

Review

Empirical Predictions on Wave Overtopping for Overtopping Wave Energy Converters: A Systematic Review

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Abstract: Over the past three decades, the development and testing of various overtopping wave energy converters (OWECs) have highlighted the importance of accurate wave run-up and overtopping predictions on those devices. This study systematically reviews the empirical formulas traditionally used for predicting overtopping across different types of breakwaters by assessing their strengths, limitations, and applicability to OWECs. This provides a foundation for future research and development in OWECs. Key findings reveal that empirical formulas for conventional breakwaters can be categorized as mild or steep slopes and vertical structures based on the angle of the slope. For the same relative crest freeboards, the dimensionless average overtopping discharge of mild slopes is larger than that of vertical structures. However, the formula features predictions within a similar range for small relative crest freeboards. The empirical formulas for predicting overtopping in fixed and floating OWECs are modified from the predictors developed for conventional breakwaters with smooth, impermeable and linear slopes. Different correction coefficients are introduced to account for the effects of limited draft, inclination angle, and low relative freeboard. The empirical models for floating OWECs, particularly the Wave Dragon model, have been refined through prototype testing to account for the unique 3D structural reflector's influence and dynamic wave interactions.

Keywords: breakwaters; empirical formulas; experimental model; overtopping; Wave Dragon



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1. Introduction

Environmental pollution and energy security issues have become increasingly prominent in recent years, while traditional fossil fuel reserves are limited and susceptible to market fluctuations. The emergence of offshore wind energy, alongside solar and ocean energy, further diversifies the renewable energy portfolio. The utilization of offshore wind energy technology has great potential, but also complexity and challenges [1,2]. Floating offshore wind turbines (FOWTs) present a concept that can efficiently and economically capture energy from deep-water offshore wind resources [3,4]. Additionally, research like [5] provides critical insights into the structural integrity and efficiency of floating offshore wind turbines. Developing these renewable energy technologies is crucial to addressing these challenges, as they can reduce greenhouse gas emissions, lessen reliance on fossil fuels and enhance the diversity and security of the energy supply.

As an emergent energy source, ocean energy is garnering significant attention and investment, with innovative approaches like [6] showcasing the broad potential of harnessing energy from diverse aquatic sources. Many countries and regions are increasing their investment in ocean energy research, development, and utilisation. There are two main types of ocean energy: wave energy and tidal energy. Wave energy currently contributes only a small fraction to the global energy supply. According to the latest data from the International Energy Agency (IEA) and other sources, wave energy and other marine energy sources still represent a small portion (less than 0.1%) of the world's total

energy generation. However, ocean wave energy stands out among renewable sources due to its abundant reserves, straightforward extraction, high energy density and minimal environmental impact [7–10].

Various types of wave energy converters (WECs) are already in use, which can be generally categorized into three types: oscillating body systems (OBSs), oscillating water columns (OWCs) and overtopping wave energy converters (OWECs) [11–15]. OWECs feature a reservoir for storing seawater, a ramp and a low-head hydraulic turbine. Waves ascend the ramp and overtop into one or more reservoirs above mean sea level. The stored seawater is then released to drive a turbine, converting the water's potential energy into electrical energy. Several WEC designs have been proposed and trialed, including TAPCHAN [16], Power Pyramid [17], Wave Plane [18], Sea-wave Slot-cone Generator [19,20], Wave Dragon [21] and WaveCat [22,23].

Over the past 50 years, many investigations have concentrated on wave overtopping on coastal structures. Since then, wave overtopping predictors for typical coastal defence structures have been continuously developed. The primary strategy to predict overtopping has been to design structures that protect humans and valuable assets from the destructive force of the sea. Coastal structures aim to avoid or minimize overtopping as much as possible, as overtopping can result in functional or structural failures. However, for OWECs, overtopping performance is one of the critical parameters determining reservoir capacity and power generation, which should be maximized, as hydraulic efficiency is directly proportional to the volume of water that overtops and enters the reservoirs. When OWECs were not proposed, overtopping research mainly focused on breakwaters with slopes. As the ramp of the overtopping device is very similar to the slope of the breakwater, studying the traditional breakwater with a slope can provide a reference basis for research on OWECs.

Currently, three main techniques are used to predict wave overtopping: physical model testing, numerical simulations and empirical formulas. Experimental and numerical analyses are indispensable in wave overtopping research. Though costly and time-consuming, experiments in wave flumes or basins offer highly reliable data by capturing complex physical phenomena such as wave breaking and turbulence. Numerical methods, such as Computational Fluid Dynamics (CFD) simulations, are more flexible and cost-effective, allowing for extensive parametric studies [24–26]. However, their accuracy heavily depends on validation against experimental data, particularly in complex, nonlinear scenarios. To ensure reliability, numerical models should be validated with experiments. After that, they can refine empirical formulas by examining various parameters (overtopping volume, overtopping probability, etc.) in overtopping devices, leading to more accurate and widely applicable predictions.

The empirical formula method provides a straightforward way to estimate overtopping discharge and volumes for OWECs. It typically involves parameterized formulas based on wave characteristics like height, period and angle, making it popular in preliminary design due to its speed and simplicity. However, the existing literature lacks a systematic review of the empirical predictions used for this purpose. This paper addresses this critical gap by comprehensively reviewing the various empirical formulas used to predict wave overtopping in OWECs. We enumerate and analyze existing empirical formulas and evaluate their strengths, limitations and applicability to different OWEC configurations. Our work is novel in highlighting the internal connections between traditional breakwater formulas and their adaptations for OWECs, which has profound implications on the accuracy and applicability of overtopping predictions.

This paper is organized as follows: Sections 2–4 summarize the empirical formulas commonly used for predicting wave runup and overtopping on bottom-mounted breakwaters, fixed OWECs and floating OWECs. Also, the strengths and limitations of various overtopping formulas are discussed in the following sections. Section 5 presents the summary and conclusion. Figure 1 presents a flowchart of the main outlines of the current work.

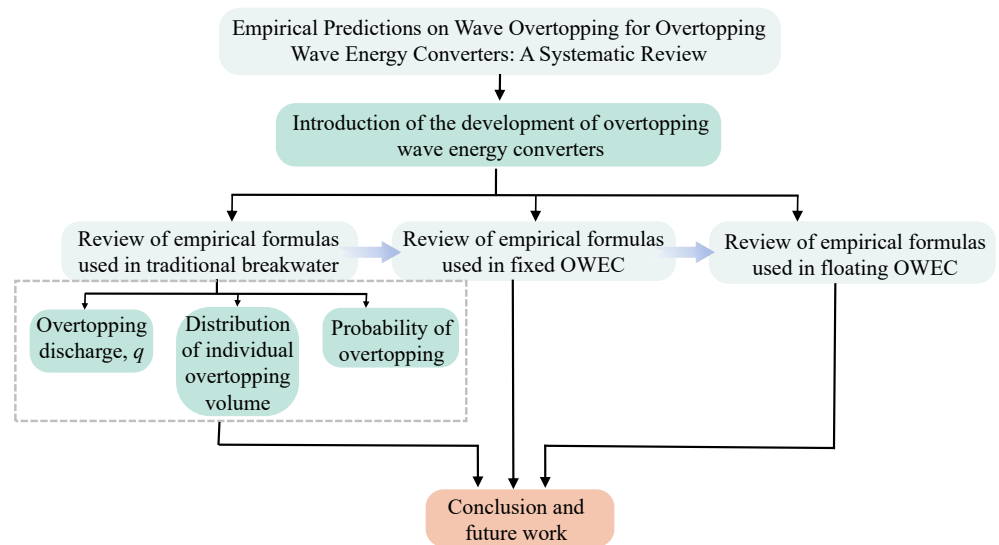


Figure 1. A flowchart of the main outlines of the present paper.

