Journal of Naval Architecture and Marine Engineering
June, 2023
https://doi.org/10.3329/jname.v20i1.55046 http://www.banglajol.info

WAVE OVERTOPPING CHARACTERISTICS OF NON-PERFORATED AND SEASIDE PERFORATED EMERGED QUARTER-CIRCLE BREAKWATER

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Abstract:

A breakwater is a structure used to dissipate the wave energy in order to protect the shore and maintain tranquility inside the harbor basin. The quarter-circle breakwater (QBW) constitutes a quarter circular front wall facing incident waves, a vertical rear wall, and a horizontal base slab placed on a rubble mound foundation. In this study, a comprehensive experimental investigation is carried out in order to examine the wave overtopping characteristics of an emerged non-perforated and seaside perforated emerged quarter-circle breakwater subjected to regular waves. A model scale of 1:30 is selected based on the limitations of testing facilities. For the current investigation, an emerged QBW models of the radius 0.50 m is utilized. The model is tested for six different perforations ranging between 0% and 20%, with a constant perforation radius of 0.016 m. The paper highlights the influence of wave steepness (H_i/gT^2), relative crest freeboard (R_c/H_i), relative water depth (d/gT^2) on the wave overtopping performance of the seaside perforated and non-perforated QBW models. An increase in wave steepness is found to increase the dimensionless mean wave overtopping discharge. Also, an exponential decrease in dimensionless mean wave overtopping discharge is observed with an increasing relative freeboard. The relative freeboard is found to be one of the predominant parameters influencing the wave overtopping discharge rate.

Keywords: Emerged quarter-circle breakwater; perforations; mean wave overtopping discharge; relative freeboard; and wave steepness

NOMENCLATURE

d	- Water depth	q	- Overtopping volume
D	- Diameter of perforations	R	- Radius of structure
g	- Acceleration due to gravity	T	- Wave period
h_s	- Height of structure	q/gH_iT	- Overtopping discharge
H_i	- Incident wave height	d/gT^2	- Depth parameter
L	- Wave length	H_i/gT^2	- Incident wave steepness
p	- Percentage of perforations	R_c/H_i	- Relative freeboard

1. Introduction

A breakwater is a structure used to attain calm conditions on its lee side. Throughout the world, different types of breakwaters are used to protect the coastal region and harbours. Research accomplishments are evolving to analyze the performance of hydrodynamic characteristics of new innovative breakwaters, which can be recommended for the prevailing economic and environmental conditions (Aburatani et al., 1996; Mane et al., 2013; Rajendra et al., 2017). The Quarter circular breakwater was proposed by Xie et al. (2006) based on the concept of the semi-circular breakwater (SBW), and the construction of QBW is almost similar to SBW, which is generally provided with a base of rubble mound foundation. A conceptual 3D view of non-perforated QBW and seaside perforated QBW is as shown in Figure 1.

1813-8535 (Print), 2070-8998 (Online) Received on: Aug., 2021

As the bottom width of QBW is half of the base width of SBW, the volume of its rubble mound foundation would be reduced to nearly half of the SBW. However, QBW still has advantages such as reducing wave force on the seaside surface against the incoming wave, easy installation as it is prefabricated on land, and an excellent aesthetic view similar to SBW. The most common difference between QBW and SBW is their rubble foundation width, which causes different stress distributions on the foundation. QBW is suitable for the places where a stronger subsoil is available, as the magnitude of stress on the foundation soil will be more. The wave overtopping phenomenon is generally the flow of seawater over a crest of the coastal structure due to wind action, wave run-up, and wave breaking (Van der Meer et al., 2016).

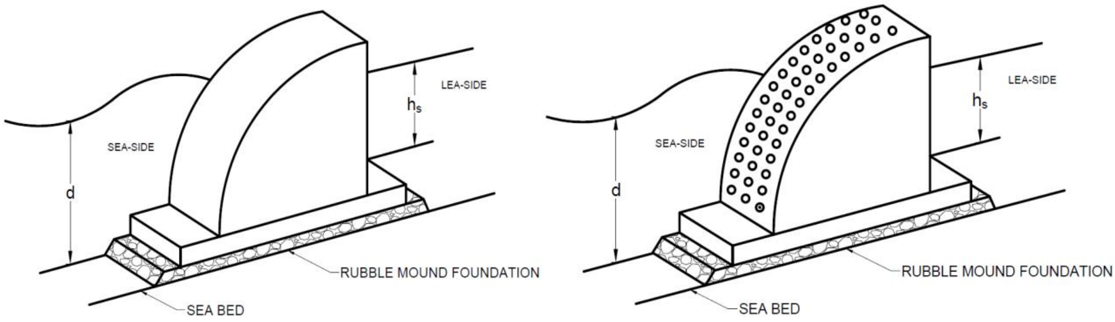

Fig. 1: Isometric view of non-perforated and perforated QBW

Many researchers studied the hydrodynamic performance characteristics of QBW, wherein they focused mainly on the investigation of dynamic wave pressures, transmission, and reflection characteristics (X. L. Jiang et al., 2017; Liu et al., 2006). Also, the characteristics of different types of breakwater models (rubble mound and vertical breakwater) were analyzed, focusing on the wave overtopping performance (Gil et al., 2015; Tuan, 2013; Van Bergeijk et al., 2019).

As expected, Shi et al. (2011) observed that the loss of wave energy for emerged breakwater is more than that for the submerged breakwater. They have concluded that the hydrodynamic performances of SBW and QBW are almost similar, resulting in the identical wave profiles of both breakwaters. Hegde and Ravikiran (2013) examined the impact of wave structure interaction for submerged QBW of the different radii, wave height, and submergence ratios. They concluded that the wave reflection increased with an increase in wave steepness. Further, Qie et al. (2013) conducted a study on the development of a wave force formula to design quarter-circular caisson breakwater. They suggested a simplified method to calculate the wave forces based on the Goda formula. Pedersen (1996) carried out experimental work to examine the crown wall's performance against the wave forces and wave overtopping. The authors developed a newly designed empirical formula to predict the mean overtopping discharge over a crown wall structure.

