

Wave Forces on Breakwater Crown Walls: Physical Model Tests and Improvement of Prediction

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Abstract: This paper collects the results of several new 2D physical model tests on wave interaction with rubble mound breakwaters. The wave forces on the crown wall are analyzed in detail, showing their dependence on the tested geometric and hydraulic configurations. The database for this analysis is obtained from three models of different rubble mound breakwaters reproduced at the new small-medium scale wave flume of the Department of Engineering of Roma Tre University, for a total of 184 random wave tests. The paper aims at improving the prediction of wave forces on the crown wall, proposing new empirical formulae and enhancing the model tests database of wave forces and overtopping discharges.

Keywords: Wave forces on crown wall, rubble mound breakwaters, overtopping, laboratory tests

1 Introduction

Rubble mound breakwaters are usually built with a concrete crown wall on the top, often partially protected by the armour layer. Crown walls can be installed for several reasons, mainly to ensure a lower overtopping discharge; to reduce the required volume of quarry material; to facilitate construction and maintenance operations. To ensure a safe operability at the rear side of the crown wall, a correct design of these structures is mandatory. The wave loads on a crown wall depend on the characteristic of both wave and breakwater geometry. There exist several empirical studies on wave loads at crown walls, analyzing the wave induced pressures or the wave forces recorded in physical model tests. Several Authors correlate non-dimensional horizontal and vertical wave forces to hydraulic and structural non-dimensional parameters and have proposed some design formula based on the fitting of experimental data. Firstly, Jensen (1984) points out that exist a linear relation between the maximum horizontal force on the wall and the significant wave height. Later, Bradbury & Allsop (1988) and Pedersen & Burcharth (1992) investigated the fitting coefficients of this linear relation, based on other experimental tests. Pedersen (1996) studied wave forces and overtopping on crown wall for rubble mound breakwaters, proposing a design method for assessing the maximum horizontal wave force based on experimental data. Other Authors, as Günback & Göcke (1984) and Martin et al. (1999) proposed pressure distributions along the wall, providing correlations with the significant wave height and other parameters such as the run-up value, the freeboard height, the height of the armour protection berm. Negro et al. (2013) carried out a state of the art review analysis, comparing the available design formula and highlighting their applicability. They outlined the heavy dispersion of results between different methods and therefore recommended to confirm the results of more than one formula with ad hoc experimental tests. Nørgaard et al. (2013) extended the formulae of Pedersen (1996) by including also data from experimental tests in shallow water conditions. Molines et al. (2018) correlate the wave forces with seven non-dimensional explanatory variables, also including the overtopping rate.

The present paper has two main goals. Firstly, to present the results of three physical model studies performed at the recently installed (2017) wave flume at the Engineering Department of Roma Tre

University. Secondly, to analyze the experimental database of the wave forces on the crown wall and to evaluate the ability of some non-dimensional explanatory variables to predict the wave loads given the hydraulic and structural parameters. Following the approach of Molines et al. (2018), we analyze different input variables to estimate wave forces on crown wall, also investigating on how to better express the non-dimensional dependent variable (wave force). Finally, the mean overtopping discharge is related to the hydraulic and structural parameter, and compared with EuroTop (2018) formula. Franco et al. (2018) presented the results of just the first of these three models, analyzing overtopping and pressures at the crown wall.

2 Description of the physical models

The laboratory facility is a small-medium scale wave flume: 20.0 m long, 0.6 m wide and 1.0 m high. The channel is equipped with a 1.35 m stroke piston for the wave generation, controlled to reproduce both regular and irregular waves. During all tests five Churchill resistive wave gauges have been used to measure the free surface oscillations along the flume, as shown in Figure 1 for the three models, hereinafter referred as model A, B and C.

Six Trafag pressure transducers (water-column pressure range of 0.0 - 2.0 m) have been used to measure the pressures induced by the waves (see also Franco et al., 2018). These instruments have been placed at the vertical wall face and at the horizontal base (slab) of the crown wall in all the three models, as represented in Figure 2. The crown wall was built in Perspex (1 cm thick), allowing easy mounting of the pressure sensors. Steel reinforcement structures were used to avoid crown wall vibrations or deformations under wave action. A tank is used to collect the overtopping water. It discharges the flow in a bucket located below the flume and connected to a force transducer for weight measurement. The mean overtopping discharge is obtained for each test.

