Wave pressure reduction on vertical seawalls / caissons due to an offshore breakwater

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The technique to reduce the wave loads on seawalls/caissons is gaining momentum around the world. Placing an offshore structure in front of seawall/caisson is expected to reduce the loads on these structures. A series of physical model tests were carried out to examine the order of wave pressure reduction for different height of the breakwater related to the local water depth. One has to be careful, when the crest of the breakwater is immersed with about 17% of the water depth, during which, water jetting effect is found to increases the pressures. Modification factor in association with Goda`s formula is proposed to estimate the shoreward pressures on the seawall in the presence of the offshore breakwater. Statistical analysis is carried out on the measured pressure data and the wave pressures for 2% probability of exceedence for different breakwater heights w.r.t. the water depth are given in this paper.

[Key words: Dynamic pressures, seawalls, low-crested breakwater, pool length, piling-up of water]
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Introduction

Vertical coastal structures such as seawalls have been used for protecting the coastal against wave induced erosion and caissons are used as breakwaters of harbors for many years. However, a large number of these structures are still being damaged by storms and sometimes can be catastrophic\(^1,2\). Extreme wave actions cause either displacement or overturning of caissons or progressive damage of structural components or foundation failures due to liquefaction or excessive base pressures. The damages are mainly associated with the impact of breaking waves during unexpected cyclones. The breaking wave induced shock pressure intensity on concrete caissons can be as high as 10 times that of the pulsating non breaking wave pressures for almost same wave height. Blackmore \& Hewson\(^3\) recorded impact pressure of 48.6 kN/m\(^2\) with rise time of 0.3 sec. Cyclone induced waves also cause severe toe scour and cause failures of seawalls/caissons by sinking or forward tilting. Scientific community in the coastal engineering area has thought of various possibilities to reduce the action of extreme waves on caissons and seawalls in order to reduce the possibility of above mentioned failures and also to increase the service life of these structures. The following methods are proposed:-

a. Introduction of additional porous wall on the seaside of the caisson/seawall in order to reduce the effective exposed area for wave attack and to accelerate more energy dissipation by turbulence.

b. Construction of horizontally composite caissons, in which the sea side of the caisson is packed with rubble stones and hence they take the load and the caisson/seawall is relatively free from severe impact. Sometimes during severe wave climate, the rubble stones are lifted and thrown off by waves, which may cause structural damage to the caisson or the properties.

c. Construction of caissons with reduced/submerged crest level. In this case, the crest level is expected to submerge well during cyclone wave activity and hence the load will reduce. However, this will result in severe overtopping of water mass on the harbor side.

d. Construction of offshore breakwater as protection structure for the existing caisson to improve its
service life or for new seawall/caisson structure (Fig.1). The offshore breakwater will be located at an appropriate distance away from the caisson/seawall towards the sea. The influence of the presence of the offshore breakwater, of a certain height with respect to the local water depth, on the wave pressure reduction on the caisson is not studied in detail so far. There are many reports available on studies related to stability, wave transmission, overtopping etc on offshore breakwaters\textsuperscript{4-11}. Gonzelz & Prud`homme\textsuperscript{12} carried out limited studies on the reduction of forces on vertical breakwater protected by a seaward submerged breakwater. The resonating effect of waves in between the vertical wall and a protecting structure was studied theoretically by Yip et al.\textsuperscript{13} using linear wave theory concepts. The theory did not consider the energy dissipation character of the protecting structure, which is one of the factors for controlling the wave energy transmission. Theoretical estimation of wave pressure on seawalls/caissons in the presence of an offshore breakwater is cumbersome and the results will not be certain for steep and breaking wave conditions. The best way in such situation is to carry out the physical model investigation with a suitable model scale, even though it is time consuming and strenuous. This is accomplished and reported in this paper.