2. Bottom-Mounted Breakwaters

2.1. Overtopping Discharge

Wave overtopping is one of the critical hydraulic responses in the context of coastal structures, such as breakwaters [27]. Designing coastal defence structures to accommodate tolerable levels of wave overtopping has become a paramount concern for researchers and practitioners [28–32]. Overtopping is primarily triggered when the most formidable waves’ run-up heights surpass the coastal structure’s freeboard [33]. Understanding and mitigating the impacts of wave overtopping is essential for ensuring the longevity and functionality of coastal structures. Given the complex and non-linear characteristics of wave overtopping, predictions of overtopping flow generally depend on empirical methods, mainly derived from data from downscaled physical model tests. The mean overtopping discharge, q , is the main parameter to quantify the overtopping obtained from quantitative measurements in a laboratory wave flume.

Many physical model tests have been performed worldwide to develop empirical methods for predicting overtopping. The empirical methods or formulas have a range of applicability and often apply to specific structures, such as smooth slopes (dikes, sloping seawalls), armoured rubble mound breakwaters or vertical structures. The principal formula used for wave overtopping is based on the original work of Owen (1980) [34] according to which wave overtopping discharge, denoted as q , typically diminishes exponentially with an increase in the crest freeboard

$$Q = \frac{q}{\sqrt{gH_{m0}}^3} = A \cdot e^{\left(-B \frac{R_c}{H_{m0}}\right)}, \quad (1)$$

where Q is the non-dimensional average overtopping flow per crest meter width, R_c is crest freeboard used for the vertical distance from mean water to the crest of the reservoir and $R = R_c / H_{m0}$ is the non-dimensional crest level. H_{m0} stands for the significant wave height estimated from the zero-moment of the wave spectrum, and g is the acceleration due to gravity. A and B are empirical coefficients that depend on the seaward slope of the seawall. One of the advantages of this formulation is its capacity to yield a non-zero and finite result for $R_c = 0$. To refine and validate the model, laboratory tests are conducted to determine the optimal values for the coefficients A and B .

In practical applications, Van der Meer and Janssen (1995) [35] generalized Equation (1) for a single-level structure based on extensive testing with both small and large-scale

models, incorporating a variety of geometries. The overtopping formula of mildly sloping dikes subjected to non-breaking waves (for $\xi_{m-1,0} \geq 2$) is expressed as follows:

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot e^{-2.6 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \gamma_b \gamma_h \gamma_\beta}} \quad (2)$$

where correction factors including those for roughness (γ_f), shallow foreshores (γ_h), wave angles (γ_β) and berms (γ_b) are multiplied together to address their respective impacts. All of these coefficients typically range between 0.5 and 1. Specifically, in the case of no berm, shallow foreshore, smooth slope (no roughness and impermeable) and head-on waves, the coefficients are expected to be at their upper limit of 1. The corresponding ranges of application for the slope angle and relative crest freeboard are $1.0 < \cot \alpha < 4.0$ and $0.5 < R_c/H_{m0} < 3.5$.

For structures with steep slopes and vertical walls in relatively deep water, Van der Meer and Bruce (2014) [36] proposed Equation (3)

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = A_{V\&B} \exp \left[- \left(B_{V\&B} \frac{R_c}{H_{m0}} \right)^{C_{V\&B}} \right], \quad (3)$$

with the following expressions for the coefficients $A_{V\&B}$, $B_{V\&B}$ and $C_{V\&B}$:

$$A_{V\&B} = 0.09 - 0.01(2 - \cot \alpha)^{2.1} \quad \text{and} \quad A_{V\&B} = 0.09 \quad \text{for} \quad \cot \alpha > 2; \quad (4)$$

$$B_{V\&B} = 1.5 + 0.42(2 - \cot \alpha)^{1.5} \quad \text{with a maximum of} \quad B_{V\&B} = 2.35 \\ \text{and} \quad B_{V\&B} = 1.5 \quad \text{for} \quad \cot \alpha > 2; \quad (5)$$

$$C_{V\&B} = 1.3. \quad (6)$$

Equation (3) extends its applicability beyond Equation (2) to include very steep slopes and vertical walls ($\cot \alpha \geq 0$), with very small and zero freeboard ($R_c/H_{m0} = 0$) under non-breaking wave conditions. Compared to Equation (2), it adds a constant power coefficient $C_{V\&B}$ inside the exponential function, indicating that the prediction follows a Weibull distribution. In Equation (3), the coefficient $A_{V\&B}$ determines the value of the dimensionless average overtopping rate $q/\sqrt{gH_{m0}^3}$ for the zero freeboard case ($R_c = 0$), while the coefficient $B_{V\&B}$ determines the shape of the prediction for the entire range of relative crest freeboards R_c/H_{m0} .

Vertical structures ($\cot \alpha \geq 0$) and zero freeboard structures ($R_c/H_{m0} = 0$) are the limit cases of steep low-crested structures. Although Equation (3) includes these limiting cases, it is essential to note that the derivation of these formulas was based on a relatively constrained dataset. Specifically, the foundational research was drawn from a set of tests with zero freeboards conducted by Smid et al. (2001) [37]. Besides the above predictions, various authors developed formulas that are only valid for vertical structures or zero freeboards. Franco et al. (1994) [38] developed the following equation based on 2D experimental tests on vertical walls in relatively deep water ($h_t/H_{m0} > 3.0$) with relative crest freeboards ($0.8 < R_c/H_{m0} < 3.0$) under non-breaking conditions

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot e^{-4.3 \frac{R_c}{H_{m0}}}. \quad (7)$$

The Eurtop (2018) manual [28] presented Equation (8) (originally proposed by Allsop et al. (1995) [39]) for vertical and composite vertical structures with influencing foreshores under non-breaking conditions valid for $R_c/H_{m0} \geq 0$.

