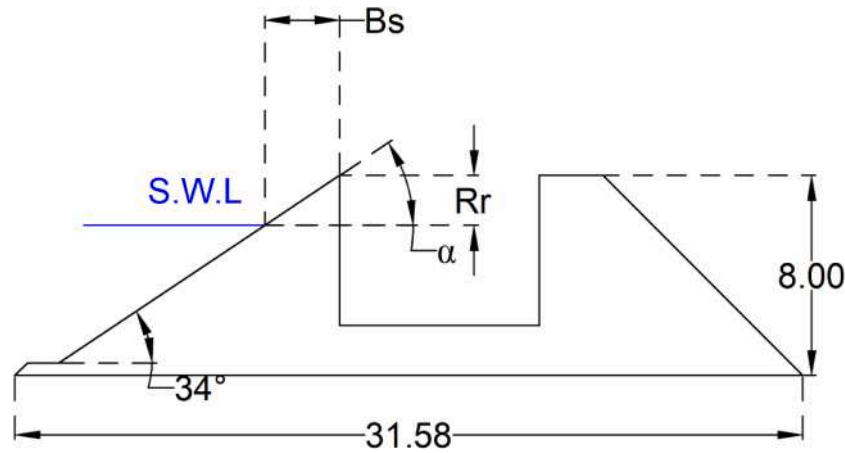


Fig. 13. Geometric parameters.

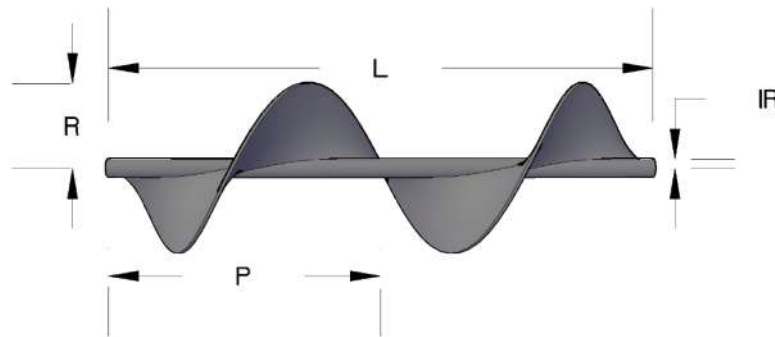


Fig. 14. Sketch of the screw turbine model with main geometric parameters.

Table 6
Geometric parameters of the turbines.

Parameter	Symbol	Model 1 (full pipe)	Model 2 (full pipe)
Turbine Radius	R	0.14 m	0.13 m
Shaft Radius	IR	0.015 m	0.015 m
Length	L	0.9 m	0.9 m
Blade inclination to shaft	-	90°	90°
Stride (pitch)	P	0.45 m	0.45 m
Turbine Density	-	7800 kg/m ³	7800 kg/m ³

4.1. Wave overtopping and reflection

Results for overtopping are shown in Fig. 15 (left panel) and are organized according to wave scenarios and structure freeboard. At higher relative freeboard values, the Van der Meer maximum formulation version 2007, Eq. (3.2) demonstrates superior agreement (RMSE = 0.0024 for 2007 vs. 0.0026 for 2018) with the numerical results. The 1 m freeboard configurations (triangles in Fig. 15) generally yield higher q^* . However, the green markers correspond to the NNW-D0.05 scenario, which exhibits lower energy (Fig. 10) and resulting in low or no q^* . According to the EurOtop 2018 maximum overtopping equation (Eq. (6)) (van der Meer et al., 2018), the predicted overtopping is higher than the numerical results, which remain below the 2018 limit. Comparing the probability densities of NNW waves for an OBREC with 1 m freeboard (red and green triangles) shows that the smaller density (approximately 10 times lower) results in an overtopping rate that is nearly 10 times higher.

$$q_{EurOtop}^* = 0.09 \times \exp \left[- \left(\frac{1.5 \times R^*}{\gamma_f \gamma_\beta \gamma_v} \right)^{1.3} \right] \quad (6)$$

Reflection coefficients (Cr) were accurately determined through a post-processing analysis using the free surface data. The incident and reflected wave components were separated using the method of Zelt et al. (1993) as later implemented by Draycott et al. (2018) in ZeltRef code.

Fig. 15 (right panel) presents the reflection coefficients plotted against the Iribarren number, allowing comparison with similar results from the literature (McBride et al., 1995). These authors proposed the following expression for smooth slopes $C_r = 0.96 \xi_m^2 / (4.8 + \xi_m^2)$, where the Iribarren number is defined as $\xi_m = \tan \alpha / s_m^{0.5}$, with $s_m = 2\pi H_s / g T_m^2$.

A few simulations appear as outliers in the dataset. In particular, the NNE_0.05–1.5 and NNW_0.05–1.0 scenarios (blue circles and green triangles in Fig. 15, respectively) exhibited anomalously high overtopping rates. In these cases, the simulations were either terminated due to numerical stability limits or produced unrealistically large overtopping values, likely associated with numerical artefacts rather than physical wave behaviour.

By contrast, the NNW waves with a density of 0.05 in the 1.5 m freeboard configuration (green circles in Fig. 15) did not produce overtopping, as they exhibited the highest reflection coefficients.

Overall, the numerical results for the full-scale structure suggest that, even with a relatively small number of waves, the predicted wave reflection agrees well with experimental wave flume results. Moreover, the overtopping rate can serve as a first indicator for evaluating the influence of ramp geometry and slope.

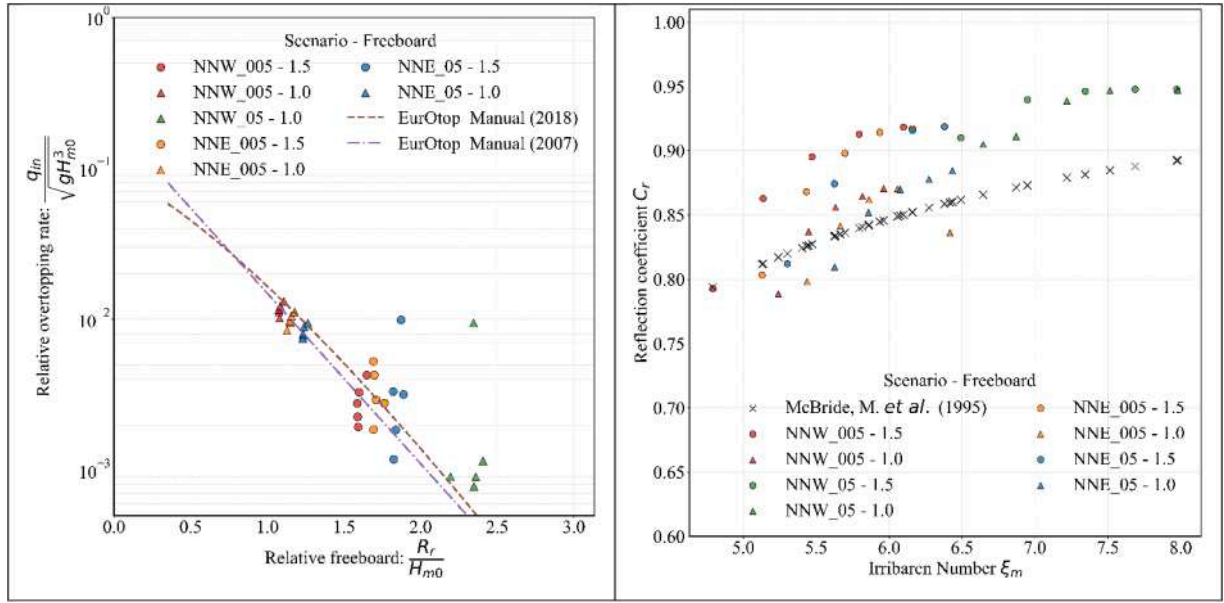


Fig. 15. Left: Numerical wave overtopping discharges. Right: Numerical reflection coefficients in full-scale study.

4.2. Available power in the conveyance system considering different outlet conditions

The present framework is sequentially coupled: overtopping simulations define the feasible inflow conditions and geometric design parameters, while the conveyance simulations isolate the effect of seaward outlet forcing on internal hydraulic power distribution under a representative basin head.

The computational domain and boundary conditions for the study of available power in the conveyance system are shown in Fig. 16. Data were extracted at six cross-sections along the pipe under different scenarios. Waves were assumed to occur at different sea level to mimic varying storm surge conditions, based on measurements from a nearby tide gauge (0, 0.1, 0.2, 0.3, and 0.4 m). In addition, two no-wave conditions with sea-levels of 0 and 0.4 m were considered, the latter representing tidal conditions. The locations of the six cross-sections, corresponding to positions between the basin outlet and the pipe outlet, are listed in Table 7. Point A is located near the basin outlet and the pipe inlet, whereas Point F is close to the pipe outlet. A uniform cell size of 0.05 m was used in all directions, resulting in a total of 1,045,506

Table 7

x-coordinates of the measurement cross-sections.

A	B	C	D	E	F
0.1 m	1.1 m	2.1 m	3.1 m	4.1 m	4.61 m

computational cells. The RNG turbulence model was applied to represent Reynolds stresses.

The basin, measuring $7.5 \times 6 \times 0.5$ m, was initialized with a water head of 1.5 m relative to the reference still water level (SWL = 0). During the conveyance simulations, no additional overtopping inflow was imposed, so the basin water level decreased progressively as discharge occurred. The pipe has a slope of -0.204 over a 5 m length along the x-direction. Wall boundary conditions were applied around the basin, while pressure boundary conditions were applied at the free surface and at the outlet of the system (Fig. 16). The outlet forcing was represented through a time-varying free-surface level at the pressure boundary. The resulting variation in downstream hydrostatic head modifies the pressure gradient along the pipe, affecting discharge capacity and producing

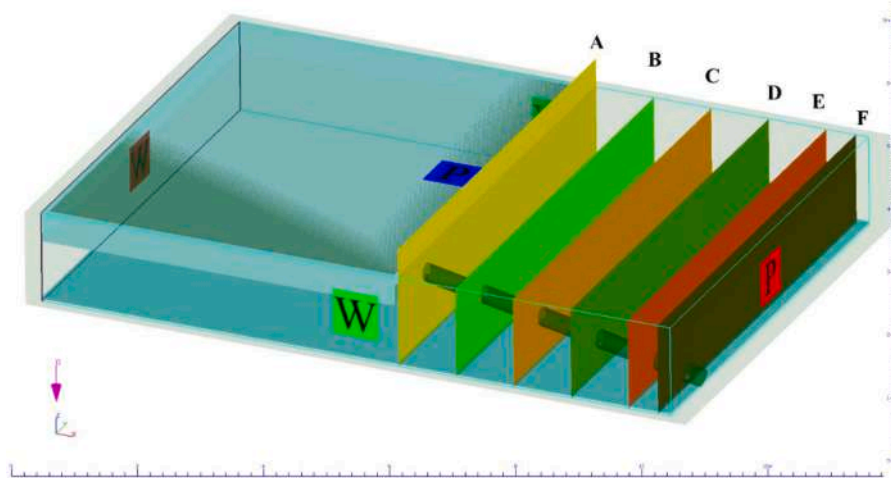


Fig. 16. Numerical domain with boundary conditions (W: wall; P: pressure boundary) and identification of the control cross-sections. Section A monitors the reservoir-to-pipe transition, while Section F captures outlet pressure feedback.

local reductions or reversals of velocity near the outlet.