Material and Methods

Physical model experiments were carried out in a wave flume 30 m length, 2 m wide and 1.7 m deep (Fig. 1). A caisson model of 1.96 m length, 1.20 m high and 0.87 m wide was firmly fixed with steel frames on the flume floor. Incident waves were measured by capacitance type wave gauges in the absence of the model. Pressure transducers (HBM P11) were used to measure the wave pressures at different points on the vertical seaward face of the caisson. Rubble mound breakwater was designed and constructed towards the seaward side of the caisson with two layers, primary and core layer. The armour weight was calculated using the Van der Meer\textsuperscript{14} formulae for low-crested and submerged breakwaters. Water depth (d) of 0.30 m and breakwater crest width (B) of 0.40 m were kept constant throughout the experiment. The height of the breakwater varied from 0.20 m to 0.40 m with an increment of 0.05 m.

Two ranges of pool length ratio, \(L_p/L\) (0.035 - 0.32 and 0.07-0.64) were chosen. Pool length, \(L_p\) is the distance between the caisson/seawall and the leeward side toe of the offshore breakwater (Figure 1). \(L\) is the local wave length. For the present study, \(L_p = 0.5\ m\) and \(1.0\ m\) were selected. Incident wave steepness, \(H_i/L\) and relative wave heights, \(H_i/d\) of regular monochromatic waves were varied from 0.003-0.06 and 0.15-0.51 respectively, where \(H_i\) is the incident wave height. Five different relative height of the breakwater, \(h/d\) were used, where \(h\) is the height of the offshore breakwater (Fig. 1).

Results and Discussion

Prediction of simple pulsating wave loads on vertical walls is relatively easy, but prediction of the occurrence and magnitude of more intense wave impact loads on such structures is more complicated and calculations of such pressures are often uncertain. Methods to calculate the wave pressures for vertical wall type structures for pulsating wave conditions are described by Goda\textsuperscript{15}. As on day, this is the most

![Fig. 1—Experimental set-up for the present study](image-url)
widely used prediction method for wave pressures on vertical walls. The important assumptions made in Goda's formula are: a) It takes care of both breaking and non-breaking wave forces simultaneously in a single equation, b) The wave pressure from still water level to the sea bed assumed varies linearly, but in reality, the variation is cos hyperbolic, c) The formula also assumes linear variation of pressure from still water level upward and becomes zero at an elevation \( \eta^* = 1.5 H_{\text{max}} \), where \( \eta^* \) is the height above still water level at which the wave pressure intensity is zero and \( H_{\text{max}} \) is the design wave height.

The Goda's formula is written as follows:

\[
p_1 = 0.5 (1 + \cos \beta) \left( \alpha_1 + \alpha_2 \cos^2 \beta \right) \rho_w g H_{\text{max}} \quad \ldots \quad (1)
\]

\[
p_2 = p_1 / \left[ \cosh \left( 2 \pi h'' / L \right) \right] \quad \ldots \quad (2)
\]

\[
p_3 = \alpha_3 \, p_1 \quad \ldots \quad (3)
\]

\[
\alpha_1 = 0.6 + 0.5 \left[ \left( 4 \pi h'' / L \right) / \sinh \left( 4 \pi h'' / L \right) \right]^2 \quad \ldots \quad (4)
\]

\[
\alpha_2 = \min \left\{ \left[ (h_b - d) / 3 h_b \right] (H_{\text{max}} / d)^2 , 2d / H_{\text{max}} \right\} \quad \ldots \quad (5)
\]

\[
\alpha_3 = 1 - h' / h'' \left[ 1 - 1 / \cosh \left( 2p h'' / L \right) \right] \quad \ldots \quad (6)
\]

Pressure coefficient \( \alpha_2 \) represents the tendency of the pressure to increase with the height of the rubble mound foundation. The coefficient \( \alpha_2 \) in Eq. (5) becomes zero, when \( h_b \) and \( d \) are equal, as in the case of present study. Hence the Eq. (1) can be written as

\[
p_{1\text{mod}} = 0.5 (1 + \cos \beta) \alpha_1 \rho_w g H_{\text{max}} \quad \ldots \quad (7)
\]