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = 0.05 \cdot e^{-2.78 \frac{R_c}{H_{m0}}}. \quad (8)$$

Figure 2 summarizes the overtopping prediction formulas reviewed in this section. For the same relative crest freeboards, the dimensionless average overtopping rates of mild slopes are larger than those of vertical structures. Furthermore, for the vertical structure, when relative crest freeboards are small ($R_c/H_{m0} < 0.6$), all the formulas feature predictions within a similar range. For large relative crest freeboards, the Allsop et al. (1995) [39] prediction, which is included in the Eurtop (2018) manual [28], Equation (8) exhibits higher overtopping rates for the same values of relative crest freeboard.

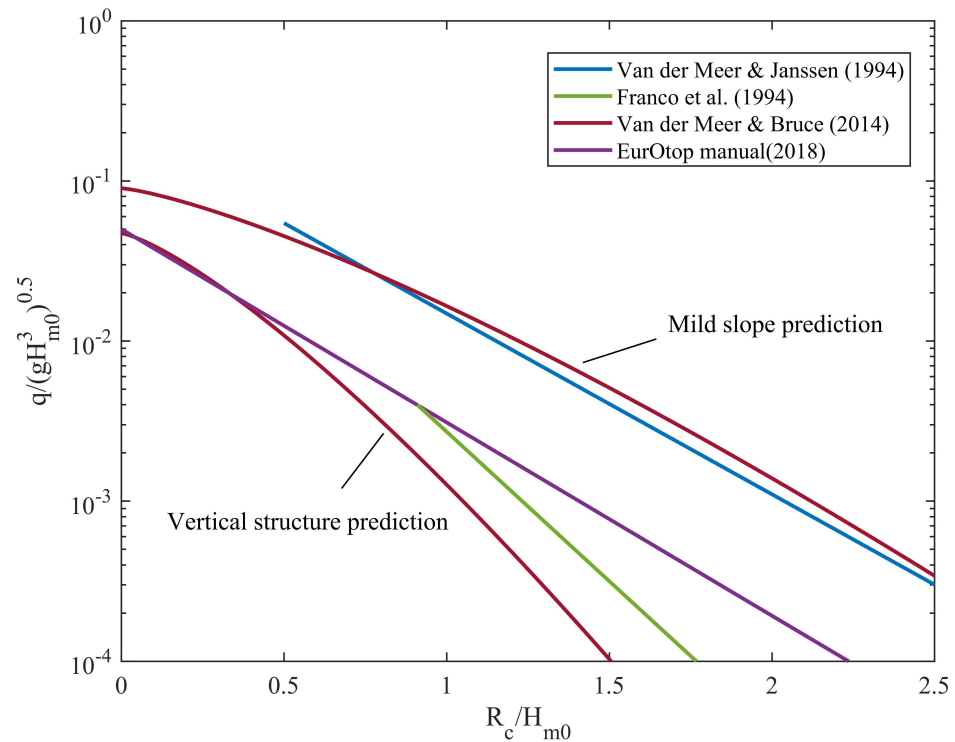


Figure 2. Average overtopping prediction formulas by [28,35,36,38] for mild slopes ($\cot \alpha > 2$) and vertical structures ($\cot \alpha = 0$).

2.2. Distribution of Individual Overtopping Volume

Van der Meer and Janssen (1995) [35] conducted an in-depth investigation into the phenomenon of wave-only overtopping on structures with sloping fronts. They revealed that the distribution of individual overtopping wave volumes was well represented by the two-parameter Weibull probability distribution given by Equation (9)

$$P_V(V_i \leq V) = 1 - \exp \left[- \left(\frac{V}{a} \right)^b \right], \quad (9)$$

where P_V is the cumulative probability that an individual overtopping volume (V_i) will be less than a specified volume (V). The two parameters of the Weibull distribution are the non-dimensional shape factor, b , which helps define the extreme tail of the distribution and the dimensional scale factor, and a , which normalizes the distribution. Van der Meer and Janssen (1995) [35] formulated the scale factor as

$$a = \left(\frac{1}{\Gamma(1 + 1/b)} \right) \cdot \left(\frac{qT_m}{P_{ov}} \right), \quad (10)$$

where T_m means average wave period, P_{ov} is probability of overtopping waves and Γ denotes mathematical gamma function.

Victor (2012) [40] and Victor et al. (2012) [41] conducted small-scale tests to study the distribution of individual wave overtopping volumes on steep, low-crested smooth

structures with $0.11 < R_c/H_{m0} < 1.69$, $0.36 \leq \cot \alpha \leq 2.75$. These conditions represented heavier overtopping on steep seaward slopes. They fit the Weibull distribution to the upper 50% of the volumes, and their analysis determined that the Weibull shape factor varied as a function of relative freeboard and seaward slope as

$$b = \exp\left(-2.0 \frac{R_c}{H_{m0}}\right) + (0.56 + 0.15 \cot \alpha). \quad (11)$$

Hughes et al. (2012) [42] re-analyzed the tests with smooth slopes reported by Van der Meer and Janssen [35], Victor (2012) [40] and Victor et al. (2012) [41], utilized varying datasets to more accurately ascertain the shape factor of the Weibull distribution. Equation (12) was developed [42] to estimate the shape factor in the range of $-2 < R_c/H_{m0} < 4$

$$b = \left[\exp\left(-0.6 \frac{R_c}{H_{m0}}\right) \right]^{1.8} + 0.64. \quad (12)$$

Zanuttigh et al. (2014) [43] investigated the shape factor (b) for both smooth-slope and rubble mound breakwater distributions. They found that rubble mound structures exhibit more significant variability in the shape factor (b) than smooth slopes. Consequently, they recommended correlating the shape factor (b) with dimensionless mean wave overtopping discharges (q) rather than the relative crest freeboard (R_c/H_{m0}), as mean overtopping discharge inherently includes details like wave steepness or slope angle. Zanuttigh et al. (2014) [43] proposed to use Equation (13) to estimate the shape factor for conventional and low-crested mound breakwaters within the ranges $0 < R_c/H_{m0} < 2$:

$$b = 0.85 + 1500 \left(\frac{q}{gH_{m0}T_{m-1,0}} \right)^{1.3}, \quad (13)$$

where $T_{m-1,0}$ is the average spectral wave period defined from spectral analysis.

2.3. Probability of Overtopping

Equation (9) is applied to the number of waves that reach the crest and cause wave overtopping and is not applied to the number of incident waves in a storm. To apply Equation (9) for a specific case, one has to calculate the probability of overtopping [35], P_{ov} , or the number of overtopping waves, which is given by

$$P_{ov} = \exp\left[-\left(\sqrt{\ln 0.02} \frac{R_c}{R_{u2\%}}\right)^2\right] \quad (14)$$

where $R_{u2\%}$ is the runup height exceeded by 2% incident waves.