The three different rubble mound structures are shown in Figure 3 at their prototype scale. Using Froude similarity a reduction scale of 1:40 has been used for the first model (model A) and of 1:30 for the other two models (B and C). Rock armour is used in breakwaters A and B, while both rocks and tetrapods are used in model C. For the breakwater models B and C the armour geometry has been varied. In details, the breakwater B is tested in a standard configuration and with a modified berm profile, made with the same rock size (see Figure 3, model B). The breakwater C is reproduced in three different configurations, shown in Figure 4, which represent three different sections of the same breakwater. The main difference between the sections is in the size of the armour layers, and in the water depth. Each of these 3 tested sections have been tested in two configurations, the standard one with layers named 1 and 2 in Fig 4 and by adding further layers of tetrapods, named as 3 and marked with thicker lines in Fig 4, and by raising up the crown wall of 1 meter, for sections 1 and 2. The database of model C therefore consists in 6 breakwater configurations.

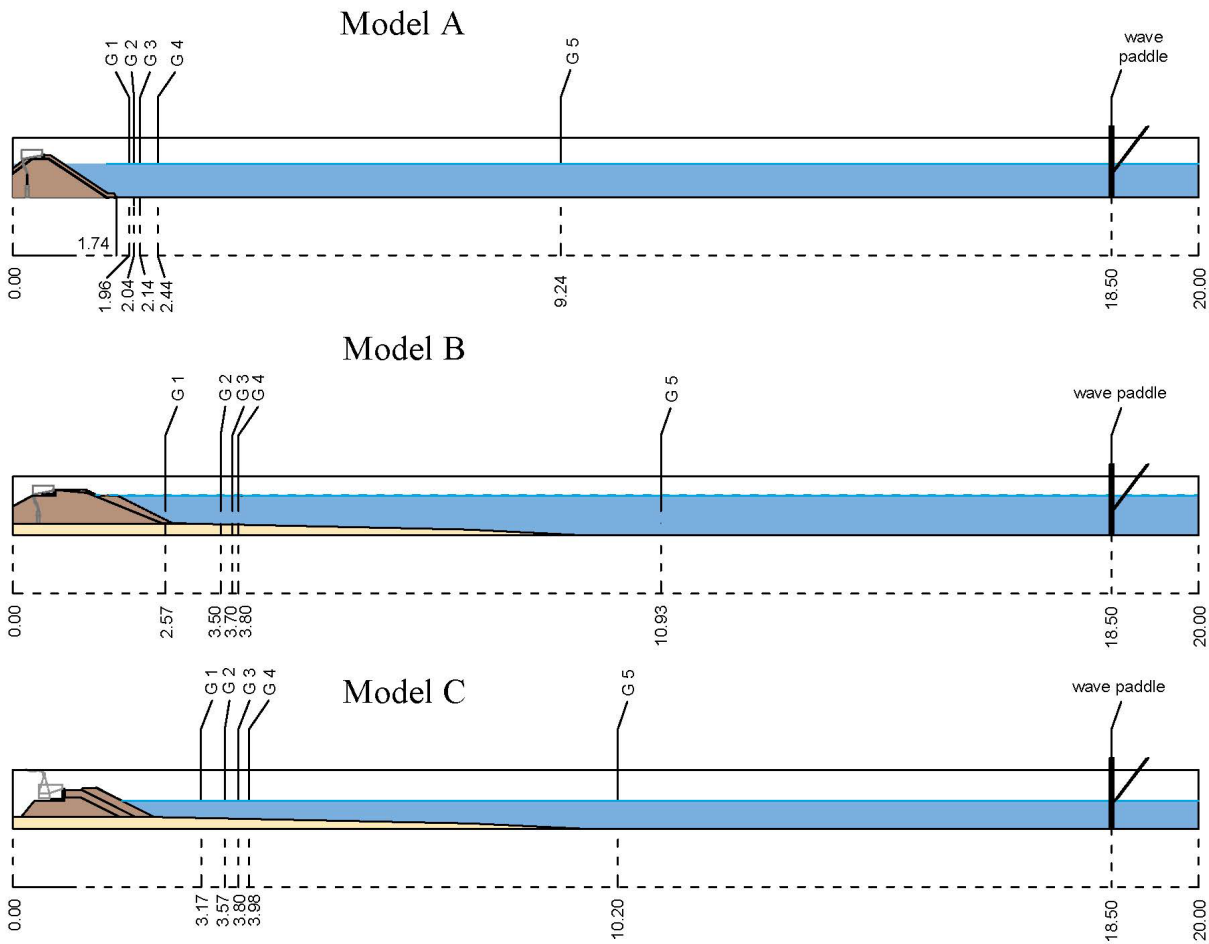


Fig. 1. Longitudinal sections of the wave flume with the three models (measures in meters).