\( p_{1\text{mod}} \) is \(< p_1 \) in Eq. (1) because the additive term \( \alpha_2 \cos^2 \beta \) vanishes. Measured shoreward wave pressure ratio at still water level (SWL) on the seawall without breakwater are compared (Fig. 2) with wave pressures predicted by the Eq. (7). The x-axis in Fig. 2 is relative water depth, \( d/L \). The measured pressures are higher than the values obtained by using Goda's formula. Sharp peaks of wave pressures are seen (typical wave pressure time series at the SWL of the seawall is shown in Fig. 3), which indicates the impact pressure due to wave breaking on the seawall.

The probable reasons, which can be attributed to these high measured wave pressures on the caisson compared to Goda's theory are:

a. Significant non-linear contribution of waves at SWL in the depth limited shallow waters.
b. Absence of rubble mound base below the upright section of caisson which is expected to help some energy dissipation and percolation.
c. The additive term \( \left( \alpha_2 \cos^2 \beta \right) \) in Eq. (1) is zero.

The time histories of the measured wave pressures at SWL on the seawall in the presence of offshore breakwater with different relative height of the breakwater, \( h/d \) is shown in Fig. 4. This figure is for \( d = 0.30 \), \( H_i = 0.11 \) m and \( T = 1.4 \) s, where \( T \) is the wave period. The measured wave pressure, especially in the presence of the breakwater can be divided into two

![Fig. 2—Comparison of the measured wave pressure on the seawall [pressure at still water level without breakwater]](image)

![Fig. 3—Typical wave pressure time series at still water level (SWL) on the seawall [without breakwater, d=0.30 m]](image)
Fig. 4—Typical wave pressure time series at SWL on the caisson for different relative breakwater heights $h/d$ [$d=0.30$ m, $H_i=0.11$ m and $T=1.4$ s]
components. One is the quasi-static pressure and the other is the dynamic pressure. The quasi-static pressure component is due to the raising of water level in between the seawall and breakwater, compared to the mean water level of the far field. The piling-up of water inside the pool area is a state of quasi-equilibrium reached between the mean rate of water flowing into the protected zone by waves breaking over the low or submerged breakwater, and that of water flowing out of the protected zone as a result of difference in mean water levels inside the pool and in the seaward side of offshore breakwater. The two flows are unsteady and periodic. The period of inflow is about 0.20 to 0.25 T, and that of outflow is of the order of 0.75 to 0.80 T (Diskin\cite{16}). Drei & Lambert\cite{17} have described this as pumping effect of submerged barriers. The actual dynamic pressure oscillates above the quasi-static pressure with its own mean value. This quasi-static pressure effect is more prominent for the case of low and submerged breakwater ($h/d = 0.66, 0.83$ and $1.0$). For the case of emerged breakwater ($h/d = 1.33$), this component is negligible due to lesser wave overtopping. The value of the quasi-static pressure can be obtained by drawing a line parallel to the x-axis through the lowest value of the pressure time series. It is interesting to note the following:- a) The pressure time series without the presence of offshore breakwater has only a monochromatic component and the pressure are almost regular; b) The pressure time series in the presence of offshore breakwater is irregular and the time series is no more monochromatic. This is the major physical influence of the presence of the offshore breakwater. For $h/d = 1.33$, the wave pressure on the seawall is mainly due to the waves generated on the pool side by the percolating energy of the incident waves through the porous breakwater.