3. Fixed OWEC

Kofoed (2002) [44] conducted model tests for an OWEC with a single-level reservoir and smooth and impermeable slopes. During model testing, the effects of various parameters on the overtopping discharge, including the slope angle, the crest freeboard and the draft, were investigated. Thus, new overtopping expressions for non-breaking waves were formulated as

$$Q = \frac{q}{\lambda_\alpha \lambda_{dr} \lambda_s \sqrt{gH_{m0}^3}} = 0.2 \cdot e^{-2.6 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \gamma_b \gamma_h \gamma_\beta}}, \quad (15)$$

where

$$\lambda_\alpha = \cos^\beta(\alpha - \alpha_m). \quad (16)$$

Equation (16) is formulated so that λ_α is equal to 1 for the optimal slope angle, which is set at $\alpha_m = 30^\circ$. This angle is identified as the one that maximizes overtopping. The value of λ_α diminishes as the ramp's inclination α deviates from this optimal angle. The

λ_{dr} coefficient, which considers the influence of limited draft, is expressed by Kofoed (2002) [44] as

$$\lambda_{dr} = 1 - \kappa \cdot \frac{\sinh\left(2 \cdot k_p \cdot d \left(1 - \frac{d_r}{d}\right)\right)}{\sinh(2 \cdot k_p \cdot d) + 2 \cdot k_p \cdot d'} \quad (17)$$

where k_p represents the wave number derived from L_p , and κ is a coefficient that governs the extent of the influence of the limited draft. κ is found to be 0.4 by best fit. The expression is based on the average ratio between the average amount of energy flux integrated from the draft up to the surface $E_{f,dr}$ and the average of energy flux integrated from the sea bed up to the surface $E_{f,d}$ as

$$\frac{E_{f,dr}}{E_{f,d}} = \frac{\int_{-d_r}^0 p^+ \cdot u \cdot dz}{\int_{-d}^0 p^+ \cdot u \cdot dz} = 1 - \frac{\sinh\left(2 \cdot k_p \cdot d \left(1 - \frac{d_r}{d}\right)\right) + 2 \cdot k_p \cdot d \left(1 - \frac{d_r}{d}\right)}{\sinh(2 \cdot k_p \cdot d) + 2 \cdot k_p \cdot d}. \quad (18)$$

Finally, the correction coefficient λ_s has been introduced to address low R_c . This adjustment was necessary due to increasing discrepancies with Van der Meer and Janssen (1995) [35] as $R = \frac{R_c}{H_s}$ decreases from 0.75 to 0. The formula for this correction factor is as follows:

$$\lambda_s = \begin{cases} 0.4 \cdot \sin\left(\frac{2\pi}{3} R\right) + 0.6 & \text{for } R < 0.75 \\ 1 & \text{for } R \geq 0.75 \end{cases} \quad (19)$$

To improve the performance of OWECs, multi-level reservoirs, as seen in the Sea-wave Slot-cone Generator (SSG) shown in Figure 3, have been suggested and studied [19,45–47]. This wave energy converter consists of three reservoirs placed one over the other (above the mean water level) that temporarily store the water of incident waves. Low-head hydraulic turbines are then used to convert the potential energy of the stored water into electric power. Compared to the single reservoir structure, using multi-level reservoirs results in a higher overall efficiency [21,48–50]. Research indicates that hydraulic power efficiency can be boosted by up to 76% when using five levels instead of a single reservoir [51]. In terms of economic feasibility, using three-level reservoirs in practice resulted in a 38–53% increase in the potential energy of the overtopping water [52]. It could attain a hydraulic efficiency of 51.8% optimized crest level [17].

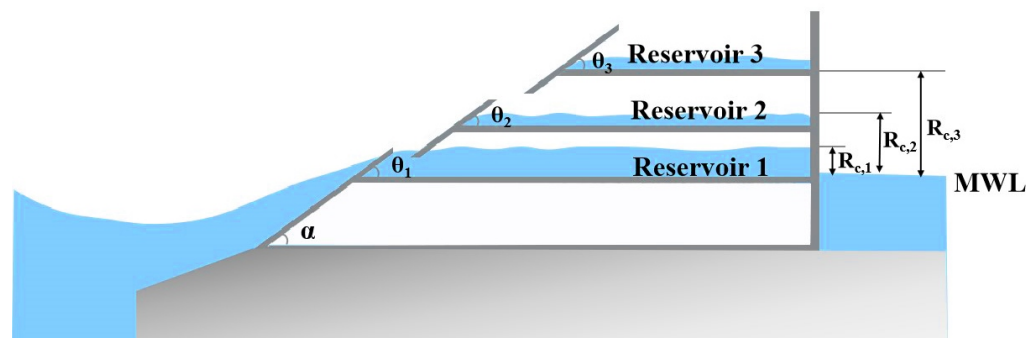


Figure 3. Cross sketch of SSG (reproduced from [52]).

In Kofoed (2002) [44], an expression for the dimensionless derivative of the overtopping discharge relative to the vertical distance is provided for SSG with multi-level reservoirs:

$$Q' = \frac{dq/dz}{\lambda_{dr} \sqrt{g \cdot H_{m0}}} = A \cdot e^{B \frac{z}{H_{m0}}} \cdot e^{C \frac{R_{c,1}}{H_{m0}}}, \quad (20)$$

where Q' is the dimensionless derivative of the overtopping discharge concerning the vertical distance z , and $R_{c,1}$ is the crest freeboard of the lowest reservoir. The coefficients A , B and C are empirical and must be fitted with experimental data measured on a scale model of the SSG. λ_{dr} is a coefficient describing the dependency of the draught. Equation (20) can

be used to calculate the overtopping discharge for each reservoir in a structure featuring multiple levels of reservoirs. Rewriting Equation (20) results in the following [44,53]:

$$\frac{dq}{dz} = \lambda_{dr} \sqrt{g \cdot H_{m0}} A \cdot e^{B \frac{z}{H_{m0}}} \cdot e^{C \frac{R_{c,1}}{H_{m0}}} \quad (21)$$

Therefore, calculated overtopping rates for the j -th reservoir can be found by integrating Equation (21) over the crest levels $R_{c,j}$ and $R_{c,j+1}$:

$$q_j = \int_{R_c}^{R_{c,j+1}} \frac{dq}{dz} dz = \lambda_{dr} \sqrt{g \cdot H_{m0}^3} \cdot \frac{A}{B} \cdot e^{C \frac{R_{c,1}}{H_{m0}}} \cdot \left(e^{B \frac{R_{c,j+1}}{H_{m0}}} - e^{B \frac{R_{c,j}}{H_{m0}}} \right) \quad (22)$$

where $R_{c,j}$ is the crest freeboard of the current reservoir and $R_{c,j+1}$ is the crest freeboard of the reservoir above. For the highest reservoir of a device with no roof, $R_{c,j+1}$ can be set to a sufficiently high value, such as twice $R_{c,j}$.

