Reflection and Dissipation Characteristics of Non-overtopping Quarter Circle Breakwater with Low-mound Rubble Base

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Abstract

Breakwaters are the coastal structures constructed either perpendicular (shore connected) or parallel (detached) to the coast. The main function of breakwater is to create a tranquil medium on its leeside by reflecting the waves and also dissipating the wave energy arriving from seaside, resulting in ease of manoeuvrability to boats or ships to their berthing places. Different types of breakwaters are being used at present, such as rubble mound breakwater, vertical wall type breakwater and composite breakwater. The objective of this paper is to investigate reflection coefficients (Kr) and dissipation (loss) coefficients (Kl) for physical models of Quarter circle caisson breakwater of three different radii of 0.550 m, 0.575 m and 0.600 m with S/D ratio of 2.5 (S=spacing between perforations, D=diameter of perforations). The models were tested in the monochromatic wave flume of the department, for different incident wave heights (Hi), Wave periods (T) and water depths (d). It was observed that reflection coefficient increased with increase in the wave steepness (Hi/gT^2) and decreased with increase in depth parameter (d/gT^2) and hs/d (Height of structure including rubble base/depth of water). The loss coefficient decreased with increase in the wave steepness and increased with increase in depth parameter and hs/d

Keywords: Reflection coefficient, Loss coefficient, Quarter circle breakwater, Wave steepness, Depth parameter, Ratio of height of structure to depth of water

1. Introduction

The idea of placing superstructure or caisson on a rubble mound breakwater is rather old. It started in 1830 in order to save the quantity of natural rock of rubble mound breakwater. Consequently, the breakwater development has moved from the high-mound to the standard type of composite breakwater, i.e. low-mound composite breakwater. If the height of the rubble foundation is more than or equal to 50% of the height of the total structure it is high mound composite breakwater, else it is a low-mound composite breakwater (Muttray et al., 1998). Semicircular breakwater was developed firstly by Ports and Harbour Research Institute of Japan in the earlier 1980’s. It had 36 m length and was built at Miyazaki harbour in Japan. Based on the semicircular breakwater section, the quarter circle breakwater was developed by China Communications First Design Institute of Navigation Engineering (Luwen et al., 2013).

Quarter circle breakwater consists of pre-cast reinforced cement concrete caisson having perforations and a bottom slab. The whole structure rests on a low-mound rubble base. Sundar and Raghu (1997) conducted experi-
ments on an impermeable semi circular breakwater subjected to monochromatic waves. They concluded that $K_r$ varied from 0.5 to 0.9 for relative water depth $h_w/L$ (water depth/wave length), ranging between 0.2 and 0.4. Dhinakaran et al. (2002) claimed that $K_r$ varied with $h_w/L$ for semicircular breakwater of 7% and 11% perforation ratios from 0.35 to 0.65 and from 0.22 to 0.59, respectively which are both less than the range for semi circular breakwater of 0% perforations, for which $K_r$ ranges between 0.5 and 0.9. They also observed that $K_r$ slightly increases with increase in $h_w/L$, decreases with increase of $h_w/ht$ (water depth/total height of breakwater). Sundar and Rao (2003) conducted research on quadrant front-face pile-supported breakwater for different $s/D$ (spacing between the piles/pile diameter) and relative water depth $d/h$ (water depth/pile height). They concluded that for a constant $s/D$, $K_r$ increases with the depth of water increases and $K_t$ increases with an increase in scattering parameter $ka=2\pi a/L$ ($L$ is the wave length and $a$ is the radius of quadrant front face) for a lesser $d/h$ and decreases with an increase in $ka$ for the higher $d/h$. They noticed that the values of $K_r$ and $K_t$ subjected to regular waves were varied from 0.25 to 0.85 and 0.5 to 0.95 respectively.

Dhinakaran et al. (2009) revealed that $K_r$ decreases with increase in percentage of seaside perforations of semi-circular breakwater from 0 to 17% and also observed increase that of rubble mound height causes reduction in reflection, transmission coefficients and runup from $h_r/ht$ (height of rubble mound to the total height of structure) 0.18 to 0.29. They also found that optimum percentage of perforation arrived in case of fully perforated semi circular breakwater was at 11%. They recommended that total height of the model was 1.25 times the water depth and height of the rubble mound ($h_r$) to be 0.29 times the total height of the model ($h_t$). Yan et al. (2011) conducted experiments on performance of quarter circle breakwater from that they concluded that the reflection coefficients increases with the increase in $h_c/H_t$ (relative freeboard height), and loss of energy was found higher in emerged than submerged condition.

Hegde et al. (2012) studied the characteristics of semi circular breakwater for different seaside perforations. They concluded that $K_r$ decreases with increases in wave steepness and depth parameters and maximum value of was found as 0.4 for relative spacing $S/D$ of 12 ($S=c/c$ spacing of perforations, $D$=diameter of perforations) on the seaside. Hegde et al. (2013) have reported results on submerged quarter circle breakwater that reflection coefficient $K_r$ increases with increase in incident wave steepness($H_i/gT^2$), decreases with decrease in depth parameter ($d/gT^2$) and also observed that it decreases with increase in $R/Hi$ (radius to incident wave height).