Franco et al. (1994) measured the wave overtopping response on various caisson breakwaters and studied the probability distribution of individual overtopping waves. The authors concluded that the overtopping discharges on deepwater vertical walls are considerably greater than that of those projected by Tanimoto and Goda (2015) and moderately lesser than those for a corresponding sloping arrangement. The arrangement of a perforated wall with a recurved crest (nose) on the front wall creates a significant overtopping drop, while rock shelter in front of the caisson up to the sea level can increase overtopping.

An experimental study was carried by Reis et al. (2008) on a two-dimensional breakwater model to investigate the impact of the test duration on mean wave overtopping. The effect of the higher waves on the overtopping discharge is strong. Even a low variance in the elevation of the involved waves in a wave train may powerfully influence the mean overtopping discharge, especially for lesser overtopping rates with a small test duration. Another researcher, Bruce et al. (2007) compared the overtopping performance for different armour units of rubble mound breakwater through experimental investigation. The inquiry concluded that the wave period has a greater influence on wave overtopping, and a larger wave period contributes more to wave overtopping.

The investigators Binumol et al. (2017) and Dhinakaran et al. (2002) explored the hydrodynamic characteristics of QBW and SBW. The authors found that the dimensionless wave run-up increases with an increase in wave steepness for various values of height of the structure to the depth of water (h_s/d) and water depth parameter (d/gT^2). Also, the non-dimensional stability parameter is always decreasing with an increase in wave steepness. Another observation was that the wave run-up (R_u/H_i) decreases with an increase in the water depth parameter (d/gT^2). It is expected that curvature influence is more pronounced due to higher water depths, which results in a lower run-up.

Jiang et al. (2018) examined the flow separation and vortex dynamics phenomenon during wave overtopping on Submerged QBW. They concluded that the instant and mean value of time vorticity fields expose a couple of vortices of conflicting marks at the breakwater structure that is likely to affect transportation, suspension, and sediment entrainment. Thus, resulting in scour on the lee side of the breakwater. Further, Salauddin and Pearson (2020) studied the comprehensive two-dimensional experimental study on the sloping walls overtopping performance undertaken on both impermeable and permeable foreshore slopes. They proposed a revised forecast tool to predict the overtopping performance at sloping structures on porous rock foreshores.

Kerpen et al. (2020, 2019) developed a reduction coefficient for a ventured revetment roughness for a broad utility scope considering its progression proportion. The obtained reduction coefficient for ventured revetments did not base on the prototype model scale. The correction factor for the prototype model scale effect for ventured revetments has not been considered, subsequently, which is likely to be influenced by scale effects.

Many researchers studied the hydrodynamic characteristics of QBW, focusing mainly on wave transmission and reflection. The available literature confirms that there are limited studies on wave overtopping characteristics of emerged QBW. Wave overtopping is an essential factor as it plays a significant role in the design of emerged QBW structure. The objective of the current study is to investigate the wave overtopping performance of non-perforated and perforated emerged QBW using physical models. The studies are conducted on emerged QBW models for varying percentages of perforation at different water levels against different wave conditions.

2. Experimental Setup and Methodology

2.1 Testing facility

A detailed investigation is carried out in a 2-dimensional wave flume equipped with a generation system for regular waves only of length 50 m, width 0.74 m, and depth 1.1 m at Marine Structures Laboratory, Department of Water Resources and Ocean Engineering, National Institute of Technology, Surathkal, Karnataka, India. The proposed quarter-circle breakwater models are tested with the existing amenities of the wave flume and suit the Mangaluru coast characteristics (Dattatri, 1993). In the wave flume, regular waves of heights varying from 0.02 m to 0.24 m and wave periods ranging from 0.8 s to 4 s can be generated. Detailed sectional views of the wave flume along with the location of the QBW model and wave probes, are shown in Figure 2.

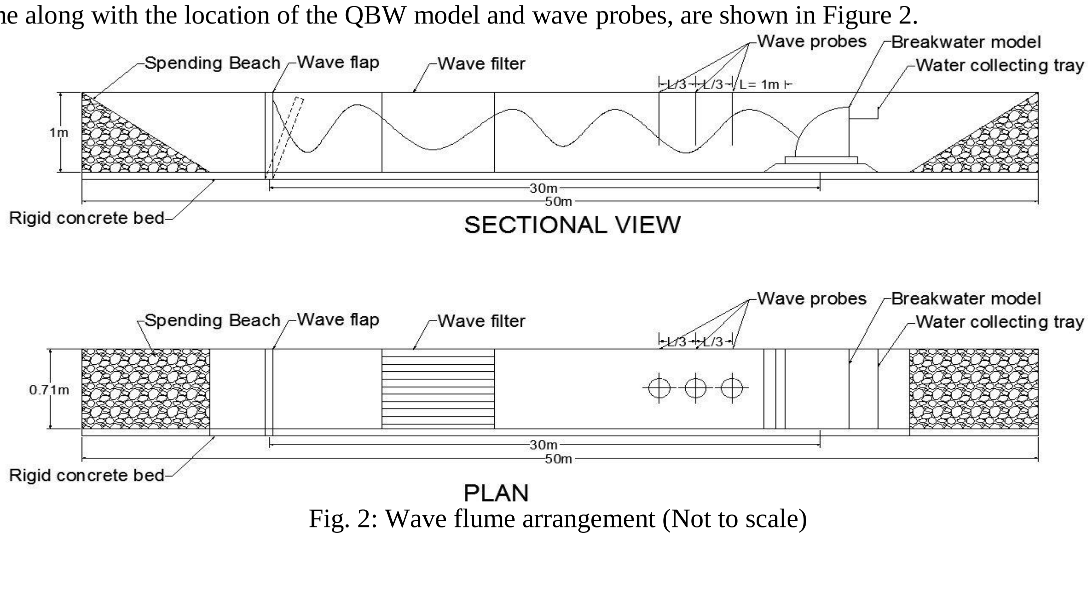

Fig. 2: Wave flume arrangement (Not to scale)

