PRESSURE DISTRIBUTIONS ON A VERTICAL BREAKWATER: EXPERIMENTAL STUDY AND SCALE EFFECTS

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This work studies the horizontal and uplift pressure distributions over a caisson founded on porous materials, and their dependence on the stone diameter and the height of the foundation. For this, tests at a wave flume with an idealized composite breakwater of rectangular section, varying the depth of the foundation of the caisson and the diameter of the stones, were carried on. Eight resistive gauges and eight pressures sensors were used to measure free surface elevations and horizontal and uplift pressure variation, respectively. Results show that: 1) there exist a "saturation" of the reflection coefficient for B/L>0.4, being B the width of the dike and L the wave length, 2) by using the total wave height measured at the toe of the dike in the analysis, the dispersion of the results is significantly reduced; 3) dimensionless run-up and pressures obtained using total wave height mainly depends on the reflection regime and on the relative height of the foundation; 4) maximum uplift and horizontal forces are not always in phase, and three regimes are identified depending on which force dominates; and 5) the relation between the dimensionless forces with the total wave height at the toe of the dike depends mainly on the reflection regime and on the relative foundation height.

Keywords: vertical breakwater; wave height; pressure distribution; porous media

INTRODUCTION

Wave action on coastal structures results from the interaction between the incident wave train and the geometry and characteristics of the structure. Thus, wave pressure distribution for a specific vertical-front structure depends on how the wave-structure interaction, including reflection and transmission of wave energy, is evaluated. Several authors have derived solutions to determine such pressure distributions for breaking and non-breaking waves (Goda, 1985). Generally, upright pressure distribution is described by a polygonal with its maximum value in the still water level that decreases above and below it. However, foundations characteristics, i.e. porosity, should affect pressure distributions and they are not considered in these formulations. Moreover, several authors have pointed out that the linear pressure gradient assumption for the uplift pressure distribution may be inaccurate (Losada et al. 1993).

As a result, several questions arise regarding the influence of the hydrodynamic behaviour of the foundation in the final design: how the breakwater width and the foundations characteristics (foundation height and stone size) modify the wave train in front of the structure? In case the influence of these factors is important, are the design assumptions valid after the structure is built? Has the breakwater been built on the safety side? Also, is it possible to optimize the design including the hydrodynamic behaviour of the breakwater-foundation?

On the other hand, as wave-generated pressures on structures are complicated functions, laboratory experiments are usually performed as part of the final design. Therefore, there is scarce information about wave induced uplift pressures and their scale effects in laboratory experiments. This work shows the results of 2D laboratory tests that have been performed to analyze horizontal and uplift pressures on vertical breakwaters under regular non-breaking wave conditions.

Therefore, the objective of this work is to study the horizontal and uplift pressure distributions dependence on the height of the foundation and/or on the size of the stones, for a caisson founded on porous materials.

BACKGROUND

To answer these questions, the "Grupo de Dinámica de Flujos Ambientales" has been working on the development of theoretical models that describe the wave interaction with the structure and its subsequent verification in the wave flume (Scarcella et al., 2006; Pérez et al., 2009).

In particular, the behaviour of a permeable vertical breakwater has been studied. The results show that wave interaction with the permeable structure mainly depends on the parameters: relative width B/L, where B is the width of the dike and L is the wavelength; relative diameter Dk, where D is the

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stone diameter and k is the wave number; and ratio diameter wave height D/H, where H is the wave height.

For the range of relative width B/L analyzed, the results show that the reflection coefficient increases to reach its maximum value; also, there is a zone of saturation adjustment and there is a saturation regime where the reflection coefficient can be considered practically constant (Pérez et al., 2009). In the first two areas, the reflection coefficient is a function of the parameters described before, while in the saturation zone, the reflection coefficient depends on the porosity of granular material.



Figure 1. Behaviour of the reflection and transmission coefficient for the vertical permeable breakwater.

Based on a potential flow model for wave interaction with permeable structure (Dalrymple et al. 1991) and a set of experimental tests, a characteristic friction diagram was obtained considering the wave energy balance. This allows for the quantification of the resistance to flow through the porous media. Furthermore, it intends to provide an adequate selection criterion for the experimental stones diameter to minimize scale effects in the laboratory.

The results show that the characteristic friction coefficient is function of the following dimensionless monomials.

$$f_c = \Psi(Dk, B/L, D/H) \tag{1}$$

The characteristic friction coefficient diagram as a function of the parameters Dk can be seen in Figure 2.

