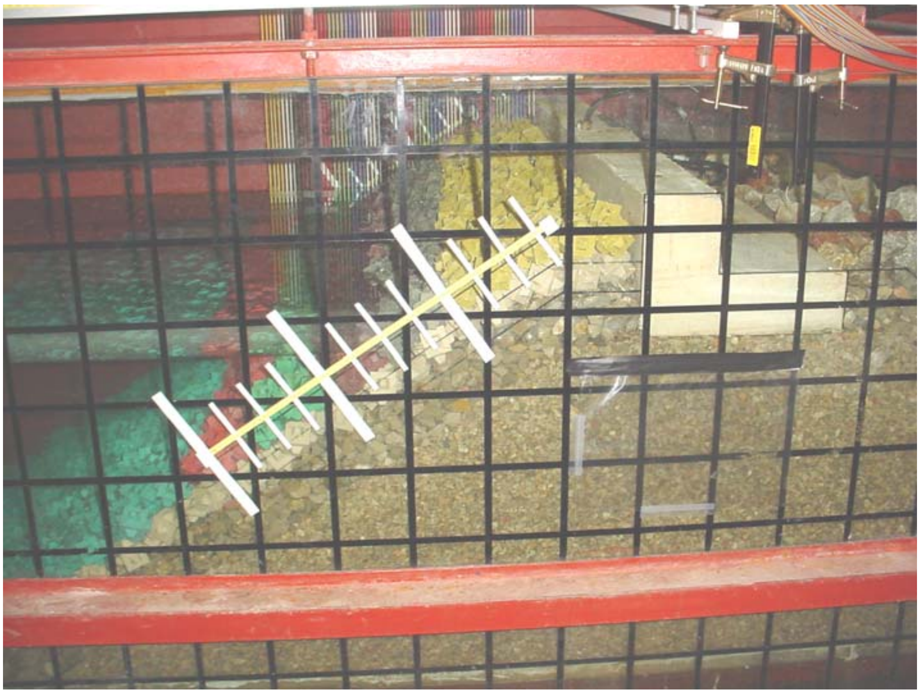
Stability of Crown Walls of Cube and Cubipod armored mound breakwaters



AUTHOR: Jorge Molines Llodrá

Laboratory of Ports and Coasts. Universidad Politécnica de Valencia

Laboratorio de Puertos y Costas LPC

Stability of Crown Walls of Cube and Cubipod armored mound breakwaters

J. Molines, Laboratory of Ports and Coasts, Universidad Politécnica de Valencia

Mound breakwaters usually have concrete crown walls to reduce overtopping. The stability of the crown wall is necessary to develop the different operations involved in port activities, sliding being the major failure mode. This paper focuses on sliding failure, using existing formulae to estimate wave forces on crown walls, e.g. Jensen (1984), Pedersen (1996), Martín et al. (1999), Berenguer and Baonza (2006), etc.

Physical model tests were carried out using two different armor units: Cubic blocks and Cubipods. The model was attacked with regular and irregular waves, measuring pressure values of the wave impacts. The analyzed formulae do not accurately represent the horizontal and up-lift forces at the same time, so a new method is proposed: estimating the maximum horizontal force and maximum up-lift force associated with the wave that generated the largest horizontal force.

After defining the variables that influence the phenomenon, test data were treated with pruned neural networks and statistical t-student analysis to obtain the new formulae to calculate the horizontal and up-lift forces. It was observed in the tests that these forces are most critical in more than 70% of the cases. The main advantages of this method are simplicity and robustness, because the formulae were obtained applying linear regressions.

Keywords: Crown Wall Forces, Crown Wall Stability, Breakwater, Crown Wall Sliding.

1.- Introduction.

Breakwaters can generally be divided into two types: vertical and mound breakwaters. The former reflect the wave energy, with the associated problems for navigation, but they are cheaper and more respectful of the environment than the mound ones. The latter absorb part of the wave energy, resisting wave action mainly by wave breaking.

Normally, mound breakwaters are crowned with a concrete superstructure resting on the mound layer, and are partially protected by the armor layer. The aim is to reduce the amount of concrete used in armor layers and increase the crest freeboard while decreasing cost. Crown walls are attacked by wave impact and earth armor-layer pressure, the former being the most important one due to the higher value.

Initially, the main layer of mound breakwaters consisted of quarrystone. As demand for space in ports grew, it became necessary to place the breakwaters deeper. At greater depths, no quarry could provide heavy enough quarrystone, so prefabricated elements of concrete such as the Cube, the Tetrapode (1950) or the Dolo (1963) appeared.

In a mound breakwater there are different types of failure modes, as shown in figure 1 by Bruun (1985): extraction of armor units, overtopping, erosion of the toe, etc. There are four common types of failure affecting the crown wall:

1. **Sliding** is the most common failure mode. It happens when the horizontal force is greater than the friction resistance, which can be altered by ascending pressures.
2. **Overturning** happens when the unstabilizing moments are larger than the stabilizing ones.
3. **Cracking** refers to the deterioration of the material throughout its lifetime.
4. **Geotechnical failure** is the failure of the foundations. It is caused when the load transmitted is higher than the load of collapse of the foundations.

Failures modes 2 (overturning) and 3 (cracking) are easily solved by the proper design of the crown wall geometry. Failure mode 4 (geotechnical failure) is related to the foundation soils rather than to the crown wall design; therefore sliding, is the most typical critical failure, since it requires building the crown wall with sufficient weight.