2.2 Model casting

A typical cross-section of seaside perforated and non-perforated QBW is shown in Figure 3. The breakwater model comprises two sections, the bottom slab and the top quarter-circle formed by a metal sheet. The model is built in two steps, with the first phase involving casting the base slab and the second involving casting the QBW to the necessary dimensions. The base slab is provided to increase the total weight of the QBW in order to form a stable base for the superstructure and the dimension of the slab is chosen accordingly to serve the purpose. A thick Galvanized Iron (G.I) sheet of thickness 0.002 m is used to fabricate the quarter-circular breakwater of radius 0.5 m and coated with cement slurry. Then, the G.I sheet is fixed to the base slab with the help of stiffeners. The dimensions for QBW are chosen to serve as an emerging type for all water depths and facilitate the overtopping of incident waves. The breakwater model is then positioned over the foundation with a rubble mound of thickness 0.05 m (minimum thickness as per CEM, 2001) and stones weighing from 50 g to 100 g.

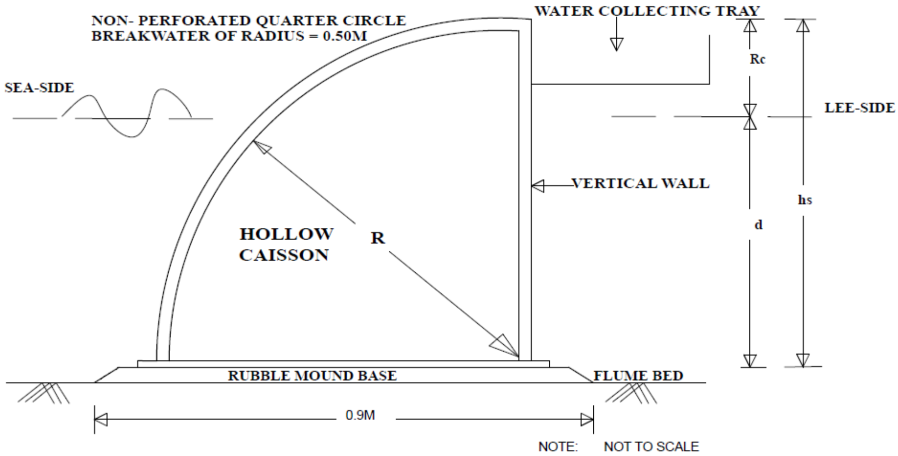

Fig. 3a: Typical cross-section of non-perforated QBW

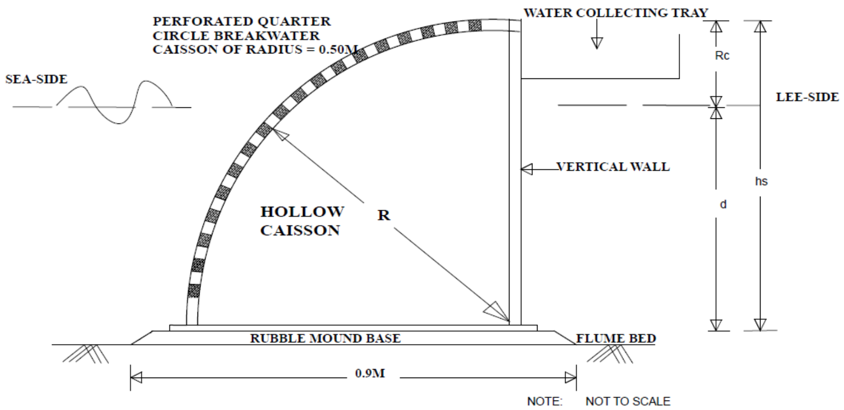

Fig. 3b: Typical cross-section of a seaside perforated QBW

The studies carried out by Binumol et al., (2017) on perforated QBW of varying sizes (0.016 m and 0.02 mm) indicated that the influence of perforation size is incognizable on the performance of the structure. Hence, under the constant size of perforations (D = 0.016 m), the percentage of perforations (p) is varied from 1.25 to 20%. The size of the perforation considered in the present study is in line with the other investigations (Binumol et al. 2017; Dhinakaran et al. 2002; Hegde and Ravikiran, 2013) on similar types of structures.

2.3 Mechanism of overtopped water collecting tank

The overtopped water volume per wave is collected in a tray attached to the breakwater models on the lee side using stiffeners. The water collecting tray length, breadth, and depth are 0.87 m, 0.73 m, and 0.13 m, respectively. Initially, trial cases are run for the maximum water depth (d = 0.50 m) and the dimensions of the water collecting tray are arrived at based on the maximum water collected by including free board. The maximum water collected in the collection tank is 0.0825 cubic meters. The waves are generated in a short burst and the total volume of overtopped water is measured after each burst. Hence, the overtopping discharge per wave is calculated as the ratio of the total volume of overtopped water collected to the number of waves overtopped. The overtopping discharge is expressed in m^3/s per m width of the breakwater. The collected water is disposed of at the end of each wave burst and the next trial is conducted. The correctness and consistency of the outcomes are confirmed by repeating all the cases by three times, according to the studies by Zhao and Ning (2018). To prevent spillage losses, a thin sheet of rubber is placed between the structure corners and the flumes side wall. Figure 4 shows the mechanism of collecting overtopped water over the quarter circle breakwater structure.

