

Article

Depth-Dependent Wave-Energy Contribution-Based Parametric Study of Submerged Ramp Design for a Caisson-Type Overtopping Wave Energy Converter: Hydraulic Efficiency and Construction-Economy Assessment

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Abstract

Breakwater-integrated caisson-type overtopping wave energy converters (OWECs) can retrofit port infrastructure with energy recovery, and their performance is strongly influenced by submerged-ramp geometry that governs underwater wave-particle motion, reflection, recirculation, and localized breaking. This study establishes a depth-selection framework based on the cumulative distribution of wave-induced kinetic energy from linear wave theory and applies weakly compressible smoothed particle hydrodynamics (WCSPH) simulations using DualSPHysics under regular waves to quantify how hydraulic efficiency responds to ramp slope and installation depth for single-slope designs. Guided by these trends, a segmented multi-angle ramp is proposed to preserve the upper-slope function required for overtopping while reducing submerged volume and foundation demand. Performance is assessed by combining hydraulic efficiency with construction-quantity-based economy indices. Results show that deeper ramps generally enhance efficiency but with diminishing returns, and that the preferred slope depends on the installation depth. In suitable depth ranges, segmented ramps provide a practical compromise between material savings and retained performance. The proposed procedure supports early-stage geometry screening and robust depth-range selection across site conditions.

Keywords: overtopping wave energy converter; caisson-type OWEC; wave kinetic energy contribution; DualSPHysics; hydraulic efficiency; construction economy

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1. Introduction

The primary purpose of coastal and harbor structures is to absorb, reflect, and block offshore waves to secure calm conditions in sheltered waters and to support safe berthing and cargo-handling operations. Waves, from the structural design perspective, are external loads that must be dissipated; however, they can also be regarded as recoverable renewable energy resources [1,2]. In particular, in sea areas where large-scale infrastructures such as breakwaters, wharves, and caissons are already installed, integrating marine-energy conversion devices into existing structures can add electricity-generation

functionality while minimizing additional infrastructure investment. Recently, beyond retrofitting and strengthening existing structures with auxiliary devices, the concept of multi-functional structures where wave-energy recovery is incorporated at the design stage of new coastal/harbor structures has been proposed [3–11].

Marine energy exists in various forms, including tidal range, tidal currents, ocean thermal gradients, salinity gradients, and waves. Among them, wave energy has relatively high energy density but also exhibits variability with seasonal and meteorological conditions. Wave energy converters (WECs) are commonly classified by operating principle into oscillating body systems, oscillating water column systems, and overtopping systems [1,2]. An overtopping-type wave energy converter (OWEC) converts wave kinetic energy into potential energy by allowing incident waves to run up along a front ramp, overtop into a reservoir, and then drive low-head turbines using the stored water head.

Many studies have reported that overtopping devices are structurally simple, enabling stable structural design, and are well-suited for integration with breakwaters [3,12,13]. Breakwater-integrated OWECs can be categorized into rubble-mound types (e.g., OBREC) that utilize the front slope and caisson-based monolithic types (e.g., OBREC-V, SSG) [14–19]. Slope-mounted types can be relatively easily linked to existing sloped faces; however, hydraulic and structural design constraints strongly depend on the placement and armor conditions of rubble-mound layers. In contrast, caisson-based monolithic systems offer advantages in standardization and mass production because they are typically fabricated onshore and then towed and installed offshore, and they can be readily integrated with the caisson-type breakwaters widely used in ports. Nevertheless, the front and submerged-slope geometries directly affect not only energy efficiency but also weight, buoyancy, stability, constructability, and fabrication cost [20,21].

A caisson-based OWEC inevitably includes submerged structural components (a submerged ramp and front underwater geometry). Overtopping discharge is governed by incident-wave conditions and the run-up process at the front, while the submerged geometry influences the underwater velocity and pressure fields, front reflection, wave deformation, localized breaking, and recirculation-zone formation, and can therefore affect energy efficiency. Despite this, existing studies on slope geometry have tended to focus on design parameters above or near the free surface, and submerged ramps have often been examined only in limited forms such as simple extensions, single-slope changes, or curvature modifications [21,22]. In particular, systematic studies linking ramp installation depth and geometry to the depth-dependent kinetic-energy distribution of waves remain relatively limited [23,24].

Wave-particle motion decays with depth, and the decay characteristics depend on wave period and water depth. For the same wave height, long-period and deep-water conditions can maintain velocity influence at relatively deeper levels, whereas short-period and shallow-water conditions tend to concentrate energy near the surface. Therefore, if the required submerged-ramp depth is uniformly set as the full water depth, unnecessary increases in structural volume may result; conversely, overly shallow design may reduce overtopping discharge or induce unfavorable reflection/recirculation. Moreover, for caisson-type OWECs fabricated onshore and installed offshore, expansion of submerged structures increases concrete volume and self-weight, leading to higher transportation/installation difficulty and increased construction cost. Thus, the submerged-ramp installation depth and geometry simultaneously control power-generation performance and fabrication/transport/installation and foundation costs, and shape optimization to minimize structure size while meeting target overtopping performance is a key design task.

Despite the growing literature on OWECs, submerged-ramp design has often been treated only through simple extensions or single-slope modifications, and the rationale

for selecting the installation depth has rarely been linked to the depth-decaying distribution of wave-induced kinetic energy. Moreover, systematic parametric maps that jointly quantify efficiency sensitivity to both installation depth and ramp slope for caisson-type OWECs remain limited. To address these gaps, this study proposes a kinetic-energy-contribution-based depth-selection procedure and performs simulations to support early-stage geometry screening of single-slope and segmented submerged-ramp designs using combined efficiency–economy indices.

This study quantitatively evaluates how submerged-ramp design parameters (installation depth and slope angle) affect hydraulic efficiency and economy in a caisson-type OWEC while keeping the previously validated upper overtopping–storage geometry fixed. First, under the design-wave condition, linear wave theory is used to compute the normalized cumulative depth distribution of kinetic energy, which is used as a baseline for selecting installation depth. Then, under regular-wave conditions, WCSPH-based DualSPHysics simulations are performed while systematically varying the slope angle and installation depth of a single-slope submerged ramp to compare efficiency sensitivity. Based on the single-slope results, representative depth ranges with strong efficiency contribution are identified, and a three-segment multi-angle ramp geometry is designed to assess material-reduction potential compared with the single-slope configuration. Finally, hydraulic efficiency is evaluated together with construction-quantity-based cost proxy indices and the cost per unit hydraulic power to compare overall performance from an efficiency–economy balance perspective. Because marine construction cost is strongly affected by constructability (e.g., formwork, segment joints, and underwater installation), material-volume reduction alone may not translate into lower total cost for a multi-segment ramp. Thus, this study uses a quantity-based CAPEX proxy (submerged area and foundation-rock length) for early-stage relative screening, and the economic results should be interpreted as comparative indicators rather than absolute cost conclusions. The results are intended to provide practice-oriented criteria applicable to early-stage geometry design and submerged-ramp design-range selection for bottom-fixed caisson-type OWECs in sea areas with similar wave conditions.

The remainder of this paper is organized as follows. Section 2 defines the depth-dependent cumulative kinetic-energy distribution based on linear wave theory and presents the WCSPH-based DualSPHysics simulation setup and performance metrics (hydraulic efficiency, cost proxy, and cost per unit hydraulic power). Section 3 compares and analyzes hydraulic efficiency and economy for single-slope and multi-segment (three-segment) submerged ramps. Section 4 summarizes the main conclusions and future research needs.

2. Methods and Theoretical Background

This section presents the theoretical background and numerical workflow used to quantify how the submerged-ramp geometry of a caisson-type overtopping wave energy converter (OWEC) influences wave-energy utilization and structural scale. The installation depth is parameterized using a depth-dependent cumulative distribution of wave-induced kinetic energy derived from linear wave theory, and systematic simulations are conducted under regular waves using DualSPHysics (WCSPH) while varying ramp slope and depth. Hydraulic efficiency based on the overtopping-stored potential energy, together with construction-economy indices reflecting submerged quantities and foundation demand, is defined to enable an integrated efficiency–economy comparison.

2.1. Depth-Dependent Kinetic-Energy Contribution Framework

2.1.1. Linear Wave Theory and Kinetic-Energy Distribution

To quantify the kinetic-energy distribution in the submerged region under the design-wave condition, linear wave theory was applied [25–27]. The coordinate system is defined such that the wave propagation direction is x and the vertical upward direction is z . The still water level (SWL) is $z = 0$, and the seabed is $z = -d$. The free-surface elevation of a regular wave is:

$$\eta(x, t) = \frac{H}{2} \cos(kx - \omega t) \quad (1)$$

where H is wave height, T is wave period, $\omega = 2\pi/T$ is angular frequency, and k is wavenumber. The velocity potential ϕ is:

$$\phi(x, z, t) = \frac{gH}{2\omega} \frac{\cosh k(z + d)}{\cosh(kd)} \sin(kx - \omega t) \quad (2)$$

and k satisfies the dispersion relationship:

$$\omega^2 = gk \tanh(kd) \quad (3)$$

Velocity components are obtained by differentiating ϕ . The horizontal component u and vertical component w are:

$$u = \frac{gHk}{2\omega} \frac{\cosh k(z + d)}{\cosh(kd)} \cos(kx - \omega t) \quad (4)$$

$$w = \frac{gHk}{2\omega} \frac{\sinh k(z + d)}{\cosh(kd)} \sin(kx - \omega t) \quad (5)$$

The kinetic-energy density per unit volume at depth z is:

$$e_k(z) = \frac{1}{2} \rho (u^2 + w^2) \quad (6)$$

To directly link $e_k(z)$ to the structural design variable (installation depth), the vertical distribution was reformulated into a normalized contribution function and its cumulative distribution. Let $D = -z$ be the depth measured downward from the free surface ($0 \leq D \leq d$). The normalized contribution function $f(D)$ is defined as:

$$f(D) = \frac{2k \cosh(2k(d - D))}{\sinh(2kd)} \quad (7)$$

Thus, $f(D)$ represents the relative contribution of the vicinity of depth D to the total kinetic energy. The cumulative distribution $F(D)$ becomes:

$$F(D) = \int_0^D f(D') dD' = 1 - \frac{\sinh(2k(d - D))}{\sinh(2kd)} \quad (8)$$

with boundary conditions $F(0) = 0$ and $F(d) = 1$. Therefore, $F(D)$ is defined as the cumulative fraction of kinetic energy from the free surface down to depth D .