Using the pipe invert as the reference datum ($z = 0$), which coincides with a zero Still Water Level (SWL = 0), the corresponding pressure distribution can be expressed as:

$$p = -\rho g z + Dp \quad (7)$$

where z is the vertical coordinate (negative below the invert) and Dp represents the dynamic wave-induced component. The dynamic contribution is defined as:

$$Dp = \rho g \eta \frac{\cosh k(h+z)}{\cosh kh} \quad (8)$$

where η , k , and h represent the free surface elevation, the wave number, and the water depth, respectively. In the numerical implementation, the time-varying free surface elevation $\eta(t)$ was prescribed at the outlet boundary, while the internal pressure field was computed by FLOW-3D through its VOF formulation. Above the mean water level, where dynamic pressure effects are negligible, hydrostatic pressure provides an adequate approximation (Dean and Dalrymple, 1991). Wetting and drying of boundary cells were handled automatically by the solver. When the instantaneous free surface dropped below the pipe invert, atmospheric pressure conditions prevailed at the outlet.

In Fig. 17 the pressure and water surface are shown for two representative SWLs conditions under the severe NNW $p = 0.005$ condition. It is observed that around 40 s, 80 s, and similar time intervals, the pressure at the outlet boundary reaches its maximum, indicating that flow blockage is likely to occur during these periods.

Fig. 18 presents the time-averaged available power, flow velocity, flow depth, and flow rate across the selected cross-sections. For cases with wave forcing applied at the outlet, results are shown for still water levels (SWLs) ranging from 0.0 m to 0.4 m. In contrast, for the no-wave cases, only two simulations were performed, corresponding to the minimum and maximum SWLs (0.0 m and 0.4 m). The figure illustrates the spatial variability along the pipe and highlights the influence of wave forcing on the hydrodynamic and energy characteristics.

In general, available power increases downstream, driven by rising flow velocities despite a reduction in flow depth. However, the presence of waves (Fig. 18, solid lines with circles) or high tide conditions (Fig. 18, purple lines) reduces power availability, with the most significant impact observed near the outlet. Specifically, after a certain

distance that increases with the SWL, power availability declines in the vicinity of the outlet, contradicting the general increasing downstream trend. Moreover, when comparing results for SWL = 0.4 m with the corresponding case of sea-level rise alone (e.g., tidal conditions), the presence of waves facilitates improved discharge and results in significantly higher mean power. This behavior can be attributed to the fact that waves generate transient pressure drops, allowing intermittent discharge from the pipe. In contrast, under tide-only conditions (dashed lines with triangles), the pressure remains constant, reducing the flow velocity near the outlet.

Without wave forcing, and in the absence of sea-level rise (SWL = 0.0 m), Section F, located near the pipe outlet, exhibits higher mean power (~ 600 W) and flow velocity (~ 4 m/s) than the corresponding case with waves. However, under increased sea-level (SWL = 0.4 m), the outlet becomes fully submerged, and both power and velocity throughout the conveyance system are significantly reduced, approaching zero.

The mean velocity follows a trend similar to that observed for power, suggesting that the mean velocity is affected by the same processes described above, due to the presence of waves and increasing SWL. For instance, under wave conditions, the highest velocity occurs at Section E for SWL = 0.0 m, reaching approximately 3.7 m/s, with a corresponding maximum power of about 550 W. In contrast, for SWL = 0.3 m, the peak shifts upstream to Section D, with a reduced velocity of approximately 2.7 m/s and a corresponding power of around 310 W.

Conversely, the impact of waves and SWL on flow depth is opposite to that observed for power and velocity. Depth initially increases along the conduit until the outlet effect predominates, where wave action and high SWLs further enhance mean flow depth. In particular, the flow depth increases with increasing SWL, approaching near-full pipe conditions that are associated with reduced flow velocities.

The mean flow rate remains approximately constant along the pipe for each individual scenario, indicating that, despite local fluctuations induced by wave forcing, the overall discharge is conserved in a time-averaged sense. However, the magnitude of the flow rate is affected by both waves and SWL, with both the maximum (~ 0.075 m³/s at SWL = 0.0) and minimum (< 0.005 m³/s at SWL = 0.4) values occurring under no-wave outlet conditions.

Fig. 19 presents the variability index of the hydrodynamic variables at Sections A–F for both wave and no-wave outlet conditions. The variability index is defined as $VI = \sigma/\bar{x}$, where σ is the standard deviation and $\bar{x}$ is the corresponding mean value of the considered variable. The VI

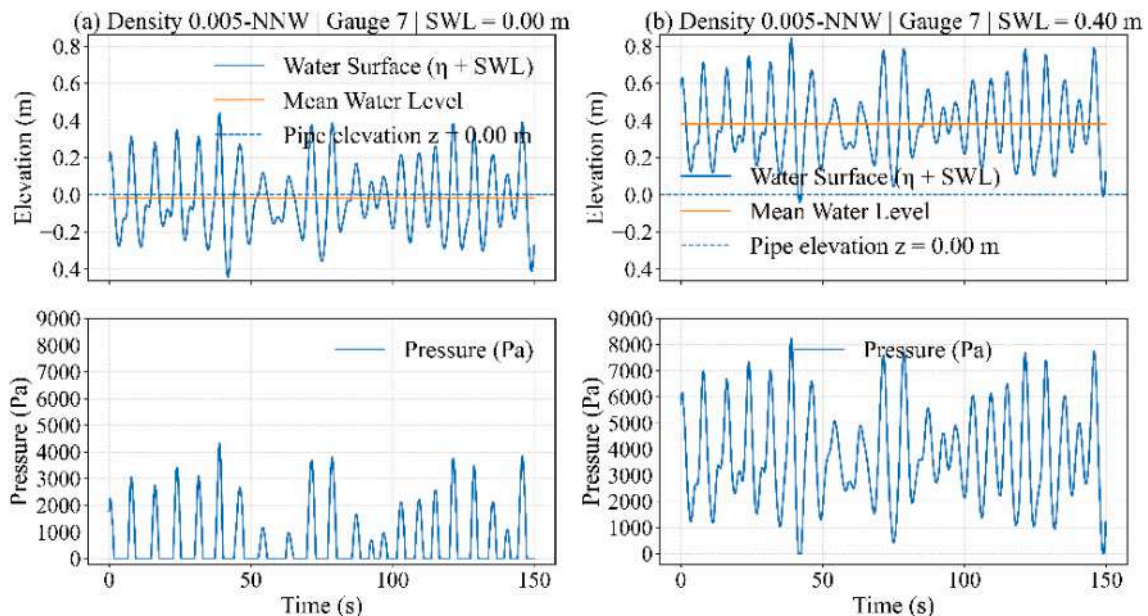


Fig. 17. Water surface fluctuations and corresponding pressures for (a) SWL = 0 m and (b) SWL = 0.4 m applied at outlet boundary condition.

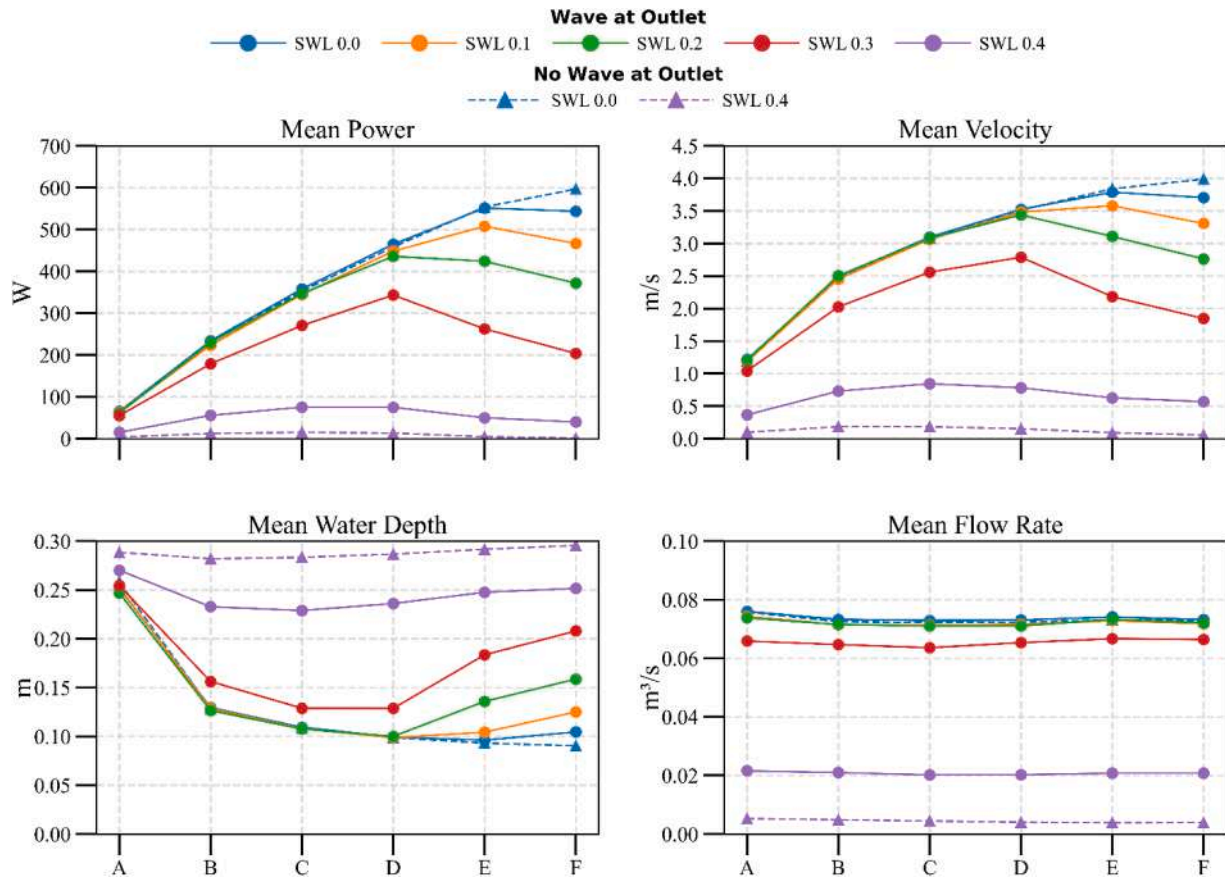


Fig. 18. Mean values of power, flow velocity, flow depth, and flow rate at Sections A–F under varying SWLs for wave-at-outlet and no-wave.

quantifies the relative fluctuations of each variable, with lower values indicating more uniform flow conditions. This index for available power increases significantly with sea-level rise. Under wave forcing, it rises from approximately 0.5 to values exceeding 5 at Section A. At Sections B and C, the variability index reaches about 2 at the highest SWL (0.4 m), which is lower than at the other cross-sections. This suggests that Section C provides more uniform power extraction conditions compared to the inlet and outlet sections for this scenario.

