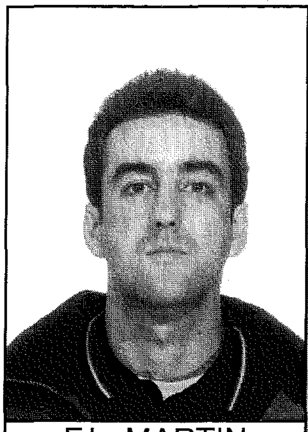

F.L. MARTIN

EXPERIMENTAL STUDY OF WAVE FORCES ON RUBBLE MOUND BREAKWATER CROWN WALLS

by

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Abstract

This paper has two main objectives: (1) to describe the experimental work carried out in order to verify the theoretical method from Martin (1995) for the calculation of wave forces on crown walls and (2) to show some results from field and lab measurements and compare these data to calculations from several analytical methods.

The Principe de Asturias Breakwater at Port of Gijon has been taken as the reference structure in this study. The experimental work (prototype and scale models) has been carried out over the same cross section, corresponding to this breakwater. Three scales have been used here: scale 1:1 (prototype measurements), scale 1:18.4 (tests done at Laboratorio de Ingeniería Marítima, UPC, Barcelona) and scale 1:90 (tests done at Laboratorio de Ingeniería Oceanográfica, Universidad de Cantabria). Therefore, data from the same phenomena in three different scales are available, which will provide the basis to analyse scale effects in the lab.

The main part of the experimental work was carried out from 1995 to 1998. Due to the large costs of such a long experimental project, several organisations (referred in the acknowledgements section) were asked for financial support to the study. This is a good example of long-term project which was possible by the joint effort of several institutions (public and private) within the European Research Framework.

In the paper, forces from the tests are compared to calculations done from the method proposed by Martín (1995). The comparison shows good agreement between the calculations and the measurements from the lab, and not-so-good agreement to prototype data. From these results, it can be stated that the method is working well as it was developed from lab data. From this study, it can be stated that differences to prototype forces are due to scale effects between lab and prototype measurements.

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1. INTRODUCTION

Most of the rubble-mound breakwaters have a crown wall on their top. These superstructures may help to control wave overtopping and to limit the height of the main layer. Moreover, they may provide access to the breakwater and give support and protection to wiring and pipelines along the breakwater crest. There are few methods to evaluate wave forces on wave screens: Iribarren et al., 1964, Günbak et al., 1984, Jensen, 1984, revisited by Bradbury et al., 1988, and Pedersen et al.,1992. None of the previous methods introduce as a calculation parameter the characteristics of the rubble mound berm (geometry and permeability), when tests reveal the influence of these parameters on the resulting pressure distributions.

Other engineers consider physical modelling as the unique reliable method. The Spanish experience is that the wave screens may withstand without failure higher waves than expected by using available engineering methods.

Generally speaking, the response of the built wave screens reveals that most of the calculating methods available overpredict the wave induced forces, with the related influence on the construction costs. Therefore, it is clear that a deeper study of these forces was needed. The Ocean and Coastal Research Group of University of Cantabria has been working for several years in the conceptualisation of the process and the study of the procedures of momentum transfer between a bore and a vertical surface. Finally, Martin (1995) developed a new method for the calculation of the pressure profiles acting in the wave screen front and base due to bores hitting the superstructure in the run-up process. This method has some empirical parameters which were experimentally evaluated from lab tests. Once the method was completed, it was necessary to verify it by comparison to different data sets done at different laboratories on different scales. Thus, reviewing the available data sets, it was clear that additional tests were needed. Therefore, the cross section of Principe de Asturias breakwater has been tested in three different scales: prototype (1:1), medium scale (1:18.4) and a small scale (1:90), see Figure 1.

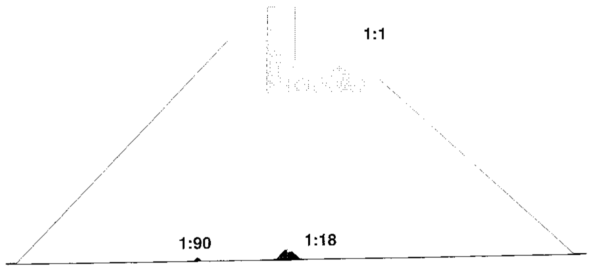


Figure 1- The three scales tested

In this paper the reader will find a brief description of the field campaign, the lab tests and the engineering methods employed in the comparison, then a qualitative check of the hypothesis employed in the calculation methods by analysing the measurements and finally a quantitative comparison of results.

2. ABSTRACT OF THE METHOD PROPOSED BY MARTIN (1995, 1999)

The variability of broken wave-induced forces on crown walls is rather less than of impact events on vertical breakwaters. Even for monochromatic tests where impacts on vertical breakwaters show quite significant variations, broken waves on crown walls show relative consistent responses. Thus, a deterministic approach to the wave-to-force transfer function can be taken.

