Effect of Runup over a Vertical Pile Supported Caisson Breakwater and Quarter Circle Pile Supported Caisson Breakwater

T. J. Jemi Jeya, V. Sriram

***Abstract*—**Pile Supported Caisson breakwater is an ecofriendly breakwater very useful in coastal zone protection. The model is developed by considering the advantages of both caisson breakwater and pile supported breakwater, where the top portion is a vertical or quarter circle caisson and the bottom portion consists of a pile supported breakwater defined as Vertical Pile Supported Breakwater (VPSCB) and Quarter-circle Pile Supported Breakwater (QPSCB). The study mainly focuses on comparison of run up over VPSCB and QPSCB under oblique waves. The experiments are carried out in a shallow wave basin under different water depths (d = 0.5 m & 0.55 m) and under different oblique regular waves (0^0, 15^0, 30^0). The run up over the surface is measured by placing two run up probes over the surface at 0.3 m on both sides from the centre of the model. The results show that the non-dimensional shoreward run up shows slight decrease with respect to increase in angle of wave attack.

***Keywords*—**Caisson breakwater, pile supported breakwater, quarter circle breakwater, vertical breakwater.

I. Introduction

THE catastrophic effect of ocean waves is one of the most challenging tasks for coastal zone development. Hence for safe maneuvering of ships and to protect the ships from waves for safe loading and unloading, breakwaters are constructed as sea defense structure, which forms the integral part of a harbor structures. Breakwaters are marine structures that break the incoming waves approaching towards it, and thereby protecting the lee side of harbor from the adverse effects of waves.

Caisson breakwater is a type of gravity breakwater structures effectively used in the protection of lee-side of a harbor in rough sea areas. These structures rest on seabed and pierce in to free surface, thereby breaking the wave incident on it.

A pile supported breakwater is constructed by placing piles placed in series of rows. Such breakwaters are constructed in the marine environment experiencing littoral drift dominant in a particular direction, pile breakwaters allow the free passage of sediments to some extent, thus reducing the shoreline erosion on its down drift side compared with what would occur with conventional rubble mound breakwaters.

The concept of circular breakwater was developed in early 90s. The first circular breakwater developed was in the form of a Semi Circular Breakwater (SCB). The model tests on the solid type of SCB subjected to regular waves concluded that the reflection coefficient K_r varied from 0.5 to 0.9 for the d/L (water depth/wave length) ranging from 0.2 to 0.4 [1]. The above success resulted in the use of the arched structures for coastal protection; this has received a great deal of interest among the engineers and researchers. Quarter circle breakwater (QCB) is an improved type of SCB. The first initiative to found the hydrodynamic performance of QCB was carried out by [2] and the results revealed that the quadrant front face pile supported breakwater experience reduced force, pressure reflection and transmission coefficient. Reference [3] conducted a series of regular and irregular wave experiments and studied the reflective and transmitting performances of SCB & QCB and concluded that the hydraulic performance of both QCB & SCB are the same. Reference [4] conducted extensive laboratory investigations and measured the distribution of forces and developed partial coefficients for the design of QCB.

Vertical front face breakwaters provided good protection on the lee side of the breakwater, they experienced increased wave reflection and high wave pressure and force in front of the breakwater. The increased reflection, force and pressure decrease the stability of the structure by causing the structure to slide (or) overturn (or) scour near the toe. These vertical front caisson breakwaters experience high horizontal wave forces and strong acting on the wall. Hence in order to reduce the effect of reflection and gradually dissipate the wave energy by shoaling, different configurations were used in various literature [1]-[3]. The prominent rough sea conditions lead to the failure of many traditional rubble mound breakwater. This resulted in researchers to focus their study on caisson breakwater. Height of caisson breakwater plays an important role in the design. As the height of caisson breakwater increases it becomes uneconomical in construction. Overtopping of waves is high during rough sea conditions. This in turn affects the stability of the breakwater. Vertical caisson breakwater will have the tendency to reflect the waves acting over it. This causes the reflected wave height to increase in front of the caisson breakwaters making it overlap with the incident waves and thereby resulting in higher wave height causing high wave pressure. This leads to severe splashing of wave over the breakwater. Although, the probability of occurrence of extreme waves is very low, the probability of failure caused due to one extreme wave is high. Therefore, a curved front caisson breakwater has to be established which has low reflection and must stable at rough

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sea conditions.