Equation (15) is an extension of the existing overtopping expression for non-breaking waves presented by Van der Meer and Janssen (1995) [35], i.e., Equation (2). Equation (2) is developed based on tests with slopes with a berm, foreshore, rough surface, short-crested and oblique waves, while Equation (15) is based on tests with smooth and non-permeable structure and linear profile. Equation (2) has been modified by applying correction factors to include the effect of the limited draft, slope angle and low relative crest freeboards. Therefore, Equation (15) not only allows for the prediction of overtopping discharge for relative crest freeboard down to 0 but also allows for the prediction of overtopping discharge for structures with limited draft. The methods for calculating these correction factors in Equations (15) and (20) were derived from separate experimental results. Despite their strengths, the above formulas have yet to consider the potential interactions between several correction factors adequately. They further analyze relationships that link the geometric correction indices.

Table 1 summarizes the advantages and limitations of traditional breakwaters and fixed OWECs. It compares the characteristics of the two structures in terms of energy attenuation, structural integrity, environmental compatibility, energy conversion efficiency, structural complexity, maintenance and cost-effectiveness. For instance, traditional bottom-mounted breakwaters reduce wave energy and provide long-term stability. Still, they may not be as efficient in energy conversion as fixed OWECs designed explicitly for energy conversion. Additionally, fixed OWECs can be optimized with specific geometries to facilitate wave overtopping but this may incur higher maintenance and construction costs.

Table 1. Advantages and limitations of traditional breakwaters and OWECs.

Feature	Traditional Breakwaters	OWEC
Advantages		
Energy Attenuation	Effective in reducing wave energy, providing a calm area for the operation of wave energy converters	Designed to allow wave energy to pass through, potentially useful for wave energy conversion
Structural Integrity	Proven design with well-understood structural characteristics, ensuring long-term stability	Can be optimized for wave energy conversion with specific geometries that facilitate overtopping

Table 1. Cont.

Feature	Traditional Breakwaters	OWEC
Environmental Compatibility	Generally has less impact on the surrounding marine environment due to their traditional design	Can be designed to minimize environmental impact while still allowing for efficient energy extraction
Limitations		
Energy Conversion Efficiency	Not specifically designed for energy conversion, may not maximize the capture of wave energy	Relies on overtopping for energy conversion, which can be less efficient than other methods like direct wave capture
Structural Complexity	Simpler structures may not provide the optimal conditions for wave energy conversion	More complex structures compared to traditional breakwaters, which can increase construction and maintenance costs
Maintenance	May require less maintenance due to their straightforward design	Potentially higher maintenance due to moving parts or more intricate design features
Cost-Effectiveness	Lower initial construction costs due to simplicity, but may not generate revenue through energy production	Potentially higher maintenance due to moving parts or more intricate design features

4. Floating OWECs

In addition to nearshore overtopping wave energy converters (WECs), innovative floating OWECs have been engineered to better accommodate the operational requirements and environmental conditions of different locations. Sea Power was one of the early devices deployed and tested in Sweden. Wave Dragon (WD) is the world's first grid-connected floating WEC of the overtopping type. It was developed originally by Erik Friis-Madsen from the Danish engineering company LSvenmark Consulting Engineers in 1986. A floating model of the WD (scale 1:50) was built in the autumn of 1998 by the Danish Maritime Institute to establish the response from the waves of different heights, the magnitude of the forces in the mooring system and the energy efficiency [54]. This model was the first generation of WD OWECs, subjected to a series of model tests and subsequent modifications at Aalborg University [48,55–57]. Based on the previous findings, Friis-Madsen and Armstrong Technology [54,58] redesigned the WD denoted as the second generation. Compared to the first-generation design, the second-generation WD has optimized overall structural geometry, focusing mainly on reflector design and ramp [59].

WD consists of two wave reflectors focusing on the incoming waves, a floating reservoir with a double curved ramp for collecting the overtopping water and a set of low-head turbines for converting the pressure head into power [21,60], as shown in Figure 4. Martinelli and Frigaard (1999b) [56] tested the Wave Dragon (both before [55] and after modifications [56]) in long-crested seas as well as in short-crested seas with the standard JONSWAP spectrum. It was found that, when the WD is subjected to 2D waves, the mean overtopping rate per unit width of the ramp q can be best described as

$$q = 0.017C_d \cdot \exp\left(-48 \frac{R_c}{H_{m0}} \sqrt{\frac{S_{op}}{2\pi}}\right) \frac{L \sqrt{g \cdot H_s^3}}{\sqrt{\frac{S_{op}}{2\pi}}}, \quad (23)$$

where $C_d = 0.9$ is the reduction coefficient accounting for directional spreading effects, L is the length of structure ramp, S_{op} is the wave steepness defined as $S_{op} = H_{m0}/L_{op}$, where L_{op} is the deep water wave length as $L_{op} = gT_p^2/2\pi$, given T_p is the peak period. A constant ratio of 1.2 between the peak and the mean period is usually assumed.

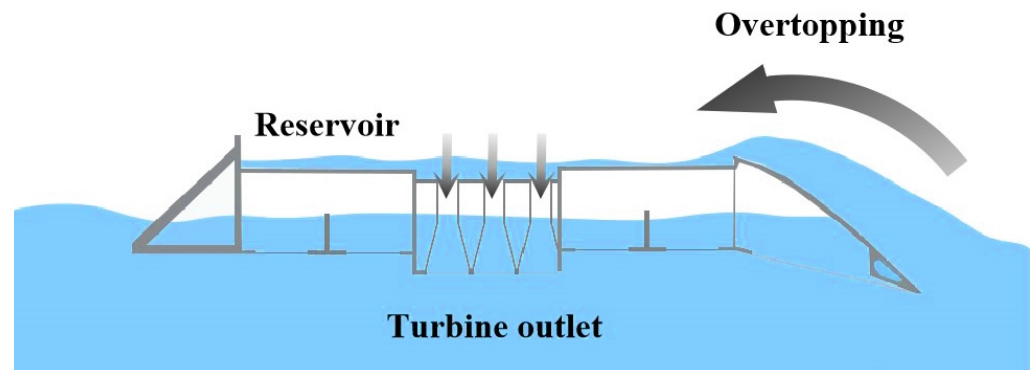


Figure 4. Sketch of the working principle of Wave Dragon, reproduced from [48].

Equation (23) is applicable exclusively under wave conditions wherein the peak period is separated significantly from the natural period of the pitch movements, and the movements of the reservoir are limited. Equation (23) cannot be well applied to wave situations with significant wave heights greater than 3 m, as it would lead to an overestimation of the overtopping rate increasingly with the degree in proportion to the increase in significant wave heights (to as much as a factor 2 for a significant wave height of 5 m). Hald and Frigaard (2001) [59,61] presented a modified overtopping equation based on the tests for the second-generation model with the doubly-curved overtopping ramp. The new overtopping equation is modified as

$$q = 0.025C_d \cdot \exp\left(-40 \frac{R_c}{H_{m0}} \sqrt{\frac{S_{op}}{2\pi}}\right) \frac{L \sqrt{g \cdot H_s^3}}{\sqrt{\frac{S_{op}}{2\pi}}}. \quad (24)$$

Figure 5 presents a comparative analysis of the overtopping rate (q) per unit width of the ramp for the Wave Dragon device. The curve-fitting of the data points is described by Equations (23) and (24), which is adjusted based on the reduction coefficient (C_d) accounting for directional spreading effects. Figure 5 shows that the overtopping rate increases exponentially with an increase in wave height, with the increase being more pronounced at higher wave steepness values. As indicated in Figure 5, the measured prototype data compare well with the expression based on laboratory tests. Compared to the first-generation design, the second generation has optimized the hydraulic performance of the WD and significantly improved the energy capture (almost double) [21].