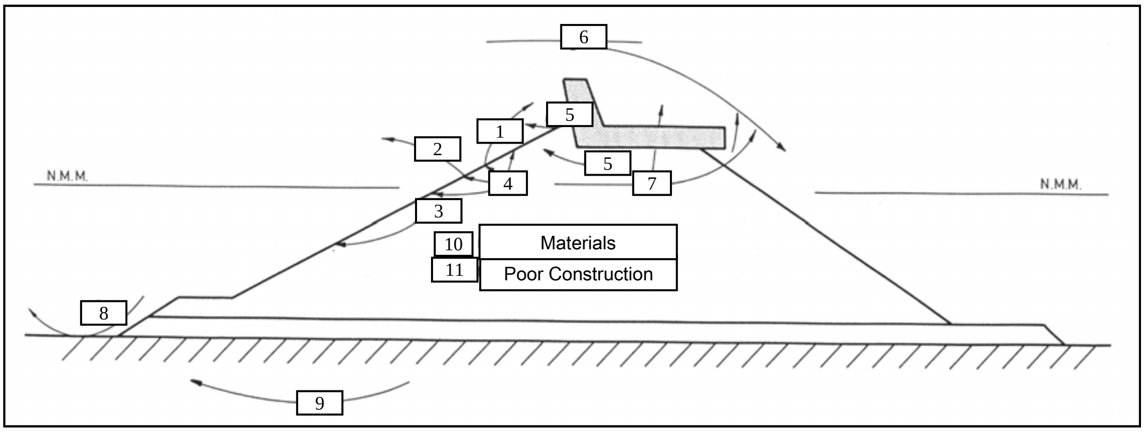


Figure 1. Failure modes of a mound breakwater (Bruun 1985).

Two processes help to understand the forces that act on a crown wall: run up and overtopping. Both phenomena are affected by the armor unit placed in the main layer; therefore, it is necessary to conduct further studies into the crown wall behavior with different armor units: Cubic blocks and Cubipods.

The Cubipod (figure 2) is a prefabricated concrete block designed to protect marine structures (Gómez-Martín and Medina 2007). The Cubipod overcomes the disadvantages of the cube while keeping its advantages: great structural resistance, simple manufacture and placement, flexible response to extreme storms, high hydraulic stability and avoiding the tendency face to face. The simple protrusion-faced design eliminates the problem of heterogeneous packing, increasing the friction with the secondary layer. In addition, thanks to these protrusions, Cubipod layers display a rougher surface than cube layers, offering greater resistance to water rise, while decreasing run-up, overtopping and forces on the crown wall.

Figure 2. The Cubipod.

Furthermore, the shape of the Cubipod allows for the use of constructive methods like those used in nowadays in cubes with minor modifications. Corredor et al. (2008) created a flanhole formwork allowing Cubipod manufacture with outputs similar to cubes. Formwork consists of two elements: a static base and a top formwork with six articulated elements to fill and vibrate in two phases which are removed after every 6 hours.

1.1.- Wave action on crown walls.

The action of waves on the crown wall of a mound breakwater is highly influenced by the process of transformation and breaking conditions of the rough slope. As the wave approaches the slope, the fast reduction of the depth and friction with the bottom makes waves increase in steepness, until they finally break.

If the waves break by plunging or collapsing, then a great amount of energy is dissipated in the slope, transmitting little energy to the crown wall. In addition, when a wave breaks over the porous slope, a certain amount of air becomes trapped in the wave in the form of bubbles. The irregularity of real swell and the theoretical complexity of these processes means that there is currently no precise and reliable analytical model to predict forces on crown walls. Therefore, to estimate the forces on crown walls, Froude similarity and physical model tests are used to obtain the empirically-related factors involved in the process and an empirical calculation method to estimate the forces.

The prediction models proposed over the years include:

1) Iribarren (1954) proposed triangular distributions (see Figure 3.a) for the dynamic and hydrostatic pressures, based on the maximum horizontal crest speed after the wave breaks on the slope.

2) Jensen (1984) studied the influence of the wave height, period and sea level. Jensen (1984) concluded that the influence of sea level variations can be expressed as the berm freeboard, and that the horizontal force is directly proportional to Hs/As. The wave period shows a clear trend: when the period increases, the forces increase too.

Jensens's formulae should only be used when the input parameters are very similar to those reported in Jensen (1984), limiting their application to situations of moderate overtopping.

Jensen (1984) did not propose a distribution of pressures, so it is not possible to get the unstabilizing moments, although overturning is not a critical failure mode.

3) Günback and Göcke (1984) proposed a method to calculate the pressures based on run up. They separated the action of the waves on the vertical wall into two simultaneous distributions: a hydrostatic one extended up to the end of the wedge run-up representing the mass of water that hits the wall, and a rectangular one associated with the kinetic energy of the wave (see Figure 3.b). They proposed a triangular distribution for the up-lift forces.

4) Bradbury (1988) investigated the influence of the slope on the loads over the superstructure, but did not drew clear conclusions about its influence. The results support those reported by Jensen (1984), i.e. proportionality between force and wave height and an increase in the forces with the wave period.

5) Hamilton and Hall (1992) conducted a parametric research through laboratory tests to determine crown wall stability when subjected to regular waves. Their main findings are:

- The increase in forces is directly proportional to wave height at moderate overtopping rates: from this point, the increase in forces decreases until approaching an horizontal asymptote.
- Forces increase with the period, but the authors do not provide clear conclusions.
- The smoother the slope, the lower the forces.
- Crown wall stability greatly decreases when placed just on the rip-rap (provided that the crown walls used in the tests conducted by Hamilton and Hall (1992) have a smooth base).
- The use of heels in the crown walls increases resistance to sliding compared to crown walls without heels; length is not relevant.

6) Pedersen and Burchart (1992) studied the influence of certain parameters on the stability of the crown wall. Their conclusions are similar to those of Hamilton and Hall (1992) and Jensen (1984):

- The higher the wave height, the higher the load on the crown wall.
- The longer the period, the stronger the actions on the crown wall.
- The Hs/As parameter displays a clear linear dependence with the force.
- Non-conclusive results are obtained regarding the influence of the berm width.
- Forces on crown walls depend on the area non-protected by the berm. When the height of the vertical wall is very high, a maximum value that depends only on the sea conditions and the sea level is reached.