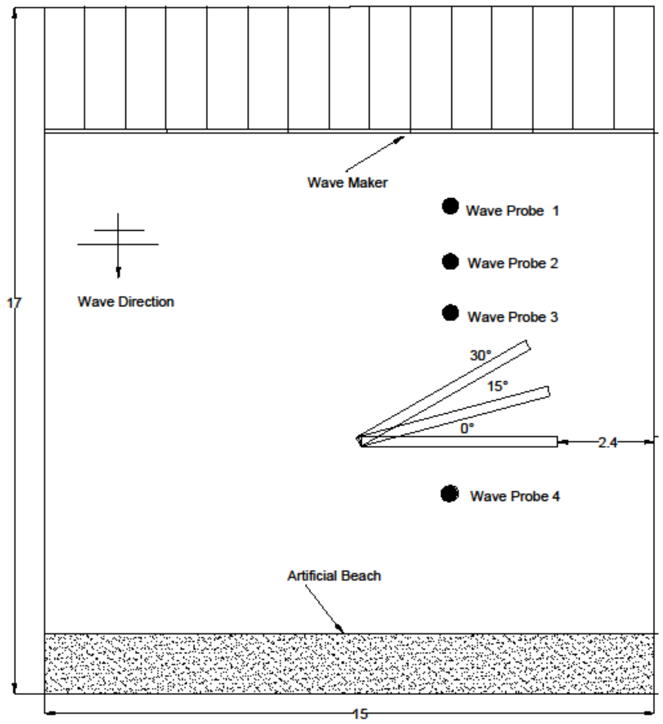

Fig. 1 Plan View of Shallow Wave Basin, position of model and wave gauges

Runup over a structure is very important to design the height and slope of the structure. As the runup over the structure increases it increases the height of caisson breakwater causing increase in wave reflection. This high wave reflection causes the increase in wave force near the structure. The increased wave force will cause scour near the bottom of the structure, which will affect the stability of the structure. Hence in order to reduce the effect of reflection, perforations are introduced. Increasing the numbers of perforated walls can reduce further reflection. This perforated caisson breakwaters need continuous monitoring and maintenance. If not maintained properly the perforations can be blocked by aquatic life. If this aquatic life is not removed from the perforation it reduces the wave dissipation capacity of caisson breakwater. So it is economical to allow the less common extreme waves to pass over the caisson breakwater and not considering them in the design of height of caisson breakwater. Hence storm surges and tides are only considered in determining the height of caisson breakwaters.

The efficiency of the breakwater as a function of the transmission, reflection and the wave energy dissipation coefficients was presented by [5]. High reflection coefficient and transmission coefficient occurs under long period waves [6]. In the Pacific, on the western coast of US, the breakwater was designed for the reduction of reflected waves on seaside and thereby transmitting waves on harbor side. The seaside consists of single row of vertical panels and the harbor side consists of double row-staggered panels of different porosity. This staggered panel design causes resistance to flow of waves, which reduces wave transmission. The efficiency of breakwater directly depends on the spacing between the breakwater walls and wave frequency. An analytical model was developed by [7] that predicts the reflection of irregular waves normally incident upon a perforated- wall caisson breakwater. The model over-predicts the reflection coefficients at larger values, and under-predicts at smaller values resulting in overestimation (or) underestimation of energy loss coefficients. The efficiency of the breakwater depends on the relationships between transmission, reflection and energy dissipation coefficients (K_t, K_r, K_L) and dimensionless parameters representing the wave and structure characteristics [8]. Reference [1] reported the transmission and reflection characteristics of SCB, whereas, [1]-[3], [9] investigated the hydrodynamic characteristics of QCB supported on piles. Similarly, [10] arrived at the transmission characteristics of partially perforated-wall caisson breakwater. Also, [12] studied the hydrodynamic characteristics of vertical pile supported caisson breakwater was analytically developed and validated experimentally.