Table 1 shows the normalized SWL pressure ratios on the seawall $P_v/P$, where $P_v$ is the average maximum wave pressure (sum of quasi-static and dynamic pressure) on the wall in the presence of the offshore breakwater and $P$ is the average maximum pressure on the wall in the absence of the offshore breakwater. Wave pressures corresponding to all wave heights and periods are used for obtaining the value of $P_v/P$. ($P_v/P)_{shore}$ is the shoreward pressure which occurs during the highest run-up on the seawall and ($P_v/P)_{sea}$ is the seaward wave pressure, which occurs during the maximum run-down on the seawall. For example, for $h/d = 1.00$, and $L_p/L = 0.035 - 0.32$, the value of $(P_v/P)_{shore}$ is 0.46. This means that the presence of the offshore breakwater of this configuration has reduced the shoreward wave pressure on the seawall to an average extent of 54%. For $L_p/L = 0.07 - 0.64$, the reduction of shoreward pressure ratio is about 58%. An important observation is that when $h/d=0.83$, the average shoreward pressure ratio $(P_v/P)_{shore}$ increased, compared to $h/d=1.0$ and $h/d=0.66$, especially for $L_p/L=0.035-0.32$. This is due to projectile action of jetting waters over the crest of the breakwater and its direct impact on the seawall surface below the SWL due to the gravitational action (Fig. 5), which has resulted in high pressures.

The reason for this is the wave power concentration due to funneling effect during the wave propagation over the submerged barriers for this particular submerged condition of the breakwater. This phenomenon of submerged barriers is used as artificial wave breaking simulators in the field for surfing sports activities, especially in Australia. This is expected to occur only for a certain range of relative submergence. This range needs to be avoided if we adopt submerged structure for force reduction. This may be the reason, why the submerged pressure sensors receive higher pressure than the one near still water level. The effects of breakwater slope and

Table 1—Effect of the relative breakwater height ($h/d$) and non-dimensional pool length ($L_p/L$) on average shoreward and seaward wave pressure ratios at SWL

<table>
<thead>
<tr>
<th>$h/d$</th>
<th>$L_p/L=0.035-0.32$</th>
<th>$L_p/L=0.07-0.64$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$(P_v/P)_{shore}$</td>
<td>$(P_v/P)_{sea}$</td>
</tr>
<tr>
<td>1.33</td>
<td>0.11</td>
<td>0.15</td>
</tr>
<tr>
<td>1.16</td>
<td>0.12</td>
<td>0.14</td>
</tr>
<tr>
<td>1.00</td>
<td>0.46</td>
<td>0.51</td>
</tr>
<tr>
<td>0.83</td>
<td>0.65</td>
<td>0.51</td>
</tr>
<tr>
<td>0.66</td>
<td>0.52</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Fig. 5—Illustration of water pumping effect of offshore breakwater for a small submergence of the crest level
amour diameter on the wave transformation were found to be relatively unimportant. Hence these two parameters were kept constant in the present study.

**Influence of the relative submergence depth h/d on wave pressures**

Figure 6 provides the effect of h/d on shoreward pressure ratio at still water level \((P_b/\rho g H_i)_{shore}\) for different relative wave heights, \(H_i/d\). The value of the pressure ratio lies between 0.0 and 1.0. Oscillating nature of pressure ratio \((P_b/\rho g H_i)_{shore}\) is observed when h/d is varied from 0.66 to 1.33. This oscillating nature of the pressure ratio is mainly due to reflected and re-reflected waves between the seawall and the offshore breakwater. The high value of pressure ratio for h/d = 0.83 is due to the wave pumping/jetting effect as described in Fig. 5. As can be seen from this figure that for h/d = 1.0, (when top level of the offshore breakwater is at still water level) the performance of the detached breakwater is reasonably good, where the breakwater crest induces significant wave energy dissipation. For h/d > 1.0, the wave energy is effectively dissipated and reflected, which results in significant wave pressure reduction on the seawall. The following wave-structure interaction processes are identified during the experimental investigations, which are explained below for the type of normalized wave pressure trend observed in Fig. 6:

a. For offshore breakwater with more submergence (say h/d = 0.66), the waves transmit freely and reflect from the seawall. These reflected waves contribute significantly for the wave pressures on the caisson/seawall.

b. For offshore breakwater with smaller crest submergence (say h/d = 0.83), the breakwater induces significant pumping/jetting effect and the overtopping jet of mass acts on the seawall/caisson kept behind the breakwater and impart higher order of pressures (Fig. 5).