2.1.2. Design Wave Conditions

The target sea area is set to the wave environment near Ulleungdo Island in the East Sea (Republic of Korea), which has been adopted and discussed as a relevant candidate condition for OWEC applications in a previous study [28]. In the present work, this site condition is used as a representative design-wave reference to enable relative comparisons and early-stage screening of submerged-ramp design variables (installation depth and slope angle), rather than to validate absolute field performance. The prototype design wave is defined as $T_p = 7.37$ s, $H_s = 2.8$ m, and $d = 18$ m. Because the present work uses

regular-wave simulations, the significant wave height H_s is treated as the representative regular-wave height H , and wave height is henceforth denoted as H . The representative period uses the spectral peak period T_p .

Numerical simulations were conducted at model scale with Froude similarity (1/20 scale). The scaled design-wave parameters are $H_m = 0.14$ m, $T_m = 1.648$ s, and $d_m = 0.9$ m (Table 1). Using Equation (8), the depth D corresponding to specific cumulative kinetic-energy ratios was computed and summarized in Figure 1 and Table 2.

Table 1. Wave conditions used in the study [19].

	T_p (s)	H (m)	d (m)
Prototype	7.37	2.8	18
Scaled (1/20)	1.648	0.14	0.9

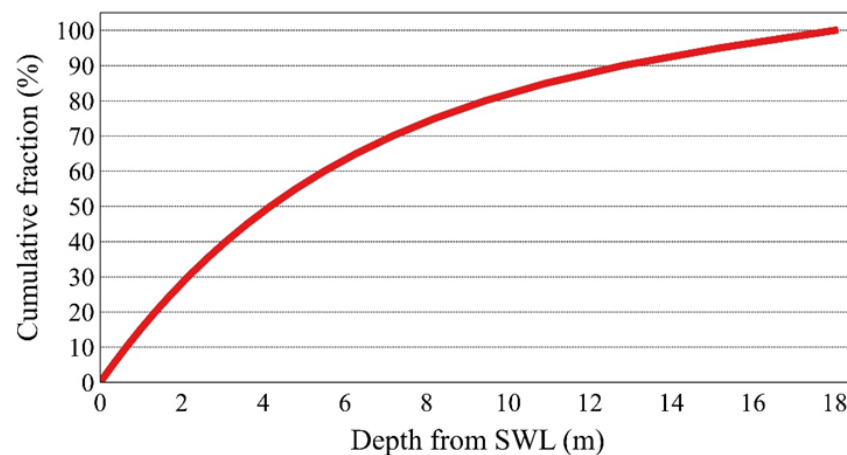


Figure 1. Cumulative distribution of wave-induced kinetic energy with depth.

Table 2. Water depth corresponding to the cumulative kinetic-energy ratio $F(D)$.

$F(D)$ (%)	$D_{prototype}$ (m)	D_{scaled} (m)	D/d
0	0.0000	0.00	0.0000
5	0.3103	0.02	0.0172
10	0.6371	0.03	0.0354
15	0.9824	0.05	0.0546
20	1.3482	0.07	0.0749
25	1.7373	0.09	0.0965
30	2.1528	0.11	0.1196
35	2.5983	0.13	0.1443
40	3.0786	0.15	0.1710
45	3.5994	0.18	0.2000
50	4.1681	0.21	0.2316
55	4.7941	0.24	0.2663
60	5.4899	0.27	0.3050
65	6.2724	0.31	0.3485
70	7.1651	0.36	0.3981
75	8.2016	0.41	0.4556
80	9.4317	0.47	0.5240
85	10.9303	0.55	0.6072
90	12.8063	0.64	0.7115
95	15.1782	0.76	0.8432
100	18.0000	0.90	1.0000

2.2. OWEC Model Configuration and Design Variables

2.2.1. Caisson-Type OWEC Model

The upper geometry of the caisson-type multi-level overtopping OWEC used in this study was adopted from a previously validated reference model [28]. This choice minimizes performance variations associated with the upper overtopping–storage mechanism so that efficiency differences caused by submerged-ramp geometry changes—the key focus of this study—can be isolated and interpreted. The submerged structure consists of a ramp connected to the caisson foundation, and it directly influences wave deformation, reflection, and localized breaking patterns by modifying the underwater velocity and pressure fields.

In particular, the submerged ramp slope angle θ_{ramp} and the submerged toe depth d_{ramp} are key geometric parameters that simultaneously change the submerged volume (material quantity) and the seabed contact length. Therefore, they can strongly affect hydraulic efficiency sensitivity. In this study, the upper slope and reservoir dimensions were fixed, while only θ_{ramp} and d_{ramp} were systematically varied. In addition, because ramp length L_{ramp} and the submerged slope area changes with these variables, geometric indices representing structural scale were also computed to enable economy-oriented comparisons. The geometry and parameter definitions are shown in Figure 2, and key dimensions are listed in Table 3.

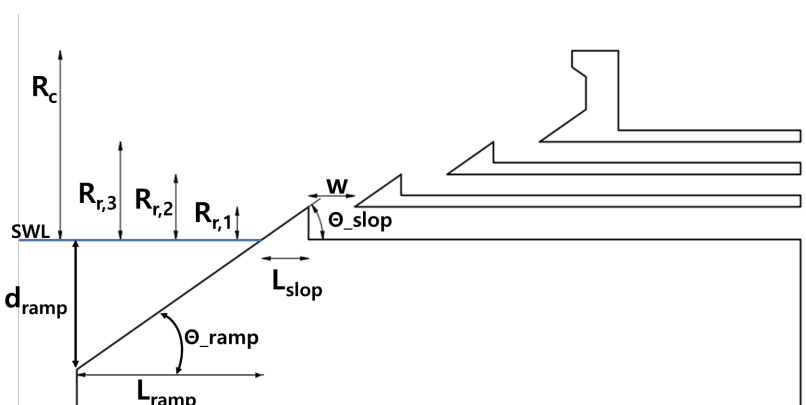


Figure 2. OWEC geometry and definition of geometric parameters [28].

Table 3. Key dimensions of the OWEC model [28].

Symbol	Prototype	Scaled (1/20)	Description
L_{ramp} (m)	15.68	0.784	Horizontal length of the submerged ramp
d_{ramp} (m)	4.0	0.2	Depth of the submerged ramp
θ_{ramp} (°)	35	35	Slope angle of the submerged ramp
L_{slop} (m)	1.42	0.071	Horizontal length of the upper slope
θ_{slop} (°)	35	35	Slope angle of the upper slope
w (m)	0.54	0.027	Slot horizontal length
R_c (m)	5.80	0.29	Crest freeboard
$R_{r,1}$ (m)	1.0	0.05	Elevation of reservoir 1
$R_{r,2}$ (m)	2.0	0.1	Elevation of reservoir 2
$R_{r,3}$ (m)	3.0	0.15	Elevation of reservoir 3

2.2.2. Parametric Cases for Single-Slope Submerged Ramps

To analyze the influence of submerged-ramp geometry on energy efficiency, the slope angle θ_{ramp} and installation depth d_{ramp} were selected as the primary design variables. The slope angle θ_{ramp} changes the run-up path length and flow components

(especially vertical velocity) in front of the structure, thereby affecting overtopping initiation, localized breaking, and recirculation formation. Previous studies on overtopping devices and similar coastal structures suggested an applicable slope-angle range of approximately 19–35° [29]. Therefore, this study adopted a range of 20–35° and discretized it at 5° intervals (20°, 25°, 30°, 35°) to screen efficiency sensitivity without excessively increasing the number of cases.

The installation depth d_{ramp} was defined using the cumulative kinetic-energy ratio $F(D)$ so that the depth-decay characteristics of wave kinetic energy are explicitly reflected. The exploration range was set to 10–80%. Ratios below 10% correspond to very shallow depths immediately below the surface, where submerged effects may excessively overlap with the upper overtopping process; ratios above 80% require extending the structure into depths with relatively small energy contribution, resulting in large structural-scale increases relative to efficiency gain. This study defined 15 depth levels at 5% increments and combined them with the four angles to form 60 single-slope cases. Each case was assigned an identifier in the form $a_{\theta_per_F(D)}$ (e.g., $a_{20_per_10}$). The full case list is provided in Appendix A1, and a schematic is shown in Figure 3.