7) Burchart (1993) presented a formula for force calculation based on the idea of Günback and Göcke (1984) extending the wedge run-up until the imaginary prolongation of the slope is reached. For simplicity, Burchart (1993) did not separate the force into an impulsive one and a hydrostatic one, but considered it as a fictitious hydrostatic force. Burchart (1993) concluded that the proposed distribution does not correctly simulate pressures in the area protected by the berm, overestimating the pressures on the base of the crown wall, so that the up-lift forces yield a very conservative value (see Figure 3.a).

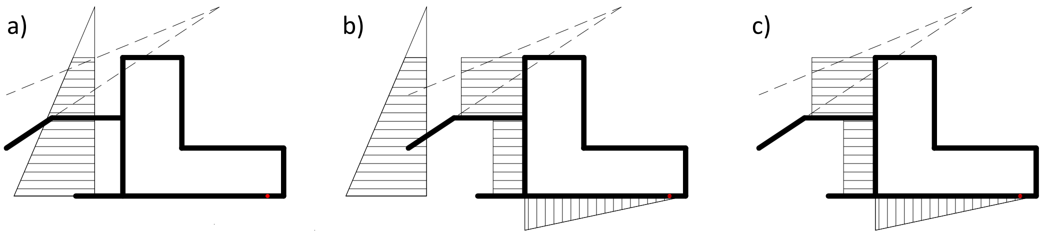


Figure 3. Pressure schemes by the different authors.

8) Martín et al. (1995) proposed a formula to calculate the forces in the case of regular waves. The method is applicable to those crown walls of mound breakwaters that are not affected by impact pressures, i.e. those in which the waves are broken or running up on the slope.

The proposed model is based on the appearance in the pressures laws of two out-of-phase peaks in time. The first peak is attributed to the horizontal deceleration of the water mass, while the second one is caused by the vertical acceleration when the accumulated water descends against the structure. Authors suggested two distributions for each pressure peak: for the first one they proposed an almost rectangular distribution, whereas for the second one they presented a nearly hydrostatic distribution (like those described by Günback and Göcke (1984), see figure 3.b).

For the up-lift pressures they proposed a triangular distribution according to the continuity of pressures; thus, the designed crown wall is on the safety side because it is supposed that wave impact occurs at the same time both on the vertical wall and on the crown wall base.

9) Pedersen (1996) reached the following conclusions:

- A linear dependency of the force with wave height exists if there is no overtopping. When overtopping begins, the force tends towards an asymptotic value.
- The force is greater with longer wave periods, assuming that the force-wavelength relation is linear.
- There is a clear linear dependency between the horizontal force and 1/Ac.
- There is a clear linear dependency between the horizontal force and 1/cot α .
- The three types of armor units placed randomly (cubes, quarystone and Dolos) show almost identical values for the horizontal force.
- When there is no overtopping, the crown wall height has no influence on the force. However, when overtopping exists, the observed forces are proportional to the square of the crown wall height.
- The influence of the berm width is not evident.

Pedersen's (1996) model of pressures, like in the dynamic distribution by Günback and Göcke (1984) and Martín et al. (1995), offered two rectangular distributions: one for the zone not protected by the berm and the other for the protected zone (see figure 3.c.). For the up-lift pressures, Pedersen (1996) proposed a triangular distribution that satisfies the pressure continuity.

10) Silva R. et al. (1998) extended Martín et al. (1995) method to irregular waves by the statistical characterization of the run-up, which fits very well to the run-up values by Van der Meer (1988).

11) Martín et al. (1999) introduced minor modifications to Martín et al. (1995), mainly in the run-up factor that directly affects the horizontal pressures and in the consideration of the up-lift pressures.

For the up-lift pressures, they proposed a trapezial distribution if the foundations are below the sea level, including the hydrostatic pressure corresponding to the foundation level.

12) Camus and Flores (2004) evaluate the formulae by Günback and Göcke (1984), Jensen - Bradbury (1984, 1988), Pedersen (1996) and Martín et al. (1999). They conclude that Pedersen (1996) method is the approach that best represents the maximum horizontal forces, whereas the methodology of Martín et al. (1999) best represents the physical phenomenon of wave impact on the crown wall.

13) Berenguer and Baonza (2006) presented a formula to calculate forces on the crown wall based on laboratory tests. This formula considers the influence of the damage level in the main layer on the wave impact intensity on the crown wall. They do not propose any distribution for the horizontal pressures, only a triangular distribution for the up-lift pressures.

2.- Experimental methodology.

2D physical model tests at a reference scale of 1:50 were carried out in the wind and wave flume of the Laboratory of Ports and Coasts at the *Universidad Politécnica de Valencia* (30x1.22x1.2 m), which is equipped with a horizontal-moved trowel impelled by a servovalve. Water depth varies according to the test, in this case 50 or 55 cm in the model zone.

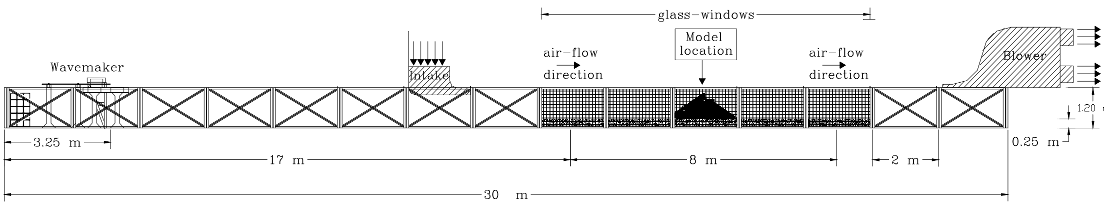


Figure 4. Longitudinal section of the wind and wave flume at the UPV. Levels in meters.

2.1.- Characteristics of the tested models.