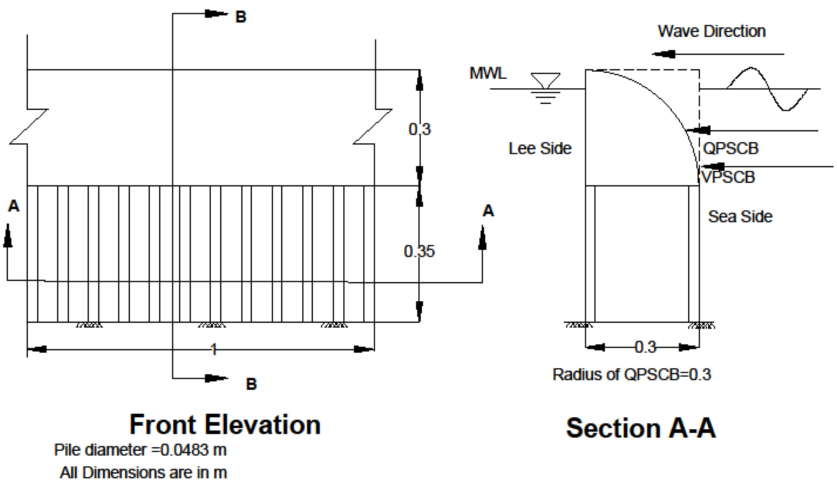

Fig. 2 Details of the pile supported breakwater models

The works done on runup over pile supported caisson breakwaters under oblique waves are very scanty. Hence this paper presents a detailed study on the runup over Pile supported caisson breakwater under different angles of wave attack.

II. Experimental Investigation

A. Experimental Facility

The present study is carried out in a shallow basin of length 19 m, width 15 m and depth 1 m, in the Department of Ocean Engineering, Indian Institute of Technology Madras, India. The wave basin consists of a wave generator containing five paddles of piston type operating at one end. A parabolic perforated Fiber-Reinforced Polymer wave absorber is located at the other end. The plan view of shallow wave basin, position of model and wave gauges are shown in Fig. 1.

B. Experimental Setup

The model scale adopted for the present study is 1:40. The model is fabricated in two parts for the ease of handling and installation. The top part is a quarter circle with a radius of 0.3 m or a vertical box of size 0.3 m and the bottom portion comprises of piles made up of 0.0269 m diameter MS pipes. The total height of the model is 0.65 m, where the height of the piles is 0.35 m and the height of quarter circle caisson is 0.3 m. The top part is made up of MS sheet and the piles are arranged in staggered manner with a longitudinal clear pile spacing of 134.5 mm with S/D = 5. The model is located at a distance of 2.4 m away from the side walls of the wave basin and 10 m in front of the wave generator. The details of the quarter circle front face pile supported breakwater model are shown in Fig. 2.

C. Instrumentation

Three wave probes of conductivity type and length 0.7 m are placed in front of the model to determine the time series of incident and reflected waves. The wave probes are placed according to the procedure explained in [11]. Two runup probes of conductive type are placed over the model at 0.3 m from both sides of the centre of the model.

D. Test Cases

The tests were conducted on both VPSCB and QPSCB with two relative water depths, d/h = 1.43, 1.57. The models are subjected to regular waves of wave period, T varying from 1-2 s at 0.2 s intervals and wave height, H varying from 0.04-0.09 m. For each wave heights three wave periods are considered. The structural parameters of the VPSCB and QPSCB remain same throughout the experiment whereas, the models are placed at 0^0, 15^0, 30^0 to the wave absorber.

III. Results and discussion

The runup is obtained from the runup probes placed over the structure. The non-dimensional runup is obtained by dividing the measured shoreward runup wave height by the incident wave height. The measured composite wave elevation of the three wave gauges from the wave maker in front of the model are separated as incident and reflected waves by [11] which can be used in evaluating reflection coefficient from the time series of two or more wave probes located at any distance in front of the model. The tests are conducted for d/L varying from 0.12 to 0.26. The runup over the structure increases with increase in d/L and indicating short period waves have more runup on both VPSCB and QPSCB.