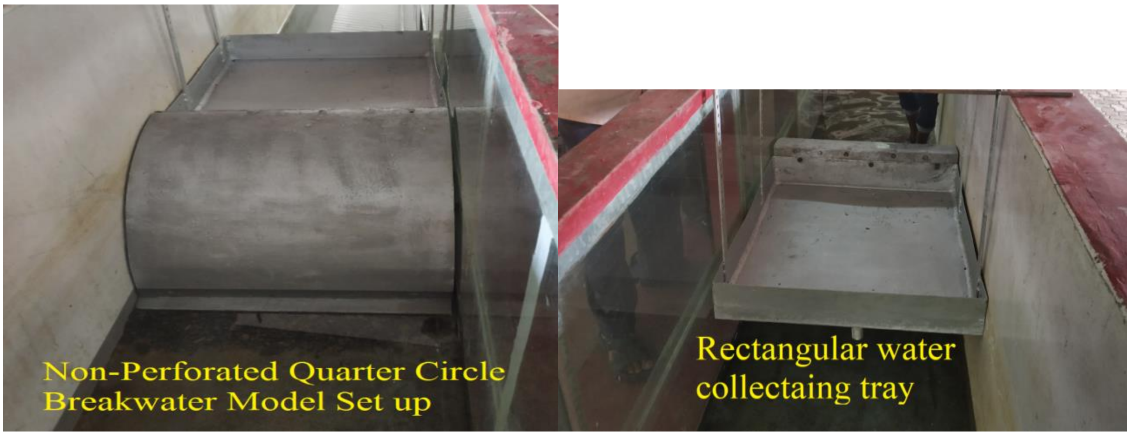

Fig. 4a: A typical front and rear view of an empty tray

Fig. 4b: A typical front and rear view of water collected in a tray

2.4 Wave characteristics

The regular waves with different wave periods and wave heights are considered for the present study is shown in Table 1. A burst of five waves is produced in order to prevent wave distortion due to wave reflection and a small amount of re-reflection from the breakwater assembly and the wave paddle. For each consecutive test run, a considerable amount of time-lapse is given in order to attain calmness with respect to still water level. The model is positioned in the wave flume at a distance of 30 m from the wave generator flap. In order to measure the incident wave heights, capacitance type wave probes are employed, and the calibration is carried out for each model setup. The spacing of the probes is in accordance with the methodology proposed by Isaacson (1991), is a function of wavelength (L) and is kept at the distance of L/3 for a particular water depth. The spacing of the first probe on the seaside is measured 1 m from the breakwater model. The wave probes arranged at suitable intervals will measure and record the incident wave heights. The wave parameters of the Mangalore coast of the Arabian Sea are used for the present study. A geometric model scale of 1:30 is considered based on the limitations of the testing facility. The wave periods ranging between 1.4 s and 2.2 s are considered in the current investigation. A maximum of six wave heights ranging from 0.08 m to 0.16 m was considered for each wave period. The models were tested for three different water depths, such as 0.45 m, 0.475 m, and 0.50 m.

Table 1: Summary of the wave and structural design parameters

Parameters	Unit	Range of investigation
Incident wave height (H_i)	m	0.08, 0.1, 0.12, 0.14, 0.16
Wave period, (T)	s	1.4, 1.6, 1.8, 2.0, 2.2
Water depth, d (m)	m	0.45, 0.475, and 0.50
Structure radius (R)	m	0.5
Diameter of perforation (D)	m	0.016
Percentage of perforation (p)	%	0, 1.25, 5, 10, 15, and 20

3. Results and Discussion

3.1 General

In the present study, the experimental tests are conducted for emerged non-perforated QBW and the seaside perforated QBW in order to compare their overtopping performance. The dimensional analysis is carried out for various wave and structural design components using Buckingham's π theorem in arriving at non-dimensional parameters. The parameters such as radius of the structure *(R)*, the height of the structure *(h_s)*, the diameter of perforations *(D)*, mean wave overtopping discharge *(q)*, water depth *(d)*, wavelength *(L)*, incident wave height *(H_i)*, wave period *(T)*, the mass density of water *(ρ)* and acceleration due to gravity *(g)* are considered for dimensional analysis. The non-dimensional π terms used in the discussion are mean wave overtopping discharge *(q/gH_iT)*, wave steepness parameter *(H_i/gT^2)*, relative water depth *(d/gT^2)*, relative freeboard *(R_c/H_i)*, and percentages of perforation *(p)*.

The experiments were conducted for non-perforated (0% perforation) and sea-side perforated (1.25%, 5%, 10%, 15%, and 20% perforation) QBW model of radius 0.50 m. The overtopping breakwater model is tested for different heights; wave periods with varying water depths, and freeboards. The effect of various sea state parameters and structural parameters on the wave overtopping of the emerged QBW models are discussed in detail. The results obtained are plotted for wave overtopping discharge against wave steepness, relative freeboard, and relative water depth. The graphs were plotted to aid in understanding the effect of influencing parameters on the mean overtopping for seaside perforated and non-perforated models.

3.2 Effect of wave steepness on wave overtopping characteristics

The mean wave overtopping discharge is plotted against wave steepness for non-perforated QBW (0%), and perforated QBW models (1.25%, 5%, 10%, 15%, and 20%) is shown in Figure 5. The results are plotted for each percentage of perforations and R_c/H_i = 0.625 to 1.875 with three water depths. The range of d/gT^2 values are 0.0095 to 0.0234, 0.0090 to 0.0221, and 0.0084 to 0.0208 corresponding to the water depths of 0.45 m, 0.475 m, and 0.50 m respectively. Figure 5 shows that q/gH_iT increases with an increase in H_i/gT^2 for all the water depths, i.e., 0.45 m, 0.475 m, and 0.50 m. This is maybe due to an increase in wave height and a decrease in relative freeboard for a particular water depth that allows the waves to pass over the model resulting in an increasing mean overtopping rate. The overtopping discharge is found to be more pronounced at higher wave steepness, in other ways, for lower wave periods and higher wave heights. The increase in wave period (wavelength) for higher wave heights with a particular relative freeboard and water depth admits the incoming waves on the lee side of the models. Also, when the wave height increases, there is an increase in wave energy, and hence overtopping.