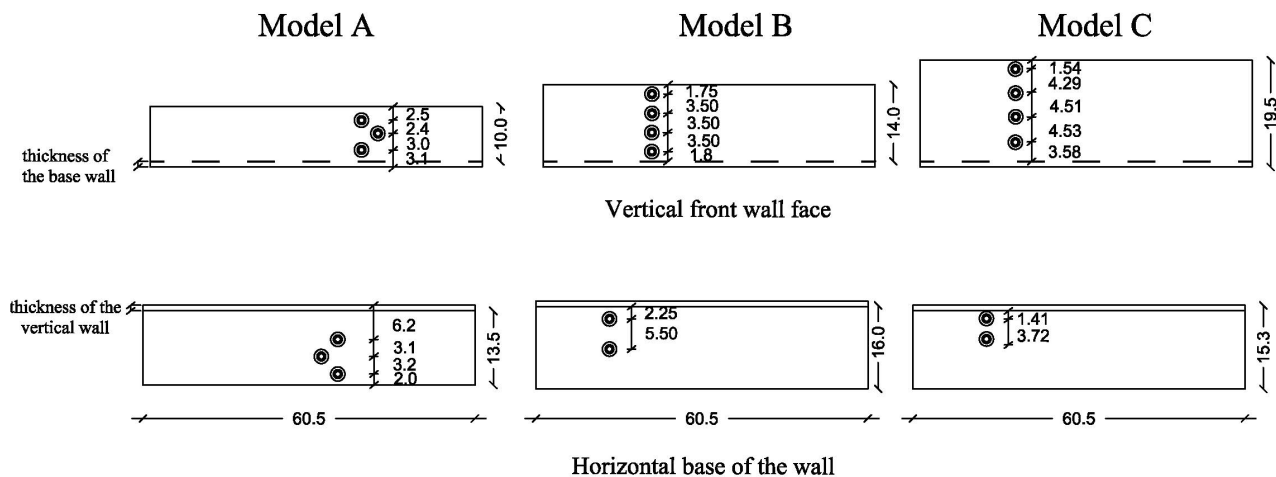


Fig. 2. Crown walls reproduced in the laboratory for the three models, with pressure sensors position (measures in cm).

In all tests the waves have been generated using Jonswap spectrum, using 3.3 as peak enhancement factor, with a duration of 1000 waves, which is in the range of 18-25 minutes. Different wave conditions have been tested with significant wave height, peak period and still water level within the ranges of: $H_{m0} = 2.25-6.46$ m; $T_p = 5.6 - 13.8$ s; s.w.l. = 0 and 0.5 m, at prototype scale.

The geometrical parameters of the three structures vary within the following ranges: $R_c = 2.70-6.00$ m; $A_c = 2.65-6.00$ m and $G_c = 3.80-15.60$ m; where R_c is the wall height with respect to s.w.l., A_c is the crest height with respect to s.w.l. and G_c is the crest width (see Fig 6). The total number of tests analyzed is 184 (38 for breakwater A, 46 for breakwater B and 100 for breakwater C).