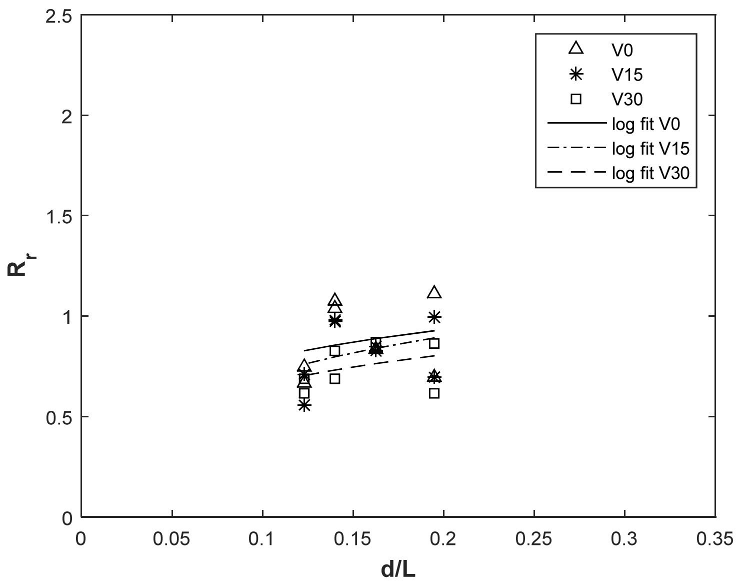

Fig. 3 Variation of non-dimensional Shoreward runup on the right side with d/L at d/h = 1.43 for VPSCB

The non-dimensional shoreward runup on the right side (R_r) of the breakwater model for VPSCB, at d/h = 1.43, increases from 0.67 to 1.1 as shown in Fig. 3, whereas, for QPSCB it increases from 1-1.5 with respect to d/L as shown in Fig. 4, when the wave trains are parallel to the breakwater. Under 15^0 wave attack R_r varies from 0.7-0.99 for VPSCB and from

0.83-1.52 for QPSCB. Under 30^0 wave attack R_r varies from 0.61-0.87 for VPSCB and from 0.89-1.43 for QPSCB. Fig. 5 shows the comparison of non-dimensional shoreward runup on the right hand side of the model for both VPSCB and QPSCB at d/h = 1.43.

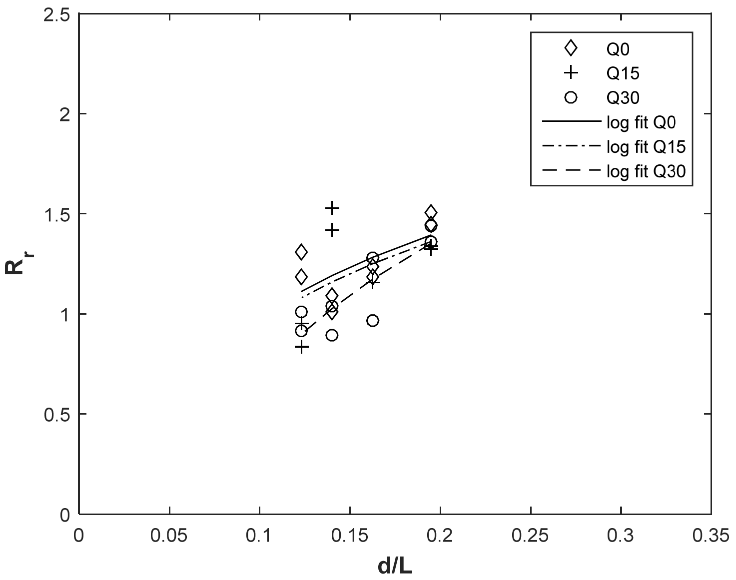

Fig. 4 Variation of non-dimensional Shoreward runup on the right side with d/L at d/h = 1.43 for QPSCB