Additionally, Sections D and C yields the highest mean available kinetic power under wave conditions at SWL = 0.3 m and 0.4 m, approximately 271 W and 75 W, respectively, compared with Section A and F. For lower sea-levels, however, the outlet region (Section F) is more favorable, as it produces higher mean power (around 543 W for SWL=0). In general, the variability index for power at Section C remains lower than at the inlet (Section A) and outlet (Section F) across all cases.

For mean power and flow velocity, the VI generally remains below unity for most sections and conditions, indicating moderate relative fluctuations. However, higher values are observed locally, particularly for power at Section A, where the VI can reach approximately 2.0. In contrast, flow velocity exhibits lower variability, with VI values typically around or below 0.5. Noticeable deviations from these trends occur at SWL = 0.4 m, where significantly higher variability is observed, with VI values exceeding 2 and reaching up to about 8 for power under no-wave outlet conditions.

For mean flow depths ranging between 0.1 and 0.27 m, the variability index remains below 0.6 for all cases, indicating relatively uniform depth conditions.

For the mean flow rate, the VI follows a trend similar to that observed for mean power and flow velocity, with relatively low values across most sections and scenarios. However, a significant increase in variability is observed at SWL = 0.4 m, where VI reaches approximately 4 under no-

wave outlet conditions and about 2.5 when wave forcing is present at the outlet.

The results for mean values and the corresponding variability indices suggest that, as the pipe becomes nearly fully submerged with an overhead of 0.1 m (SWL = 0.4 m), the available power decreases substantially and exhibits greater non-uniformity along the pipe.

In Fig. 20, snapshots at $t = 79$ s for the different outlet boundary conditions are presented. The results show that the combined influence of the incoming waves and the imposed sea-level rise affects a long section of the pipe. As the still water level (SWL) increases, the flow depth near the outlet rises, while the streamwise velocity within the pipe decreases.

Table 8 summarizes the mean kinetic power along the pipe at three cross-sections. With waves, at sections F and A, increasing sea level consistently leads to a reduction in mean available kinetic power.

To evaluate the hydraulic consistency of the system, the discharge entering the basin due to wave overtopping was compared with the discharge capacity of the conveyance system under the prescribed outlet boundary conditions. Based on full-scale overtopping simulations of the OBREC system, Model 3 in Table 5, with a basin width of 7.5 m under NNW waves with a density of 0.005, yields a mean inflow of 0.06 m³/s. At all six cross sections, the mean flow rates fluctuate mainly around 0.065–0.07 m³/s, indicating that a basin width of 7.5 m may not ensure stable hydraulic performance under all scenarios except SWL equal to 0.4 m.

If both conveyance systems are intended to operate simultaneously (one discharging toward the protected area and the other toward the sea), a total mean flow rate of around 0.15 m³/s is required. Therefore, to achieve stable operation, either the basin width should be increased to provide the necessary inflow, or a control valve should be installed within the conveyance system to regulate the discharge and balance it

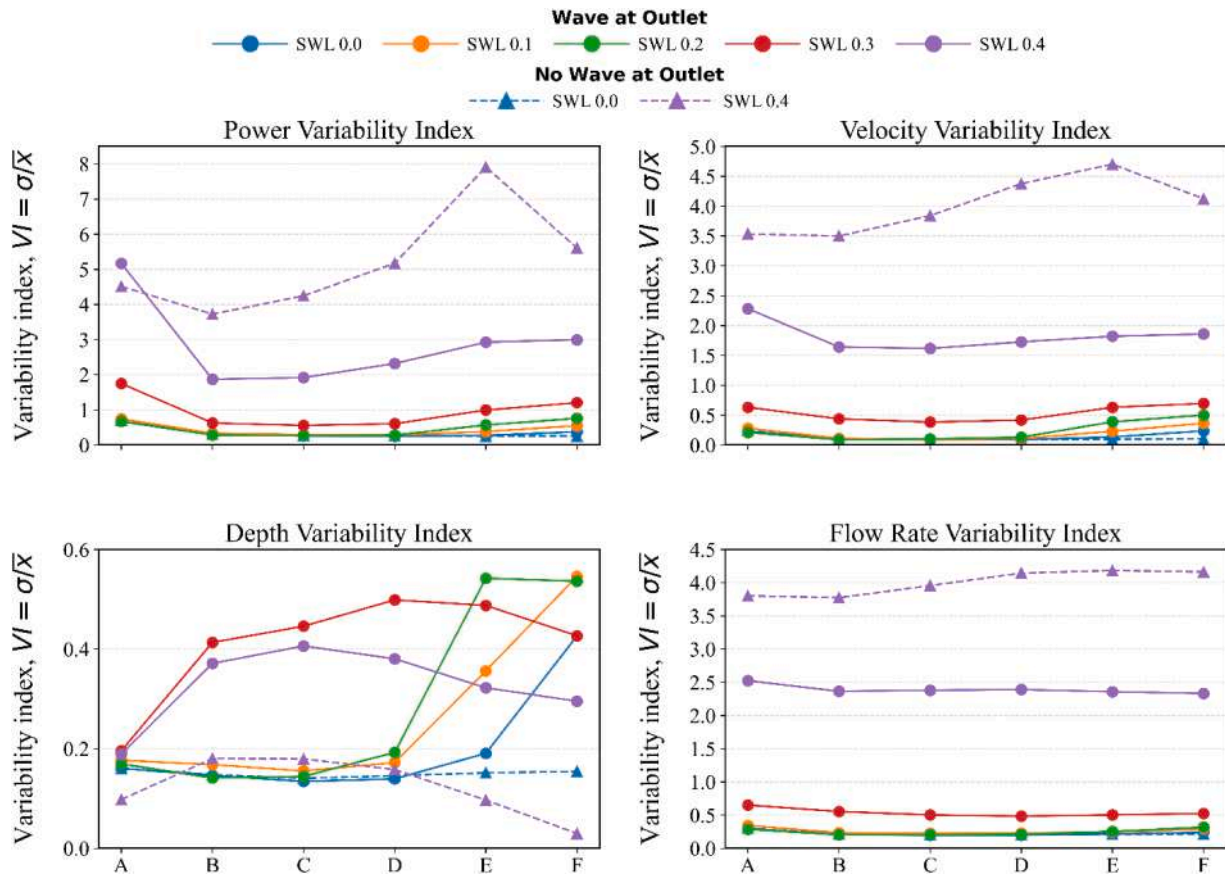


Fig. 19. Variability index ($VI = \sigma/\bar{x}$) of hydrodynamic variables at Sections A–F under different SWLs for wave and no-wave outlet conditions.

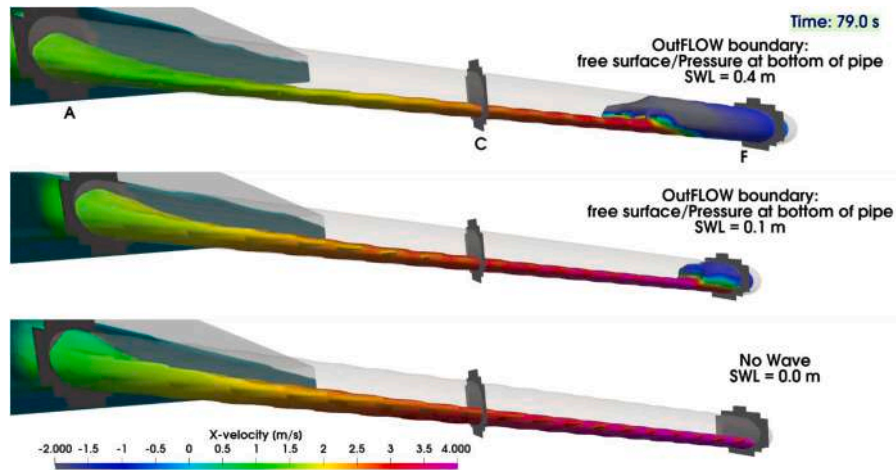


Fig. 20. Comparison of x-velocity fields inside the pipe at $t = 79$ s for the different outlet-pressure boundary conditions.

with the overtopping inflow.

To evaluate the effect of more frequent wave forcing at the conveyance-system outlet, the free-surface elevation predicted by FUNWAVE-TVD for NNW waves with $p = 0.05$ at Gauge 7 ($H_s = 0.48$ m, $T_p = 6.4$ s) was applied as the outlet boundary condition. The results are shown in Fig. 21.

Based on the $p = 0.05$ results (Fig. 21) for SWL = 0.0–0.2 m, Sections A–D show only minor changes compared with the severe $p = 0.005$ condition (Figs. 18 and 19). The main differences occur near the outlet at Sections E and F, where mean power and velocity are generally higher

and flow depth is lower. This indicates that the more frequent, lower-energy wave forcing mainly affects the downstream part of the conveyance system, while the upstream sections remain relatively stable.

However, at SWL = 0.3 and 0.4 m, the available power becomes lower than under the severe $p = 0.005$ condition. At high SWL, the reduced hydraulic head limits seaward discharge, while the smaller wave fluctuations provide less temporary enhancement of flow through the submerged pipe.

Table 8
Mean kinetic available power (W).