c. For the case of the offshore breakwater with crest level flushing with the still water level (h/d = 1.0), more of the interacting energy is expected to be dissipated on the crest of the structure and hence the wave pressure reduction is significant.

d. For the offshore breakwater with less emergence (h/d = 1.16), the waves run over the breakwater and predominantly overtops, and the energy available with this overtopping water mass imparts pressures on the seawall. The wave energy dissipation due to the interaction with the breakwater reduces due to the significant overtopping processes.

e. For the offshore breakwater with significant emergence of the crest (h/d = 1.33), overtopping will be prevented for most of the waves and the waves may be allowed to transmit through the pores of the breakwater. The energy available with this transmitted wave imparts pressures on the rear side structures.

Figure 6 with this understanding, gives a clear picture why the pressure ratio variation is oscillatory in nature with increased h/d.

Figure 7 shows the effect of non-dimensional pool length \((L_p/L)\) on the variation of average shoreward pressure ratio \((P_b/\rho g H_i)_{shore}\) at SWL for h/d = 1.0. The value of pressure ratio is fluctuating with increased \(L_p/L\), and making any solid conclusion is doubtful. The only useful information obtained is that the value of pressure ratio varies from 0.5 to 1.5. Figure 8 is a similar plot but for average seaward pressure ratio

![Fig. 6—Effect of relative height of breakwater, h/d on wave pressure ratio at SWL \([L_p/L=0.15, d/L=0.092, d/B=0.75]\)](image1)

![Fig. 7—Variation of shoreward pressure ratio with non-dimensional pool length, \(L_p/L\) [pressures are at still water level)](image2)
Here, the maximum value of the seaward pressure ratio is only about 0.3. It shows that the seawall is pushed by the wave more rigorously than pull, in the presence of an offshore breakwater. The significant oscillating trends in Figs 7 and 8 are due to the resonating behavior of the water mass in between the seawall and the offshore breakwater. The interaction between the incident wave and reflected waves from vertical wall and submerged breakwater is expected to increase the pore water pressure amplitude within the seabed, which is another important research area in connection with the foundation design of the present system. Such an interaction will create a shorter wave. The chaotic pressure fluctuations on the seawall in the presence of the offshore breakwater even for regular waves warrant statistical analysis of the measured data to obtain meaningful conclusions for the purpose of design.

It is important to see how the pressure is varying along the depth of the seawall in the presence of offshore breakwater. Figure 9 gives the vertical distribution of shoreward pressure on the seawall/caisson for different relative breakwater heights, $h/d$ for a typical $d/L = 0.092$ and $H_i/d = 0.46$. In this figure, the $y$-axis is $z/d$, where $z$ is zero at SWL and positive upward. It is found that the increase in $h/d$ has reduced the value of wave pressures significantly. The maximum value of $(P/\rho g H)$ is about 2.10 for the case of without breakwater and is only 0.6 for the case of with the breakwater for the $h/d$ of 1.33.

**Effect of the relative pool length ($L_p/L$) on the pressure ratio**

As mentioned in the previous sections, two ranges of pool lengths ($L_p/L = 0.035-0.32$ and $L_p/L = 0.07-0.64$) were studied in the present investigation. In order to get meaningful design information, a statistical analysis of all pressure values for a particular pool length range is required. Figure 10 gives the probability of non-exceedence of shoreward pressure ratio $[P/\rho g H]$ for two different pool lengths and without breakwater. [Thick line is for $L_p/L = 0.035-0.32$, thin line for $L_p/L = 0.07-0.64$]
Probability analysis