It is also observed that q/gH_iT increases with an increase in the water depth and decrease of the relative freeboard. This may be because, at higher water depths, the curvature effect is more pronounced, resulting in higher overtopping rates. Figure 5 shows that the wave overtopping discharge increases with H_i/gT^2 for all the considered depth parameters and perforations cases. In case of non-perforated model, the maximum and minimum values of q/gH_iT observed are 3.56×10^{-3} at $H_i/gT^2 = 8.32\times10^{-3}$ for $d/gT^2 = 0.026$ and 3.21×10^{-4} at $H_i/gT^2 = 1.68\times10^{-3}$ for $d/gT^2 = 0.009$ respectively. The maximum and minimum values of overtopping discharge rates for QBW with various percentages of perforations considered are shown in Table 2.

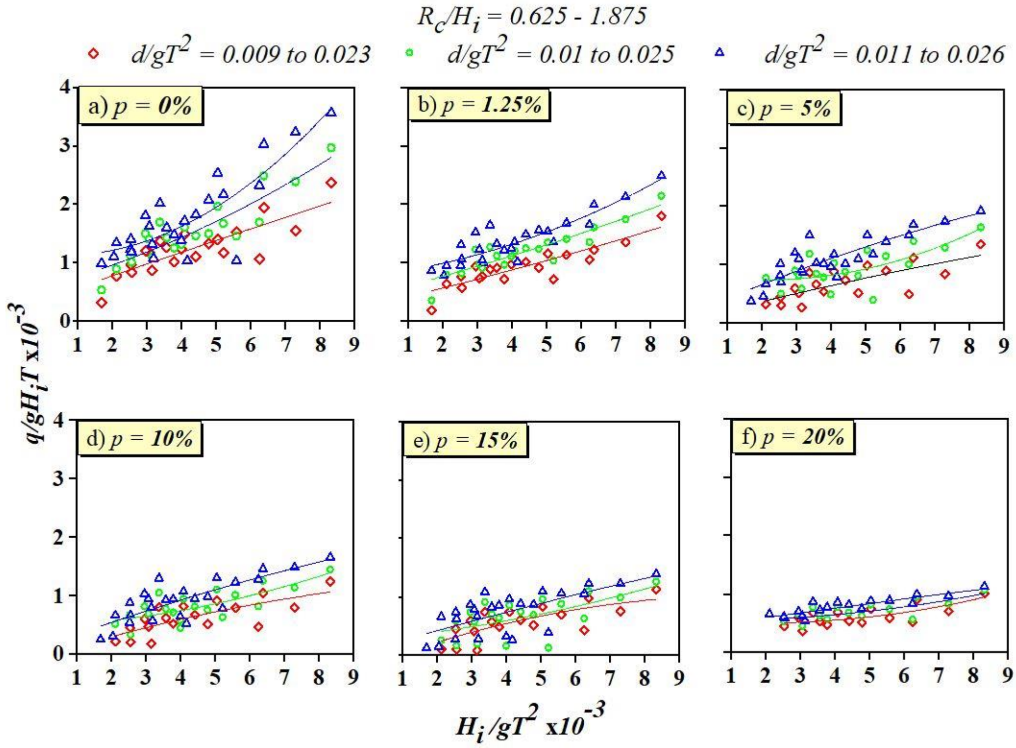

Fig. 5: Variation of q/gH_iT with H_i/gT^2 for different perforations

Table 2: Summarised results

Percentage of perforation	q/gH_iT		H_i/gT^2	R_c/H_i	d/gT^2
1.25	Min.	1.91E-04	1.68E-03	1.875	0.009
	Max.	2.49E-03	8.32E-03	0.625	0.026
5	Min.	2.74E-04	3.15E-03	1.500	0.014
	Max.	1.92E-03	8.32E-03	0.625	0.026
10	Min.	1.82E-04	3.15E-03	1.500	0.014
	Max.	1.66E-03	8.32E-03	0.625	0.026
15	Min.	9.12E-05	3.15E-03	1.500	0.014
	Max.	1.40E-03	8.32E-03	0.625	0.026
20	Min.	3.70E-04	3.06E-03	1.250	0.011
	Max.	1.13E-03	8.32E-03	0.625	0.026

3.3 Effect of relative freeboard on wave overtopping characteristics

Figure 6 shows the plot of q/gH_iT against the relative freeboard parameter (R_c/H_i) for non-perforated QBW (0%) and perforated QBW models (1.25%, 5%, 10%, 15%, and 20%) with a fixed radius of 0.50 m. Figure 6 shows that q/gH_iT decreases with an increase in R_c/H_i for all the water depths considered. The variations observed from the data points shown have the same trend as described by Troch et al. (2014) and Bradbury et al. (1988). As water depth increases, the relative freeboard decreases; the decrease in water depth increases the relative freeboard makes a lesser contribution to the overtopping discharge rate. Also, it becomes heavier for the waves to cling over the structure and overtop. The variation in q/gH_iT is found to be primarily dependent on the freeboard. In general, the relative freeboard (R_c/H_i) is found to be indirectly proportional to the wave overtopping discharge (q/gH_iT). It is expected that because of the higher water depth, the influence of curvature is more pronounced, which results in higher overtopping. For non-perforated QBW, the maximum value for

q/gH_iT observed is 3.56×10^{-3} at R_c/H_i = 0.625 for d/gT^2 = 0.026. Similarly, the minimum q/gH_iT observed is 3.21×10^{-4} at R_c/H_i = 2.5 for d/gT^2 = 0.009. The maximum and minimum rates of q/gH_iT at both parameters for varying d/gT^2 values are summarised in Table 2.