2. Methodology and Experimental Set Up

Physical models were prepared to study the behaviour of the quarter circle breakwater. Due to the predominant gravity effect in the free surface wave motion, Froude’s model law was used for the physical modelling. A scale of 1:30 was used for testing of all physical models considering the Arabian Sea wave climate. Three models of different caisson radii ($R$) 0.550 m, 0.575 m and 0.600 m were prepared.

The physical model consists of a quarter circle caisson with concrete base slab. The caisson has a curved surface with 12% perforations (kept a constant) and an impermeable vertical wall. The preparation of the models was done in two steps; firstly concrete base slab was casted, secondly- moulding of caisson with perforations. The dimensions of concrete base slabs were - 0.72 m $\times$ 0.65 m $\times$ 0.05 m, 0.72 m $\times$ 0.675 m $\times$ 0.05 m and 0.72 m $\times$ 0.70 m $\times$ 0.05 m for 0.550 m, 0.575 m and 0.600 m radii breakwaters respectively. Caissons were moulded by Galvanized Iron (G.I) sheets of 0.002 m thick and perforations were made by a drilling bit of diameter 16 mm. Finally, the caissons were attached to the concrete base slabs using stiffeners of angle section as shown in Fig. 1. The model was placed on a low rubble mound base of thickness 0.05 m consisting of granite stones of weight ranging from 50–100 gm.

The total height of the structure including the rubble base ($h_s$ as shown in Fig. 2) is the sum of the radius (in m) of caisson, thickness of the concrete base slab (0.05 m thickness) and rubble base foundation (0.05 m thick) and hence is given by Eq. (1).

$$h_s = R + 0.05 + 0.05$$ (1)
Table 1. Different parameters used and their range

<table>
<thead>
<tr>
<th>Wave specific parameters</th>
<th>Experimental range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incident wave height, $H_i$ (m)</td>
<td>0.06, 0.08, 0.10, 0.12, 0.14, 0.16, 0.18</td>
</tr>
<tr>
<td>Wave period, $T$ (s)</td>
<td>1.4, 1.6, 1.8, 2.0, 2.2, 2.5</td>
</tr>
<tr>
<td>Depth of water, $d$ (m)</td>
<td>0.35, 0.4, 0.45</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Structure specific parameters</th>
<th>Experimental range of values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radius of Quarter circle breakwater caisson (m)</td>
<td>0.550, 0.575 and 0.600</td>
</tr>
<tr>
<td>S/D ratio (percentage of perforations)</td>
<td>2.5 (12%)</td>
</tr>
<tr>
<td>Spacing between perforations (m)</td>
<td>0.04</td>
</tr>
<tr>
<td>Diameter of perforation (m)</td>
<td>0.016</td>
</tr>
</tbody>
</table>

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$$h_s = R + 0.05 + 0.05 \quad (1)$$

The models were tested in a two dimensional monochromatic wave flume for different wave and structure specific parameters (Table 1) of the Department of Applied Mechanics and Hydraulics, at National Institute of Technology Karnataka, Surathkal, India. The monochromatic wave flume used had dimensions of 50 m × 0.73 m × 1.1 m as shown in Fig. 3. Waves of different heights ($H_i$) ranging from 0.02 m to 0.24 m and wave periods (T) of 0.8 to 4.0 s can be generated by bottom hinged flap type in different water depths (d). The flap is connected by means of bar-chain link connected to the flywheel which is mounted on the shaft of the induction motor of 11 kW power at 1450 rpm. This motor is regulated by an inverter drive of frequency ranging from 0–50 Hz, with a speed ranging from 0–155 rpm. The desired wave heights and wave periods can be obtained by changing the eccentricity of bar-chain link and by changing the frequency of inverter respectively.

![Fig. 1. Isometric view of quarter circle breakwater](image1)

![Fig. 2. Cross section of quarter circle breakwater](image2)
Incident wave heights ($H_i$) and reflected wave heights ($H_r$) are measured by the capacitance type wave probes. The probes are equally spaced at one third of the wavelength ($L$) as given by Issacson, 1991. To minimize the successive reflections, bursts of five waves each, are generated in the flume. The digital voltage signals are converted into wave heights and wave periods using the laboratory wave recorder software provided by EMCON (Environmental Measurements and Controls), Kochi, India.