Kofoed (2002) [44] evaluated the impact of extremely low R_c and limited draft d_r on Q , and extended the applicability of the model to $0.58 \leq \cot(\alpha) \leq 2.75$ and $0.15 \leq R \leq 2$. Based on Equation (15), the choice was to introduce some dedicated offset-correcting factors depending on features of the structure and the waves while assuming all gain correction factors are 1. The general formula applicable to floating OWEC is as

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot \lambda_{dr} \cdot \lambda_s \cdot e^{(-2.6 \cdot R)}. \quad (25)$$

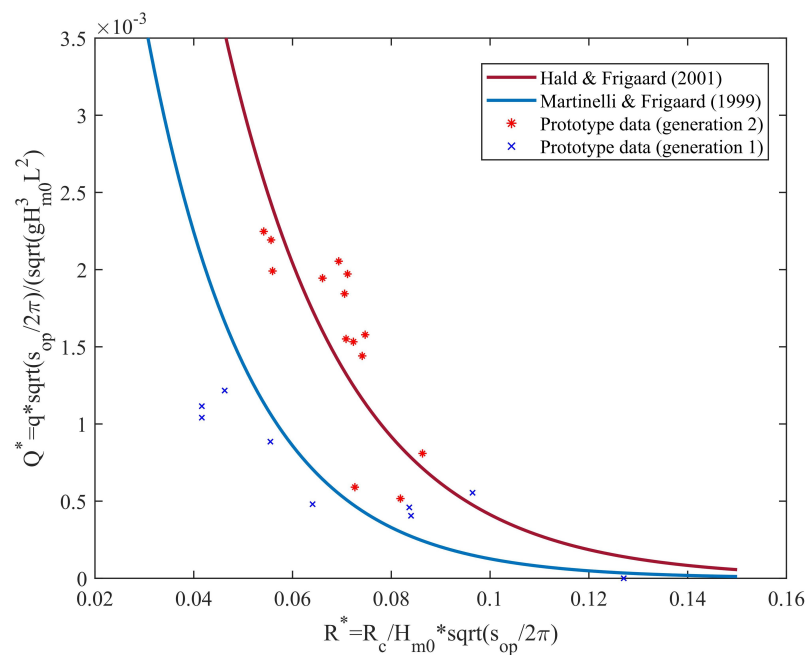


Figure 5. Comparison of preliminary prototype data and overtopping expression (Prototype data Ref. Hald et al. (2001) [59] and Kofoed et al. (2004) [62]. * is superscript indicating dimensionless variable).

Although the models presented by Kofoed (2002) [44] were developed for overtopping wave energy converters (OWECs), they do not fully represent the Wave Dragon's geometry. These models fail to account for the unique 3D structure of the Wave Dragon, the effects of its reflector wings and its dynamic response to waves. All these factors must be considered to develop a tool for more accurate predictions of Wave Dragon overtopping. To achieve this purpose, Parmeggiani et al. (2013) [63] conducted a series of tank testing experiments on a 1:51.8 scale model of a 4 MW Wave Dragon between 2011 and 2012. The study comprised four phases [63], each performing a distinct sensitivity analysis to identify the influence of specific features of the setup or the waves on Q . They established an updated version of the overtopping model (Kofoed (2002) [44]) refined for the Wave Dragon device, which has accounted for the effects of real 3D geometry, low crest height and yielding by introducing more offset-correcting factors

$$Q = \frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \cdot \lambda_{dr} \cdot \lambda_s \cdot \xi_p \cdot \lambda_{WD} \cdot e^{(-2.6\gamma_{WD} \cdot R)}, \quad (26)$$

where λ_{WD} and γ_{WD} are experimentally obtained and describe the effect of the 3D Wave Dragon geometry on Q , with offset-correcting factor $\lambda_{WD} = 0.1502$ and gain-correcting factor $\gamma_{WD} = 0.8275$. The effect of peak wave steepness is included through the peak breaking parameter ξ_p as

$$\xi_p = \frac{\tan(\alpha)}{\sqrt{H_{m0}/L_p}}, \quad (27)$$

where α is the slope of the ramp and L_p is the peak wave length.

Finally, including the effect of the reflector arms, the relation for the overtopping discharge reads

$$Q = \frac{q}{\sqrt{gH'_{m0}{}^3}} = 0.2 \cdot \lambda_{dr} \cdot \lambda'_s \cdot \xi'_p \cdot \lambda_{WD} \cdot e^{(-2.6\gamma_{WD} \cdot R')}, \quad (28)$$

where the amplified wave height H'_{m0} due to the reflector arms is used rather than the undisturbed wave height in the above relations, which is given as

$$H'_{m0} = H_{m0} \cdot a \cdot \zeta_{dr,R} \cdot \zeta_{op}. \quad (29)$$

Here, the fitting coefficient $a = 1.13$ is a crucial parameter that signifies that the maximum wave amplification is achievable. $\zeta_{dr,R}$ and ζ_{op} are non-dimensional correcting functions of the opening and the reflectors' draft, ranging from 0 to 1. $\zeta_{dr,R}$ is provided by Equation (17), utilizing a specific value of $\kappa = 0.1$ and substituting $d_{r,R}$ for d_r . It adjusts the model to reflect the impact of the draft of the reflectors on the overtopping process. The opening effect is incorporated through the function $\zeta_{op} = \cos(\theta_{op} - K_{op})$, where K_{op} represents the optimal opening angle at which wave amplification is maximized. The best-fit value for K_{op} is determined to be 40° . This factor accounts for the influence of the opening angle on the device's performance. Parmeggiani et al. (2013) [63] modified specific features in each phase of the study to ensure that differences in the measured Q could be entirely attributed to those changes. Equations (26) and (28) inserted correcting factors in the model formulation described by Equation (25).

Floating OWECs like Wave Dragon present unique challenges due to their dynamic interaction with waves. The first-generation Wave Dragon model was tested by Martinelli and Frigaard (1999a) [55], leading to Equation (23), which describes the mean overtopping rate per unit width of the ramp. However, this equation overestimated the overtopping rate for significant wave heights greater than 3 m. To address this, Hald and Frigaard (2001) [59] proposed a modified equation, Equation (24), which better captures the hydraulic performance of the second-generation Wave Dragon with its doubly curved ramp. Kofoed (2002) [44] further refined the model by introducing Equation (25), which considers the impact of extremely low crest freeboards and limited draft. This equation was later updated by Parmeggiani et al. (2013) [63] to include the effects of the 3D geometry of the Wave Dragon, the reflector wings and the peak wave steepness, as shown in Equations (26) and (28).