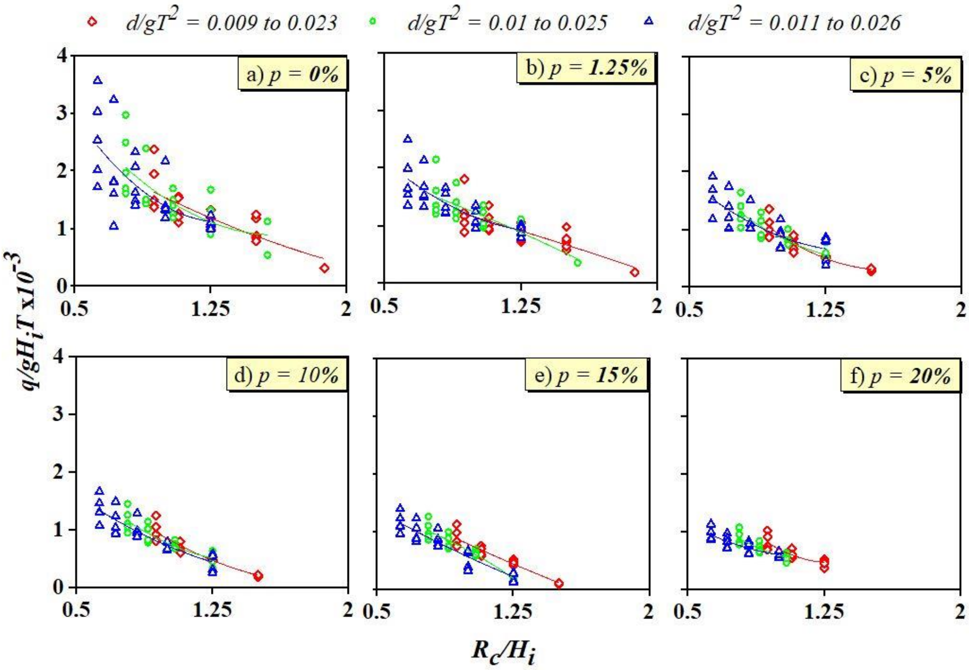

Fig. 6: Variation of q/gH_iT with R_c/H_i for different perforations

3.4 Effect of perforations and relative water depth on wave overtopping characteristics

For a particular water depth, the mean wave overtopping discharge rate is plotted against the relative water depth for all the percentages of perforation considered in order to examine the effect of perforation. The variation of q/gH_iT plotted as the function of d/gT^2 for all the percentages of perforation *(p)* is shown in Figure7. It is observed that an increase in d/gT^2 increases q/gH_iT for all the water depths considered. The increase in the percentages of perforation from 0 to 10% has a larger influence on decreasing q/gH_iT.

A further increase in the percentages of perforation has a lesser impact on q/gH_iT. The decrease in overtopping discharge (q/gH_iT) with an increase in the perforations *(p)* maybe because of the dissipation of wave energy due to the turbulence inside the chamber. As the water enters the QBW through the perforations, it flows back out of the perforations, which encounters the next incoming wave resulting in partial energy dissipation, accomplishes even before that wave reaches the breakwater. The other reason would be the waves of smaller wave periods ride over the arched surface upon which most of the incident wave energy gets reflected. Thus, lesser overtopped discharge rates are available.

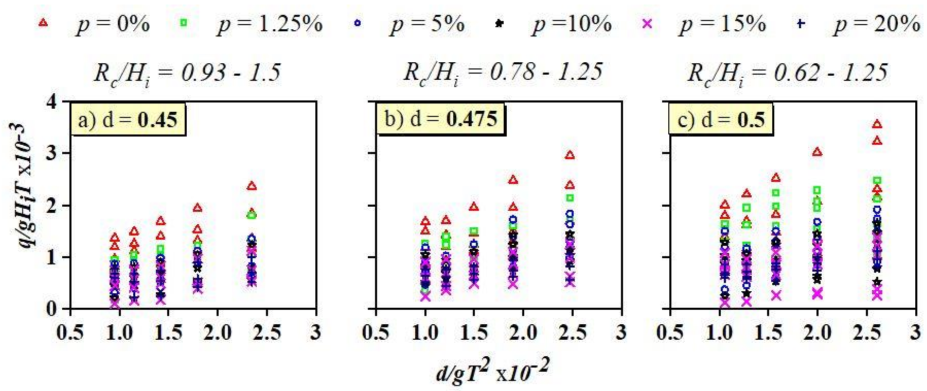

Fig. 7: Variation of q/gH_iT with $d/gT^2$for different perforations

Also, it is observed that an increase in water depth decreases R_c/H_i resulting in a larger mean overtopping rate. The values of q/gH_iT are higher for non-perforated QBW compared with perforated QBW due to less dissipation of wave energy in the non-perforated breakwater. In the case of non-perforated QBW, the range of variation of q/gH_iT is found to be varying from 1.18×10^{-4} to 3.56×10^{-3} with a range of H_i/gT^2= 1.69×10^{-3} to 8.32×10^{-3} and R_c/H_i = 1.875 to 0.625. For H_i/gT^2 = 1.69×10^{-3} to 8.32×10^{-3} and R_c/H_i = 1.875 to 0.625, the maximum and minimum discharge rates varies from 1.91×10^{-4} to 2.49×10^{-3} for 1.25% perforation, 2.74×10^{-4} to 1.92×10^{-3} for 5% perforation, 1.82×10^{-4} to 1.66×10^{-3} for 10% perforation, 9.12×10^{-5} to 1.40×10^{-3} for 15% perforation and 3.70×10^{-4} to 1.13×10^{-3} for 20% perforation.