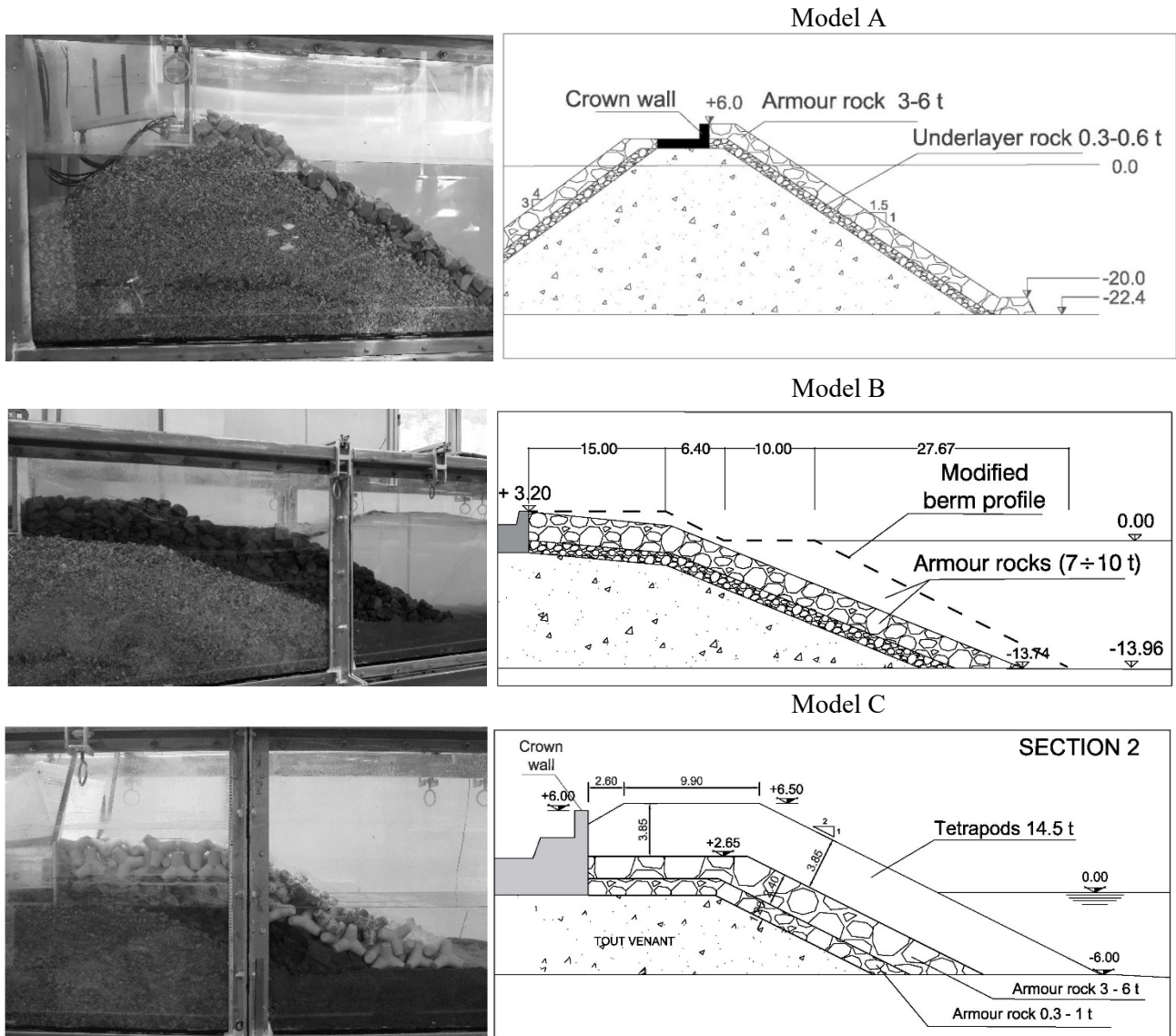


Fig. 3. Vertical sections of the three rubble-mound breakwaters reproduced in laboratory (dimensions in meters at prototype scale).

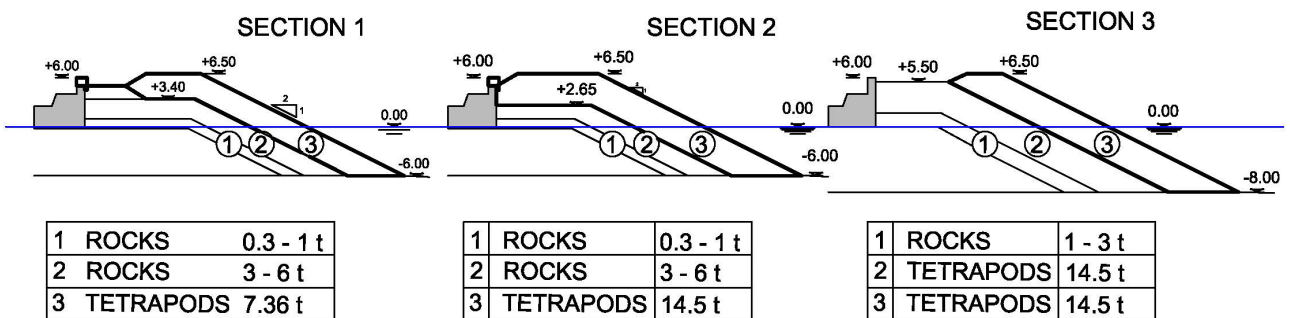


Fig. 4. Vertical sections of the three different rubble-mound breakwater Model C; thick lines indicate the tetrapod armour layers added to the standard configuration.

3 Analysis of the model results

For all the 184 tests performed, the novel nonlinear methods proposed by Lykke Andersen et al. (2017) and Eldrup and Lykke Andersen (2019) have been applied to separate the incident and reflected wave components, using the free-surface measurements at 4 gauges.

The pressures recorded at the transducers placed on the vertical face of the crown wall have been employed to calculate the total horizontal force acting on the wall, by using a basic integration

method, considering the vertical positions of the sensors along the wall and a unit wide area (Figure 2). A similar approach is used for the total vertical force.