	Cross Section	With Waves			Without Waves		
		A	C	F	A	C	F
Still water level	SWL = 0 m	65.09	357.94	543.27	63.47	352.52	596.66
	SWL = 0.1 m	63.70	344.69	466.40			
	SWL = 0.2 m	61.17	346.72	371.48			
	SWL = 0.3 m	55.08	270.52	203.71			
	SWL = 0.4 m	14.89	75.22	39.88	3.6	14.90	1.02

4.3. Turbine functioning

Fig. 22 (top) illustrates the numerical model setup for Model 1, including the applied boundary conditions, mesh blocks, grid resolution, and the x -velocity field at $t = 7.0$ s. Models 1 and 2 have gap widths of 0.01 m and 0.02 m, respectively. Efficiency decreases when water escapes through the gap instead of interacting with the screw surface. This effect is more pronounced in smaller screws, where leakage represents a larger fraction of the total flow. As screw diameter increases, the gap becomes proportionally smaller, reducing leakage-related losses (e.g., Lubitz et al., 2014; Shahverdi et al., 2020).

Fig. 22 (bottom left) shows the semi-submerged turbine integrated within the complete conveyance system, while (bottom right) presents the corresponding system without the turbine under identical boundary conditions, subjected to a hydraulic head of 1.5 m and with no wave forcing applied at the outlet boundary. Snapshots of the x -velocity (u_x) show a system-wide reduction in flow velocity due to the presence of the turbine, accompanied by an increase in flow depth downstream of the device (cross-section D). At this location, u_x decreases to approximately 1 m/s, compared to about 3 m/s in the no-turbine case.

The efficiency curves of the analysed ASTs are shown in Fig. 23, together with best-fit curves obtained using a Nonlinear Least Squares regression (dashed lines), whose equations are reported in Table 9, together with the maximum efficiency values and the corresponding TSRs. In full-pipe condition, ASTs of Models 1 and 2 have similar numerically predicted efficiencies, close to 0.3, with an absolute difference of about 0.083 in C_p compared with the free-stream AST proposed in Zitti et al. (2020). This comparison is used only as a reference because the geometry and operating conditions differ. Further, pipe-installed ASTs reach optimal performance at higher TSR, i.e. at lower flow velocities.

In addition to the fully submerged configurations, the AST of Model 1 was simulated under semi-submerged operating conditions, in a location between sections C and D of the OBREC structure (Fig. 16). A 3 s time window (between 2 s and 5 s of simulation time) was used for performance evaluation, with a zero-pressure outlet boundary condition, consistent with the no-wave case (SWL = 0.0) shown in Fig. 18 and Table 8, to represent water discharge toward the protected area. During this interval, the turbine angular velocity remained approximately within the range 26–30 rad/s, indicating a short quasi-steady operating phase. However, because the simulation duration was limited, the resulting efficiency should be interpreted as a preliminary estimate rather than a statistically stationary performance value.

The results show that this configuration induces semi-submerged operating conditions. During the selected 2–5 s interval, the estimated efficiency of the semi-submerged AST was 0.16 at the sampled operating condition of TSR = 1.37, as highlighted by the red triangle in Fig. 23. These AST efficiency values should therefore be interpreted as preliminary numerical estimates, since real OBREC operation will involve unsteady overtopping inflow, basin-level fluctuations that are not fully

reproduced in the present simulations.

5. Discussion and conclusions

The analyses and results proposed above serve two main purposes: 1) provide an assessment of the modalities of a radically new use of turbines within shore-protection structures (essentially of shore-connected type), 2) give some brief indications of the energetic significance and viability of the newly proposed approach.

The analysis has been carried out using semi-analytical modelling and in relation to conditions specific to the Port of Ancona (Italy), which is the planned location for the construction of a full-scale OBREC system to be used for field experiments on the basis of the proposed analysis.

1) The study investigates the influence of outlet conditions on the performance of a seaward-discharge OBREC under representative storm-wave conditions at the Port of Ancona. Wave characteristics were first compared between buoy measurements and Copernicus data. Poor directional agreement entailed the use of observed data to identify the dominant wave directions and select the representative wave conditions.

The propagation of offshore wave conditions toward the breakwater—evaluated via Goda's model and simulated using FUN-WAVE-TVD—identified the severe wave climate scenario from the NNW direction as the most relevant for energy production, as it remains fully compatible with the structural constraints.

The near-field FLOW-3D model was benchmarked against laboratory data from Iuppa et al. (2019). The model provided acceptable engineering-scale predictions for overtopping, especially at low relative freeboard and higher overtopping discharge, although larger discrepancies occurred at higher R^* . The full-scale OBREC was then simulated for ten ramp geometries under four wave scenarios. A ramp slope of 18° produced the highest overtopping and lowest reflection, although steeper slopes may be preferable from a construction perspective. A 1 m freeboard increased discharge but only meets the minimum head required by the Archimedes turbine and may limit efficiency. Reflection coefficients showed good agreement across the range of Iribarren numbers.

However, the prototype-scale mesh sensitivity analysis showed that the dimensionless overtopping discharge q^* remains affected by an uncertainty of approximately 35% between the two finest meshes. Therefore, the comparison among ramp geometries should be interpreted with this uncertainty in mind, particularly when ranking the configurations based on overtopping discharge. In particular, the conclusion that the 18° ramp produces the highest overtopping should be considered within this uncertainty range, while the reflection-coefficient trends are less affected.

The full 3D basin and conveyance system were simulated to evaluate available power under waves and sea-level rise (0–0.4 m). The results show that the optimal turbine location depends strongly on the downstream water level. For SWL increases up to 0.2 m, the outlet remains the most favourable location; at higher SWLs, the optimal position shifts upstream toward the basin. Under the highest SWL, outlet power drops from approximately 543 W to approximately 40 W, corresponding to a reduction of about 90%. The reported reduction should therefore be interpreted as specific to the Port of Ancona geometry and the severe outlet-forcing condition considered here, rather than as a universal value for all seaward-discharge OBREC systems.

At higher SWLs (0.3–0.4 m) under severe climate (NNW $p = 0.005$), Sections C and D form the most favourable practical installation zone, providing the highest mean available kinetic power together with lower variability than the inlet and outlet sections. Because Sections C and D are separated by approximately 1.0 m and the investigated turbine is 0.9 m long, they represent a common practical installation zone rather than distinctly different turbine positions. Additional simulations under the

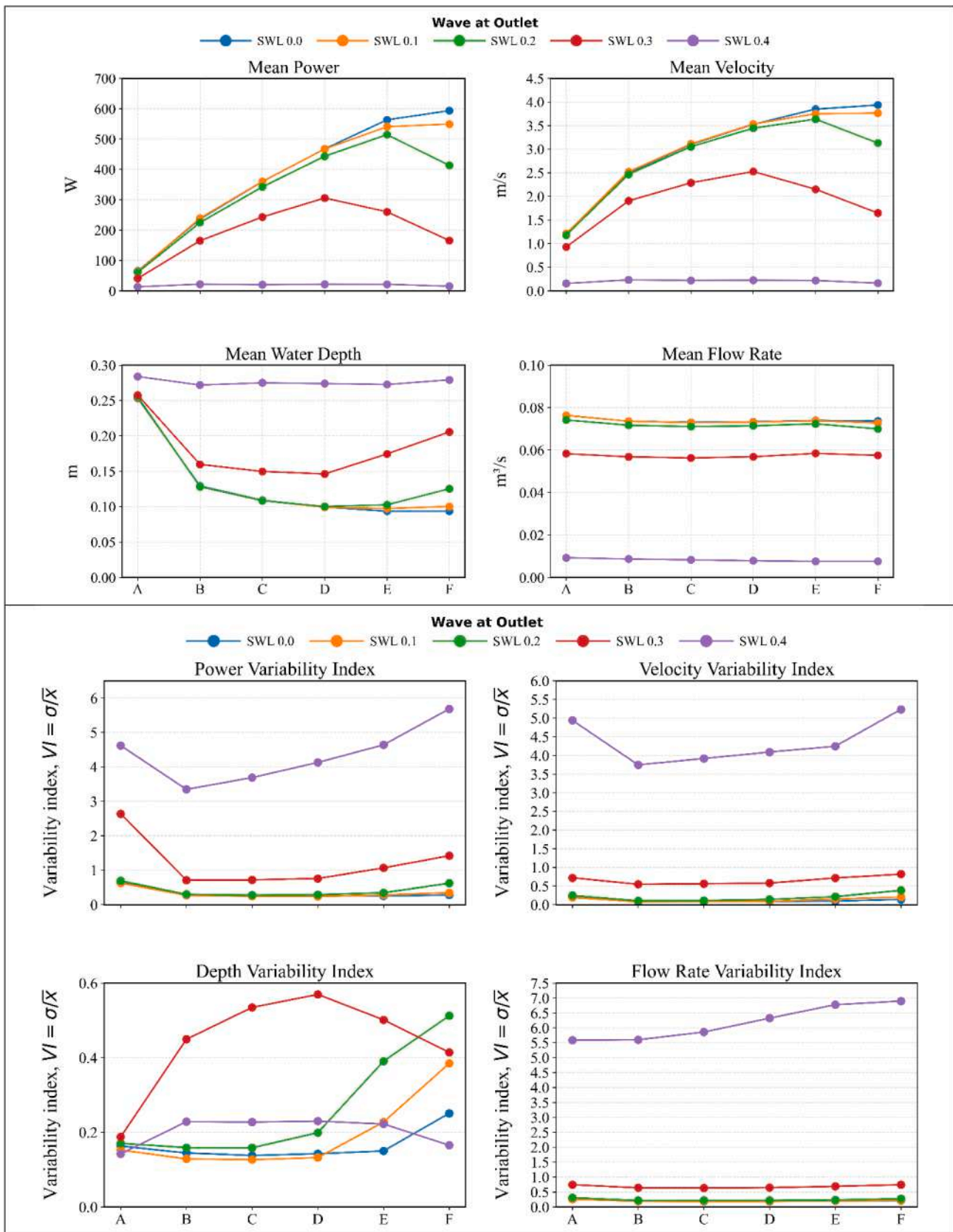


Fig. 21. Top: Mean power, flow velocity, flow depth, and flow rate under the NNW $p = 0.05$ condition. Bottom: Corresponding variability indices.