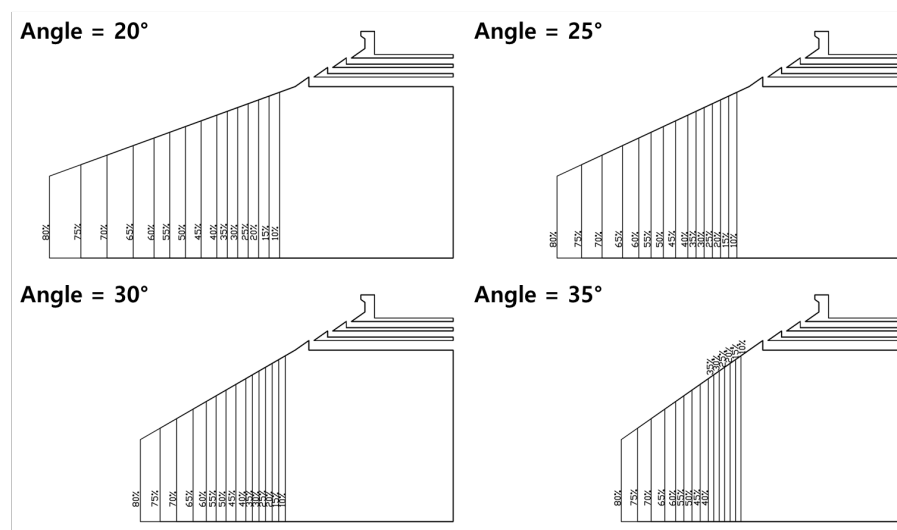


Figure 3. Schematic of single-slope ramp configurations.

2.3. WCSPH Numerical Simulations Settings

Numerical simulations were conducted using DualSPHysics v5.4, an open-source solver based on WCSPH [30]. The modeling setup and computational parameters follow a previously validated numerical environment [28], applying the same numerical wave-tank concept and procedures (Figure 4). Simulations were performed using a two-dimensional (x–z) model representing a vertical section along the wave-propagation direction.

The initial particle spacing was set to $dp = 0.005$ m to adequately resolve flow gradients near boundaries and overtopping flow. The smoothing length was defined as $h = coefh \cdot dp$, and $coefh = 1.4$ was used to balance interaction radius and numerical stability. Seawater density was set to $\rho = 1025$ kg/m³ and gravitational acceleration to $g = 9.81$ m/s². Other detailed parameters—such as the equation of state for pressure, time-integration algorithm, boundary treatment, viscosity and density-diffusion terms—follow the validated settings in the previous work [28]. This ensures that performance changes obtained in this study can be mainly attributed to changes in submerged-ramp design variables.

The numerical wave tank is a two-dimensional recirculating tank (Figure 4). The total tank length is $L_T = 26$ m, and the water depth is fixed at $d = 0.9$ m. Wave generation

and reflected-wave control were implemented using DualSPHysics relaxation zones, enabling simultaneous wave generation and active absorption [31,32]; the relaxation-zone length is $L_{RZ} = 6$ m. The structure front was placed 11 m downstream of the relaxation zone to secure a sufficient wave-development distance. Damping areas 1 and 2 were placed at the front and rear of the tank to suppress numerical reflections and nonphysical vortices at the recirculation-channel boundaries.

To prevent particle-number reduction in the main tank due to reservoir accumulation of overtopped water, the rear of the structure was idealized as a drainable condition, and a circulation path was included so that drained particles re-enter the main tank through the recirculation channel. Overtopping discharge was measured using the DualSPHysics FlowTool, which counts the volume of particles passing through reservoir control sections and converts it into cycle-averaged discharge.

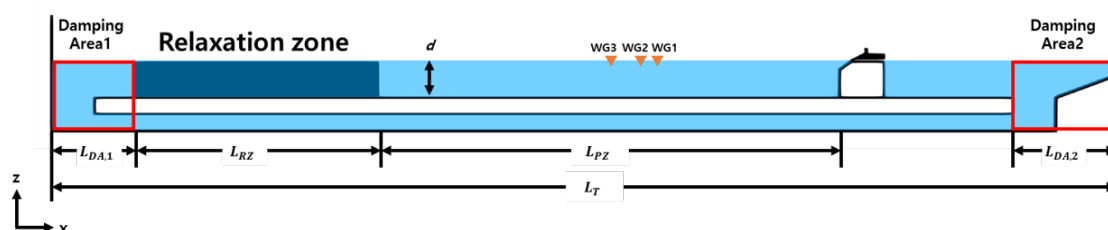


Figure 4. Numerical wave tank configuration [28].

2.4. Performance Metrics

To compare and evaluate optimization results, both hydraulic-efficiency and economy-related indices were employed. Hydraulic efficiency, a representative performance index for OWECS, was used, and economy was quantified using a construction-cost proxy based on structural quantities and a cost-per-unit-hydraulic-power index. All indices were computed from regular-wave simulation results obtained using DualSPHysics (WCSPH).

2.4.1. Definition of Hydraulic Efficiency

Hydraulic efficiency η_{hyd} is defined as the ratio of the potential-energy flux stored by overtopping P_{crest} to the incident wave-energy flux per unit width P_{wave} [16–18,33]:

$$\eta_{hyd} = \frac{P_{crest}}{P_{wave}} \quad (9)$$

For a multi-level OWEC, P_{crest} is computed by summing the potential-energy contributions from overtopping discharge into each reservoir:

$$P_{crest} = \sum_{j=1}^n \rho g \bar{q}_j R_{c,j} \quad (10)$$

where ρ is seawater density, g is gravitational acceleration, $\bar{q}_j$ is the cycle-averaged overtopping discharge into the j -th reservoir, and $R_{c,j}$ is the vertical distance from SWL to the reservoir inlet. The cycle-averaged discharge is computed from the instantaneous discharge $Q(t)$ measured by DualSPHysics FlowTool:

$$\bar{q} = \frac{1}{BT} \int_{t_0}^{t_0+T} \max(Q(t), 0) dt \quad (11)$$

In this study, the cycle-averaged overtopping discharge was evaluated by averaging over 20 wave periods under regular-wave conditions. The incident wave-energy flux per unit width at the structure front is:

$$P_{wave} = \frac{1}{8} \rho g H^2 C_g \quad (12)$$

where C_g is group velocity, given by the product of celerity C and group coefficient n :

$$C_g = nC = \frac{1}{2} \left[1 + \frac{2kd}{\sinh(2kd)} \right] \frac{\omega}{k} \quad (13)$$

Here ω , k , and d denote angular frequency, wavenumber, and water depth, respectively, with k obtained from the dispersion relation (3) and Table 1 values.

2.4.2. Construction-Cost Proxy and Cost per Unit Hydraulic Power

Economic evaluation of WECs is typically performed using unit-cost metrics such as LCOE (Levelized Cost of Energy), which divides life-cycle cost by energy production [1,3,5]. However, at the geometry-optimization stage, accurate LCOE estimation is difficult due to large uncertainties in PTO (turbine/generator), grid connection, OPEX, installation, offshore construction conditions, etc. Therefore, this study defines a cost proxy (CAPEX proxy) using structural-quantity indicators directly dependent on geometry changes (submerged area and foundation-rock length) and combines it with hydraulic efficiency to build relative screening indices for economic comparison.

The construction-cost proxy C_{proxy} is defined using the average submerged ramp cross-sectional area A_{Ramp} (idealized as concrete) and the required foundation-rock length L_{Ramp} . Since construction cost is generally estimated as quantity $\times$ unit price [33], the concrete portion is estimated as $c_{rc}A_{Ramp}B$, and the foundation-rock portion is approximated as $c_{rock}k_{sec}L_{Ramp}B$, where $k_{sec} = 1.0$ m is a representative sectional factor. Thus:

$$C_{proxy} = c_{rc}A_{Ramp}B + c_{rock}k_{sec}L_{Ramp}B \quad (14)$$

where B is the unit width ($B = 1$ in this study). Because the focus is on relative comparisons, common geometry components among cases were excluded from metric calculations. The unit-cost coefficients c_{rc} and c_{rock} were set based on data from the Ministry of Oceans and Fisheries of Korea [34]. The coefficients are interpreted as relative cost weights rather than absolute construction-cost predictions.

Because candidate geometries change both hydraulic performance and structural/foundation quantities, the cost per unit hydraulic power index CI_p is defined to mitigate the limitations of comparing designs solely by efficiency ratios:

$$CI_p = \frac{C_{proxy}}{P_{crest}} = \frac{c_{rc}A_{Ramp}B + c_{rock}k_{sec}L_{Ramp}B}{\eta_{hyd}P_{wave}} \quad (15)$$

Under identical regular-wave conditions, P_{wave} is constant across all designs; therefore,

$$CI_p \propto \frac{c_{rc}A_{Ramp} + c_{rock}k_{sec}L_{Ramp}}{\eta_{hyd}} \quad (16)$$

3. Results

This section compares the hydraulic-efficiency distributions associated with variations in installation depth and ramp slope for single-slope submerged ramps, identifying depth ranges with strong sensitivity and angle-dependent performance trends. Based on the selected representative depth regions, the submerged ramp is reconfigured into a segmented multi-angle geometry, and its ability to retain overtopping performance while reducing submerged volume and foundation demand is evaluated. Finally, design candidates are ranked using combined efficiency and economy indices, providing practical criteria for early-stage geometry screening and design-range selection.