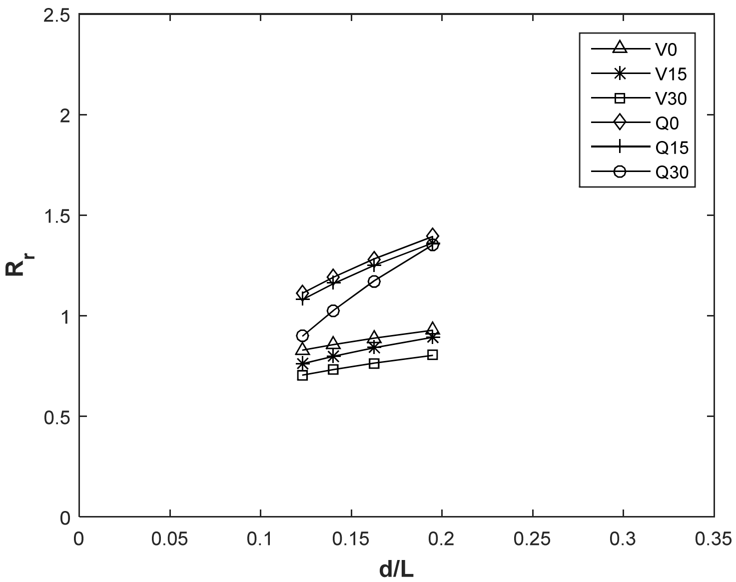

Fig. 5 Comparison of non-dimensional Shoreward runup on the right side with d/L at d/h = 1.43 for both VPSCB and QPSCB

The non-dimensional shoreward runup on the right side (R_r) of the breakwater model for VPSCB, at d/h = 1.57, increases from 0.71 to 1.23 as shown in Fig. 6, whereas, for QPSCB it increases from 1.3-3 with respect to d/L as shown in Fig. 7, when the wave trains are parallel to the breakwater. Under 15^0 wave attack R_r varies from 0.62-1.19 for VPSCB and from 1.04-2.54 for QPSCB. Under 30^0 wave attack R_r varies from 0.57-1.09 for VPSCB and from 1.09-2.62 for QPSCB. Fig. 8 shows the comparison of non-dimensional shoreward runup on the right hand side of the model for both VPSCB and QPSCB at d/h = 1.57.

The non-dimensional shoreward runup on the left side (R_L) of the breakwater model for VPSCB, at d/h = 1.43, increases from 0.71 to 1.23 as shown in Fig. 9, whereas, for QPSCB it increases from 0.98-1.47 with respect to d/L as shown in Fig. 10, when the wave trains are parallel to the breakwater. Under 15^0 wave attack R_L varies from 0.65-1.17 for VPSCB and from 0.75-1.05 for QPSCB. Under 30^0 wave attack R_L varies from 0.63-1.05for VPSCB and from 0.67-1.08 for QPSCB. Fig. 11 shows the comparison of non-dimensional shoreward runup on the left side of the model for both VPSCB and QPSCB at d/h = 1.43.

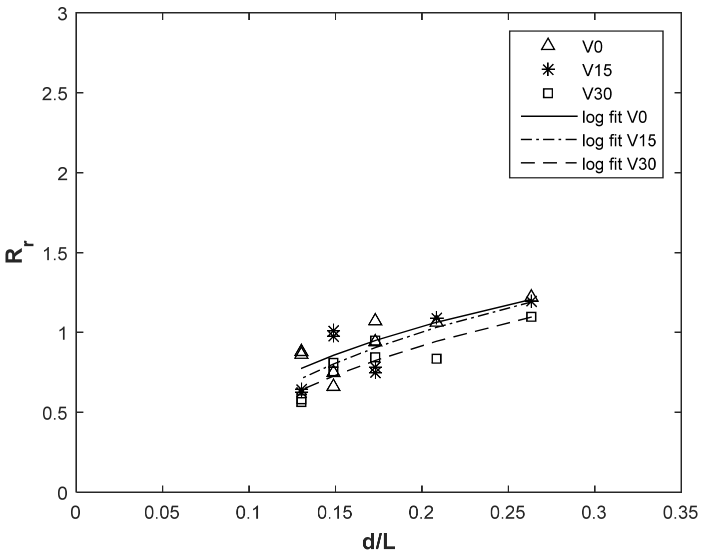

Fig. 6 Variation of non-dimensional runup on the right side with d/L at d/h = 1.57 for VPSCB