Statistical analysis of shoreward pressure ratio was carried out to propose appropriate design value of the wave pressures on the caisson/seawall protected by an offshore breakwater. The cumulative probability or probability of non-exceedence of \( \frac{P}{\rho g H_i} \) for the measured wave pressures at still water level for all \( H_i \) (\( H_i/d = 0.15 - 0.51 \)) and wave periods for \( L_p/L = 0.07 \) to \( 0.64 \) and for different relative height of the offshore breakwater is shown in Fig. 11. The normalized pressure value corresponding to 98% non-exceedence (i.e. 2% exceedence) can be taken for the purpose of design of the caisson/seawall. It is seen that when the caisson is without breakwater, the 2% exceedence value of \( \frac{P}{\rho g H_i} \) is 2.75, whereas this value is reduced to 2.26, 1.98, 1.27, 1.11 and 1.15 when \( h/d \) is varied to 0.66, 0.83, 1.0, 1.16 and 1.33 respectively. This clearly brings out the relative benefit of increasing the height of breakwater in a given water depth.

Modification factor \((S_n)\) for shoreward force

Modification factor is proposed to estimate the shoreward pressure on the seawall protected by offshore breakwater. After analyzing the influence of the non-dimensional parameters on shoreward pressure a modification factor is derived from multiple regression analysis. The multiplication of the modification factor with Goda's formula will give the pressure on the seawall, when it is protected by the offshore breakwater.

\[
[P]_{\text{shore}} = S_n \cdot [P]_{\text{Goda}} \quad \cdots (8)
\]

\([P]_{\text{shore}}\) is the shoreward wave pressure on the seawall in the presence of the offshore breakwater, \( S_n \) is the modification factor and \([P]_{\text{Goda}}\) is the wave pressure on the seawall based on Goda's formula. Equation (9) is the result of the multiple regression analysis. It is seen that the modification factor is more sensitive for the change of relative breakwater height, \( h/d \). In the above equation \( L_p/L \) takes care of the effect of wave period since \( L_p \) has taken as constant. Figure 12 shows the comparison of the observed and estimated \([S_n]_{\text{shore}}\) using Eq. (9) for different runs (different wave heights, periods and \( h/d \)) for pressures at still water level. For most of the runs, the modification factor is less than 1.0 indicating the presence of offshore breakwater. Only for few runs, the modification factor is more than 1.0, indication the water jetting effects on the offshore breakwater.

Conclusion

The effect of the presence of an offshore breakwater on wave pressures on a vertical seawall is investigated using physical model studies. The following conclusions are obtained:-

a. The measured pressures on the seawall in the absence of any protection structure are in general higher than that predicted by Goda’s formulae\(^{15}\).

b. The presence of the breakwater in front of the seawall induces irregular wave pressures on the seawall even for regular wave inputs.

c. Normally it is expected that the wave pressure on the seawall reduces when the height of the protecting breakwater is increased for a given
water depth. However, when the relative height of the breakwater \( h/d \) is closer to 0.83, and for certain hydrodynamic input conditions, the average shoreward pressure ratio is more than when \( h/d = 1.0 \) due to water jetting effect over the breakwater. Such a condition need to be avoided in order to reduce the wave load on the seawall in the actual prototype case.

d. The shoreward wave pressure on the seawall is higher by at least 4 times compared to the seaward pressure in the presence of offshore breakwater. Hence shoreward pressure governs the design of such system.

e. 2% exceedence value of shoreward pressure ratio, \( (P/\rho gH)^{shore} \) at SWL on the seawall without breakwater is 2.83. When the breakwater is introduced with relative breakwater heights \( h/d = 0.66, 0.83, 1.0, 1.16 \) and 1.33 the pressure ratio become 2.26, 1.98, 1.27, 1.11 and 1.15 respectively. This result can be used for design of prototype system.

f. A cost benefit analysis by using the present results is required to select the optimum relative breakwater height.

g. The results of this study can be used for rehabilitation of partially damaged seawalls or design of new seawall with offshore breakwater as protecting structure.

h. Empirical formula for pressure modification factor is proposed to estimate the shoreward pressure on the seawall, when it is protected by the low-crested breakwater. This modification factor, when multiplied with Goda's formula yields the shoreward wave pressure on the seawall protected by the offshore breakwater.

References