Building upon the foundational research on overtopping wave energy converters (OWECs), such as the Wave Dragon and conical floating body designs, a novel device known as the Circular Ramp Overtopping Wave Energy Converter (CROWN) has been conceptualized. The Korea Research Institute of Ship and Ocean Engineering (KRISO) initially proposed this innovative design. As shown in Figure 6, the CROWN device resembles an inverted bowl positioned in the ocean. The device's main structure consists of a cylindrical reservoir and a circular ramp for wave run-up and overtopping. By using a circular ramp, CROWN better adapts to the incident wave directions. A discharge pipe extends downward from the center of the reservoir, connecting the reservoir to the surrounding water pool. Guide vanes are installed on the ramp to guide more seawater into the reservoir and enhance the overtopping discharges. Liu et al. (2017) [64] defined a dimensionless average overtopping discharge Q as a function of the reservoir shape and overtopping behavior.

$$Q = \frac{q}{\pi d_m \sqrt{g \cdot d_f^3}} \quad (30)$$

where d_m is diameter of circular reservoir open mouth and d_f is waterline of reservoir.

Tables 2–4, respectively, present summaries of the overtopping prediction formulas for traditional breakwater, fixed OWECs and floating OWECs reviewed in this paper with their range of applications. The empirical formulas for traditional breakwaters are mostly based on observations and data analysis of specific types (breakwaters with mild or steep slopes and vertical structures) under varying wave conditions, apart from a few that consider the influence of factors such as wave angle, roughness, foreshore and wave crest. Formulas for truncated breakwaters expand on the traditional ones, often incorporating structural draft limitations, slope angles and reduced freeboard heights, with predictions refined through coefficient adjustments. Floating devices have seen further development on this

foundation, with particular improvements to accommodate dynamic wave interactions and complex structural forms, such as the Wave Dragon model, which specifically accounts for its unique three-dimensional structure and reflector effects.

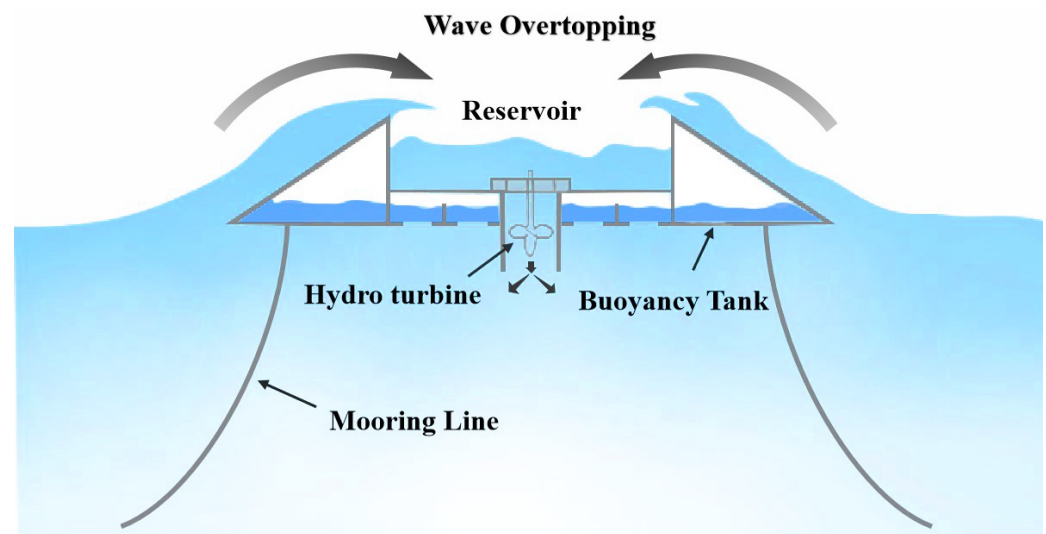


Figure 6. Sketch of the working principle of CROWN (Redrawn from [64]).

Table 2. Average overtopping prediction formulas for traditional breakwaters and the applicability range of parameters.

Sources	Structures	Overtopping Model	Dimensionless Overtopping Discharge Q	Dimensionless Freeboard R	Applicability Range of Parameters
[34]	Impermeable smooth, rough, straight and bermed slopes	$Q = Ae^{-bR}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	
[35]	Berm, foreshore, rough surface, mild slopes	$Q = 0.2 \cdot e^{-2.6R}$ (for $2 \leq \zeta_{m-1,0} \leq 5$)	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_f \gamma_f \gamma_\beta}$	Slope angle $1 \leq \cot(\alpha) \leq 4$ $0.5 \leq R \leq 3.5$
[36]	Steep slopes and vertical walls for relative deep water	$Q = Ae^{-bR}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	$0 \leq \cot(\alpha)$ $0 \leq R$
[38]	Vertical walls for relative deep water	$Q = 0.2 \cdot e^{-4.3R}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	$0.8 \leq R \leq 3.0$
[28]	Steep slopes and vertical walls for relative deep water	$Q = 0.05 \cdot e^{-2.78R}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	$0 \leq R$

Table 3. Average overtopping prediction formulas for fixed OWECs and the applicability range of parameters.

Sources	Structures	Overtopping Model	Dimensionless Overtopping Discharge Q	Dimensionless Freeboard R	Applicability Range of Parameters
[44]	A single reservoir with smooth and non-permeable surface and linear profile, low relative crest freeboard	$Q = 0.2 \cdot e^{-2.6R}$	$\frac{q}{\lambda_R \lambda_{dr} \lambda_s \sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}} \frac{1}{\gamma_b \gamma_f \gamma_f \gamma_\beta}$	
[44]	Multi reservoirs with fronts and 35° linear slope	$Q = Ae^{\frac{B}{H_s} + \frac{C}{H_s}}$	$\frac{dq/dz}{\lambda_{dr} \sqrt{g \cdot H_{m0}}}$		

Table 4. Average overtopping prediction formulas for floating OWECs and the applicability range of parameters.