For design purposes, the calculation sea state must be defined by the significant wave height, Hs, peak period, Tp and duration, at the toe of the breakwater. Then, the calculation wave height, Hc, must be determined. For preliminary design, it is recommended to use Hc = H99.8%. If H99.8% can not be determined from the wave distribution in the calculation sea state, Hc 1.8 Hs can be accepted (it must be verified that Hc is compatible with the calculation water depth).

Next, it must be verified that waves can not plunge directly to the vertical structure generating shock pressures. Only surging, collapsing or broken waves are considered in this method. The criterion for deciding whether the method can be applied is the occurrence of shock impact events. Once the wave height (Hc) and wave period (Tp) are known, the impact event due to this wave does not occur, in the following cases :

1. If the design wave breaks before reaching the breakwater toe.

2. If Ir > 3, where

   $Ir = \tan \alpha / S_c^{1/2}$
   $S_c = H_c / L_{po}$

   where L_{po} is the peak wave length in deep water and is the breakwater slope angle.
   In these cases, the wave breaks on the breakwater slope as a collapsing or surging breaker.

3. For other cases a simple method by Martin (1995) (Figure 2) identifies the regions of shock impact and non-impact events as a function of relativeberm width (B_b/H_c) and relative berm crest height (A_c/H_c), where B_b is the berm length and A_c is the berm crest height, above design sea level.

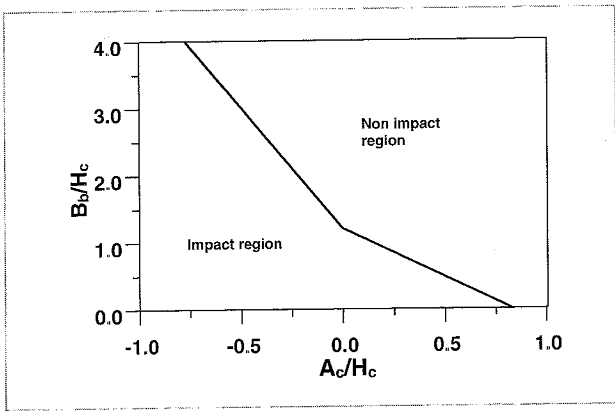


Figure 2 - Definition of shock impact and non-impact regions (empirical)

It has been noted that a single wave may generate two peaks of force on the vertical structure (see Fig. 3). For wave impact events, the initial peak (impact force) is always larger than the second peak (pulsating force). For crown walls and other walls subjected to broken waves, whether the impact force is larger than the pulsating force depends on the wave and armour characteristics. Thus, the engineer must consider both load situations in the analysis, and select as design load the one which produces the lowest safety factor.

Simplified pressure distributions corresponding to the two load situations due to the calculation wave (defined by wave height, Hc, peak wave period, T_p, and water depth, h) are shown in Figure 4. A detailed description of the method and its basis can be found in Martin (1995) and Martin et al. (1999).

a) Impact pressures.

The pressure distribution in this loading case is determined by P_{so} and C_{w2}. Over the unprotected region of the crown wall (above A_c) the pressure is :

$$P_i(z) = P_{so} = C_{w1}\ \rho_w\, g\, S_o \qquad A_c + S_o > z > A_c$$

$$C_{w1} = 2.9\ [(R_u/H_c) \cos \alpha\ ]^2$$

$$S_o = H_c\ (1 - A_c/R_u)$$

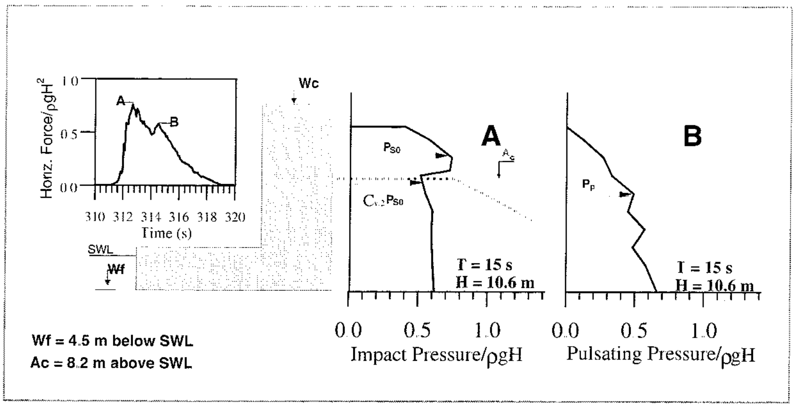


Figure 3 - Vertical distribution of pressures (empirical)