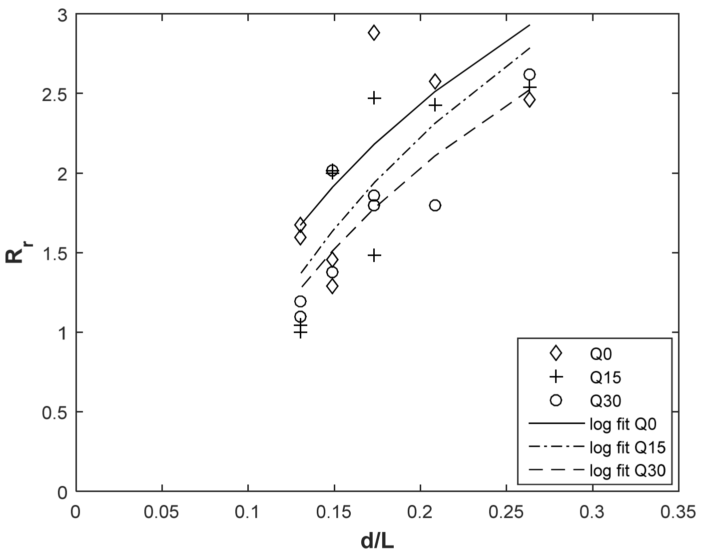

Fig. 7 Variation of non-dimensional runup on the right side with d/L at d/h = 1.57 for QPSCB

The non-dimensional shoreward runup on the left side (R_L) of the breakwater model for VPSCB, at d/h = 1.57, increases from 0.75 to 1.2 as shown in Fig. 12, whereas, for QPSCB it increases from 1.47-2.48 with respect to d/L as shown in Fig. 13, when the wave trains are parallel to the breakwater. Under 15^0 wave attack R_L varies from 0.72-1.14 for VPSCB and from 0.93-2.064 for QPSCB. Under 30^0 wave attack R_L varies from 0.53-1.19 for VPSCB and from 0.59-2.123 for QPSCB. Fig. 14 shows the comparison of non-dimensional shoreward runup on the right hand side of the model for both VPSCB and QPSCB at d/h = 1.57. This is because of the fact that when the model is parallel to wave train there is no interruption of wave train causing higher runup over the structure. Similarly at this stage both the right and left runup gauges will record almost similar runup.

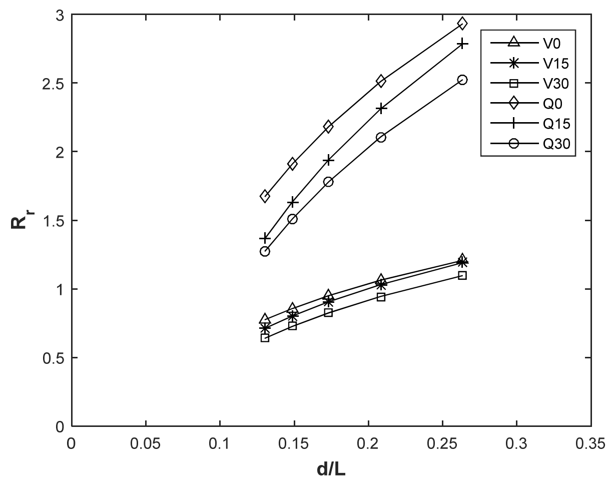

Fig. 8 Comparison of non-dimensional runup on the right side with d/L at d/h = 1.57 for both VPSCB and QPSCB

When the wave attack is normal to the structure, the variation of runup with respect to d/h increases at about 11% and 9% and about 42% and 28% in QPSCB the runup gauge located in the right and left side of the model. Similarly at 15^0 it increases at about 10% and 17% in VPSCB and 33% and 63% in QPSCB. Also at 30^0 it increases at about 38% and 68% in QPSCB, but the trend gets reversed in QPSCB, the runup slightly decreases at about 0.31% and 7% in VPSCB. This is mainly due to reflection and diffraction occurring at the edges of the breakwater. It is clear from Figs. 3-14 that the variation of non-dimensional runup with respect to d/h is very high in QPSCB, when compared to VPSCB due to the presence of convexity in the seaward face of the breakwater.