3.1. Performance of Single-Slope Ramp

Table 4 and Figure 5 show hydraulic-efficiency results for the single-slope submerged ramp as a function of kinetic-energy cumulative depth percentage $F(D)$ and slope angle. Hydraulic efficiency generally increases as $F(D)$ increases, and the increase is particularly steep in the range $F(D) = 10$ –35%. For example, for $\theta = 20^\circ$, efficiency increases from $\eta_{hyd} = 19.95\%$ at $F(D) = 10\%$ to 34.29% at $F(D) = 30\%$, indicating that efficiency can recover substantially even within relatively shallow depth ranges.

Angle-dependent dominance varies with $F(D)$. In the range $F(D) = 10$ –35%, 20° consistently yields the highest efficiency among all angles. However, at $F(D) = 40\%$, 25° (36.87%) slightly exceeds 20° , indicating a transition in the angle–efficiency trend. At $F(D) = 55\%$, 30° reaches the maximum efficiency (40.61%), showing that as the cumulative-energy depth becomes deeper, larger slope angles become advantageous over wider ranges. The heatmap in Figure 6 also shows broad high-efficiency regions around $F(D) = 45$ –65% and 25 – 35° , and efficiency tends to saturate rather than increase indefinitely with $F(D)$. This is interpreted as a diminishing-benefit characteristic: as depth increases, the additional kinetic-energy fraction included decreases, and efficiency recovery benefits from structural expansion become limited. To provide a physical interpretation of the depth- and angle-dependent efficiency trends, the time evolution of the air–water interface and velocity contours for a representative single-slope case is presented in Appendix A.2 Figure A1. The snapshots over one wave period illustrate the sequence of run-up, overtopping, and recirculation near the ramp, supporting the mechanistic explanation of the efficiency variations summarized in Table 4 and Figures 5 and 6.

Based on the single-slope results, representative design depth regions for subsequent multi-angle (three-segment) designs were set to $F(D) = 30\%$, 40% , 55% .

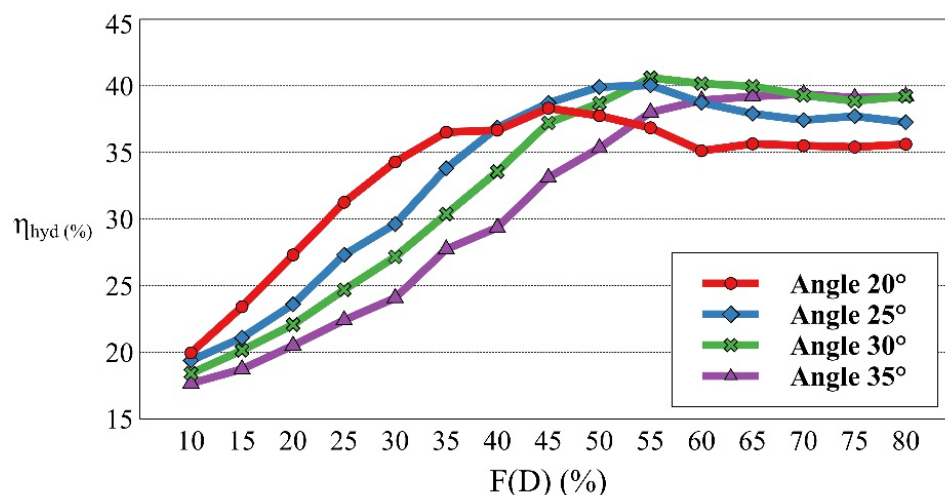


Figure 5. Hydraulic efficiency of single-slope ramp cases as a function of $F(D)$.

Table 4. Hydraulic efficiency for single-slope submerged ramps as a function of slope angle and cumulative kinetic-energy ratio $F(D)$.

$F(D)$ (%)	Angle = 20°	Angle = 25°	Angle = 30°	Angle = 35°
10	19.95%	19.37%	18.40%	17.66%
15	23.42%	21.09%	20.14%	18.73%
20	27.31%	23.59%	22.07%	20.50%
25	31.25%	27.31%	24.69%	22.40%
30	34.29%	29.62%	27.16%	24.07%
35	36.52%	33.81%	30.37%	27.73%
40	36.67%	36.87%	33.56%	29.35%

45	38.32%	38.72%	37.21%	33.11%
50	37.75%	39.92%	38.70%	35.34%
55	36.85%	40.04%	40.61%	38.01%
60	35.12%	38.74%	40.17%	38.91%
65	35.65%	37.91%	39.97%	39.21%
70	35.51%	37.43%	39.29%	39.39%
75	35.40%	37.72%	38.86%	39.10%
80	35.62%	37.27%	39.22%	39.21%

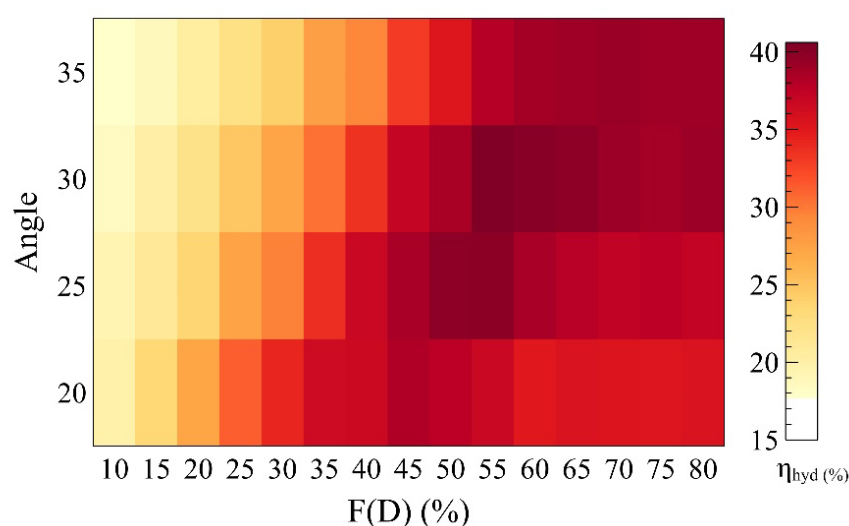


Figure 6. Heatmap of hydraulic efficiency over slope angle and $F(D)$.

3.2. Performance of Multi-Segment Submerged Ramps

3.2.1. Model Design

Multi-angle (three-segment) ramps were defined by dividing the submerged ramp into three segments and assigning different slopes to each segment while maintaining the same overall $F(D)$ (Figure 7). Segment division depths ($d_{ramp,1} \sim d_{ramp,3}$) were fixed to represent each design depth region, and only the combination of segment slope angles ($\theta_{ramp,1} \sim \theta_{ramp,3}$) was varied. Based on single-slope results, three angles showing relatively good performance under the corresponding $F(D)$ condition were selected, and six permutations were generated. For $F(D) = 30\%$ and 40% , the $20^\circ\text{--}25^\circ\text{--}30^\circ$ set was used; for $F(D) = 55\%$, the $25^\circ\text{--}30^\circ\text{--}35^\circ$ set was used. The geometric definitions (d_{ramp} , θ_{ramp} , L_{ramp} , $AREA$) and case IDs are listed in Table 5.

The key intent of applying multi-segment geometry is to maintain the function of the upper slope that is critical for overtopping formation while reducing submerged volume and seabed contact length, thereby improving constructability and economy. Comparisons were performed using hydraulic performance as well as cost-weighted indices based on submerged area and foundation-rock length, and the cost per unit hydraulic power index.

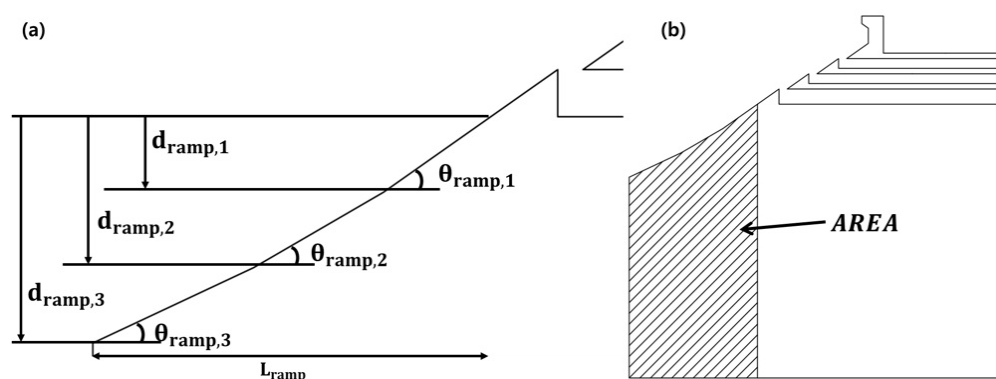


Figure 7. Definition of the multi-segment ramp geometry, where (a) represents the ramp angle, segment division depth, and ramp length, and (b) represents the submerged area of the ramp.

Table 5. Geometry of multi-segment ramp cases.