3.5 Regression analysis

The experimental results on mean overtopping discharge rates under regular waves are subjected to multiple regression analysis based on the least square method. The empirical equations have arrived for q/gH_iT as a function of independent variables H_i, T, R_c, d, and g for the non-perforated model. For the perforated models, in addition to the above said independent variables, a new independent variable p is considered in the arriving empirical equation. These developed empirical equations can be elementary used to forecast overtopping rates. The empirical equation of q/gH_iT for non-perforated and perforated models is given in equations (3.1) and (3.2). These equations are based on regular wave test conditions only.

$$O_d=\{0.146\ (H_i/gT^2)\}+\{0.0164(R_c/H_i)\}-\{0.01(d/gT^2)\} + 0.0019 \quad (3.1)$$
$$O_d=\{0.083\ (H_i/gT^2)\}-\{0.0006(R_c/H_i)\}-\{0.001\ (d/gT^2)\} - 0.0003p + 0.0018 \quad (3.2)$$

Where O_d is a dimensionless wave overtopping discharge (q/gH_iT), the non-dimensional test ranges for the above equations are mentioned and shown in Table 3.

Table 3: Test ranges

Parameters	**Range**
Wave steepness (H_i/gT^2)	1.69×10^{-3} to 8.32×10^{-3}
Relative freeboard (R_c/H_i)	1.875 to 0.625
Relative depth (d/gT^2)	0.009 to 0.026
Percentage of perforation *(p)*	0% to 20%

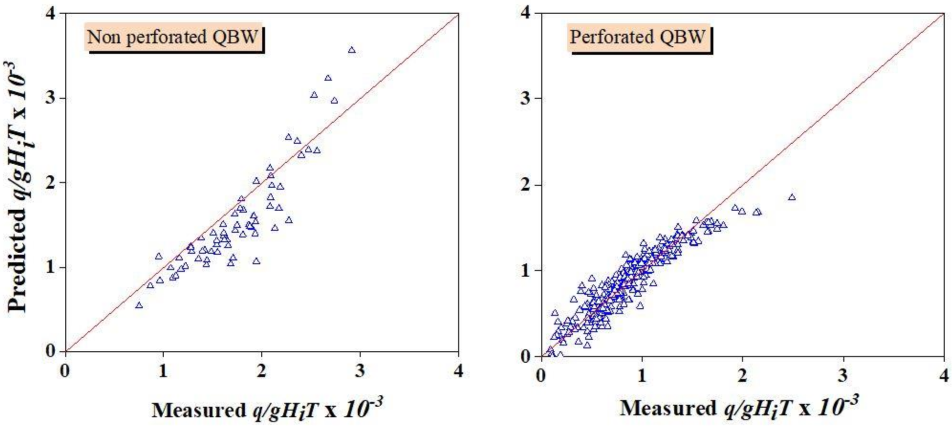

Fig. 8: Comparison of measured and Predicted q/gH_iT for non-perforated and perforated QBW

The comparison of measured and predicted q/gH_iT from the above-derived equations is shown in Figure 8a for the non-perforated and Figure 8b for the perforated models. The line of equality is superposed in the plot. It is observed that the measured and predicted q/gH_iT for both the models are reasonably good in agreement. The correlation coefficient for the predicted values of q/gH_iT is found to be 0.89 for the non-perforated model and 0.91 for the perforated model.

3.6 Uncertainty analysis

Uncertainty is an evaluation of experimental error. Generally, whenever experimentation is involved, there is a possibility of some errors creeping in a while making measurements. With the help of uncertainty analysis, it is possible to conduct experiments scientifically and predict the accuracy of the result (S.C. Misra, 2001). The width of the confidence interval is a measure of the overall quality of the regression line. The 95% confidence interval limits must always be estimated, and this concept of confidence level is fundamental to uncertainty analysis.

The methodology used is the method of confidence bands. Confidence interval may be constructed from the mean response at a specified value x, say X_o. This is a confidence interval about $E\left(\frac{y}{x_o}\right) = \hat{\mu}_{\frac{y}{x_0}}$ and is often called a confidence interval about the regression line. A 100(1-α) percent confidence interval about the mean response at the value of x = X_o, say $\hat{\mu}_{\frac{y}{x_0}}$, is given by,

$$\hat{\mu}_{\frac{y}{x_0}} \pm t_{\alpha/2,n-2}\sqrt{\hat{\sigma}^2\left[\frac{1}{n}+\frac{(x_0-\bar{x})^2}{S_{xx}}\right]} \qquad \text{.................................. (3.3)}$$

Where, $\hat{\mu}_{\frac{y}{x_0}}$ = $\beta_o + \beta_1 X_o$ computed from the fitted regression model,

α = significance level used to compute the confidence level,

$\hat{\sigma}^2$ = variance,

X = sample size,

$\bar{x}$ = sample mean.

$$S_{xx} = \sum_{i=1}^{n} x_0^{\;2} - \frac{(\sum_{i=1}^{n} x_i)^2}{n} \qquad \text{...................................... (3.4)}$$

The 95% confidence and prediction band for variation of wave overtopping discharge *(q/gH$_i$T)* with incident wave steepness *(H$_i$/gT^2)*for perforated and non-perforated quarter-circle breakwater models is shown in Figure. 9. The wave parameters tested with a range of T = 1.2 s to 2.2 s, H = 0.06 m to 0.18 m, and d = 0.45 m and 0.475 m are considered in the current investigation.