The model consists of a mound breakwater under non-breaking conditions with a crown wall on the top, a slope 1/1.5 on the face exposed to waves, whose crest level was designed to avoid overtopping damaging the main layer. Several models were constructed with the same nucleus, filter and crown walls, but different armor units for the main layer: conventional double-layer cube armor of D_n=6 cm, double-layer Cubipod armor of D_n=3.82 cm and single-layer Cubipod armor of Dn = 3.82 cm.

Initially, 4 cm cubes and 3.82 cm Cubipods were to be tested, but the 4 cm cubes were too unstable when the waves produced overtoppin g.

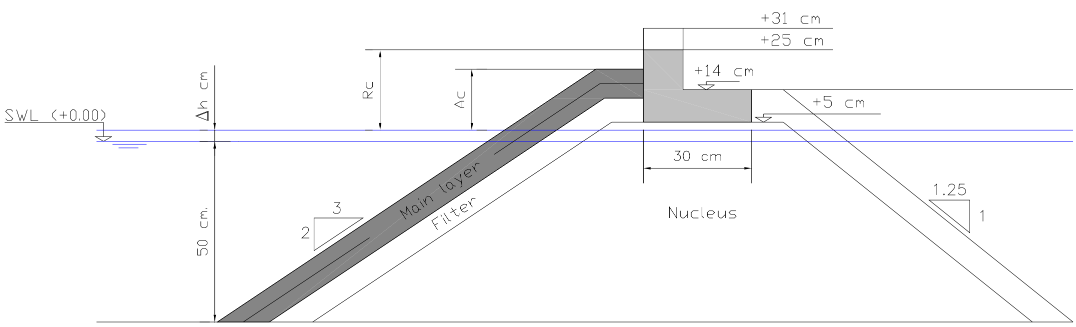


Figure 5. Section of the model tested.

2.2.- Constructive process and data acquisition.

First the channel walls are cleaned and the cross section of the model is drawn on both sides of the channel. Then, the nucleus is placed with the correct slope and crest level.

Next, on the top of the nucleus, the crown wall is placed and a filter constant thickness (about 6.7 cm) is also installed. In the protected area of the breakwater, heavy material is placed to avoid the washing of the filter and nucleus by overtopping.

Finally, the main layer is placed; it may consist of one or two layers, depending on the armor unit: Cubipods are placed in single and double layers whereas cubes are only placed in double layers. When the double-layer breakwater is tested, the lower one is white, whereas the upper one is divided into bands of colors to easily identify where a piece has been removed. The elements are placed randomly, simulating the real crane placement (not placing them in a certain position). As a consequence, the double-layers cube and Cubipod armored breakwater has a porosity of 37 % and the single-layer Cubipod armored breakwater has a porosity of 39 %.

In order to collect the necessary data for a complete analysis, 8 level sensors were used (separated according to Mansard and Funke's (1980) arrangement), 2 for run-up, one scale for overtopping and 7 pressure sensors (3 on the base and 4 on the vertical wall). The sampling of the gauges is 20 Hz, except for the scale, which is 5 Hz.

Incident and reflected wave separation was calculated using the LASA-V program (Figueres and Medina 2004). The LASA-V program determines the incident and reflected wave in the time domain using an approximated model of non-linear Sotkes-V wave through simulated annealing processes.

Ascendent or descendent trends were detected in the pressure sensors because the gauges were not able to return immediately to zero deformation after wave impact. To eliminate these tendencies, mobile averaging techniques were applied (usually applied in time series).

The constructed mound breakwater was attacked with regular and irregular waves of normal incidence maintaining an approximate Iribarren number (Ir=2.0, 2.5, 3.0, 3.5 and 4.0 were tested). During the tests, run up, overtopping, crown wall stability and main layer stability were recorded and analyzed, as the wave action on the crown wall was the main objective of the present study.

The objective of the regular tests was to understand the response of the model's section to incident waves, using the data obtained to better fit the wave height of zero damage of the irregular tests. The irregular tests were generated with 1000 waves and JONSWAP spectrum with a peak parameter of 3, simulating the real duration of extreme wave conditions.

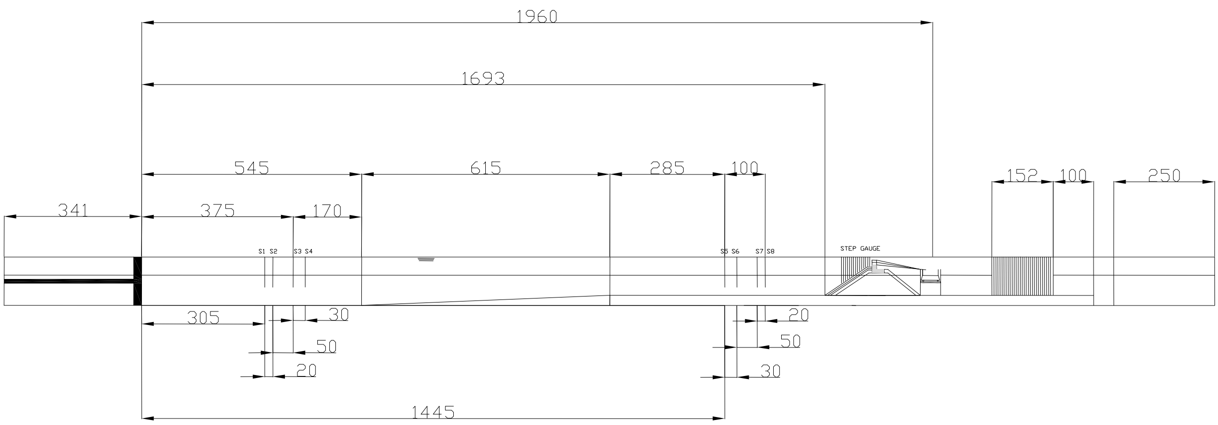


Figure 6. Scheme of the sensor distribution.

3.- Data analysis.