3.1 Statistical wave forces correlation

A statistical analysis of the forces has been carried out for the 184 tests, from both horizontal and vertical force time series each peak values have been extracted, producing a statistical sample for further analysis. It is assumed that each of the 1000 waves in each tests impacts the crown wall, although most of them do not reach the crest of the structure, providing a negligible or null force. On this basis, the largest peak force is referred to as the F_{\max} , or $F_{1/1000}$, and parameters similar to those typical of the short-term wave statistics can be calculated. In order to set the basis for the analysis of the next sections, where a statistical correlation is searched between hydraulic and structural parameters and peak forces, the results of each test are represented with one single, robust, wave force descriptor. Similarly to other studies on similar problems (Van Doorslaer et al.; 2017 and Bellotti et al.; 2014) the average of the largest four peak forces is selected, and referred to as the $F_{1/250}$.

Since the following analysis is carried out using $F_{1/250}$, but for design purposes, it might be important to obtain also the F_{\max} , a correlation between these statistical values is evaluated in Fig. 5. Therefore, a linear relation is provided by fitting the data with a least-squares solver. As noted in Fig 5, the data are well aligned with a line, for both horizontal and vertical forces.

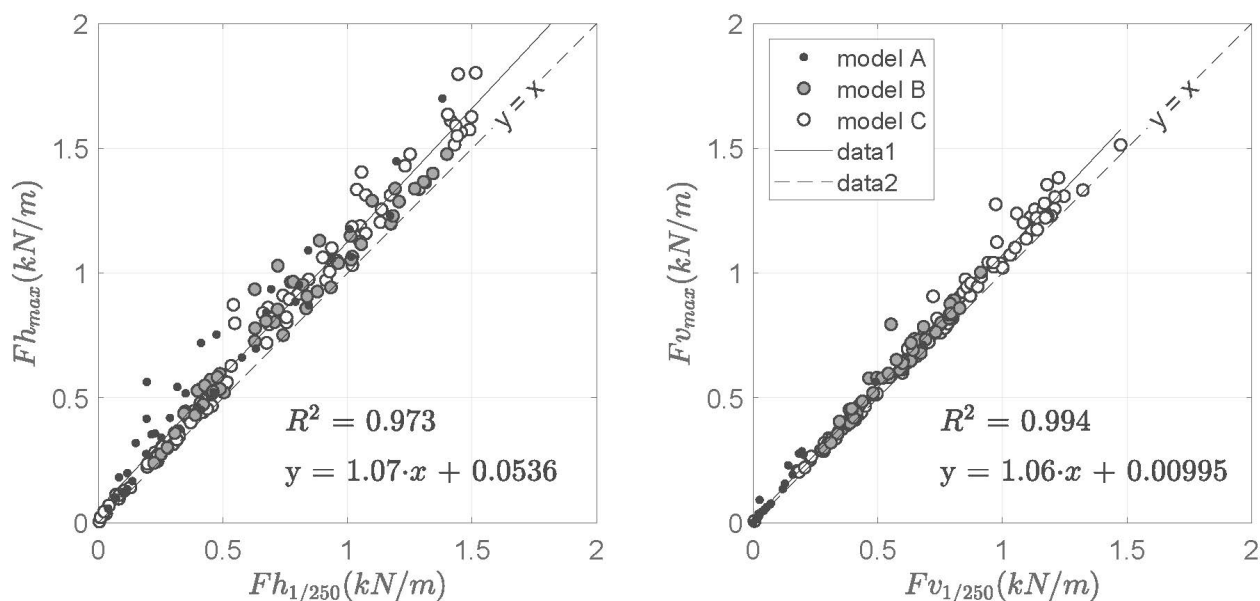


Fig. 5. Correlation between statistical values of the forces (F_{\max} and $F_{1/250}$) at the crown wall. Horizontal forces on the left and vertical forces on the right.

3.2 Empirical formula for horizontal wave forces on crown wall

In order to correlate the horizontal forces on the crown wall with the hydraulic and geometrical parameters some of the explanatory variables, as proposed by Molines et al. (2018), are used. Table 1 (left) resumes these non-dimensional variables. We also investigated on four possible explanatory non-dimensional formula for the horizontal wave force, shown in Table 1 (right).

Tab 1. Four explanatory non-dimensional variables (X_i) and non-dimensional horizontal force on the crown wall (Y_i).