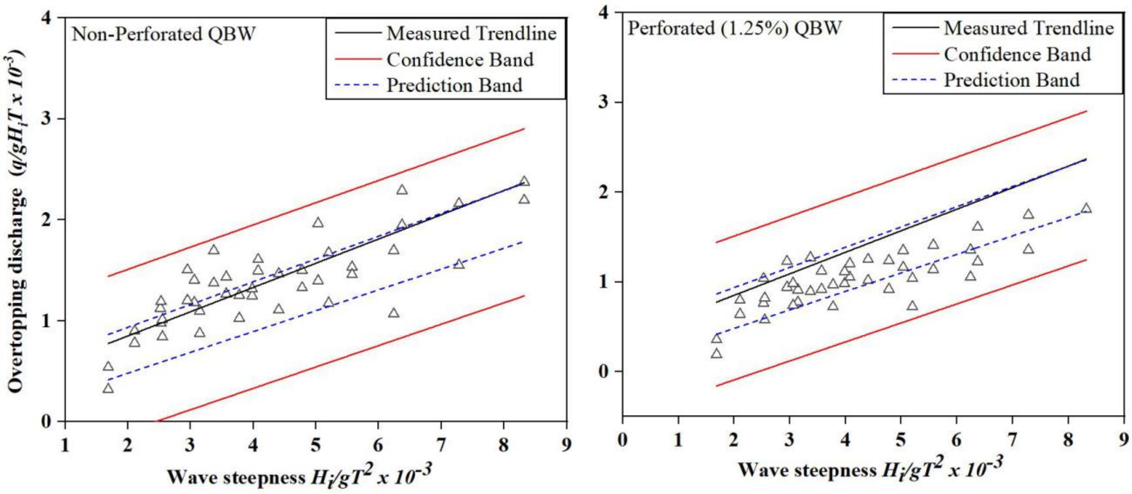

Fig. 9: A plot of 95% confidence and prediction bands for the variation of *q/gH$_i$T* for both non-perforated and perforated (1.25%) QBW

The figures show that the trend line showing q/gH_iT variation with H_i/gT^2 lies within the 95% confidence bands, and data points lie within the 95% prediction bands drawn. Also, from the figures, it is observed that more than 80% of experimental data lie within the 95% confidence bands. The regression coefficient, R^2, is found to be 0.87. Therefore, the results obtained may be analyzed with 95% confidence, i.e., the conclusions drawn from these graphs are 80% reliable. Further, from the above-said Figure 9, it may be visualized that all the experimental data points are found to be close to the 95% confidence level limits.

3.7 Comparative analysis

This section deals with the comparison of the results of current experimental work with results obtained by other researchers on similar work collected from the literature. Shankar and Jayaratne (2002) describe the wave steepness demonstrates a suitable parameter for defining the combined influence of wave height and wave period on wave overtopping discharge along with the relative crest height parameter. Their objectives are to explore the effect of wave and structural parameters on wave overtopping discharge for the sloped permeable and impermeable breakwater. Within the laboratory facilities limitations, the author chosen wave parameters ranges are wave height *(H)* = 0.05 to 0.12 m, wave period *(T)* = 0.8 to 1.2 s, depth parameter (d/gT^2) = 0.019 to 0.073, and wave parameter (H_i/gT^2) = 0.006 to 0.011. Figure 10shows the comparative analysis of overtopping discharges on an impermeable breakwater model (1:2) with the current experimental results. From Figure 10, it can be noticed that the present experimental data points are shown in good agreement with the smooth, impermeable experimental data points.

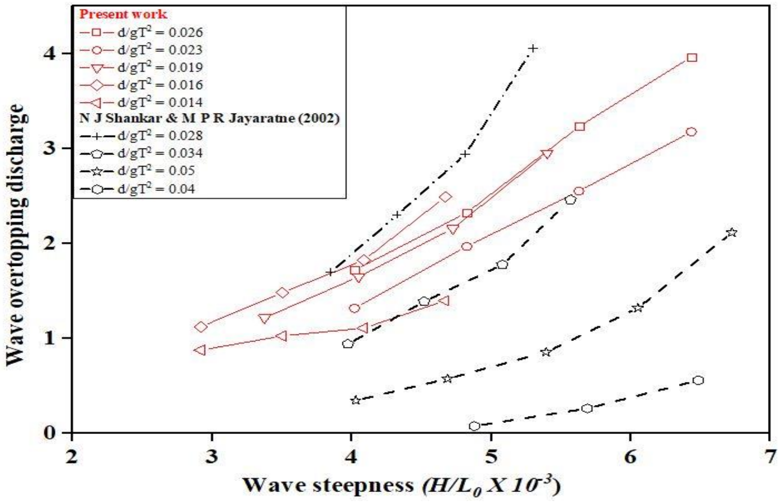

Fig.10: Comparative analysis of present work with sloped and impermeable breakwater

4. Conclusions

The study explored the investigation of mean wave overtopping discharges on the emerged seaside perforated and non-perforated quarter-circle breakwater subjected to regular waves of different wave heights and wave periods. Based on the analysis of the results of the current study, the following conclusions are drawn. The present study observed that an increase in the percentage of perforations results in a decrease in the mean wave overtopping discharge rate. The mean wave overtopping discharge increases with an increase in wave steepness and increases with the relative water depth parameter. Also, the mean overtopping discharge decreases with an increase in relative freeboard for all the water depths. The values of q/gH_iT are higher for non-perforated QBW when compared with perforated QBW due to the lesser dissipation of wave energy in the non-perforated breakwater.

The percentage of reduction in q/gH_iT for 1.25% perforated QBW is varied from 7% to 38% compared to the non-perforated model. Similarly, the decrease in q/gH_iT for 5% perforated QBW is varied from 13% to 63%. Whereas, for 10%, 15%, and 20% perforated QBW, the decrease in q/gH_iT is varied from 30% to 72%, 40% to 73%, and 43% to 74% respectively. The developed empirical equation has reproduced the experimental results with desirable accuracy. The proposed empirical equation can be extended for predicting the overtopping discharge of QBW within the test limit with an appropriate engineering judgment based on the site conditions.

Acknowledgments

The authors are thankful to the Director, National Institute of Technology Karnataka (NITK), Surathkal, and the Head, Department of Water Resources and Ocean Engineering, NITK, Surathkal, for the encouragement laboratory and testing facilities provided to carry out the investigations.

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