LABORATORY TESTS

Experiments were conducted in the wave flume of "Centro Andaluz de Medio Ambiente" (23x0.65x1m). The model consisted of a vertical breakwater with rectangular cross section (Figure 3). The caisson foundation was built with uniform grain diameter. Three different model lengths (B1 = 0.14 m, B2 = 0.50 m and B3 = 1.5 m) were tested to analyze a wide range of relative widths (0.009 < B/L < 1.02). Water depth has been kept constant and equal to h = 0.4 m. Foundation depth (hb) was varied (hb/h = 1, hb/h = 0.5 and hb/h = 0.25) and five granular nominal diameters were used (D = 12 mm - D = 110 mm) in order to obtain different flow regimens into the porous media.

Monochromatic waves were generated with the wave absorption system (AWACS®) activated. Wave period was varied from T = 1 - 3 seconds with increments of 0.25 seconds; for each period the wave height was varied from HI = 0.04 to 0.010 m with increments of 0.02 m.

Eight resistive gauges and eight pressures sensors were used to measure free surface elevations and pressure variation, respectively (Figure 3). Two sets of three sensors were placed to calculate reflection and transmission coefficient using the method proposed by Baquerizo (1995), one between the paddle and the model (WG 1, 2 and 3) and one between the model the dissipation ramp (WG 6, 7 and 8). To assess the total wave height in front of the structure an additional sensor was placed (S4). Finally, sensor S5 was placed to assess water level variations at the rear side of the structure.



Figure 3. Scheme of the experimental set up.

RESULTS

Reflection and transmission coefficients

Figure 4 and Figure 5 presents the effect of the relative width B/L on the experimental results of the wave transmission and reflection coefficients for the three foundation depths tested. The results show the same behaviour as reported by Perez et al. (2009) for a vertical porous dike. The following behaviour can be observed:

- For the three foundation height tested, it can be observed that for increasing breakwater relative widths B/L, the reflection coefficient grows until a near constant value is achieved at $(x/L)_c$ where x is the wave propagating distance inside the breakwater. For larger widths, the coefficient does not change significantly (or does slightly) and the reflection process reaches a "saturation" regime. For the experimental results the "saturation" regime can be established at x/L>0.4.
- In the saturation region the value of the reflection coefficient depends on the stone diameter (porosity) and weakly on B/L and D/H. As a result, it can be accepted that the reflection coefficient is constant in the saturation region; under resonant conditions (B/L=1/2, 1/4, 3/4), when perfect coupling of the transmitted and reflected waves inside the porous structure occurs, the reflection coefficient gives maxima and minima values around the "saturated" value.
- From Figure 4 it can be observed that the two flow regions identified by Losada et al. (1995) can be distinguished: 1) a transition region x/L < 0.4, where K_R depends on wave parameters,

breakwater geometry and stone characteristics, and 2) a transmitted region, $x/L > (x/L)_c$ (leeward of the saturation point).

- As pointed out by Losada et al. (1995) for small breakwater widths (B/L) < 0.4 the saturation regime is never reached; the reflection, transmission and dissipation coefficients are mainly governed by the abrupt change in the wave flow regime, similar to the local head loss of a perforated plate or the abrupt change of a channel section.
- On the contrary, for larger breakwater widths $(B/L > (x/L)_c)$ the transmission coefficient always decreases with the exponential decay of porous medium pressure amplitude, as pointed out by Burcharth et al. (1999).

For the three height foundations tested, the transmission coefficient presents an exponential decay with the increased of the relative width B/L.

It can be observed that the transmission coefficient presents lower values with the decrease of the stone diameter.



Figure 4. Behaviour of the reflection coefficient. Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/H_p<2.42; 0.08<h/L<0.27



Figure 5. Behaviour of the transmission coefficient. Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/Hp<2.42;0.08<h/L<0.27

Pressure distribution

Regarding the pressure distribution over the vertical structure, it was verified that the pressure distribution over the mean water level is proportional to the sea water elevation over the wall, as given by the following expression

$$P = \rho g \eta \tag{2}$$

Where ρ is the water density and g is the gravity acceleration. Figure 7 shows the results of P1 measured at the mean water level (Figure 6) versus P2 measured at the toe of the structure.



Figure 6. Scheme of the pressure distribution over the vertical breakwater.

Experimental results show that the maximum pressure value is approximately constant except for resonant conditions. Moreover, data tend to group in terms of the relative depth h/L.

Variables have been dimensionless with the total wave height to include the effects of reflection. That is the reason why less dispersion is shown.