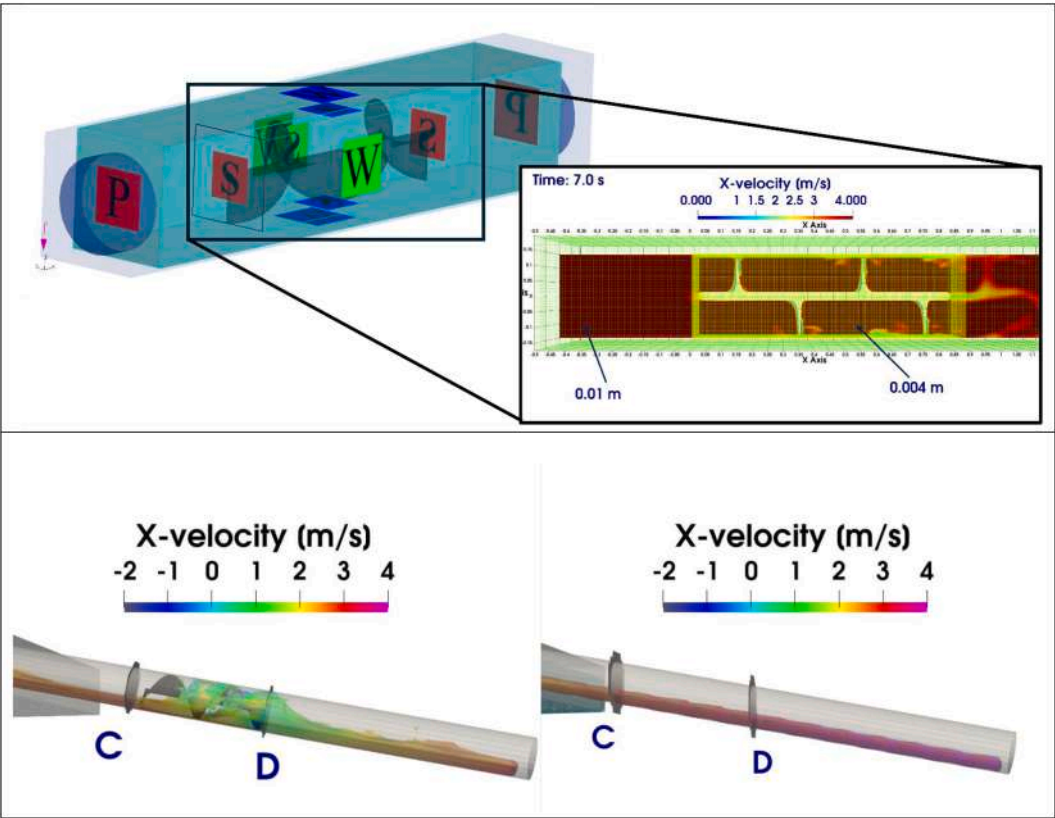


Fig. 22. Top: Numerical setup showing boundary conditions, mesh blocks, and grid resolution, together with the x-velocity field at $t = 7.0$ s under fully submerged conditions. Bottom: Numerical setup of the conveyance system: (left) semi-submerged turbine configuration representing field conditions; (right) baseline configuration without the turbine, subjected to identical boundary conditions (absence of wave forcing at the outlet boundary).

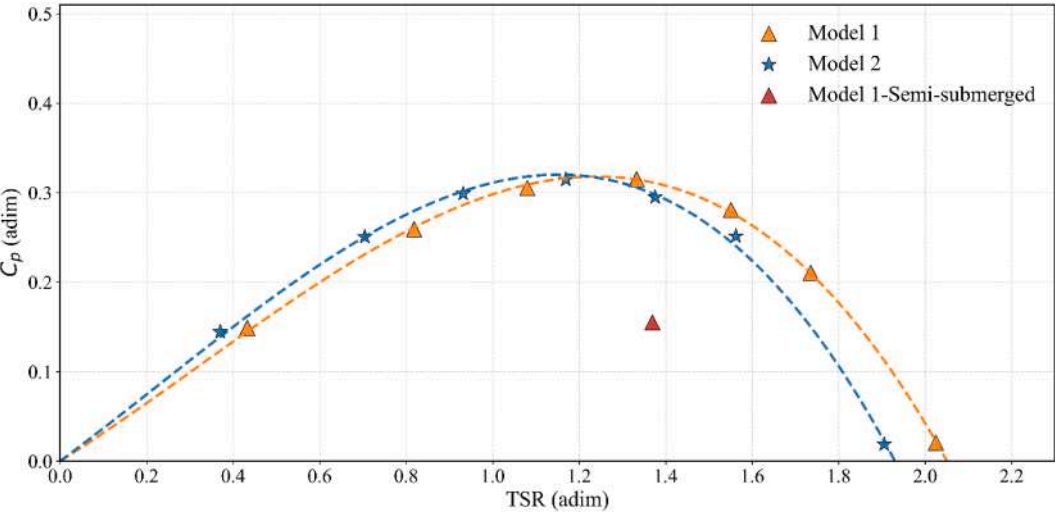


Fig. 23. Efficiency of full-pipe turbines (current study) and semi submerged condition.

Table 9
Summary of turbine performance metrics for fully submerged operating conditions.

	Model 1 (full pipe)	Model 2 (full pipe)
Max C_p	0.32	0.32
TSR at Max C_p	1.24	1.15
Fit Function	$C_p = TSR(-0.14TSR^2 + 0.13TSR + 0.3)$	$C_p = TSR(-0.15TSR^2 + 0.17TSR + 0.36)$

more frequent NNW $p = 0.05$ condition show similar or slightly higher available power at SWL=0.3–0.4 m, but lower power at SWL=0.3–0.4 m than under the severe $p = 0.005$ condition. Overall, wave and sea-level forcing modify the pressure gradient along the conveyance system, reduce the available power near the outlet, and shift the most suitable turbine location upstream. The conveyance simulations also indicate that the present basin width of 7.5 m may be insufficient for stable simultaneous operation of both discharge systems, suggesting the need for increased basin width or flow regulation.

For the Archimedes screw turbine, the numerical results show that lower rotational speeds improve performance. Fully submerged in-pipe configurations reach C_p values close to 0.32 under idealized conditions, while during the selected 2–5 s interval, the estimated efficiency of the semi-submerged AST integrated into the OBREC system was 0.16 at the sampled operating condition of $TSR = 1.37$. These values should be interpreted as preliminary numerical estimates, since real OBREC operation will involve unsteady overtopping inflow, basin-level fluctuations.

2) To provide a quantitative comparison between the two conveyance configurations (seaward discharge and discharge toward the protected area), under the severe condition considered, an estimate has been performed based on a simple, but not negligible order of magnitude approach. This is carried out by comparing the power that could be extracted from both water discharging methods, assuming that classical shore-connected rubble-mound breakwaters for coastal protection might be equipped with an overtopping device discharging seaward, while port structures might be equipped with systems discharging into a protected area. Thus, the illustrative scaled ratio between systems discharging to sea $P_{seaward}$ and towards protected basins $P_{protected}$ has been estimated by multiplying the specific power achievable through the two methods ($P_{seaward}^s$, $P_{protected}^s$) times the length of the structures where these systems could be installed, defined as $L_{seaward}$ = total length of shore-connected breakwaters and $L_{protected}$ = total length of port dikes. For simplicity and methodological adequacy, we compute the specific powers by referring to the specific system here analysed, i.e. to the power data of Table 8, while for structure lengths we refer to those of the Italian coasts. The total length of shore-connected breakwaters along the Italian coastline is approximately 500 km.⁴ Within this broader coastal protection framework, the total length of port dikes along the Italian coastline is estimated at approximately 24 km. This figure corresponds to the combined length of the principal breakwater arms protecting nine major commercial and regional ports (Genoa,⁵ La Spezia,⁶ Augusta,⁷ Marsala,⁸ Catania,⁹ Trapani,¹⁰ Taranto,¹¹ Civitavecchia,¹² and Reggio Calabria¹³), selected based on the availability of publicly documented structural data and their representative role within the national port system. The estimate was derived by aggregating available length data for primary breakwaters enclosing harbour basins, restricted to shore-connected structures directly integrated into port infrastructure. Detached offshore breakwaters, shore-parallel coastal defence works, groynes, non-port seawalls, and erosion control structures were excluded.

$$\frac{L_{seaward}}{L_{protected}} \sim \frac{500}{24} \sim 21$$

In relation to the specific powers, Table 8 allows us to get the maximum and minimum ratio of maximum available power for the seaward discharge configuration relative to the protected discharge configuration is approximately 0.24, 0.21, and 0.067 at cross-sections A (near the basin), C (between the inlet and outlet), and F (at the outlet of the conveyance system), respectively. These ratios are estimated based on a conservative comparison, defined as the maximum available power under the highest still water level with wave forcing ($SWL = 0.4$ m, wave condition) divided by the corresponding value under the baseline condition without wave forcing ($SWL = 0.0$ m, no-wave condition). Considering the most conservative case (cross-section F), the illustrative scaled ratio can be approximated as:

$$\frac{P_{seaward}}{P_{protected}} = \frac{P_{seaward}^s}{P_{protected}^s} \times \frac{L_{seaward}}{L_{protected}} = 0.067 \times \frac{500}{24} = 1.40$$

If the other cross-sections are considered, the same scaling gives higher indicative ratios, approximately $0.21 \times 500/24 \approx 4.4$ for Section C and $0.24 \times 500/24 \approx 5.0$ for Section A.

The values of 1.4–5.0 should be interpreted only as illustrative results associated with different locations along the conveyance pipe in the Ancona configuration.

This simplified scaling suggests that the greater potential installation length of Italian shore-connected breakwaters may partly offset the lower available power per unit device associated with seaward discharge. However, this comparison should be interpreted only as a first-order preliminary estimate for the Ancona-specific power ratios and the assumed structure lengths. It does not account for site-specific wave climate, breakwater orientation, local bathymetry, realistically deployable length, capacity factor, device availability, maintenance requirements, or economic costs. Therefore, it should not be interpreted as a national energy estimate, evidence of competitive advantage, or direct compensation in terms of annual energy production or economic performance.

Several limitations of the present study should be noted. The modelling framework is sequentially coupled, meaning feedback between the overtopping basin and the conveyance system is not captured within a single simulation. The conveyance runs adopt an idealised initial basin head as a representative boundary condition rather than a dynamically evolving one. Overtopping estimates carry higher uncertainty at large freeboard values, where discharge rates are small and sensitive to grid resolution, as reflected in the mesh sensitivity results.

Future work will extend the present framework toward the development of dimensionless design charts relating optimal turbine placement to inlet and outlet hydraulic forcing condition, enabling generalisation beyond the specific geometry analysed here.

These numerical findings will serve as the basis for the upcoming full-scale deployment at the Port of Ancona. Field measurements from the prototype will be used to examine the hydraulic performance and energy efficiency of the seaward-discharge OBREC system under real operating conditions, supporting future validation and refinement of the numerical framework.

CRedit authorship contribution statement

Saeed Osouli: Writing – review & editing, Writing – original draft, Validation, Methodology, Investigation, Formal analysis, Data curation, Conceptualization. **Ivan Sabbioni:** Supervision, Project administration, Funding acquisition, Data curation. **Maurizio Brocchini:** Writing – review & editing, Supervision, Project administration, Methodology, Investigation, Funding acquisition. **Gianluca Zitti:** Writing – review & editing, Methodology, Investigation, Data curation. **Matteo Postacchini:** Writing – review & editing, Supervision, Project administration, Methodology, Investigation, Funding acquisition.