CaseID	$F(D)$	$d_{ramp,1}$	$d_{ramp,2}$	$d_{ramp,3}$	$\theta_{ramp,1}$	$\theta_{ramp,2}$	$\theta_{ramp,3}$	L_{ramp}	AREA
Per_30_Case 1	30	0.04	0.07	0.11	20	25	30	0.244	0.207
Per_30_Case 2					20	30	25	0.248	0.210
Per_30_Case 3					25	20	30	0.237	0.201
Per_30_Case 4					25	30	20	0.248	0.208
Per_30_Case 5					30	20	25	0.237	0.200
Per_30_Case 6					30	25	20	0.244	0.204
Per_40_Case 1	40	0.05	0.1	0.15	20	25	30	0.331	0.276
Per_40_Case 2					20	30	25	0.331	0.275
Per_40_Case 3					25	20	30	0.331	0.274
Per_40_Case 4					25	30	20	0.331	0.272
Per_40_Case 5					30	20	25	0.331	0.272
Per_40_Case 6					30	25	20	0.331	0.271
Per_55_Case 1	55	0.08	0.16	0.24	25	30	35	0.424	0.335
Per_55_Case 2					25	35	30	0.424	0.333
Per_55_Case 3					30	25	35	0.424	0.333
Per_55_Case 4					30	35	25	0.424	0.328
Per_55_Case 5					35	25	30	0.424	0.329
Per_55_Case 6					35	30	25	0.424	0.326

3.2.2. Performance at $F(D) = 30\%$

Results for the $F(D) = 30\%$ design depth region are summarized in Figures 8 and 9 and Table 6, and representative time-series SPH snapshots illustrating the free-surface evolution and overtopping process are provided in Appendix A.2 (Figures A2 and A3). For the six multi-segment cases, η_{hyd} ranged from 29.76% to 30.75%, and case-to-case variation was limited to ~1 percentage point. Case 4 showed the maximum efficiency of 30.75%, indicating that the ordering of segment slopes (upper–middle–lower) can contribute to efficiency improvement to some extent even under the same $F(D)$ condition.

However, among single-slope reference cases at the same $F(D)$, Angle_20 yielded 34.29%, confirming that a single-slope ramp can be more favorable for overtopping formation. Multi-segment cases exhibited ~3–4% lower efficiency than Angle_20, but they also offer potential economy improvement through reduced structural scale. In Table 6, multi-segment cases showed $C_{proxy} \approx 79.19 - 82.99$ (unit: 10^6 KWR/m^3) and $CI_p \approx 263.46 - 278.35$ (unit: 10^6 KRW/W). In contrast, Angle_20 had $C_{proxy} = 101.09$ and $CI_p = 294.85$, indicating higher cost indices, consistent with the tendency that higher efficiency is accompanied by larger submerged quantities.

From an economic perspective, Case 5 yielded the lowest $CI_P = 263.46$, providing an advantage in cost–efficiency balance, although its efficiency (30.06%) was lower than Case 4. Therefore, under $F(D) = 30\%$, Angle_20 is dominant in maximum efficiency, but multi-segment cases can be competitive when the economy is included.

Table 6. Performance metrics for $F(D) = 30\%$ cases.

Case	η_{hyd}	$c_{rc}A_{Ramp}$	$c_{rock}k_{sec}L_{Ramp}$	C_{proxy}	CI_P
Case 1	30.43%	58.86	22.92	81.78	268.73
Case 2	29.81%	59.68	23.31	82.99	278.35
Case 3	29.76%	57.17	22.35	79.52	267.21
Case 4	30.75%	59.20	23.31	82.51	268.33
Case 5	30.06%	56.84	22.35	79.19	263.46
Case 6	30.33%	58.05	22.92	80.97	266.98
Angle_20	34.29%	72.65	28.45	101.09	294.85
Angle_25	29.62%	56.70	22.20	78.91	266.36
Angle_30	27.16%	45.80	17.93	63.73	234.65

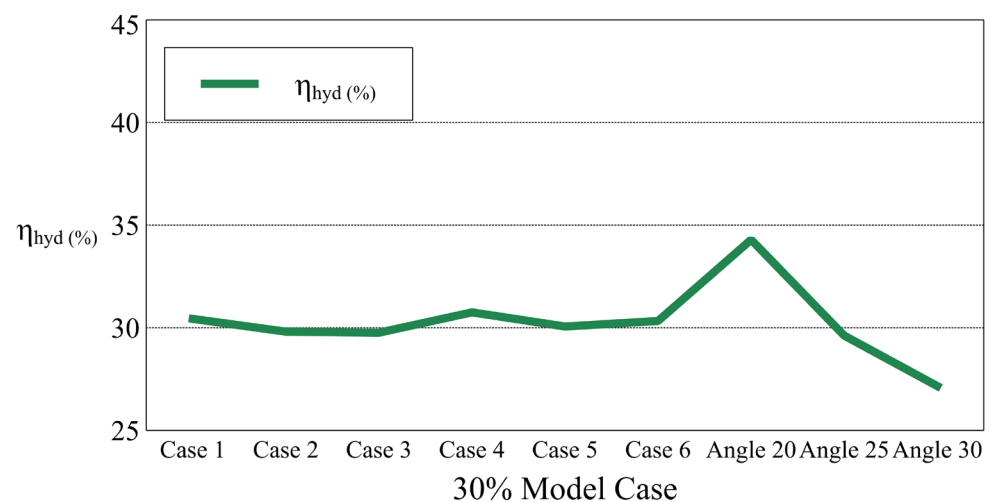


Figure 8. Hydraulic efficiency for $F(D) = 30\%$ cases.

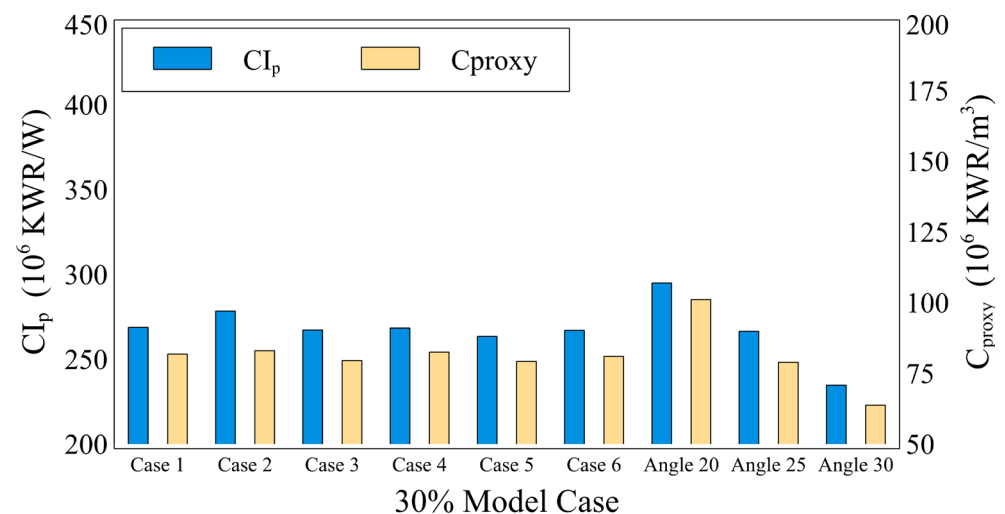


Figure 9. Cost proxy and cost-efficiency index for $F(D) = 30\%$ cases.

3.2.3. Performance at $F(D) = 40\%$

Results for $F(D) = 40\%$ are summarized in Figures 10 and 11 and Table 7, and representative time-series SPH snapshots illustrating the free-surface evolution and overtopping process are provided in Appendix A.2 (Figures A4 and A5). Multi-segment cases showed $\eta_{hyd} = 35.80\text{--}37.15\%$ with a $\sim 1.35\%$ p variation. Case 4 exhibited the maximum efficiency (37.15%), while Case 1 showed the minimum (35.80%), indicating that segment slope combinations influence run-up and overtopping formation even under the same depth condition.

For economy indices, C_{proxy} ranged from 108.03 to 109.49, showing only small differences, whereas CI_P ranged from 291.59 to 305.88, producing a clearer relative ranking. In particular, Case 4 achieved both the highest efficiency (37.15%) and the lowest CI_P , making it an excellent candidate from an efficiency–economy balance perspective.

Among single-slope references, Angle_25 showed high efficiency (36.87%) and good economy ($CI_P = 286.11$); Angle_30 had lower efficiency (33.56%) but the lowest cost index ($CI_P = 253.86$). In contrast, Angle_20 had relatively high efficiency (36.67%) but very high-cost burden ($C_{proxy} = 135.14$, $CI_P = 368.55$), indicating that maximizing efficiency alone does not guarantee rational design.

Notably, multi-segment Case 4 reduced C_{proxy} and CI_P to approximately 20% lower than Angle_20 while maintaining equal or higher efficiency, demonstrating the practical value of multi-segment designs under real constraints where submerged quantities strongly affect feasibility. It was also confirmed that the arrangement of segment slopes can influence AREA and L_{ramp} , thereby controlling cost indices. Thus, under $F(D) = 40\%$, combinations exist that simultaneously increase η_{hyd} and decrease CI_P , and Case 4 was selected as the key candidate in this study.

Table 7. Performance metrics for $F(D) = 40\%$ cases.