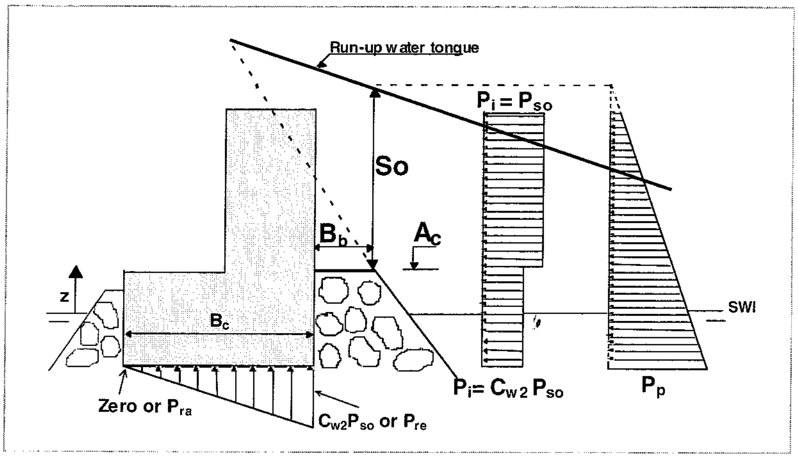


Figure 4 - Pressure distributions, impact and pulsating

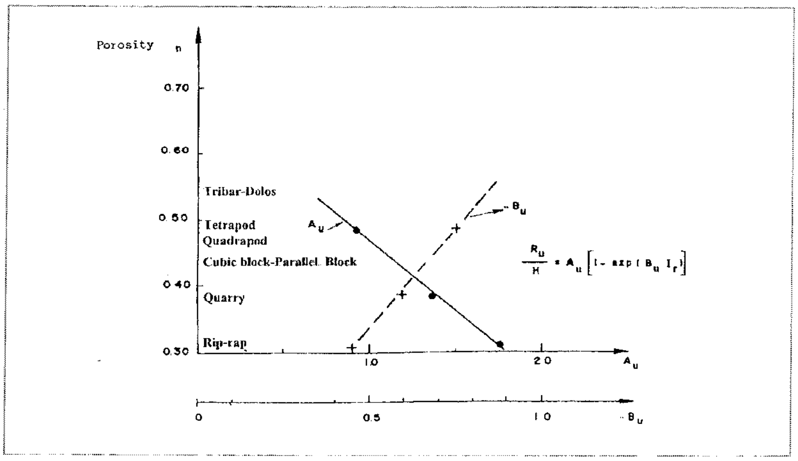


Figure 5 - Run-up parameters A_u and B_u (after Losada, 1992)

where :

z is the vertical coordinate, referred to a design SWL, positive upwards;
R_u is the run-up height of the calculation wave (H_c, T_p) in a straight-infinite slope;
A_c is the level of the armour berm above the design SWL;
ρ_w is the water density;
g is gravitational acceleration.

To calculate R_u, Losada et al. (1981), based on experimental work under monochromatic waves and normal incidence, proposed the following expression for R_u on an infinite slope :

$$R_u/H_c = A_u \left[1 - \exp(- B_u \, Ir) \right]$$

where, A_u and B_u (Fig. 5) are experimental coefficients, which depend on the type of armour unit and the Iribarren Number.

Over the region of the crown wall protected by the armour berm, the pressure distribution is:

$$P_i(z) = C_{w2} \, P_{so} = C_{w1} \, C_{w2} \, \rho_w \, g \, S_o$$
$$w_f < z < A_c$$

where w_f is the foundation level of the crown wall (above SWL) and C_{w2} is an empirical non-dimensional parameter calculated for $0.03 < H_c/L_p < 0.075$, given by :

$$C_{w2} = 0.8 \exp(-10.9 \, B_b/L_p)$$

where B_b is the armour berm width at A_c level and Lp is the local peak wave length.

b) Pulsating pressures.

The pressure distribution in this case is determined by :

$$P_p(z) = C_{w3} \, \rho_w \, g \, (S_o + A_c - z)$$

where C_{w3} is a non-dimensional parameter evaluated empirically from monochromatic wave tests as :

$$C_{w3} = a \exp(Co)$$
$$Co = c \, (H_c/L_p - b)^2 \quad (0.03 < H_c/L_p < 0.075)$$

Table 1 - Fitting coefficients for a, b and c

B_b/D_{n50}	a	b	c
1	0.446	0.068	259.0
2	0.362	0.069	357.1
3	0.296	0.073	383.1

where D_{n50} is the equivalent size of the armour units forming the berm.

c) Uplift pressures

The following values are adopted:

Seaward edge :

impact pressure $=C_{w2}\ P_{SO}$
pulsating pressure $= P_p(z = w_f) = P_{re}$

Heel :
negligible impact pressure, $P_{ra}= 0$
pulsating pressure $= P_{ra}$, from Fig. 6.