⁴ https://www.isprambiente.gov.it/files/pubblicazioni/statoambiente/tematiche-2012/Cap.5_Mare_ambiente_costiero.pdf.

⁵ <https://www.fincos.it/fincosit/nuova-diga-foranea-di-genova-il-piu-grande-intervento-di-potenziamento-della-portualita-italiana/>.

⁶ <https://www.assoporti.it/media/6852/deasp-adsp-mar-ligure-orientale.pdf>.

⁷ <https://www.adspmaresiliciliaorientale.it/port-of-augusta/>.

⁸ <https://va.mite.gov.it/it-IT/Oggetti/Info/1387>.

⁹ <https://www.iris.unict.it/retrieve/dfc4d22e-c759-bb0a-e053-d805fe0a78d9/10392-Article%20Text-36645-1-11-20201230.pdf>.

¹⁰ <https://trapaniwelcome.it/en/nautical-tourism/>.

¹¹ <https://michelangelolentini.com/it/project/port-of-taranto-new-breakwater/>.

¹² https://it.wikipedia.org/wiki/Porto_di_Civitavecchia.

¹³ <https://www.portidellostretto.com/en/ports-of-the-strait/port-of-reggio-calabria/docks/#:~:text=Banchina%20di%20Ponente>.

Declaration of competing interest

There are no conflicts of interest to disclose.

Acknowledgements

The authors acknowledge the financial support provided by the

European Union's (EU) Horizon Europe Framework Programme (HORIZON) via SEDIMARE (grant agreement no 101072443), an MSCA Doctoral Network (HORIZON-MSCA-2021-DN-01).

The authors also thank Prof. Claudio Iuppa for sharing the experimental data used for the validation of the overtopping simulations.

Appendix

A. FUNWAVE-TVD

Among the main governing equations taken into account by the solver, the volume conservation is as follows:

$$\eta_t + \nabla \cdot \mathbf{M} = 0 \quad (\text{A.1})$$

$\mathbf{M}$ is the horizontal volume flux.

The depth-averaged horizontal momentum is described by the following:

$$u_{\alpha,t} + (u_{\alpha} \cdot \nabla) u_{\alpha} + g \nabla_{\eta} + V_1 + V_2 + V_3 + R = 0 \quad (\text{A.2})$$

R : diffusive and dissipative terms (e.g. bottom friction, sub-grid lateral turbulent mixing)

V_1 and V_2 are terms representing the dispersive Boussinesq terms (function of z_{α}).

V_3 is a contribution of order $O(\mu^2)$ $\mu = \frac{h}{z_{\alpha}} = \zeta h + \beta \eta$, where ζ and β are constant terms.

u_{α} denotes the velocity at a reference elevation $z = z_{\alpha}$.

B. FLOW-3D

The numerical simulations were conducted using FLOW-3D HYDRO, which solves the Reynolds-Averaged Navier–Stokes (RANS) equations on a staggered Cartesian grid using a combination of finite-volume and finite-difference methods (Flow Science, Inc., 2023). Complex geometric boundaries including the ramp, basin, turbines and pipe walls were represented through the FAVOR™ (Fractional Area/Volume Obstacle Representation) technique, which assigns fractional open area and volume values to fixed Cartesian cells, avoiding the need for body-fitted meshes (Hirt and Sicilian, 1985).

The free surface was tracked using the Volume-of-Fluid (VOF) method, which consists of three components: definition of the fluid volume fraction F , solution of the VOF transport equation, and application of boundary conditions at the free surface. Turbulence closure was provided by the RNG k - ϵ model for the ramp, basin, and conveyance system simulations, and by the k - ω model for the standalone turbine efficiency simulations.

Time advancement employed an adaptive time-step algorithm governed by the Courant–Friedrichs–Lewy (CFL) stability criterion. At each cycle the solver evaluates the advective stability condition:

$$\delta t \leq \text{CON} \cdot \min \left(\frac{V_F \delta_x}{A_x u}, \frac{V_F \delta_y}{A_y v}, \frac{V_F \delta_z}{A_z w} \right) \quad (\text{B.1})$$

Where V_F is the open volume fraction, (u, v, w) are velocity components, (A_x, A_y, A_z) are fractional open face areas, and the safety factor CON is a user-defined safety factor for the advective stability limit (Flow Science, Inc., 2023). These conditions are evaluated at every cell at each cycle, and the minimum qualifying time step is selected automatically.

Fluid–structure interaction for the rotating turbine was modelled using the General Moving Object (GMO) module, which employs a six-step iterative implicit coupling algorithm: the fluid dynamics equations and the rigid-body dynamic equations (for both translational and rotational degrees of freedom) are solved within a cyclic loop until the solution converges at each time level (Flow Science, Inc., 2023). The governing equations for VOF, FAVOR™, RNG k - ϵ , and GMO, along with their mathematical formulations, are described in Osouli et al. (2025), where the FLOW-3D HYDRO framework was validated against physical model data for a floating breakwater application.

For the standalone turbine efficiency simulations, the k - ω turbulence model (Wilcox, 1998) was employed, which solves transport equations for turbulent kinetic energy k and specific dissipation rate ω ($= \epsilon/k$, where ϵ is the turbulent dissipation rate); the FLOW-3D implementation follows (Flow Science, Inc., 2023). This model offers superior performance near wall boundaries and in flows with streamwise pressure gradients, making it well-suited for the rotating turbine geometry.

Sensitivity analysis for full-scale simulations

The energy spectrum of the NNW waves, with a density of 0.005, was extracted from FUNWAVE-TVD at gauge four. Each simulation was performed for approximately 150 s (around 20 cycles), with output data recorded at a time interval of 1/15 s. The elevation of the free surface was captured throughout the entire simulation duration.

The snapshot of a typical simulation is shown in Fig. 24. The locations of the six wave gauges in the FLOW-3D numerical domain are listed in Table 10, with G5 located at the toe of the structure. The domain dimensions are 232 m in length, 1 m in width, and 10 m in height, with a still water depth of 6 m. The geometry is identical to model 5 in Table 5. The k - ϵ (RNG) turbulence model was employed to represent Reynolds stresses due to its enhanced accuracy over the standard k - ϵ model and its significantly lower computational cost compared to large eddy simulation (LES) for wave-field analysis (Yin et al., 2023). In addition, the current study shows that the RNG model provides reliable results (Table 4 and Fig. 12). The Flow-3D

simulations were conducted on a workstation equipped with 128 GB of RAM and an Intel® Core™ i9–14900KF processor (3.20 GHz, 8 cores).

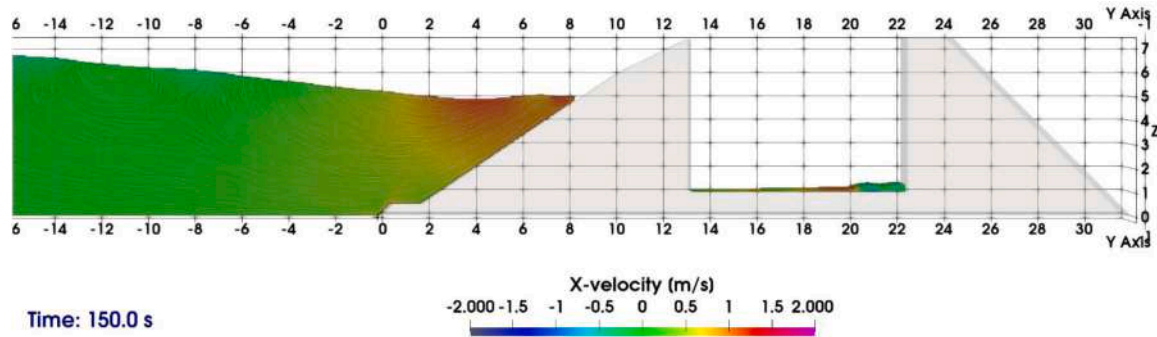


Fig. 24. Numerical domain.

Table 11, Figs. 25 and 26 present the results of the mesh independence study for a prototype scale of the ramp. Uniform cell sizes were applied over the entire domain using a single mesh block. In one model, two mesh blocks were employed, with a cell size of 0.05 m over the whole domain and a finer resolution of 0.03 m in the vicinity of the structure. As the total number of cells increased, the calculated reflection coefficients and H_{m0} converged, reaching a stable value at approximately 928,000 cells. Therefore, this cell count was adopted for the remaining simulations. The corresponding mesh resolution is 0.05 m in both x and z directions, while only one cell was used in the y direction, resulting in a two-dimensional numerical model.

Palma et al. (2020) modelled an OBREC using OpenFOAM and, without studying the mesh sensitivity, used ratios of $l/\Delta x \approx 100$ and $H_s/\Delta z \approx 10$, with the later ratio reaching 60 near the structure (Palma et al., 2020). In the current study, being $l \approx 50\text{m}$ and $H_s \approx 1\text{m}$, the ratios are $l/\Delta x \approx 1000$ and $H_s/\Delta z \approx 20$, respectively. The mesh-independence study at laboratory scale used the ratio of ~ 200 and 8 respectively (Table 4).

For the reflection coefficient (Cr), the mesh-independence study converged to a value of 0.79. Similarly, H_{m0} converged to approximately 0.91 m. However, for q^* , the differences between successive mesh refinements remain on the order of 30–35%, indicating a higher sensitivity of overtopping discharge to grid resolution.

Table 10

Locations of wave gauges in the FLOW-3D numerical domain.

G1	G2	G3	G4	G5 (toe of structure)	G6
−32.5	−30	−27.5	−25	0	−150

Table 11

Mesh independence study.