The different formulae were applied to the collected data to obtain the maximum amount of information possible from the tests (in some formulae not all conditions of application were fulfilled). Figure 12 shows all the relative mean square errors (RMSE (1)).

$$RMSE = \frac{1}{N} \cdot \sum_{i=1}^{N} \frac{(o_i - e_i)^2}{Var(o_i)} \tag{1}$$

None of the analyzed models satisfactorily represented both the horizontal and the up-lift forces; thus, a new methodology is proposed. First, the most relevant variables of the process are studied, and then the pertinent formulae are given using statistical studies and pruned neural networks.

These formulae estimate the maximum horizontal force (Fh) and the up-lift force (Fv(Fh)) associated with the wave that caused Fh (Figure 7). A statistical analysis of the sliding failure mode (the most frequent failure) was conducted, which showed that the combination (Fh, Fv(Fh)) is the most unfavorable situation in more than 70 % of the cases (Figure 8).

The crown wall was considered to fail when an event larger than the resistant force occurs, no matter the duration of the event, being on the safety side.

3.1.- Initial statistical analysis.

The formulae that were obtained can serve to estimate the maximum horizontal force (Fh) and the maximum up-lift force corresponding to the wave that generated the maximum horizontal force (Fv(Fh)). These actions, although separated some tenths of a second, were considered to occur at the same time on the safety side.

In order to know if the design using Fh and Fv(Fh) is correct, the crown wall behavior against sliding was studied (the most common failure in crown walls). This is:

- Force data were recorded every 0.05 seconds. Hence, the failure function can be calculated for every moment in time $S(t) = (W - Fv(t))\mu - Fh(t)$.
- For each test, S(t) values were ordered from greater to lower, thus obtaining the most unfavorable case of the test: the smallest S(t), from now on Sd.
- On the other hand, the experimental values of Fh and Fv(Fh) were obtained for each test, and the failure function $S_1 = (W - Fv(Fh))\mu - Fh$ could be calculated.

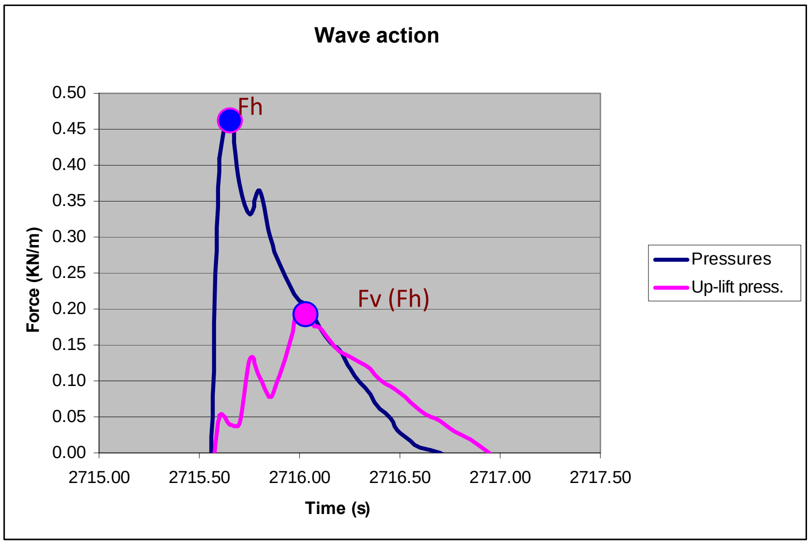


Figure 7. F h and Fv (Fh) values of the cube test; Hs=17 cm;low crown wall=20 cm; h=55 cm; Ir = 4.

- Therefore, the paramater $\frac{Sd - S_1}{Fh}$ can be obtained. If $\frac{Sd - S_1}{Fh} > 0$, then the (Fh, Fv(Fh)) estimator pair is on the security side, since $S_1 < Sd$.

The formulation of sliding failure mode is very sensitive to the μ value, a parameter that presents a great variability: Goda (1985) and the Japanese standards (BSI (1986)) suggest 0.6; ROM 0.5-2005 recommends a value of 0.7, etc. For this reason, different values were tested (0.5, 0.55, 0.6, 0.65 and 0.7) to cover the most typical range of values.

For each μ and armor unit (double-layer cube armor, double-layer Cubipod armor and single layer Cubipod armor), the best fitting probability density function was obtained (using EasyFit 4.3). In most cases Gumbell's distribution accurately represented the phenomenon (being a distribution widely used for the extremal characterization).

The table below (Figure 8) shows the probability for the different μ and elements to be over zero.

	Probability $\frac{Sd - S_1}{Fh}$ **>0**		
μ	**cb**	**cp1**	**cp2**
0.500	0.719	0.775	0.854
0.550	0.712	0.767	0.849
0.600	0.707	0.760	0.845
0.650	0.703	0.756	0.840
0.700	0.700	0.749	0.836

Figure 8. Probability that S1 is the most unfavorable case.

Since in at least 70 % of the cases the estimator S_1 is on the security side, it is reasonable to obtain the formulae that represent these forces.

3.2.- Significant variables.

The main variables that control the horizontal and up-lift force phenomena are shown in figure 9. These variables were obtained from the existing formulae.

- RU is the Ru/Rc variable. RU indicates whether overtopping exists or not and is related to the value of the run-up. RU also represents the higher level of water that reaches the crown wall.
- RA is the (Rc-Ac)/hf variable. RA represents the crown wall zone which is not protected by the berm versus the protected one. This variable only depends on the cross section geometry.
- WC is the Wc/hf variable. WC represents the distance between the foundations of the crown wall and the mean sea level. This variable depends only on the cross section geometry.
- γ_f is the overtopping roughness factor, which depends on the armor unit. Smolka (2008) obtained these factors in the overtopping formulation. This factor considers the roughness and permeability of the structure, and depends on the armor unit among other factors. This variable is introduced in RU, directly affecting the wave height of the run-up, common in overtopping and run-up models: γ_f RU.
- BL is the $\sqrt{\frac{L_{01}}{Ba}}$ variable. BL represents the berm width and time period, and it affects the water flow that arrives at the crown wall through the armor layer.