Case	η_{hyd}	$C_{rc}A_{Ramp}$	$C_{rock}k_{sec}L_{Ramp}$	C_{proxy}	CI_P
Case 1	35.80%	78.32	31.17	109.49	305.88
Case 2	36.23%	78.00	31.17	109.18	301.3
Case 3	36.32%	77.87	31.17	109.04	300.23
Case 4	37.15%	77.15	31.17	108.32	291.59
Case 5	35.89%	77.28	31.17	108.45	302.21
Case 6	36.58%	76.85	31.17	108.03	295.28
Angle_20	36.67%	96.35	38.79	135.14	368.55
Angle_25	36.87%	75.21	30.28	105.48	286.11
Angle_30	33.56%	60.74	24.45	85.2	253.86

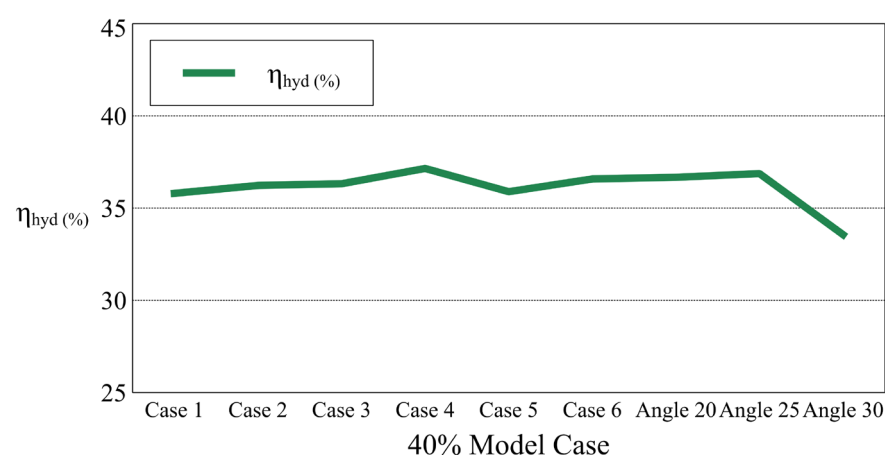


Figure 10. Hydraulic efficiency for $F(D) = 40\%$ cases.

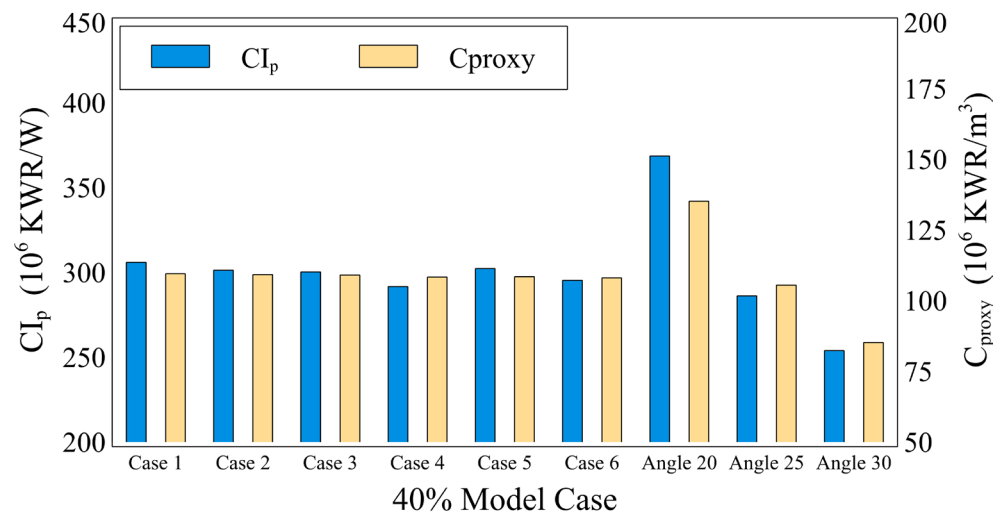


Figure 11. Cost proxy and cost-efficiency index for $F(D) = 40\%$ cases.

3.2.4. Performance at $F(D) = 55\%$

For $F(D) = 55\%$, η_{hyd} , C_{proxy} , and CI_p were computed and summarized in Figures 12 and 13 and Table 8, and representative time-series SPH snapshots illustrating the free-surface evolution and overtopping process are provided in Appendix A.2 (Figures A6 and A7). Multi-segment cases achieved $\eta_{hyd} = 39.38\text{--}40.55\%$, indicating a generally high efficiency level around 40%. Case 5 showed the maximum efficiency (40.55%), while Case 1 showed the minimum (39.38%), with a case-to-case variation of $\sim 1.17\%$ p. For economy, $C_{proxy} = 132.59\text{--}135.20$ and $CI_p = 328.83\text{--}343.31$, confirming that both efficiency and cost indices vary simultaneously. Case 5 provided a favorable high-efficiency–low- CI_p combination within the multi-segment set.

Among single-slope references, Angle_30 exhibited the highest efficiency (40.61%) and also good economy ($CI_p = 323.22$). Angle_35 showed low-cost indices ($C_{proxy} = 108.22$, $CI_p = 284.70$) but reduced efficiency (38.01%), implying economy improvement accompanied by efficiency loss. Angle_25 achieved high efficiency (40.04%) but large cost indices ($C_{proxy} = 162.50$, $CI_p = 405.84$), demonstrating that even at similar efficiency levels, increased quantities can constrain design selection.

Overall, multi-segment cases formed an alternative set that can approach the single-slope high-efficiency conditions while partially alleviating quantity burdens through slope arrangement. Therefore, at $F(D) = 55\%$, the optimal choice can vary depending on whether the design goal prioritizes maximum η_{hyd} or minimum CI_p , and this study quantitatively illustrates the selection structure through within-percent comparisons.

Table 8. Performance metrics for $F(D) = 55\%$ cases.

Case	η_{hyd}	$c_{rc}A_{Ramp}$	$c_{rock}k_{sec}L_{Ramp}$	C_{proxy}	CI_p
Case 1	39.38%	95.26	39.94	135.2	343.31
Case 2	39.66%	94.70	39.94	134.65	339.52
Case 3	39.84%	94.51	39.94	134.45	337.49
Case 4	40.06%	93.20	39.94	133.15	332.34
Case 5	40.55%	93.40	39.94	133.34	328.83
Case 6	40.29%	92.65	39.94	132.59	329.11
Angle_20	40.04%	114.06	48.44	162.5	405.84
Angle_25	40.61%	92.12	39.12	131.24	323.22
Angle_30	38.01%	75.96	32.26	108.22	284.7

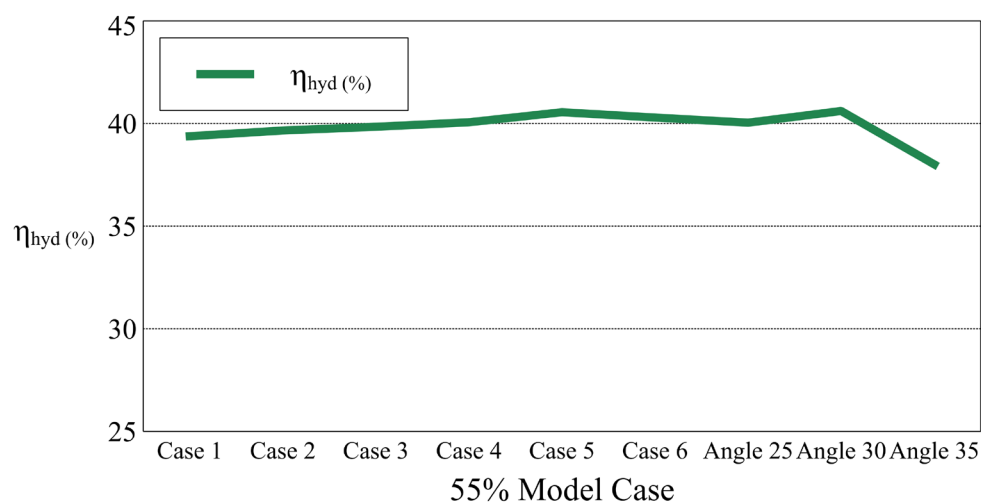


Figure 12. Hydraulic efficiency for $F(D) = 55\%$ cases.

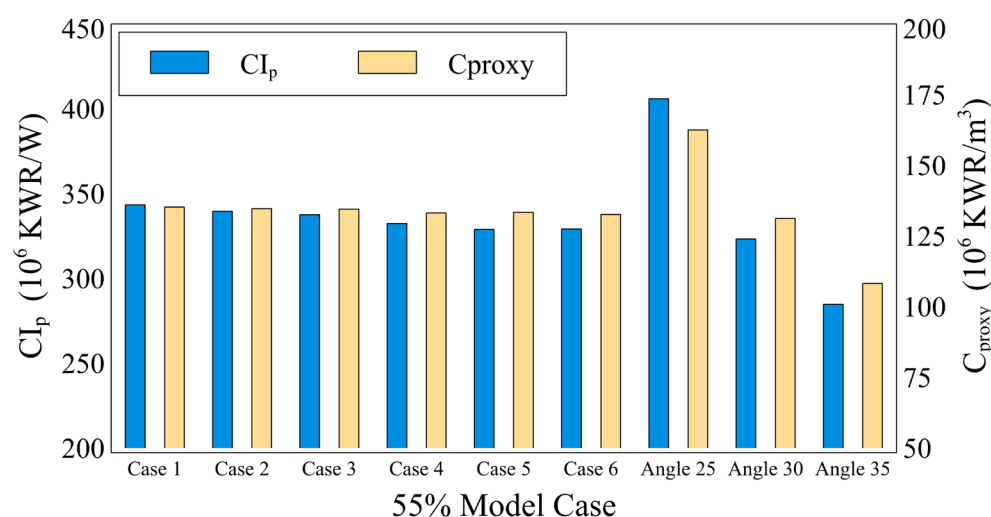


Figure 13. Cost proxy and cost-efficiency index for $F(D) = 55\%$ cases.