Cell size	1 m	0.5 m	0.1 m	0.05 m	0.05 - 0.03 m
Number of cells	2,320	9,280	232,000	928,000	1,278,649
H_{m0}	0.8421	0.8679	0.9066	0.9102	0.9102
Cr	0.6	0.66	0.79	0.79	0.79
Overtopped Volume (m^3)	2.1	2.69	1.49	1.74	2.38
q^*	0.0058	0.0071	0.0037	0.0043	0.0058
Simulation Time (mins)	6	9	70	324	1,441

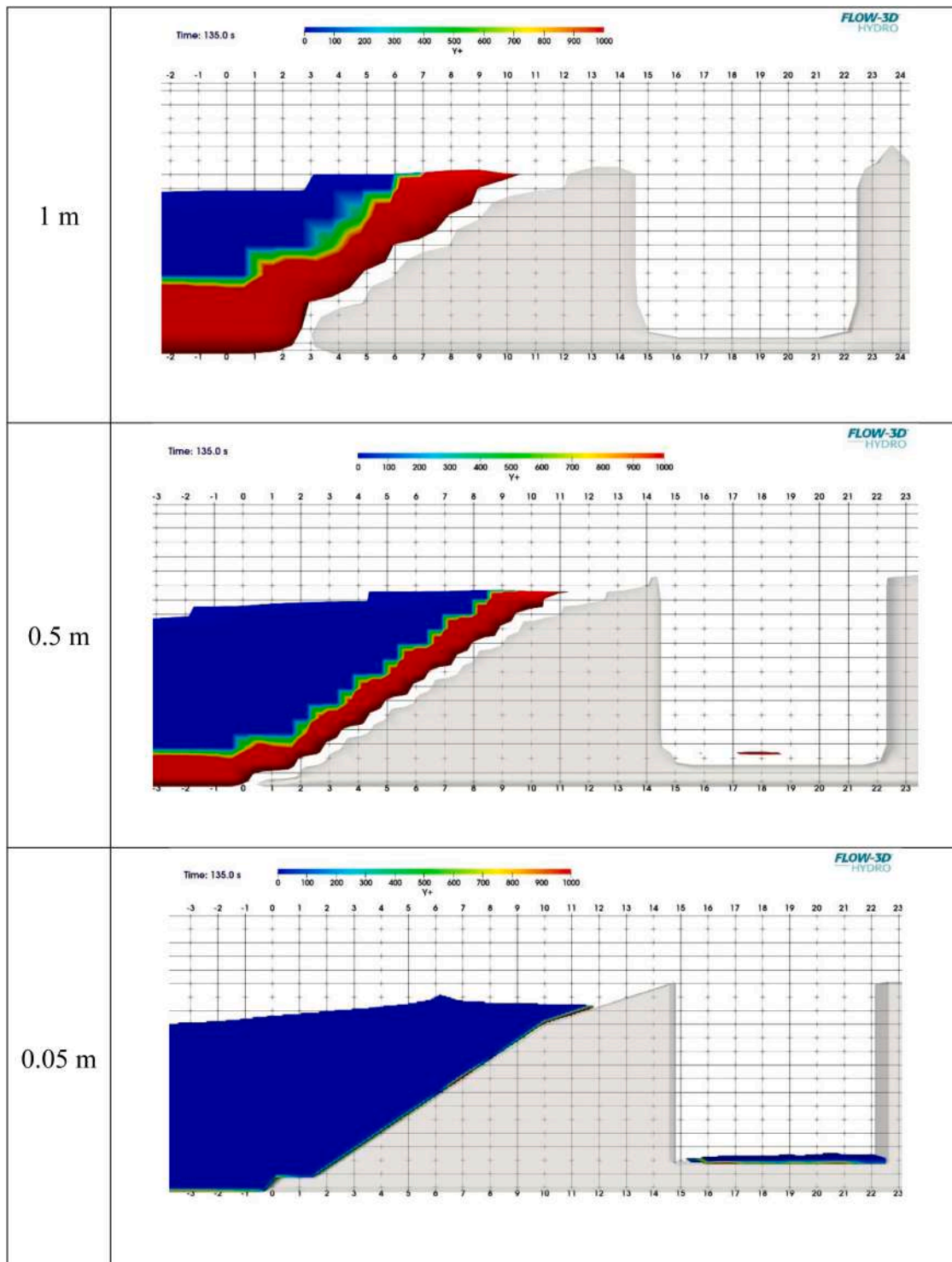


Fig. 25. Y^+ (dimensionless wall distance) values in the vicinity of the OBREC for different cell sizes.

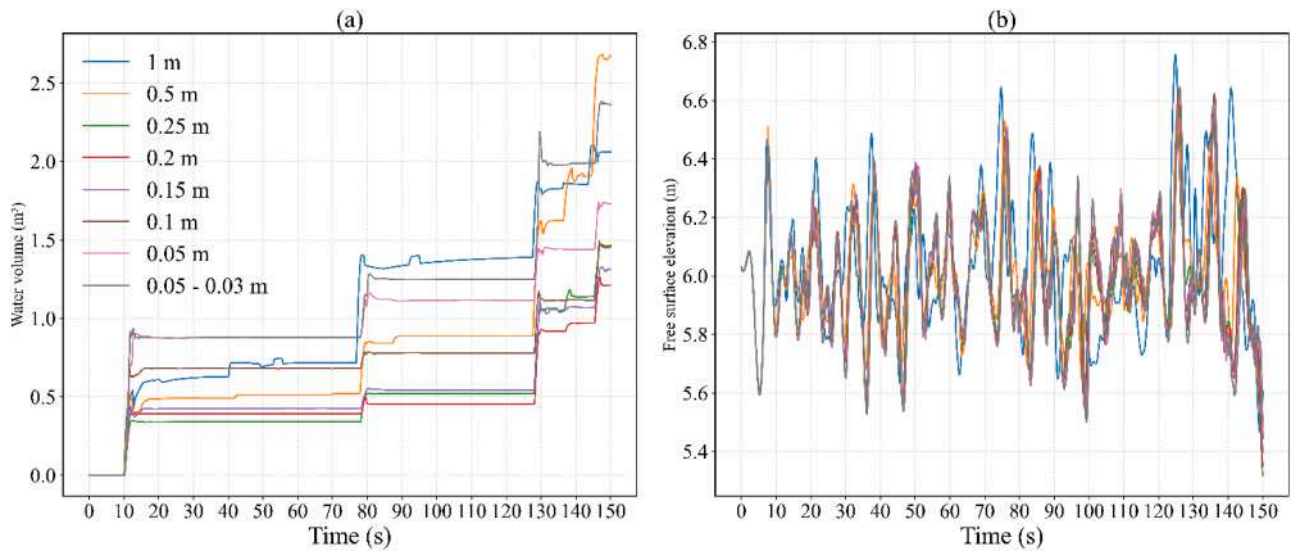


Fig. 26. Overtopped volume (a) and free-surface elevation at the toe of the device for different cell sizes (b).

Wave overtopping: alternative visualization

Fig. 27 illustrates the overtopping results in a different order than in Fig. 15, highlighting that a slope of 18° under the NNW scenario with probability density of 0.005 and 1 m freeboard with 7.5 m width produces the maximum discharge of $0.27 \text{ m}^3/\text{s}$, whereas a slope of 26° with 1.5 m freeboard under the same scenario yields a flow rate of $0.06 \text{ m}^3/\text{s}$. EurOtop suggests that, for wind waves and a relative freeboard of 2.5, slopes from 1:2 to 1:3 (27° to 18°) yield the largest overtopping (van der Meer et al., 2018).

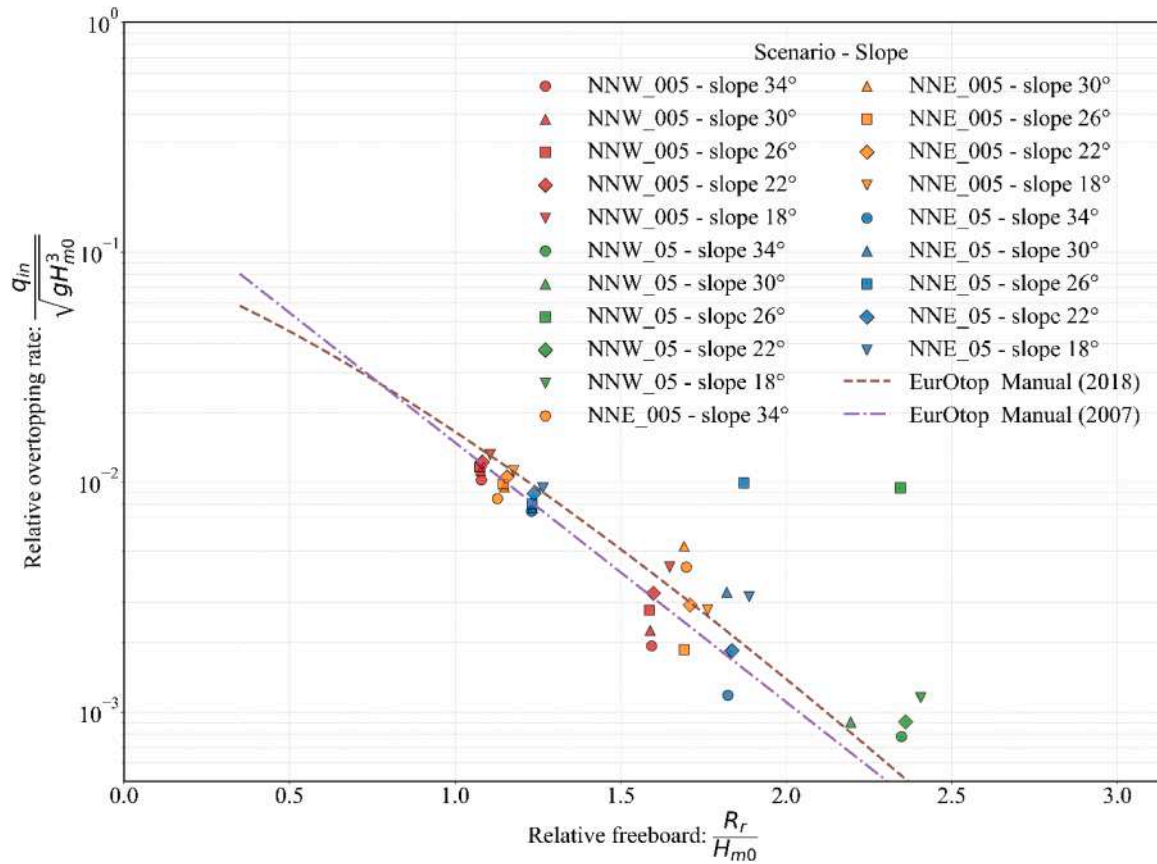


Fig. 27. Numerical wave overtopping discharges grouped by slope and scenario.