Sources	Structures	Overtopping Model	Dimensionless Overtopping Discharge Q	Dimensionless Freeboard R	Applicability Range of Parameters
[55]	1. generation Wave Dragon with linear ramp inclination 45°	$Q = 0.017e^{-48R}$	$\frac{q\sqrt{S_{op}/2\pi}}{\sqrt{gH_{m0}^3L}}$	$\frac{R_c}{H_{m0}}\sqrt{\frac{S_{op}}{2\pi}}$	Significant wave height $H_{m0} \leq 3$
[59]	2. generation Wave Dragon with doubly-curved ramp	$Q = 0.025e^{-40R}$	$\frac{q\sqrt{S_{op}/2\pi}}{\sqrt{gH_{m0}^3L}}$	$\frac{R_c}{H_{m0}}\sqrt{\frac{S_{op}}{2\pi}}$	
[44]	Wave Dragon, low crest freeboard and limited draft	$Q = 0.2\lambda_{dr}\lambda_s e^{-2.6R}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	Slope angle, α $0.58 \leq \cot(\alpha) \leq 2.75$ $0.15 \leq R \leq 2$
[63]	2 generation Wave Dragon	$Q = 0.2 \cdot \lambda_{dr} \cdot \lambda'_s \cdot \tilde{\zeta}'_p \cdot \lambda_{WD} \cdot e^{(-2.6\gamma_{WD} \cdot R')}$	$\frac{q}{\sqrt{gH_{m0}^3}}$	$\frac{R_c}{H_{m0}}$	$0.2 \leq R \leq 1.5$

5. Conclusions and Further Work

This review addresses the effectiveness of current empirical formulas in predicting wave overtopping for overtopping wave energy converters (OWECs). This paper analyzed various empirical approaches to identify their strengths, limitations and applicability to various OWEC configurations. Our findings reveal that, for traditional breakwaters, the empirical formulas are based on observations and data analysis of the wave overtopping behavior of specific types (breakwaters with mild slope or steep slope and vertical structures) under varying wave conditions. These formulas usually contain key parameters such as the wave height, period and geometry of the breakwaters (e.g., slope, freeboard). These formulas are often based on simplifying assumptions such as wave regularity and the uniformity of the breakwater for ease of application, and each formula has its specific conditions and scope of use.

The empirical formulas used for predicting overtopping in fixed and floating OWECs draw extensively from the principles established for traditional breakwaters, characterized by their smooth and impermeable linear slopes. The formula proposed by Van der Meer and Janssen (1995) [35] is a foundational reference for developing models such as SSG and Wave Dragon. Different correction coefficients are introduced to account for the effects of limited draft, inclination angle and low relative freeboard. The current empirical formulas for floating OWECs are derived from prototype tests based on the WD model. The second-generation WD model proposes some modified empirical formulas based on the first-generation model by considering the effect of WD's unique 3D structural reflector and the response to dynamics. While these formulas have been primarily distilled from simplified, small-scale experimental setups tailored to specific research objectives, their applicability across diverse scenarios necessitates further scrutiny.

We acknowledge that this paper has several limitations. Due to the vast and rapidly evolving body of literature, the review may not encompass all existing empirical models. In addition, many of the reference studies are simplified small-scale experiments based on specific experimental purposes and may not fully reflect large-scale conditions.

Although these existing empirical models have been validated over many physical model experiments and prototype testings, significant gaps remain in their applicability and accuracy. Existing models often derive from reduced-scale experiments, leading to potential inaccuracies in full-scale applications. Many formulas are based on idealized conditions, which do not fully capture the variability of real-world ocean environments. Many models do not adequately represent the unique geometric configurations of OWECs, such as Wave Dragon's 3D structure.

Moving forward, emerging technologies and methodologies offer promising solutions to improve empirical prediction models. Machine learning and artificial intelligence can analyze large datasets from experiments and field measurements to uncover new patterns and

enhance prediction accuracy. High-fidelity computational fluid dynamics (CFD) models can provide detailed insights into wave-structure interactions. At the same time, advanced sensors and real-time monitoring systems in OWECs can refine and validate empirical models with continuous data. Future research should focus on full-scale experimentation in diverse ocean conditions, developing integrated models that combine empirical, numerical and machine learning approaches, and fostering interdisciplinary collaboration between engineers, oceanographers and data scientists.

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Abbreviations

The following abbreviations are used in this manuscript:

FOWT	Floating Offshore Wind Turbine
CROWN	Circular ramp overtopping wave energy converter
IEA	The International Energy Agency
KRISO	The Korea Research Institute of Ship and Ocean Engineering
OBSs	Oscillating body systems
OWCs	Oscillating water columns
OWECs	Overtopping wave energy converter
PTO	Power take-off
SSG	Sea-wave Slot-cone Generator
TAPCHAN	Tapered Channel
WD	Wave Dragon

Nomenclature

Q	Non-dimensional average overtopping flow per crest meter width
R_c	Crest freeboard used for the vertical distance from mean water to the crest of the reservoir
R	Non-dimensional crest freeboard
H_{m0}	Significant wave height estimated from the zero-moment of the wave spectrum
W	Crest width
g	Acceleration due to gravity
A	Weibull scale parameter
B	Weibull shape parameter
$\xi_{m-1,0}$	Breaker parameter
γ_b	Influence factor for a berm
γ_f	Influence factor for roughness elements on a slope
γ_β	Influence factor for oblique wave attack
γ_h	Influence factor for shallow foreshore
$A_{V\&B}$	Coefficient in equation of van der Meer and Bruce
$B_{V\&B}$	Coefficient in equation of van der Meer and Bruce
$C_{V\&B}$	Coefficient in equation of van der Meer and Bruce
a	Weibull scale parameter
b	Weibull scale parameter
P_{ov}	Probability of overtopping
P_V	Probability of an individual wave volume
V_i	Individual wave volume
V	Specified volume
α	Slope angle

V_{max}	Maximum overtopping volume
γ_{α}	Influence factor for slope angle
γ_{dr}	Influence factor for limited draft
$E_{f,dr}$	Average amount of energy flux integrated from the draft up to the surface
$E_{f,d}$	Average of energy flux integrated from the sea bed up to the surface
γ_s	Correction coefficient to address low-crest freeboard
Q'	Dimensionless derivative of the overtopping discharge relative to the vertical distance
z	Vertical distance
$R_{c,1}$	Crest freeboard of the lowest reservoir
$R_{c,j}$	Crest freeboard of the current reservoir
$R_{c,j+1}$	Crest freeboard of the reservoir above
q_j	Overtopping rates for the j-th reservoir
q	Overtopping discharge
C_d	Reduction coefficient accounting for directional spreading effects
L	Length of structure ramp
S_{op}	Wave steepness
L_{op}	Deep water wave length
T_p	Peak period
λ_{WD}	Offset-correcting factor that describe the effect of the 3D Wave Dragon geometry
γ_{WD}	Gain-correcting factor that describes the effect of the 3D Wave Dragon geometry
ξ_p	Peak breaking parameter
H'_{m0}	Amplified wave height due to the reflector arms
θ_{op}	Non-dimensional correcting functions of the opening of the reflectors
$\theta_{dr,R}$	Non-dimensional correcting functions of the draft of the reflectors
K_{op}	Optimal opening angle at which wave amplification is maximized
d_m	Diameter of circular reservoir open mouth
d_f	Waterline of reservoir
*	Superscript indicating dimensionless variable

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