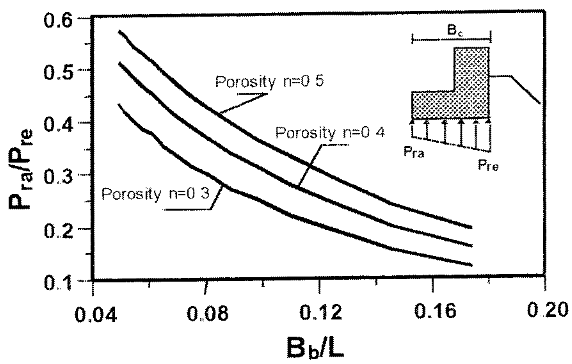


Figure 6

where B_c is the width of the crown wall foundation. For design purposes the porosity selected must represent the porosity of the material on which the crown wall is founded. A linear law between the seaward edge and the heel is proposed.

3. DESCRIPTION OF EXPERIMENTAL SETUP

3.1. Scale 1:1. Prototype measurements

3.1.1. The Principe de Asturias breakwater

Gijón is located at the Bay of Biscay, in the north of Spain. It is exposed to storms from N-NW, which are the most severe sea states in that zone. The 100-year return period significant wave is 10.5 m which can lead to wave heights greater than 18 m. The Principe de Asturias breakwater is the main protective structure of El Musel Port, located at Gijon harbour (see Fig. 7). It was built in successive steps, from 1892 to 1976. The section instrumented (indicated in the figure) was built in 1976.

The total length of the breakwater is 1,750 m, and it provides protection to the commercial and the fishing ports. The plan view clearly shows the three different parts of the breakwater built in 1890, 1950 and 1976, respectively. Together with the progressive increment of commercial traffic and the evolution of the vessels along the XX century grew the need of larger depths, more mooring lines and additional land area. Therefore, the extension of El Musel Port had to increase extraordinarily throughout the century. This Port was formerly settled in its present location because of the protection of Cape Torres, which defended the Port from NW storms. After the first enlargement of the harbour (1880), and due to the requirements of the vessels and activities developed in the Port, the protective effect of Cape Torres was insufficient. Thus, the first part of the Principe de Asturias breakwater was built as additional protection to the harbour area (1892). The increment of mining activities in Asturias during the forties and the fifties originated the need of larger vessels (larger depth) and resulted into the building of the second part of the breakwater. This part provided protection to the new mooring lines installed in the southern part of the harbour, reaching depths up to 10.0 m. Finally, in the former seventies, the need of even larger depths and more land area resulted into the building of the last part of the breakwater, reaching depths of 20 m. This is the part of the breakwater where the sensors are installed.

In this part of the breakwater, the core is built of parallelepipedic blocks of 90 tons while the armour is built of 120-ton parrallelepipedic blocks. The crown wall base level is 0.0 m over the low tide level (zero datum), the level of the rubble berm is +13.5 m (+12.2 m before 1995) and the level of the crown wall top is +18.35 m. The width of the crown wall is 18.72 m and the berm width is 3.75 m, which means that the berm is built of one unit of 120 tons. The armour layer slope is 1:1.5. The water depth at the breakwater toe is 21.0 m at LLWL, and the maximum tidal range in the area is 4.5 m.

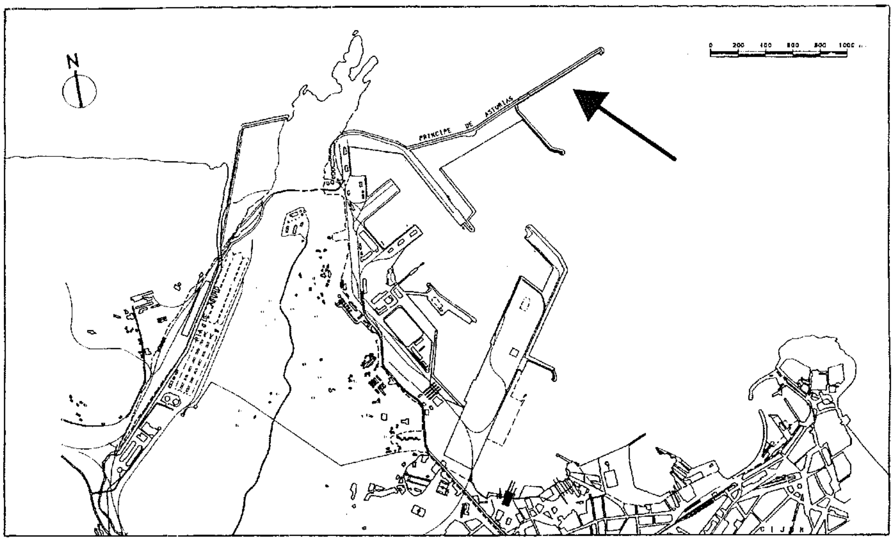
3.1.2. Instrumentation

Figure 8 shows the arrangement of the equipment installed in the Principe de Asturias breakwater.