References

- Aref, R.M., Shehata, A.S., Khairy, Y., Khairat, M.M., 2023. Applying an overtopping wave energy converter device on San Stefano breakwater Alexandria Egypt. In: AIP Conf. Proc. American Institute of Physics Inc. <https://doi.org/10.1063/5.0129517>
- Barbosa, D.V.E., Santos, A.L.G., dos Santos, E.D., Souza, J.A., 2019. Overtopping device numerical study: openfoam solution verification and evaluation of curved ramps performances. *Int. J. Heat Mass Transf.* 131, 411–423. <https://doi.org/10.1016/j.jheatmasstransfer.2018.11.071>.
- Cavallaro, L., Iuppa, C., Castiglione, F., Musumeci, R.E., Foti, E., 2020. A simple model to assess the performance of an overtopping wave energy converter embedded in a port breakwater. *J. Mar. Sci. Eng.* 8, 858. <https://doi.org/10.3390/jmse8110858>.
- Contestabile, P., Crispino, G., Di Lauro, E., Ferrante, V., Gisonni, C., Vicinanza, D., 2020. Overtopping breakwater for wave energy conversion: review of state of art, recent advancements and what lies ahead. *Renew. Energy* 147, 705–718. <https://doi.org/10.1016/j.renene.2019.08.115>.
- Contestabile, P., Di Lauro, E., Buccino, M., Vicinanza, D., 2017a. Economic assessment of overtopping Breakwater for energy conversion (OBREC): a case study in Western Australia. *Sustainability* 9, 51. <https://doi.org/10.3390/su9010051>.
- Contestabile, P., Iuppa, C., Di Lauro, E., Cavallaro, L., Andersen, T.L., Vicinanza, D., 2017b. Wave loadings acting on innovative rubble mound breakwater for overtopping wave energy conversion. *Coast. Eng.* 122, 60–74. <https://doi.org/10.1016/j.coastaleng.2017.02.001>.
- Darma, Y.Y.E., Rahmawati, S., Rosyid, D.M., Pratama, R.H., Kusuma, A.I., 2026. Overview of advancements in overtopping breakwater for wave energy conversion (OBREC): harnessing coastal power for renewable energy and enhanced coastal protection. *Renew. Sustain. Energy Rev.* 226, 116399. <https://doi.org/10.1016/j.rser.2025.116399>.
- Dean, R.G., Dalrymple, R.A., 1991. *Water Wave Mechanics For Engineers and Scientists*. WORLD SCIENTIFIC. <https://doi.org/10.1142/1232>.
- Di Lauro, E., Lara, J.L., Maza, M., Losada, I.J., Contestabile, P., Vicinanza, D., 2019. Stability analysis of a non-conventional breakwater for wave energy conversion. *Coast. Eng.* 145, 36–52. <https://doi.org/10.1016/j.coastaleng.2018.12.008>.
- Di Lauro, E., Maza, M., Lara, J.L., Losada, I.J., Contestabile, P., Vicinanza, D., 2020. Advantages of an innovative vertical breakwater with an overtopping wave energy converter. *Coast. Eng.* 159, 103713. <https://doi.org/10.1016/j.coastaleng.2020.103713>.
- Draycott, S., Steynor, J., Davey, T., Ingram, D.M., 2018. Isolating incident and reflected wave spectra in the presence of current. *Coast. Eng. J.* 60, 39–50. <https://doi.org/10.1080/05785634.2017.1418798>.
- Flow Science, Inc., 2023. FLOW-3D, Version—2023R1. <https://www.flow3d.com/>.
- Goda, Y., 2010. *Random Seas and Design of Maritime Structures*.
- Hirt, C.W., Nichols, B.D., 1981. Volume of fluid (VOF) method for the dynamics of free boundaries. *J. Comput. Phys.* 39, 201–225. [https://doi.org/10.1016/0021-9991\(81\)90145-5](https://doi.org/10.1016/0021-9991(81)90145-5).
- Hirt, C.W., Sicilian, J.M., 1985. A porosity technique for the definition of obstacles in rectangular cell meshes. In: *Fourth International Conference on Numerical Ship Hydrodynamics*. Office of Naval Research, Washington, DC, pp. 450–468.
- Iuppa, C., Cavallaro, L., Musumeci, R.E., Vicinanza, D., Foti, E., 2019. Empirical overtopping volume statistics at an OBREC. *Coast. Eng.* 152, 103524. <https://doi.org/10.1016/j.coastaleng.2019.103524>.
- Iuppa, C., Contestabile, P., Cavallaro, L., Foti, E., Vicinanza, D., 2016. Hydraulic performance of an innovative breakwater for overtopping wave energy conversion. *Sustainability* 8, 1226. <https://doi.org/10.3390/su8121226>.
- Jäger, W.S., Nagler, T., Czado, C., McCall, R.T., 2019. A statistical simulation method for joint time series of non-stationary hourly wave parameters. *Coast. Eng.* 146, 14–31. <https://doi.org/10.1016/j.coastaleng.2018.11.003>.
- Kralli, V.E., Theodossiou, N., Karambas, T., 2019. Optimal design of overtopping breakwater for energy conversion (OBREC) systems using the harmony search algorithm. *Front. Energy Res.* 7. <https://doi.org/10.3389/fenrg.2019.00080>.
- Lubitz, W.D., Lyons, M., Simmons, S., 2014. Performance model of Archimedes screw hydro turbines with variable fill level. *J. Hydraul. Eng.* 140. [https://doi.org/10.1061/\(asce\)hy.1943-7900.0000922](https://doi.org/10.1061/(asce)hy.1943-7900.0000922).
- McBride, M.~W., Allsop, N.~W.~H., Besley, P., Colombo, D., Madurini, L., 1995. *Vertical Walls and Low Reflection Alternatives: Results of Wave Flume Tests*. <https://eprints.hrwallingford.com/357/>.
- Musa, M.A., Roslan, M.F., Ahmad, M.F., Muzathik, A.M., Mustapa, M.A., Fitriadhy, A., Mohd, M.H., Rahman, M.A.A., 2020. The influence of ramp shape parameters on performance of overtopping breakwater for energy conversion. *J. Mar. Sci. Eng.* 8, 875. <https://doi.org/10.3390/jmse8110875>.
- Osouli, S., Hakimzadeh, H., Kardan, N., 2025. Study on the effect of the triangular cogged floor of floating breakwaters on wave transmission coefficient. *J. Ocean Eng. Mar. Energy* 11, 1095–1118. <https://doi.org/10.1007/s40722-025-00425-6>.
- Palma, G., Contestabile, P., Zanuttigh, B., Formentin, S.M., Vicinanza, D., 2020. Integrated assessment of the hydraulic and structural performance of the OBREC device in the Gulf of Naples, Italy. *Appl. Ocean Res.* 101, 102217. <https://doi.org/10.1016/j.apor.2020.102217>.
- Palma, G., Formentin, S.M., Zanuttigh, B., Contestabile, P., Vicinanza, D., 2019. Numerical simulations of the hydraulic performance of a breakwater-integrated overtopping wave energy converter. *J. Mar. Sci. Eng.* 7, 38. <https://doi.org/10.3390/jmse7020038>.
- Pullen, T., 2007. *Wave Overtopping of Sea Defences and related Structures : Assessment Manual*. Boyens.
- Rohmer, J., Knittel, D., Sturtzer, G., Flieller, D., Renaud, J., 2016. Modeling and experimental results of an Archimedes screw turbine. *Renew. Energy* 94, 136–146. <https://doi.org/10.1016/j.renene.2016.03.044>.
- Romano, A., Bellotti, G., Briganti, R., Franco, L., 2015. Uncertainties in the physical modelling of the wave overtopping over a rubble mound breakwater: the role of the seeding number and of the test duration. *Coast. Eng.* 103, 15–21. <https://doi.org/10.1016/j.coastaleng.2015.05.005>.
- Scott, D.W., 2015. *Multivariate Density Estimation: Theory, Practice, and Visualization*, 2nd ed. John Wiley & Sons, Hoboken, NJ.
- Serinaldi, F., Briganti, R., Kilsby, C.G., Dodd, N., 2022. Sailing synthetic seas: stochastic simulation of benchmark sea state time series. *Coast. Eng.* 176, 104164. <https://doi.org/10.1016/j.coastaleng.2022.104164>.
- Shahverdi, K., Loni, R., Ghabadian, B., Gohari, S., Marofi, S., Bellos, E., 2020. Numerical optimization study of Archimedes screw Turbine (AST): a case study. *Renew. Energy* 145, 2130–2143. <https://doi.org/10.1016/j.renene.2019.07.124>.
- Shahverdi, K., Loni, R., Maestre, J.M., Najafi, G., 2021. CFD numerical simulation of Archimedes screw turbine with power output analysis. *Ocean Eng.* 231, 108718. <https://doi.org/10.1016/j.oceaneng.2021.108718>.
- Shi, F., Harris, J.C., Kirby, J.T., Grilli, S., 2011. FUNWAVE-TVD Fully Nonlinear Boussinesq Wave Model with TVD Solver Documentation and User's Manual (Version 3.0) Release Dec 2016 WITH TVD SOLVER DOCUMENTATION AND USER'S MANUAL (VERSION 1.0). <https://github.com/fengyanshi/FUNWAVE-TVD>.
- J.W. van der Meer, N.W.H. Allsop, T. Bruce, J. De Rouck, A. Kortenhaus, T. Pullen, H. Schüttrumpf, P. Troch, B. Zanuttigh, *EurOtop: manual on wave overtopping of sea defences and related structures : an overtopping manual largely based on European research, but for worldwide application*, 2018.
- Vicinanza, D., Contestabile, P., Ferrante, V., Contestabile, P., 2011. Estimation of the Wave Energy in the Italian Offshore. www.italywavenergy.it.
- Vicinanza, D., Contestabile, P., Quvang Hærck Nørgaard, J., Lykke Andersen, T., 2014. Innovative rubble mound breakwaters for overtopping wave energy conversion. *Coast. Eng.* 88, 154–170. <https://doi.org/10.1016/j.coastaleng.2014.02.004>.
- Wilcox, D.C., 1998. *Turbulence Modeling for CFD*. DCW Industries, Incorporated. <https://books.google.it/books?id=VwIRAAAMAAJ>.
- Yin, Z., Ni, Z., Luan, Y., Zhang, X., Li, J., Li, Y., 2023. Hydrodynamic characteristics of a box-type floating breakwater with restrained piles under regular waves. *Ocean Eng.* 280, 114408. <https://doi.org/10.1016/j.oceaneng.2023.114408>.
- Zelt, J.A., Skjelbreia, J.E., Technologies, W., 1993. Estimating incident and reflected wave fields using an arbitrary number of wave gages. In: *Proceedings of the Coastal Engineering Conference*, 1, pp. 777–788. <https://doi.org/10.1061/9780872629332.058>.
- Zitti, G., Fattore, F., Brunori, A., Brunori, B., Brocchini, M., 2020. Efficiency evaluation of a ductless Archimedes turbine: laboratory experiments and numerical simulations. *Renew. Energy* 146, 867–879. <https://doi.org/10.1016/j.renene.2019.06.174>.