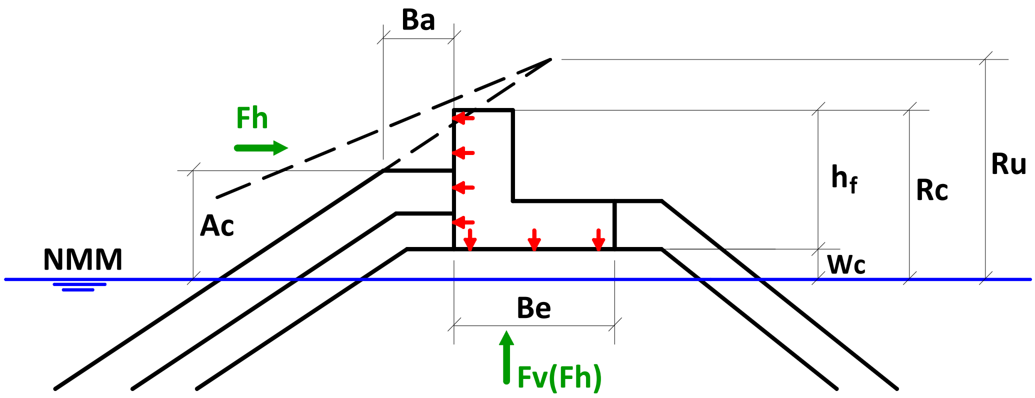


Figure 9. Significant horizontal and up-lift pressures variables. In red, pressure sensors.

The forces were made dimensionless as shown:

- Fh, dimensionless horizontal force: $\frac{Fh}{\rho \cdot g \cdot hf^2 \cdot 0.5}$
- Fv(Fh), dimensionless up-lift force: $\frac{Fv(Fh)}{\rho \cdot g \cdot hf \cdot Be \cdot 0.5}$, with Be being the base width of the crown wall.

The forces were calculated considering that the pressure at each point of the crown wall takes the value from the nearest pressure sensor. This calculation does not assume any specific pressure distribution, since there are as many rectangular distributions as pressure sensors.

3.3.- New formulae.

A linear dependence of some of the variables was observed, so the initial linear formula for Fh was:

$$FhRL = a1 + b1 \cdot \gamma_f \cdot RU + c1 \cdot RA + d1 \cdot BL + e1 \cdot WC \tag{2}$$

Model (2) was tested with t-student analysis (with a 0.05 alpha) to eliminate variables (eliminating WC), used later as input in a pruned neural network analysis (Medina et al. (2002)). Neuroport 2.1 eliminated variables RA, BL and $\gamma_f \cdot RU$, leaving only FhRL as a significant variable transformed by two hidden neurons. Using simulations, a quadratic relationship between FhRL and the prediction of the neural network was observed. Therefore, the phenomenon was fitted by the square root of the dimensionless data, eliminating the quadratic tendency observed. Thus, after the pruned neural network model, a new statistical analysis was conducted using XL-STAT with the form (3):

$$\sqrt{FhRL} = a2 + b2 \cdot \gamma_f \cdot RU + c2 \cdot RA + d2 \cdot BL \tag{3}$$

The up-lift force process was similar to that of Fh, with the same problems (quadratic relationship) and also solved using pruned neural network models. The main difference is that Fv(Fh) depends on all the input variables, including WC.

The final formulae corresponding to the central estimation are the following:

$$Fh = \rho \cdot g \cdot hf^2 \cdot 0.5 \cdot \left(-1.25 + 1.80 \cdot \frac{\gamma_f \cdot Ru}{Rc} + 0.82 \cdot \left(\frac{Rc - Ac}{h_f} \right) + 0.16 \cdot \sqrt{\frac{L_{01}}{Ba}} \right)^2 \tag{4}$$

$$Fv(Fh) = \rho \cdot g \cdot hf \cdot Be \cdot 0.5 \cdot \left(-0.6 + 0.40 \cdot \frac{\gamma_f \cdot Ru}{Rc} + 0.27 \cdot \left(\frac{Rc - Ac}{h_f} \right) + 0.16 \cdot \sqrt{\frac{L_{01}}{Ba}} - 1.03 \cdot \frac{Wc}{h_f} \right)^2 \tag{5}$$

Where:

$$Ru = R_{u0.1\%} = \begin{cases} 1.12 H_s \xi_m & \xi_m \le 1.5 \\ 1.34 H_s \xi_m^{0.55} & \xi_m > 1.5 \end{cases} \quad \text{with } \xi_m = \tan\alpha / \sqrt{H_s / L_{01}} \qquad (6)$$

Being limited to $Ru = 2.48 \cdot H_s$ for permeable structures (CIRIA/CUR 1991)

Rc Crest freeboard of the crown wall in meters.

Ac Crest freeboard of the berm in meters.

γ_f Roughness factor. $\gamma_f = 0.5$ (cb), $\gamma_f = 0.44$ (cp2), $\gamma_f = 0.46$ (cp1)

Ba Berm crown width in meters.

$L_{01} = \dfrac{g \cdot T_{01}^{\,2}}{2\pi}$ in meters.

Hs Significant wave height at the breakwater toe in meters.

$\tan\alpha$ Slope.

Wc Foundation level of the crown wall in meters.

Be Base width of the crown wall in meters.

hf Crown wall height in meters.

Fh Maximum horizontal force in N/m.

Fv Maximum up-lift force associated with the wave that has generated the maximum horizontal force in N/m.

ρ Water density in kg/m^3.

g Gravity acceleration in m/s^2.

The application range for the variables is shown in Figure 10

Application range		
0.3001	<γ_fRU<	0.9605
0.0665	<RA<	0.5890
0.0127	<WC<	0.2665
3.1342	<BL<	6.5938

Figure 10. Application range of the equations.