$X_1 = \frac{R_c}{\gamma_f H_{m0}}$	$Y_1 = \frac{Fh_{1/250}}{\rho g C_h^2}$
$X_2 = \frac{R_c}{\gamma_f H_{m0} \xi_{op}}$	$Y_2 = \frac{Fh_{1/250}}{\rho g R_c^2}$
$X_3 = \frac{\gamma_f R_{u0.1\%}}{R_c}$	$Y_3 = \frac{Fh_{1/250}}{\rho g C_h R_c}$
$X_4 = \log(Q)$	$Y_4 = \frac{Fh_{1/250}}{\rho g H_{m0} R_c}$

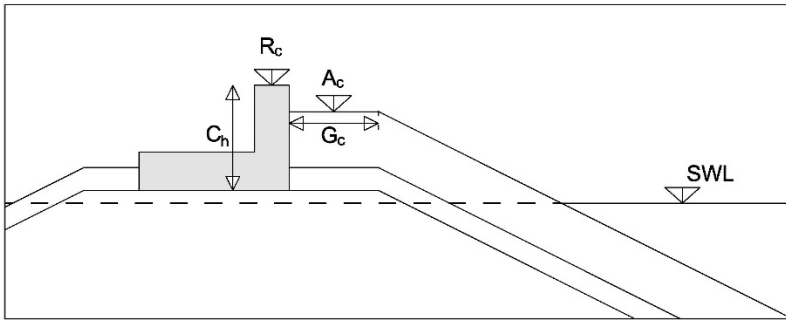


Fig. 6. Geometrical parameters of the breakwater

In Table 1, γ_f is the roughness factor of the armour layer; ξ_{op} the Iribarren's number; Q is the dimensionless overtopping discharge per unit width; C_h the crown wall height and $R_{u0.1\%}$ the virtual run-up estimated as in Molines et al (2018) and the geometrical parameters of the breakwater are defined in Figure 6.

Figure 7 shows the correlation between the 4 x 4 variables. The data are fitted with a least-squares method, interpolating the data with a best-fit function, which is exponential for the first two variables X_1 and X_2 , and linear for variables X_3 and X_4 . In each graph of Fig 7, the value of R^2 provides a measure of how well the data are fitted by the curve. In all plots of Fig 7, some correlation between the variables can be noted, although the data are quite scattered. This fitting analysis leads to the conclusion that for design approach all these correlations can be used to estimate $Fh_{1/250}$. Given specific values for the main parameters included in the variables X_i , 16 predicted horizontal wave forces could be obtained using the interpolating functions. The average of these predictions, weighted with the level of uncertainty related to each correlation, provides an estimate of the horizontal wave load. The best correlation is the one between the dimensionless variables X_1 and Y_2 and is given by the formula:

$$\frac{Fh_{1/250}}{\rho g R_c^2} = 0.0081 \cdot \exp\left(-1.298 \frac{R_c}{\gamma_f H_{m0}}\right) \quad (1)$$

Figure 8 shows a scatter plot of the non-dimensional wave forces measured from all the 184 tests and calculated with the empirical formula (1).