4. Conclusions

This study investigated a submerged-ramp design for a caisson-type overtopping wave energy converter (OWEC). Under the design-wave (representative regular-wave) condition, installation depth was quantified based on the depth-dependent cumulative kinetic-energy distribution, and both hydraulic efficiency and construction economy were evaluated for single-slope and multi-segment (three-segment) ramp geometries. Numerical simulations were performed using DualSPHysics, a WCSPH-based solver capable of reproducing strong free-surface deformation and overtopping/breaking processes. Performance metrics included hydraulic efficiency (η_{hyd}), a construction-cost proxy based on construction quantities (C_{proxy}), and the cost per unit hydraulic power index (CI_p). For readability, economy indices were presented in units of 10^6 KRW, and unit-cost coefficients were interpreted as relative comparison weights rather than absolute cost predictions.

From the single-slope ramp results, the submerged geometry in an overtopping OWEC is not merely a factor determining structural scale; it effectively defines the depth range of wave kinetic energy that can be utilized under the design wave, acting as a key design variable. When installation depth is normalized using the cumulative kinetic-energy distribution, the method provides high design applicability because it can

consistently identify the depth range with effective energy contribution even under different site conditions. The design-depth regions selected in this study ($F(D) = 30\%, 40\%, 55\%$) were determined based on ranges where both efficiency sensitivity and structural-scale change are prominent. In shallow ranges, even small geometry changes can significantly affect the overtopping contribution. In intermediate ranges, the angle-dependent efficiency trend shifts, implying design flexibility to reduce submerged quantities while maintaining performance. In deeper ranges, high efficiency can be achieved, but it is accompanied by increased structural scale, indicating that design suitability cannot be determined from efficiency alone.

The proposed multi-segment submerged ramp maintains the upper-slope characteristics critical for overtopping formation while reducing submerged volume and foundation quantities. Therefore, it can serve as a meaningful alternative from an efficiency–economy balance perspective when jointly considering η_{hyd} , C_{proxy} , and CI_P . In the representative design-depth regions, multi-segment configurations enabled selection of optimal alternatives not solely by maximizing efficiency but by balancing efficiency and economy; specifically, under $F(D) = 40\%$, Per_40_Case 4 was identified as the best overall geometry. In summary, this study presents a design framework that directly links submerged design variables (installation depth and slope geometry) to the wave-energy distribution and supports decision-making at the early-stage geometry design of caisson-type OWECs by jointly evaluating η_{hyd} , C_{proxy} , and CI_P .

Because this study is based on two-dimensional WCSPH simulations under regular-wave conditions, future work should validate performance under irregular waves and long-term wave climates and assess three-dimensional effects (lateral flow, localized turbulence, and air entrainment). In particular, a targeted sensitivity study with modest variations in wave period and height around the design condition should be conducted to assess the robustness and generality of the proposed $F(D)$ -based depth-selection framework. In addition, if C_{proxy} and CI_P are further refined by coupling with realistic construction processes, unit-cost systems, and integrated constraints, including PTO and structural/constructability limitations, the field applicability and decision-support value of the proposed geometries are expected to improve.

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Conflicts of Interest: The authors declare no conflicts of interest.

Appendix A

Appendix A.1. Detailed Case List for Single-Slope Submerged Ramps

Table A1 presents the case identifiers for all simulations included in the single-slope submerged-ramp parametric study, and summarizes, for each case, the ramp slope angle (θ_{ramp}) and installation depth (d_{ramp}) together with the corresponding ramp length (L_{ramp}) and submerged cross-sectional area (AREA). This table provides the geometric input values used to compute the hydraulic efficiency (η_{hyd}) and the economy indices (C_{proxy} and CI_p) reported in the main text, thereby supporting traceability and reproducibility of the simulation conditions.

Table A1. Detailed single-slope ramp cases.

Case ID	θ_{ramp} (°)	d_{ramp} (m)	L_{ramp} (m)	AREA(m ²)
a_20_per_10	20	0.03	0.0824	0.0729
a_20_per_15		0.05	0.1374	0.1203
a_20_per_20		0.07	0.1923	0.1666
a_20_per_25		0.09	0.2473	0.2118
a_20_per_30		0.11	0.3022	0.2557
a_20_per_35		0.13	0.3572	0.2983
a_20_per_40		0.15	0.4121	0.3392
a_20_per_45		0.18	0.4945	0.4006
a_20_per_50		0.21	0.5770	0.4592
a_20_per_55		0.24	0.6594	0.5144
a_20_per_60		0.27	0.7418	0.5658
a_20_per_65		0.31	0.8517	0.6330
a_20_per_70		0.36	0.9891	0.7130
a_20_per_75		0.41	1.1265	0.7828
a_20_per_80		0.47	1.2913	0.8577
a_25_per_10	25	0.03	0.0643	0.0569
a_25_per_15		0.05	0.1072	0.0939
a_25_per_20		0.07	0.1501	0.1300
a_25_per_25		0.09	0.1930	0.1653
a_25_per_30		0.11	0.2359	0.1996
a_25_per_35		0.13	0.2788	0.2328
a_25_per_40		0.15	0.3217	0.2648
a_25_per_45		0.18	0.3860	0.3127
a_25_per_50		0.21	0.4503	0.3584
a_25_per_55		0.24	0.5147	0.4015
a_25_per_60		0.27	0.5790	0.4416
a_25_per_65		0.31	0.6648	0.4941
a_25_per_70		0.36	0.7720	0.5565
a_25_per_75		0.41	0.8792	0.6110
a_25_per_80		0.47	1.0079	0.6695
a_30_per_10	30	0.03	0.0520	0.0459
a_30_per_15		0.05	0.0866	0.0758
a_30_per_20		0.07	0.1212	0.1050
a_30_per_25		0.09	0.1559	0.1335
a_30_per_30		0.11	0.1905	0.1612
a_30_per_35		0.13	0.2252	0.1880
a_30_per_40		0.15	0.2598	0.2138
a_30_per_45		0.18	0.3118	0.2525
a_30_per_50		0.21	0.3637	0.2895

a_30_per_55		0.24	0.4157	0.3243
a_30_per_60		0.27	0.4677	0.3567
a_30_per_65		0.31	0.5369	0.3990
a_30_per_70		0.36	0.6235	0.4495
a_30_per_75		0.41	0.7101	0.4935
a_30_per_80		0.47	0.8141	0.5407
a_35_per_10		0.03	0.0428	0.0379
a_35_per_15		0.05	0.0714	0.0625
a_35_per_20		0.07	0.1000	0.0866
a_35_per_25		0.09	0.1285	0.1101
a_35_per_30		0.11	0.1571	0.1329
a_35_per_35		0.13	0.1857	0.1550
a_35_per_40		0.15	0.2142	0.1763
a_35_per_45	35	0.18	0.2571	0.2082
a_35_per_50		0.21	0.2999	0.2387
a_35_per_55		0.24	0.3428	0.2674
a_35_per_60		0.27	0.3856	0.2941
a_35_per_65		0.31	0.4427	0.3290
a_35_per_70		0.36	0.5141	0.3706
a_35_per_75		0.41	0.5855	0.4069
a_35_per_80		0.47	0.6712	0.4458

Appendix A.2. Flow-Field Snapshots for a Representative

Appendix A.2 provides representative DualSPHysics (WCSPH) snapshots to visualize the time evolution of the air–water interface and velocity-magnitude contours during the overtopping process under the design regular-wave condition (Table 1). In addition to the single-slope reference case a_35_per_55 (Figure A1), three depth-region comparisons at $F(D) = 30\%$, 40% , and 55% are presented by pairing single-slope angle cases (a_25_per_30, a_20_per_40, and a_30_per_55) with the selected multi-segment cases (Per_30_Case 4, Per_40_Case 4, and Per_55_Case 5) (Figures A2–A7). All figures show eight snapshots ($t = nT/8$ ($n = 1–8$)) over one model-scale wave period ($T = 1.648$ s).

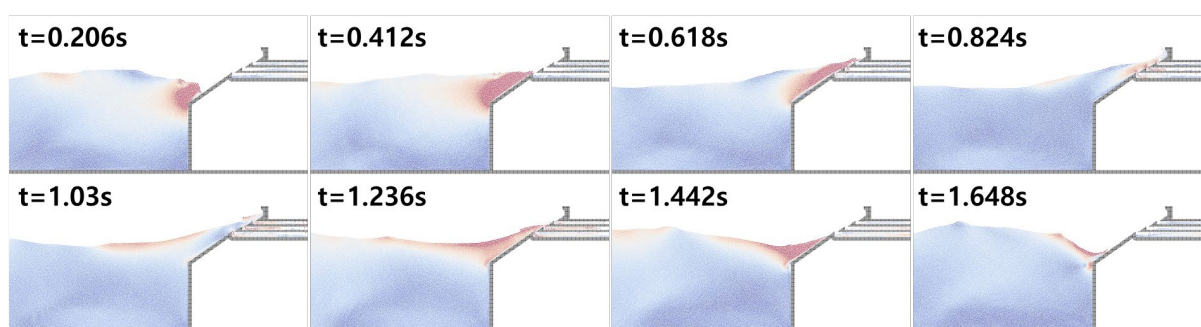


Figure A1. Time evolution of the air–water interface and velocity-magnitude contours for the single-slope case a_35_per_55. (Snapshots are shown at $t = nT/8$ ($n = 1–8$) over one model-scale wave period ($T = 1.648$ s).