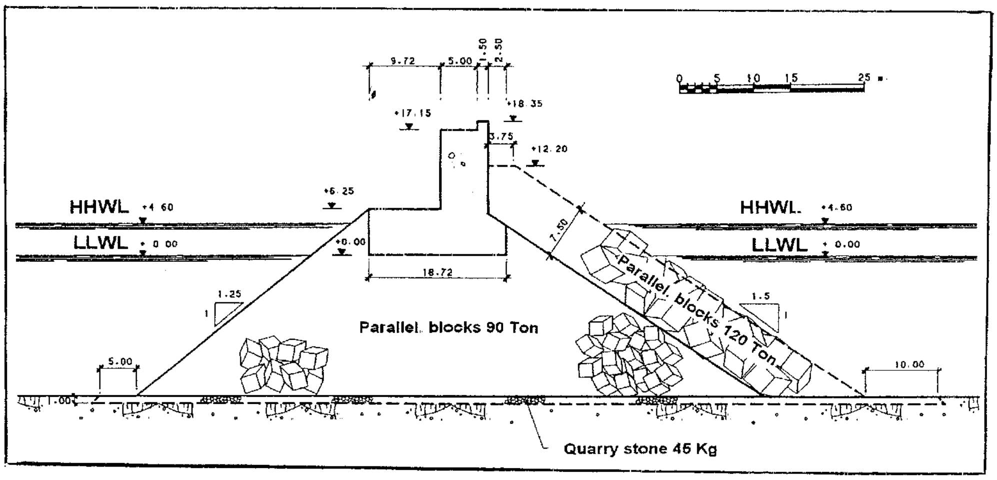
To know the incident wave climate, three wave recorders (W1-W3) were installed in front of the breakwater to be able to separate the incident and the reflected wave trains. One directional wave recorder (W4) is placed at the leeside of the instrumented section to identify the transmitted energy across the breakwater and the diffracted energy around the breakwater head. Moreover, there are two wave riders installed close to the breakwater by Puertos del Estado (Ministry of Public Works) continuously recording wave heights and periods.

In order to study wave forces on the vertical face of the crown wall, five specially designed pressure sensors were placed in the wall front (P1-P5). These sensors were installed in a vertical line, between contour elevation 7.2 and +16.0. Given that the pressure measured by the sensor could be affected by perturbances introduced by an intrusive sensor, the sensors were placed in such a way to prevent any alteration of the original vertical surface (flushing surface) and therefore, they would not affect the flow.

The knowledge of uplift forces below the breakwater is of vital importance in order to establish the net forces acting on the superstructure. In the case of the Principe de Asturias breakwater, the nature of the core (90 ton blocks) makes a detailed study of uplift pressures particularly necessary. Therefore, three pressure cells were drilled across the wall base up to the foundation level to record the uplift pressures (S1- S3) (Fig. 7b).



Plan view



Cross section

Figure 7a

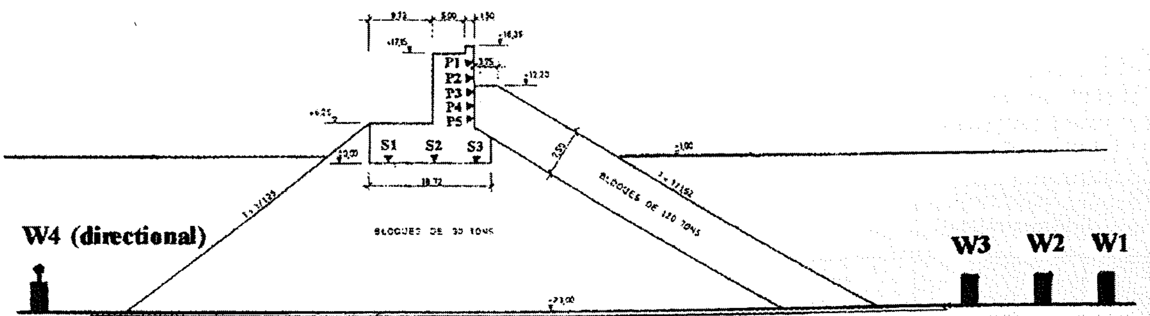


Figure 7b - Instumentation setup

The sampling rate of pressures in the crown wall was 20 Hz, which is enough to record the main characteristics of interest of the pressure time series being researched. The system was continuously logging data in 45-min. bursts. After the burst finished, the system checked whether a given threshold of pressures was exceeded. If so, the data were stored and if not, they were deleted. The sampling rate for the wave recorders was 0.5 Hz.

Additional data about prototype instrumentation can be found in Martin et al. (1997).

The system was set up in 1995 and was continuously working until April 1998.

3.1.3. Data processing

Once the data are collected, they must be processed in order to be easily analysed and stored. First, from the complete data set must be selected the data interval with the representative information. It can be said that, in this instrumentation, the data selection is done by the acquisition system itself, as the data are stored when the signal exceeds a given threshold level.