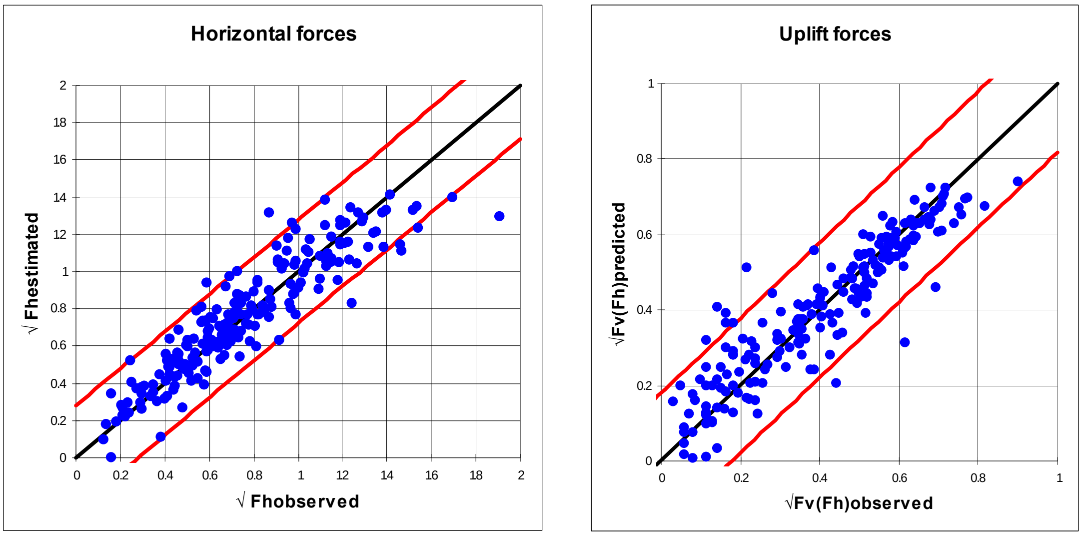


Figure 11. Comparison between dimensionless forces estimated by the formulae and the forces observed in the laboratory tests. In red, 95 % bands of confidence.

RMSE (1) associated with the square root fitting (Figure 11) is 0.16 for Fh (r^2 = 0.84) and 0.14 for Fv (r^2 = 0.86).

Relative mean square errors for each of the non-dimensionless formulae is given in Figure 12 below:

Horizontal forces		Up-lift forces	
Author	RMSE	Author	RMSE
Pedersen (1996)	0.207	Molines (2010)	0.171
Molines (2010)	0.215	Jensen (1984)	0.510
Jensen (1984)	0.320	Berenguer and Baonza (2006)	0.516
Martín et al. (1999)	0.328	Martín et al. (1999)	1.034
Günback (1985)	0.651	Pedersen (1996)	1.189
Burchart (1993)	0.763	Günback (1985)	2.031
Berenguer and Baonza (2006)	1.189	Burchart (1993)	35.735

Figure 12. Relative mean square error for each one of the formulae.

4.- Example of application.

Figure 13 shows the application of the formulae in one laboratory test, which compares the results to those obtained with the formulae reviewed in the literature. The model consists of a cube armored breakwater, high crown wall (hf = 0.26 m), water level (h=0.55 m), Ir=3, Hs = 0.16 m, Rc = 0.2633 m, Ac = 0.19 m, $\gamma_f = 0.5$, Ba = 0.12 m, L_{01} = 3.396 m, Wc = 0.0033 m and Be = 0.3 m. The experimental forces calculated from the test pressure records were Fhexperimental = 257 N/m and Fvexperimental = 123 N/m.

Replacing the data in formulae (4) and (5), the results are Fh = 290 N/m and Fv (Fh) = 122 N/m. In figure 13, a summary of all results obtained using the different authors' formulae is shown. Formulae IV, V and VI were applied despite their not fulfilling all the application conditions, just to check if some additional information about the phenomenon could be deduced.

The forces to compare are the same in all the cases, because for tests of 1000 waves, $Fh_{0.1\%}$ = $Fh_{max,,}$ being these values the ones estimated by all the authors. Estimated up-lift pressure value in all the cases is the one obtained by the pressure law continuity at the crown wall base.

	Fh (N/m)	Fv (Fh) (N/m)
Test	257	123
I. Pedersen (1996)	216	195
II. Molines (2010)	290	122
III. Jensen (1984)	202	116
IV. Martín et al. (1999)	95	130
V. Günback (1985)	214	4
VI. Burchart (1993)	150	314
VII. Berenguer and Baonza (2006)	151	150

Figure 13. Application of the different formulae to a given test.

Formulae I, II and III show similar values, the formula developed in this investigation being the one that best fits the up-lift pressures.

It should be taken into account that each formula (including the one developed in this paper) was designed for a given test and therefore present the intrinsic errors associated with an empirical method. All of them should be applied carefully considering the limitations in their application.

5.- Conclusions.

The formulae developed in this paper are easily applied, in comparison with the other existing formulae. The text below describes the logical process to calculate the crown wall using the proposed formulae:

- From overtopping limitations, Ac and Rc are obtained.
- Crown wall height (hf), base width (Be) and foundations level (Wc) are proposed.
- With the previous values and wave characteristics at the breakwater toe, horizontal and up-lift forces can be calculated using the formulae developed in this paper.
- Active and passive earth pressure of the armor units adjacent to the crown wall should be calculated using the procedure presented in ROM 0.5-2005.
- Once the forces are calculated, the minimum weight of the crown wall can be obtained using ROM 0.5-2005, limiting the sliding security coefficient to 1.5:

$$CSD = \frac{(Crown\ wall\ weight - \Sigma Up - lift\ forces)\mu}{\Sigma Horizontal\ forces} = 1.5 \tag{7}$$

- Once the minimum weight calculated, the crown wall must be checked to make sure it is possible to achieve this weight with the proposed base and height dimensions. If the minimum weight cannot be obtained, the calculation process should be repeated changing Be and hf values.