3. Results and Discussion

Dimensional analysis was carried out to find the non-dimensional π terms involved in the process using Buckingham’s π theorem. The non-dimensional π terms obtained from the analysis are $H_r/H_i$ (ratio of reflected wave height to incident wave height=$K_r$), $H_i/gT^2$ (incident wave steepness), $d/gT^2$ (depth parameter) and $h_s/d$ (Height of structure including rubble base/depth of water). Loss coefficient ($K_l$) was found by Eq. (2), as the breakwater is non-overtopping and hence there is no transmission and also due to the fact that the vertical wall of the caisson is impervious.

$$K_1 = \sqrt{1 - K_r^2} \quad (2)$$

3.1 Reflection Coefficient

Graphs were plotted for the variation of $K_r$ with different non-dimensional π terms. It may be observed from Fig. 4, that reflection coefficient ($K_r$) increases with increase in the wave steepness ($H_i/gT^2$). It may be understood that as wave height increases, the wave tends to plunge to narrow area of curved surface, hence result in increase in reflection. Also from the same figure, one can observe that as wave period decreases (increase in values of $H_i/gT^2$) the value of reflection coefficient increases. Further, it is observed that as $h_s/d$ increases, reflection coefficient decreases. This is because as the structure height increases, total perforations occupied by the wave increases and hence reflection decreases. This trend is found to be true for all values of $d/gT^2$ values from the Fig. 1. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00570$ Fig. 2, Fig. 17.

![Fig. 1. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00570$](image1)

![Fig. 2. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00652$](image2)
Fig. 3. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00733$

Fig. 4. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00737$

Fig. 5. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00842$

Fig. 6. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00891$

Fig. 7. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00947$

Fig. 8. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0101$

Fig. 9. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0110$

Fig. 13. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0114$
Fig. 10. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0125$

Fig. 11. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0139$

Fig. 12. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0141$

Fig. 13. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0159$

Fig. 14. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0179$

Fig. 15. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0182$

Fig. 16. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0205$

Fig. 17. Variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.0234$
The variation of reflection coefficient ($K_r$) with wave steepness ($H_i/gT^2$) for different depth parameters ($d/gT^2$) has been studied. Fig. 19 and Fig. 20 show the variation of $K_r$ with $H_i/gT^2$ for $d/gT^2=0.00570-0.0182$, $0.00652-0.0205$, and $0.00733-0.0234$ respectively. It may be observed that $K_r$ increases with the increase in wave steepness. Further, it is observed that $K_r$ decreases with the increase in the depth parameters. The trend was found to be logarithmic with, $R^2=0.85$, $0.82$, $0.84$, $0.80$, $0.81$, $0.67$, and $0.77$, $0.63$, $0.58$ from Fig. 19 and Fig. 20 respectively.

### 3.2 Dissipation Coefficient

The dissipation ($K_l$) decreases with the increase of wave steepness ($H_i/gT^2$) as seen from Fig. 21. It is due to high reflection of waves and with increase in the wave steepness, the dissipation was found to decrease. Also from the same figure, it may be observed that as wave period decreases (increase in values of $H_i/gT^2$), the value of loss coefficient decreases. Further, it is observed that as $h_s/d$ increases, dissipation increases which is a reverse trend in the case of reflection. This trend is found to be true for all values of $d/gT^2$ values from Fig. 22-Fig. 38.
Fig. 24. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.00737$

Fig. 25. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.00842$

Fig. 26. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.00891$

Fig. 27. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.00947$

Fig. 28. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.0101$

Fig. 29. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.011$

Fig. 30. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.0114$

Fig. 31. Variation of $K_i$ with $H_i/gT^2$ for $d/gT^2=0.0125$
Fig. 32. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0139$

Fig. 33. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0141$

Fig. 34. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0159$

Fig. 35. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0179$

Fig. 36. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0182$

Fig. 37. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0205$

Fig. 38. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0234$

Fig. 39. Variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.00570-0.0182$
The variation of loss coefficient ($K_l$) with wave steepness ($H_i/gT^2$) for different depth parameters has been drawn. Fig. 39 show the variation of $K_l$ with $H_i/gT^2$ for $d/gT^2=0.0057-0.0182$, $0.00652-0.0205$, and $0.00733-0.0234$ respectively. It may be observed that $K_l$ decreases with the increase in the wave steepness. Further, it is observed that $K_l$ increases with the increase in the depth parameter. The trend was found to be logarithmic with $R^2 = 0.71, 0.71, 0.62, R^2 = 0.53, 0.74, 0.84$, and $R^2 = 0.58, 0.89, 0.75$ from Fig. 39 respectively.

4. Conclusions

The following conclusions have been drawn from the discussion on results obtained from experimental work carried out on the non-overtopping Quarter circle breakwater models of different radii of 0.550 m, 0.575 m and 0.600 m and having 12% perforations.

- With increase in the wave steepness ($H_i/gT^2$), there is increase in reflection coefficient ($K_r$) and decrease in loss coefficient ($K_l$).
- With increase in $h_s/d$ (Height of structure including rubble base/depth of water), the wave reflection decreases and dissipation increases.
- With the increase in depth parameter ($d/gT^2$), reflection coefficient ($K_r$) decreases and loss coefficient ($K_l$) increases.

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References