Generally speaking, there are two main ways for time series analysis: the statistical analysis and the spectral analysis. The former way takes into account the effect of each single occurrence of the physical process while the latter provides the overall description of the process. For the study of the transmission and reflection of energy induced by the breakwater, the latter way is preferred while for the study of pressures and forces on the wave screen the former is selected, analysing occurrence to occurrence.

From the data recorded by the wave recorders located in front of the breakwater, and by using a modified Mansard and Funke method (1980), the incident and reflected wave trains are separated. The numerical method provides the incident and reflected spectra and the time series of both wave trains.

From the data collected by the pressure gages, the pressure profiles in the wave screen front face and in the base are calculated. By integrating the pressure profiles (trapezoidal integration), the total force acting in the front face and the uplift force are straightforwardly calculated.

Once the wave data and the pressure data are analysed, the incident wave characteristics and the related pressures on the superstructure are known. The wave field is defined by the spectrum and the duration. The pressure of force data are defined by its statistical distribution. As the time series of pressure are always 45 min. long, the output of the prototype measurements are the coupled pairs wave spectra (45 min. duration) - force distribution, for a given tidal level.

Date	Significant Wave height	Peak period	Tidal range
16//2/95	5.9 m	20 s	4.1 m
10/2/96	6.0 m	19 s	3.9 m
19/2/96	5.5 m	16 s	4.3 m
19/11/96	5.8 m	18 s	3.4 m
1/1/98	5.5 m	16.5 s	3.8 m

3.2. Scale 1:18.4. Model tests carried out at UPC funded by EU TMR programme Access to large facilities activity.

Due to the need of further data on pressures on crown walls on large scale models, EU was asked by the author for funding to develop large scale tests in a wave flume. Because of the scientific interest of the study, EU decided to give financial support to the project under the EU TMR programme Access to large facilities activity. Tests were carried out between March and April '98 at the large flume (CIEM) of the LIM (Universidad Politécnica de Cataluña). It was necessary to select this large flume not only because of the large scale of model 1:18.4 but also because of the need of large constant-spectra waves. CIEM flume has an active reflection absorber in the wave paddle, necessary to keep the spectra constant a long time. Wave paddle is sliding-wedge type, driven by a hydraulic piston which is controlled by a computer.

3.2.1. Overall description of the scale model

The scale model was built of a core of parallelepipedic blocks of 14.5 Kg, with a steel crown wall in its top. As the crown wall stability was not under study, it was ballasted until the stability against the larger waves was assured. The armour was built of 20 Kg parallelepipedic blocks. The total weight of the model was about 18 tons.

Due to the large permeability of the breakwater core, the energy transmission across the breakwater is large. A rubble mound dissipating slope was built in the harbourside of the breakwater to reduce large oscillations induced by transmission. In the seaside of the breakwater, an impermeable slope was built corresponding to the local sea bottom slope in front of the breakwater.

3.2.2. Sensors installed and test procedure

Capacitive-type wave gauges were installed to measure the wave time series in front of the breakwater. To separate incident and reflected wave energy, a modified Mansard and Funke method (1980) was used. Therefore, three wave gauges were installed in front of the wave paddle and three wave gauges were installed in front of the breakwater toe.

There is an intrinsic complication in the definition of maximum wave run-up along a rough-permeable slope. The complication appears in the establishment of a criterion to define the free surface-armour slope interface in the maximum run-up instant. First of all, it is very difficult to define the free surface in a bore : are bubbles air or water? Furthermore, there is a large imprecision when defining the position of the maximum run-up in a highly-irregular rough-permeable slope (much easier in smooth-impermeable slopes).

To avoid this problem, in the present study two methods were used to measure the run-up :

1) A capacitance wave gauge was installed lying upon the armour slope. Obviously, the gauge measurement will be affected by a high concentration of air bubbles in the later running up the slope. The maximum run-up measured by the gauge will not correspond to the maximum level of the water tongue, but will be defined by the position of the X% air concentration in the water mass. This X% is unknown, but remains constant for all the tests if the gauge is not changed.

2) All the tests are recorded by a video camera, to have a visual reference of the process. In the side wall of the flume (made of glass), a grid is drown to have a detailed spatial reference for further processing.

Strain-gauge pressure cells were installed to measure pressures on the crown wall front and uplift pressures under the base. Figure 8 shows the position of the pressure cells (measured in mm). The pressure cells in the crown wall front and the first three in the base are installed in the same relative position than in the prototype