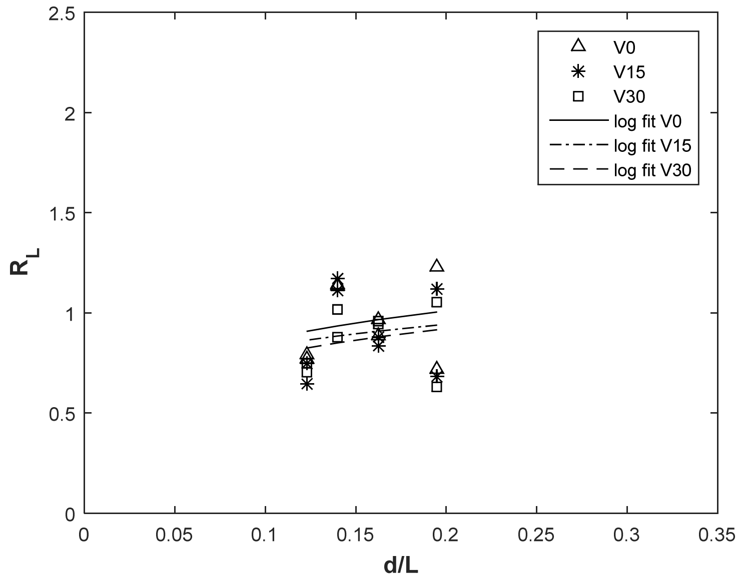

Fig. 9 Variation of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.43 for VPSCB

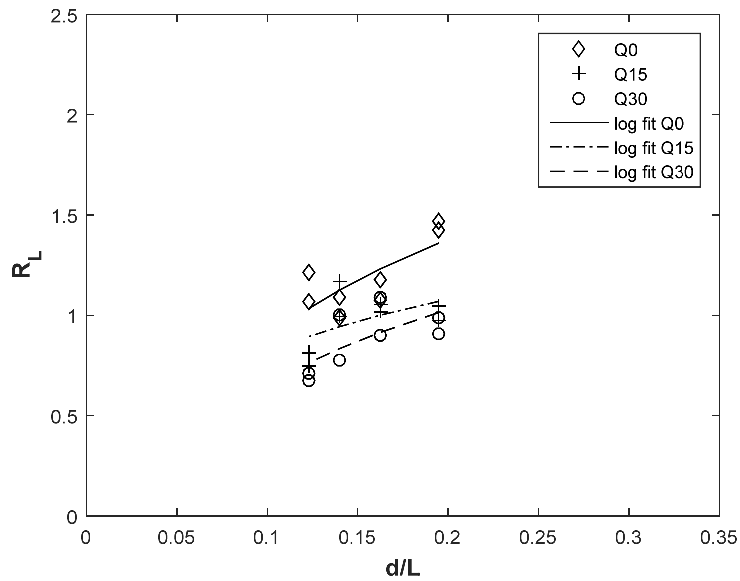

Fig. 10 Variation of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.43 for QPSCB

QPSCB is very effective in increasing the runup of the structure by reducing the wave reflection on the front face of the breakwater. The runup gauges located at the right side of the model experiences slightly higher runup because of the fact that, at oblique wave attack, the wave trains first impinge on the right side of the model. Hence, if it reaches the left side of the model, the wave energy gets dissipated and runs less over the structure. Similarly, the runup is high, when the freeboard is less, i.e. at higher water depth. Also at d/h = 1.57 the wave runup in QPSCB is more than 65% of VPSCB at all angle of wave attack in both runup gauges whereas, at d/h = 1.43 the minimum increase in wave runup between VPSCB and QPSCB is even 2.2% on both the two runup gauges.

IV. Summary and Conclusion

The following are the conclusions made in the detailed experimental study on comparison of the effect of runup on VPSCB and QPSCB under oblique regular wave attack:

- The curvature in QPSCB increases the runup length about 5 times greater than of that of the VPSCB.
- The non-dimensional shoreward runup is found to be high when the model is oriented at 0^0 and reducing up to 30^0 oblique wave attack for both VPSCB and QPSCB.
- The non-dimensional shoreward wave runup is found to be high for QPSCB when compared to VPSCB due to the presence front curvature.

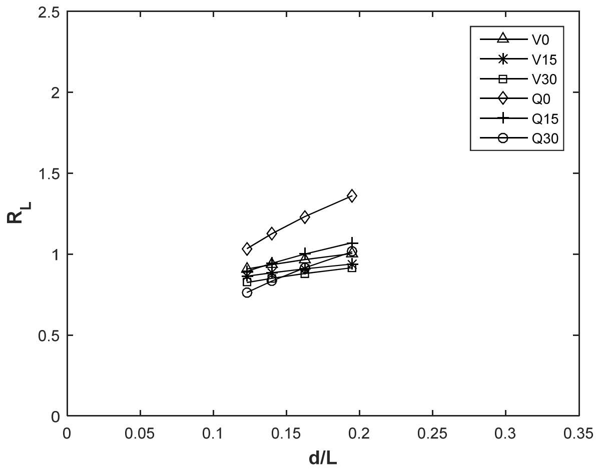

Fig. 11 Comparison of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.43 for VPSCB and QPSCB