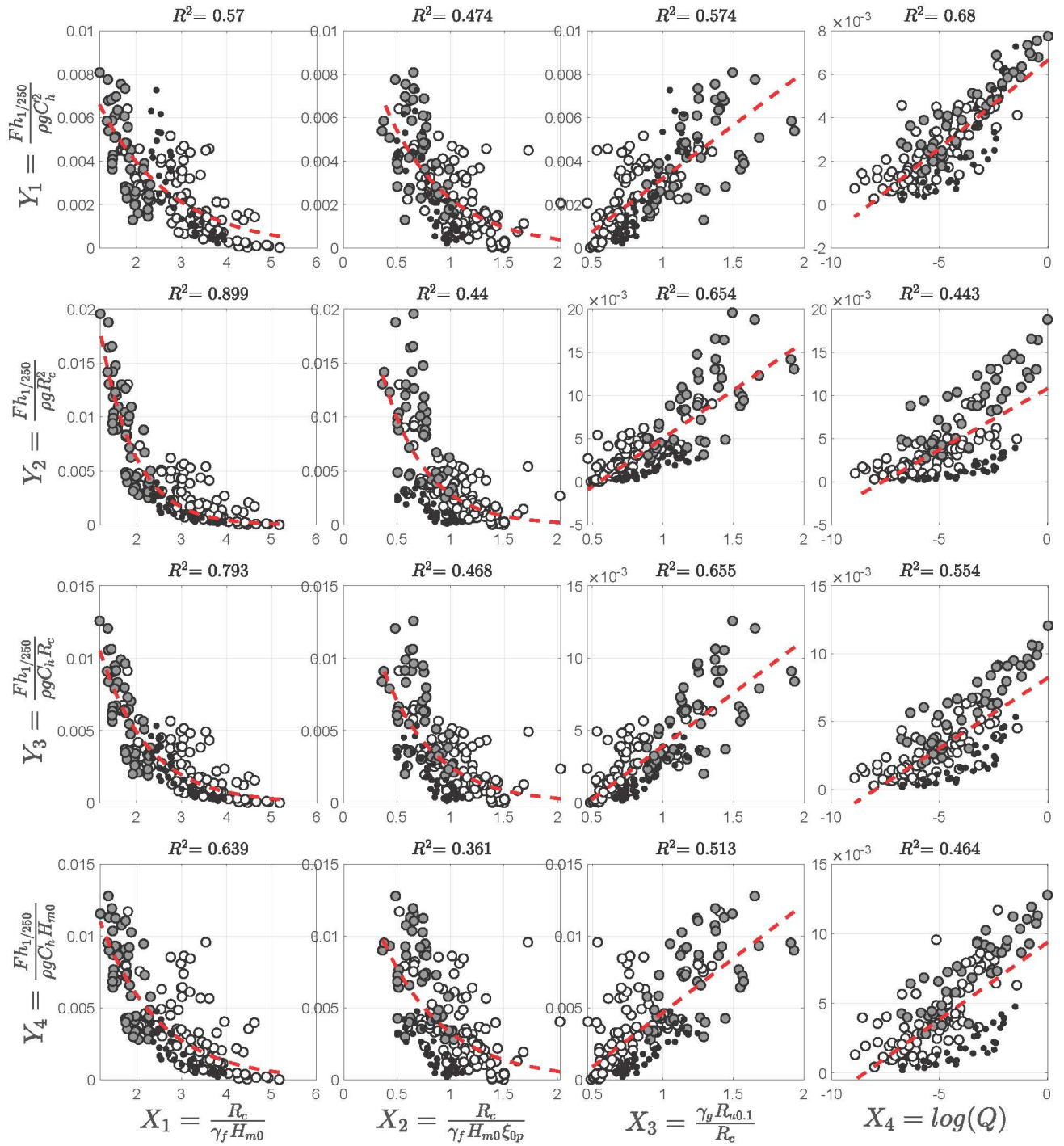


Fig. 7. Four dimensionless horizontal forces versus four explanatory variables for the three tested models (black dots, model A; gray circles, model B and white circles, model C).

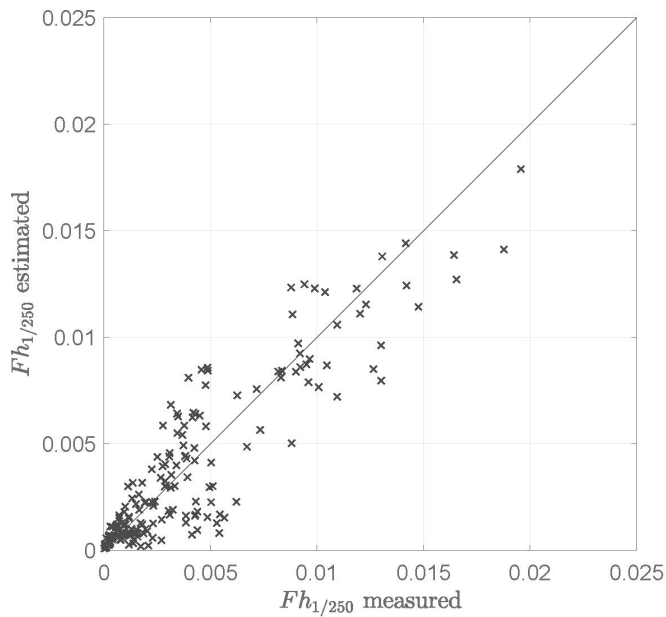


Fig. 8. Comparison between measured and estimated non-dimensional horizontal forces, for the three models.

3.3 Wave overtopping

The overtopping rate is measured for most of the 184 tests performed. The non-dimensional mean overtopping rate per unit width Q , is plotted against the dimensionless freeboard (R_c/H_{m0}) in Fig 9 (upper panel).