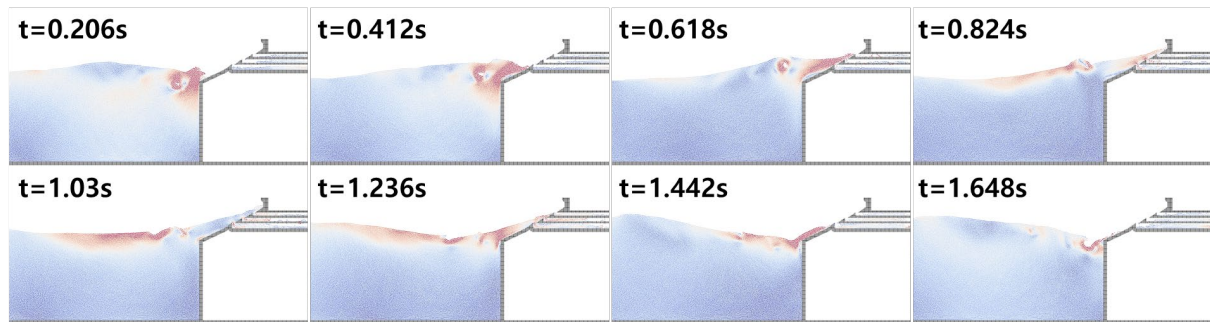


Figure A2. Time evolution of the air–water interface and velocity-magnitude contours for the single-slope case a_25_per_30. (Snapshots are shown at $t = nT/8$ ($n = 1-8$) over one model-scale wave period ($T = 1.648$ s).

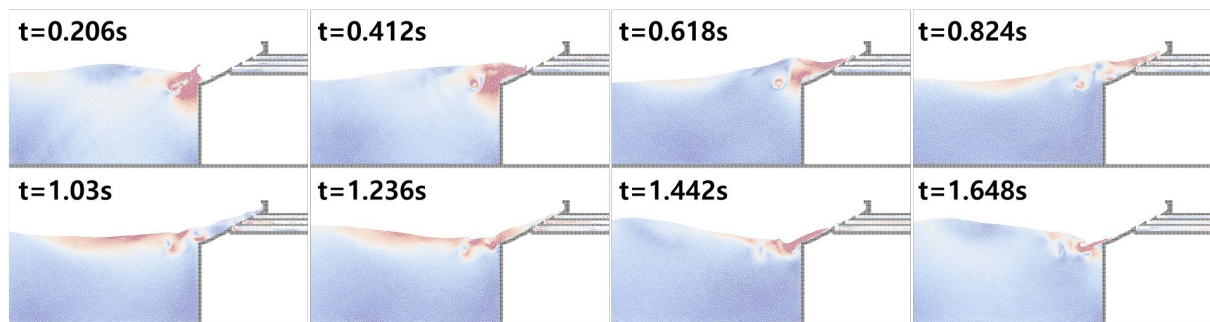


Figure A3. Time evolution of the air–water interface and velocity-magnitude contours for the multi-segment case Per_30_Case 4. (Snapshots are shown at $t = nT/8$ ($n = 1-8$) over one model-scale wave period ($T = 1.648$ s).

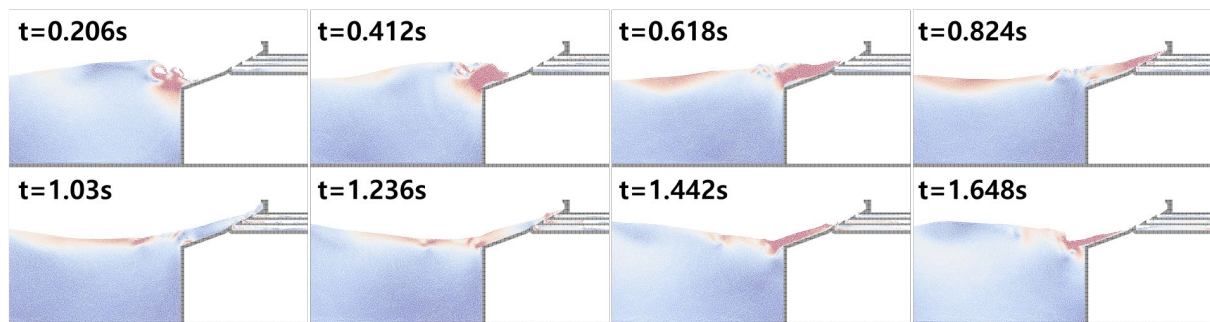


Figure A4. Time evolution of the air–water interface and velocity-magnitude contours for the single-slope case a_20_per_40. (Snapshots are shown at $t = nT/8$ ($n = 1-8$) over one model-scale wave period ($T = 1.648$ s).

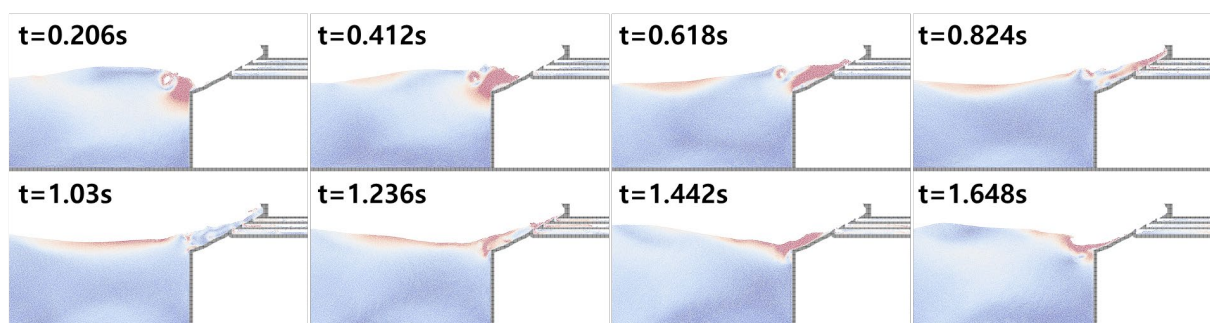


Figure A5. Time evolution of the air–water interface and velocity-magnitude contours for the multi-segment case Per_40_Case 4. (Snapshots are shown at $t = nT/8$ ($n = 1-8$) over one model-scale wave period ($T = 1.648$ s).

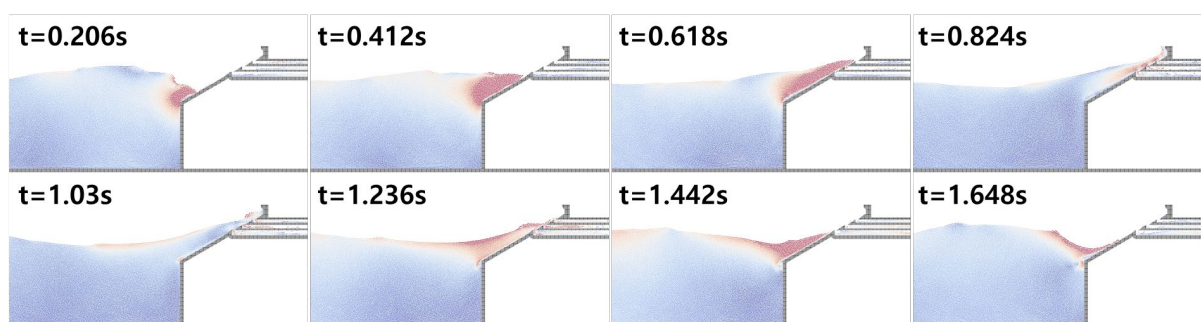


Figure A6. Time evolution of the air–water interface and velocity-magnitude contours for the single-slope case a_30_per_55. (Snapshots are shown at $t = nT/8$ ($n = 1–8$) over one model-scale wave period ($T = 1.648$ s).

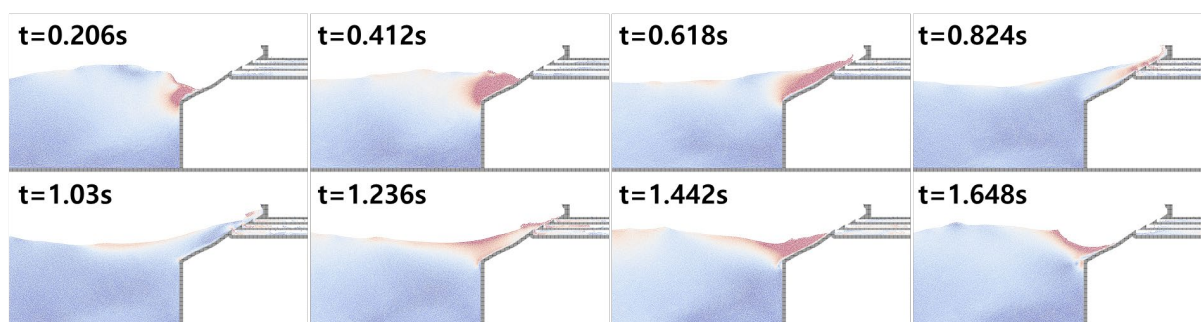


Figure A7. Time evolution of the air–water interface and velocity-magnitude contours for the multi-segment case Per_55_Case 5. (Snapshots are shown at $t = nT/8$ ($n = 1–8$) over one model-scale wave period ($T = 1.648$ s).

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