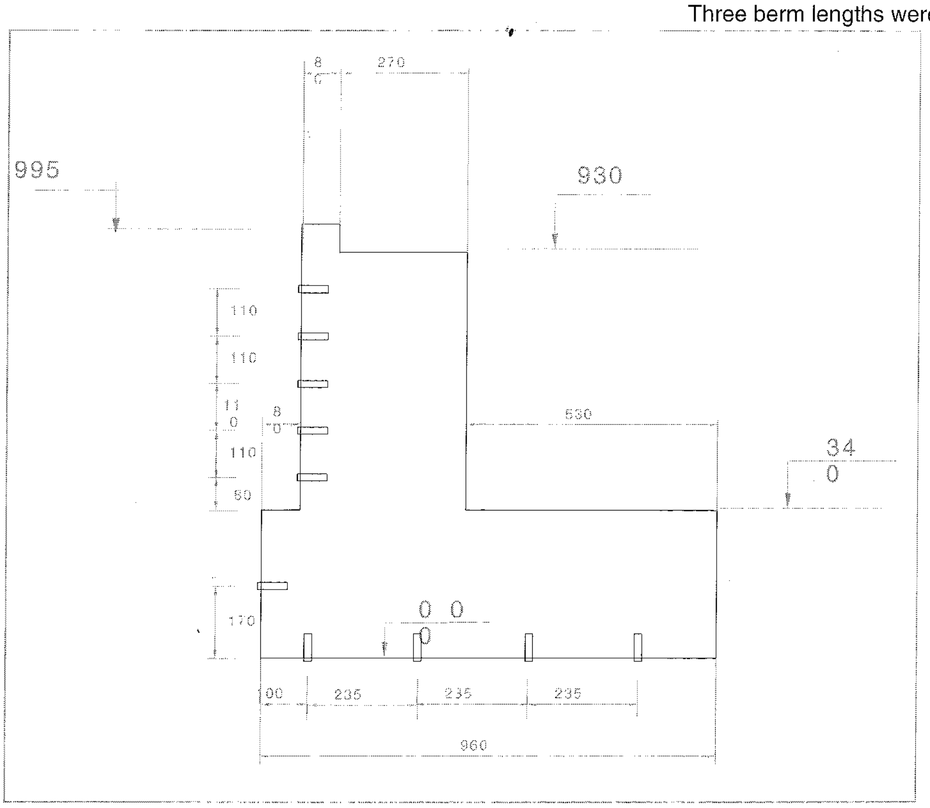


Figure 8

Monochromatic and random waves were generated for three tide levels: low tide, medium tide and high tide. Moreover, sea states measured in the prototype were simulated in a 1/18.4 scale, corresponding to the three larger storms measured in Gijon along the field campaign. Data rate in the scale tests were 20 Hz.

3.3. Scale 1:90. Model tests carried out at the University of Cantabria

Model scale lab tests were conducted in later in 1994 at the 60 m long, 2 m wide, 2 m high wave flume at the Ocean and Coastal Engineering Lab at the University of Cantabria. The test model consists of a 1/90 scale section of the Príncipe de Asturias breakwater at the Port of Gijón shown in Figure 2. The water depth was set to correspond to high tide level in the prototype. Cnoidal waves were generated by a piston-type wave paddle. Wave heights ranging from 9 to 13.5 m and periods from 11 to 17 s were tested. Moreover, irregular wave series were generated. The targets of these tests were :

1) to know the spatial distribution of wave induced maximum pressures at the breakwater to design the sensors to be installed in the prototype;

2) to identify and quantify the effect of the berm characteristics on the resulting pressures.

3.3.1. Brief description of the scale model

Three berm lengths were tested, corresponding to the length of 1 mound unit, 2 units and 3 units. Two types of parallelepipedic blocks were used corresponding to 90 and 120 tons. The core was built of parallelepipedic bocks of 125 gr while the armour was built of parallelepipedic blocks of 165 gr. The crown wall was built of wood and ballasted until the stability under the larger waves was assured. A single SWL was employed in the tests, corresponding to 23.5 m of water depth at the toe of the breakwater.

A piston-type wave paddle was used to generate waves. The paddle is controlled by a computer.

3.3.2. Sensors installed and test procedure

Free surface in front of the structure was measured by three capacitance wave gauges and a reflection analysis of the free surface time series was done. The transmitted wave height was measured by one free surface gauge located 1 m from the leeside toe of the breakwater. Four strain-gauge type pressure gauges were installed in the wave screen basement while eight gages were fixed to the structure front (see Fig. 9).

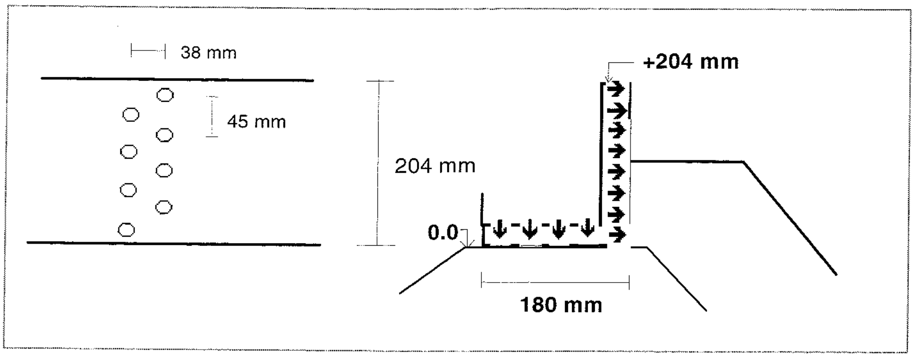


Figure 9

values.