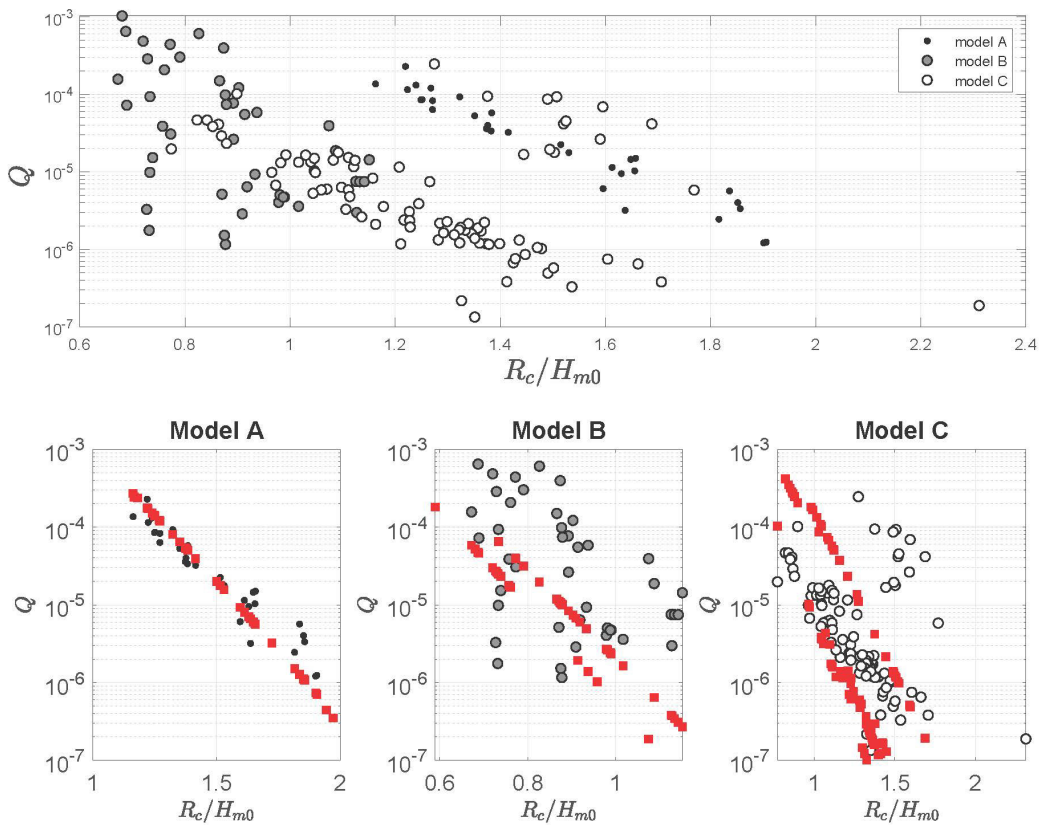


Fig. 9. Mean overtopping discharge measured for the three models (top). Comparisons of each model results with prediction of EuroTop formulas, marked in red squares (bottom).

Two families of data can be identified in the top plot of Fig 9. The upper-right cloud of data are the tests of model A and some tests of model C, which present higher values of overtopping discharge for the tested dimensionless freeboards: this is due to the absence of a berm for the breakwater of model A and, for model C, it is related to the configuration without the tetrapods armour layer.

The laboratory results are compared with those derived from the EuroTop (2018) formula for rubble mound slopes of 1:2 to 1:1.33:

$$Q = \frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp \left[- \left(1.5 \frac{R_c}{H_{m0} \gamma_f \gamma_\beta} \right)^{1.3} \right] \quad (2)$$

where γ_β is the wave obliquity factor. In Fig. 9 lower panels, the formula (2) has been applied considering $\gamma_f=0.45$ for rock armour and $\gamma_f=0.38$ for tetrapods armour (as indicated in Eurotop 2018). The wave obliquity factor is set to 1 as the tests are 2D and wave attack is perpendicular. Since most of the geometrical configurations of the breakwaters presents an armour crest berm larger than H_{m0} , the EuroTop manual suggests multiplying the mean overtopping discharge by a reduction factor C_r :

$$C_r = 3.06 \cdot \exp \left[-1.5 \frac{G_c}{H_{m0}} \right] \quad (3)$$

where G_c is the crest width.

4 Conclusions

Three small-scale rubble mound breakwater models have been tested in a 2D flume, with different sea states, water levels, geometrical and structural conditions. The database, made of 184 tests, provides consistent results. The horizontal forces induced by the waves at the crown wall have been tentatively correlated with some possible nondimensional explanatory variables that were used as input variables in the Neural Network of Molines et al. (2018). For these new tests, the best correlation appears to exist between the dimensionless variables Y_2 and X_7 . This analysis confirms a correlation of the wave forces with some hydraulic and structural parameters, however the data are quite scattered. Indeed, as recommended by Negro et al. (2013), empirical methods can be applied for preliminary design, while for final design it is recommendable to use specific physical model tests. Since the empirical methods are mainly based on laboratory tests with their own range of applicability, this study aimed at extending this range, providing 184 new results.

It can be concluded that it is difficult to find a unique prediction formula that best fits all the parameters with the horizontal wave force. Indeed, all the main hydraulic and geometrical parameters have an important effect; therefore, for design purposes, all the correlations proposed in Fig 6, each with a specific level of uncertainty, could be used as preliminary estimations of the horizontal wave load.

Further investigations need to be carried out on the vertical uplift forces under the crown slab, considering the vertical forces occurred during the recording of the maximum horizontal forces, and considering the overturning moment time series, to estimate the extreme design moment on the crown wall.

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