Figure 7. Results of P_1 vs P_2 dimensionless. Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/H_p<2.42;0.08<h/L<0.27

Uplift pressure distribution

The relationship between the pressure P2 and the uplift pressure at the entrance of the porous media Pu is established by the relative height of the foundation hb/h and the reflection regime. This can be seen in Figure 8:

- When $h_b/h = 1$ and the reflection coefficient is in the saturation regime, data presents a linear behaviour with a slope of 45° approximately.
- Following the relative height foundations less than one, out of the saturation region, data do not present a clear behaviour, having cases where the uplift pressure at the entrance of the porous media is higher than the horizontal pressure at the rear of the caisson. This behaviour may be due to flow characteristics or local effects.
- Finally, in cases where $h_b/h = 1.00$, and the pressure P_2 is equal to the maximum pressure at the mean sea level, it can be observed that pressure values presents a constant behaviour; this may be the case of a crown wall. For this case, pressure at the toe is the same that the pressure at the SWL. Additionally, pressure is dimensionless using total wave height. This means that pressure is a constant times the run-up, and variations in the run-up are included on the total wave height.



Figure 8. Results of P_{2max} versus P_{u1max} dimensionless with total wave height. Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/H_p<2.42;0.08<h/L<0.27

Horizontal and vertical force

To obtain the horizontal and vertical force values exerted by the wave train in the caisson, pressure transducers were placed in the front and the base of the caisson. Integrating the pressure time series for each time step, the time variations of the vertical and horizontal force per unit width were obtained. The maximum horizontal and vertical forces were identified along with the characteristics of the forcing wave and vice versa (maximum vertical force along with the horizontal force).

The first conclusion is that maximum horizontal and vertical forces are not always in phase (Figure 9). Thus, if this behaviour is not considered in the design of the vertical breakwater, the vertical forces may be underestimated and the structure failure modes will not be properly established.

Moreover, results show the existence of three zones:

- A first zone were horizontal forces dominate and the vertical forces value are lower. This region corresponds to relative height foundations lower than one and small breakwater widths.
- An intermediate zone where both forces have the same importance.
- And a third zone where vertical forces dominated and horizontal forces are negligible. This region corresponds to foundation heights equal to the water depth.

Analyzing the influence of the relative width of the caisson, the following is observed:

- When *B/L*<0.25, data are grouped in two curves. One corresponds to values of *h_b/h=1* and, the other one, with exponential behaviour, corresponds to *h_b/h*<1.
- When B/L > 0.25 data are grouped in three curves, one for each wave height foundation tested.
- In both regions of B/L it can be observed that for $h_b/h = 1$, vertical forces are higher that horizontal forces. This phenomenon is due to trapped air during wave propagation through the porous media.



Figure 9. Results of F_{Hmax} - F_V and $F_{HT}F_{Vmax}$. Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/H_p<2.42;0.08<h/L<0.27

The behaviour described above can be seen in Figure 10.

The results obtained with the horizontal and vertical dimensionless forces with the total wave height at the toe of the structure show lower dispersion (see Figure 11)

Analyzing data according to the parameters of the problem, it can be observed that for all cases where hb/h<1, data are grouped in B/L intervals with a lineal behaviour in all cases. Also, data are grouped in three curves, each one for the reflection regime established.

For hb/h=1, horizontal forces are approximately constant and vertical forces decreases when B/L decreases.

CONCLUSIONS

The main conclusions of this work are:

- For all tested configurations a saturation regime of the reflection coefficient exists, as was previously observed for a porous media. Whether the dike is in the saturation regime or not mainly depends on the relative width.
- Using the total wave height (incident plus reflected) in the analysis allows for the inclusion of the phenomenon of reflection. Under this condition, the results show a smaller dispersion.
- It was proved that the run up is a function of the reflection coefficient and the studied parameters.
- Relation among vertical and horizontal pressures at the toe of the dike depends on the regime of the reflection coefficient; on saturation regime the relation is linear.
- Regarding the forces, it is noted that maximum uplift and horizontal forces are not always in phase, and three zones are identified depending on which force predominates.
- The relation between the dimensionless forces with the total wave height at the toe of the dike depends mainly on the reflection regime and on the relative foundation height.



Figure 10. Results of F_{Hmax} - F_V as a function of the parameters B/L y h_b/h . Range of experimental parameters: 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05< D/H_p <2.42;0.08<h/L<0.27



Figure 11. Results of $F_{v}F_{Hmax}$ dimensionless with total wave height as a function of B/L y h_b/h_c Range of experimental parameters 0.03<B/L<1.02; 0.15<Dk<0.47; 0.05<D/H_p<2.42;0.08<h/L<0.27

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