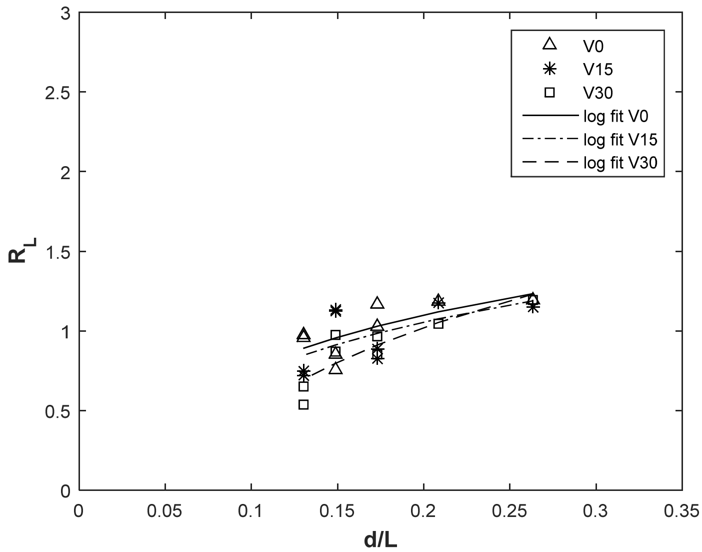

Fig. 12 Variation of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.57 for VPSCB

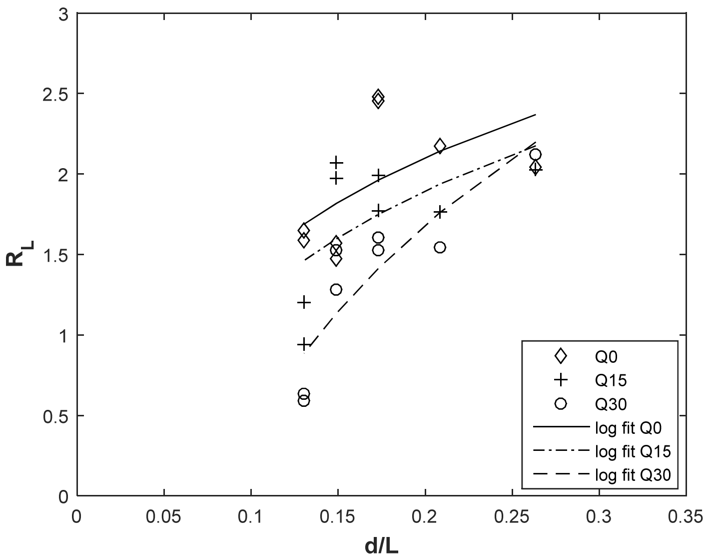

Fig. 13 Variation of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.57 for QPSCB

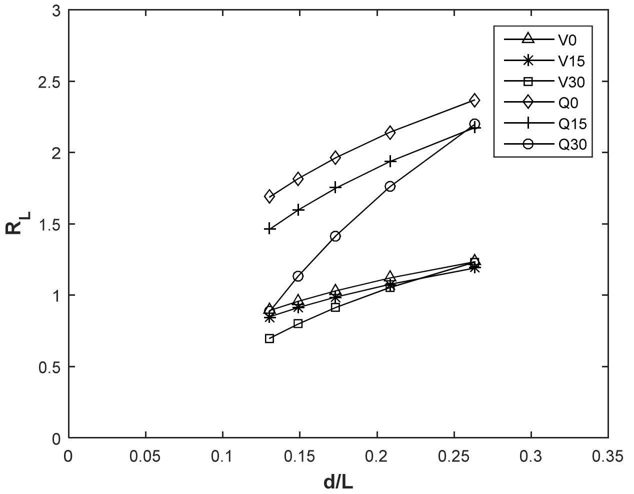

Fig. 14 Comparison of non-dimensional Shoreward runup on the left side with d/L at d/h = 1.57 for VPSCB and QPSCB

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