Finally, the load transmitted from the crown wall to the foundations should be checked to make sure it does not surpass the bearing capacity obtained with Brinch Hansen's formula (ROM 0.5-2005). Shear stress should not exceed material resistance in the concrete joint according to article 47.2 EHE (2008).

The new methodology presented in this paper is simple and was obtained using irregular laboratory tests by means of linear regressions: therefore, it is a robust method that considers the group waves effect.

Cube-Cubipod comparison shows that Cubipod armored mound breakwaters present lower forces (and therefore a smaller crown wall size) than the cube armored ones. The result is the lower cost of the Cubipod armored breakwater because of the smaller amount of concrete used.

Future research will focus on the study of the cyclical forces that affect the crown wall foundations as a consequence of the sine-crest wave action. This issue should be addressed through numerical models that simulate the whole crown wall-soil system, using the necessary input parameters.

The obtained formulae allow for the complete characterization of sliding failure, although the pressure distribution is not defined. The distribution most similar to our tests is that presented by Pedersen (1996) (Figure 3.c). Although the critical type of failure is totally defined with Fh and Fv(Fh), it is interesting to define the pressure distribution to characterize the stability of the whole crown wall.

All the exisiting formulae (including the one developed in this paper) should be applied carefully considering the limitations on their application.

Pressure distribution is currently one of our research topics, as well as the use of pruned neural network models to improve crown wall design considering wave and geometric conditions.

6.- References.

Berenguer, J.M. and Baonza, A. (2006). Diseño del espaldón de los diques rompeolas. Libro de ponencias del *II Congreso Nacional de la Asociación Técnica de Puertos y Costas, pp. 35-56 (in Spanish).*

Bradbury, A.P., Allsop, N.W.H. and Stevens R.V. (1988*).* *Hydraulic performance of breakwater crown walls*. Technical Report SR 146, Hydraulics research, Wallingford, UK.

British Standards Institution (1986). BS 8004: Code of practice for foundations. BSI, UK.

Bruun, P. (1985). *Design and Construction of Mounds for Breakwaters and Coastal Protection.* Elsevier Science Publishing Company Inc, Amsterdam, The Netherlands.

Camus, P. and Flores, J. (2004). Wave forces on crown walls: Evaluation of existing empirical formulae*. Proc. 29th ICCE, pp. 4087-4099*.

CIRIA/CUR (1991). *Manual on the use of rock in coastal and shoreline engineering.* The Netherlands. CUR, London, UK, CIRIA.

Corredor, A., Torres, R., Miñana, J.V., Fernández, E., Menéndez, C.F., Santos, M., Gómez-Martín, M.E., Goumy, R. and Medina, J.R. (2008). CUBÍPODO: Ensayos de estabilidad hidráulica 2D y 3D, estudio del remonte y rebase, diseño del encofrado y ensayos de caída de prototipos. Libro de ponencias del *III Congreso Nacional de la Asociación Técnica de Puertos y Costas, pp. 187-211.*

Figueres, M. and Medina, J.R. (2004). Estimating incident and reflected waves using a fully nonlinear wave model . *Proc. 29th ICCE, pp. 594-603.*

Goda, Y. (1985). *Random seas and design of maritime structures.* University of Tokyo Press.

Gómez-Martín, M.E. and Medina, J.R. (2007). Cubipod concrete armor unit and heterogeneous packing. *Proc. Coastal Structures'07*, World Scientific Publishing, 140-151.

Günback, A.R. and Göcke, T. (1984). Wave screen stability of rubble mound Breakwaters. Int. Symposium of Maritime Structure in the Meditarrean Sea. Athens, Greece, pp. 2099-2112.

Hamilton, D.G. and Hall K.R. (1992). Preliminary analysis of the stability of rubble mound breakwater crown walls. *Proc. 23rd ICCE, pp. 1217-1230.*

Jensen, O. J. (1984). *A monograph of rubble mound breakwaters*. Danish Hydraulic Institute, Hórsholm, Denmark.

Mansard, R. and Funke, E. (1980). The Measurement of Incident and Reflected Spectra Using a Least Squares Method. *Proc.17th ICCE.*

Martín, F. L., Vidal, C., Losada, M. A. and Medina, R. (1995). Un método para el cálculo de las acciones del oleaje sobre los espaldones de los diques rompeolas. *Ingeniería del agua, Vol. 2, Núm. 3, pp. 37-52.*

Martín, F. L., Losada, M. A. and Medina, R. (1999). Wave loads on rubble mound breakwater crown walls. *Coastal Engineering Vol. 37, pp. 149-174.*

Medina,J.R., González-Escrivá, J.A., Garrido, J.M. and De Rouk, J. (2002). Overtopping analysis using neural networks. *Proceedings ICCE 2002, ASCE, 2165-2177.*

Pedersen, J. and Burchart, H.F. (1992). Wave Forces on Crown Walls. *Proc. 23rd ICCE. Pp. 1489-1502.*

Pedersen, J. (1996). Wave Forces and Overtopping on Crown Walls of Rubble Mound Brakwaters. Series Paper 12 / Hydraulic & Coastal Engineering Laboratory, Department of Civil Engineering, University of Aalborg, Denmark.

Puertos del Estado (2005). Recomendaciones para Obras Marítimas. ROM 0.5. Recomendaciones geotécnicas en el proyecto de Obras Marítimas y Portuarias. Programa ROM. MOPU.

Silva, R., Govaere, G. and Martín F. (1998). A statistical tool for breakwater design. *Proc. 26th ICCE, pp. 1921-1933.*

Smolka, E. (2008). *Estudio Experimental del Remonte y Rebase de un dique en talud de Cubípodos: Aplicación al Nuevo Dique de Abrigo de Punta Langosteira (A Coruña)*. Proyecto Fin de Carrera de la ETSICCP de Valencia, Sep. 2008.