It is clear how the reflection coefficient shows small variation, ranging from 0.2 to 0.3 for all Ir values. Some overall oscillations can be identified for Ir values between 3 and 6 and for Ir between 12 and 15. This oscillation could be explained by resonance effects on the main layer slope (see Sawaragy et al., 1983) or in the breakwater core, built by 90 ton parallelepipedic blocks.

The logging data rate was 20 Hz. As no wave can plunge directly onto the crown wall, this data rate is fast enough to measure the main characteristics of the pressure time series, with a truncating error smaller than 3% in this scale.

Tests were recorded by a video camera and a grid was placed in the flume glass wall. This grid provided the spatial reference for future data processing.

4. SOME RESULTS

4.1. Waves in front of the breakwater

As it was said before, for the analysis of the wave reflection induced by the breakwater, a modified version of the Mansard et al. method (1980) was employed. During field campaign '95, one wave recorded was lost and one more was displaced from its original location. There are different possibilities to explain this small catastrophe but the most reliable one is the heavy fishing activity around the breakwater (the high fish production in Gijon Breakwater armour is a very good example that can be useful to change the common mind of the environmentalists about the breakwater building impact). As three wave recorders are required for the incident-reflected analysis and due to this loss of data, the wave analysis can only be done from April 95. In the measuring periods from this date, no severe storm (H_S > 5 m) was recorded. From the available data, two data sets were selected to make the incident-reflected analysis: the first is on March 28-30th, in a sea state of H_s = 2.2 m and peak periods ranging from 8-10 s. These sea states defined a clear SEA conditions. The second wave series corresponds to April 3-5th, with H_s = 3.0 m and peak periods ranging from 14-18 s, and is a well defined SWELL sea state.

Figure 10 shows the results of the reflection analysis of both wave states in 17 min. series versus the Iribarren number. The squared symbols are Lab results in the 1/90 scale model, included to complete the figure in the low Iribarren number (Ir)

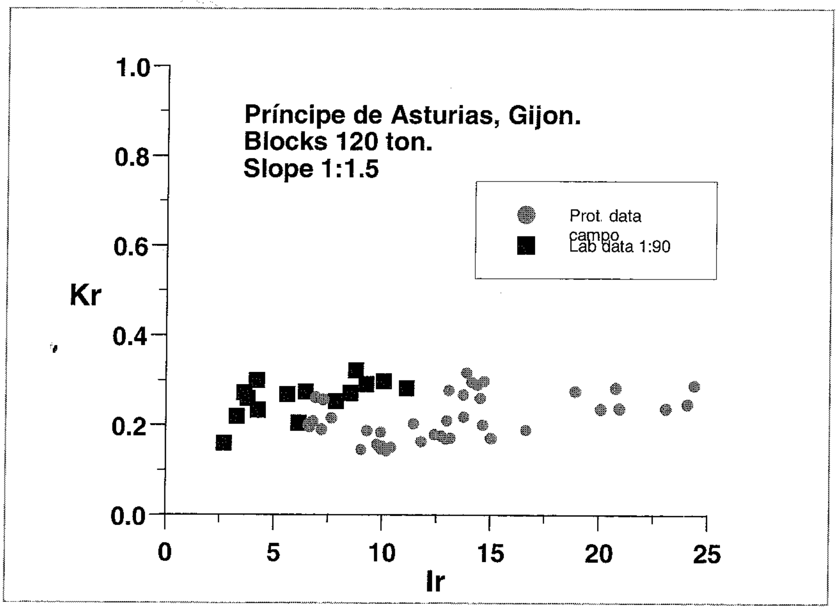


Figure 10

From the wave data analysis done under storm conditions, it is found that the Hmax/H_s ratio ranges from 1.5 to 1.7 for sea states H_s > 1.5 m (see Fig 11), while for small wave sea states the ratio can grow up to values above 2.0.

One more useful detail observed in the wave processing is the good correlation observed between the wave recorders located in front of the breakwater and the wave buoy installed by Puertos del Estado in the Gijon Harbour entrance as can be noticed in Figure 12.

4.2. Wave induced pressures on the crown wall

In this paper, the results from the lab and field campaigns are compared to the methods proposed by Jensen, 1984, Günbak et al., 1984 and Martín, 1995. The methods of Martin, 1995, Martin et al., 1999, and Günbak et al., 1984, are defined wave to wave and provide the pressure profiles in the maximum force instant, while the method of Jensen provides the force of 0.1% of probability of occurrence under a given sea state defined by the significant wave height (H_s). As was said before, the method proposed by Jensen neither predicts the pressure profiles nor the uplift pressures and therefore, can not be used to predict the overturning momentum on the wave screen.

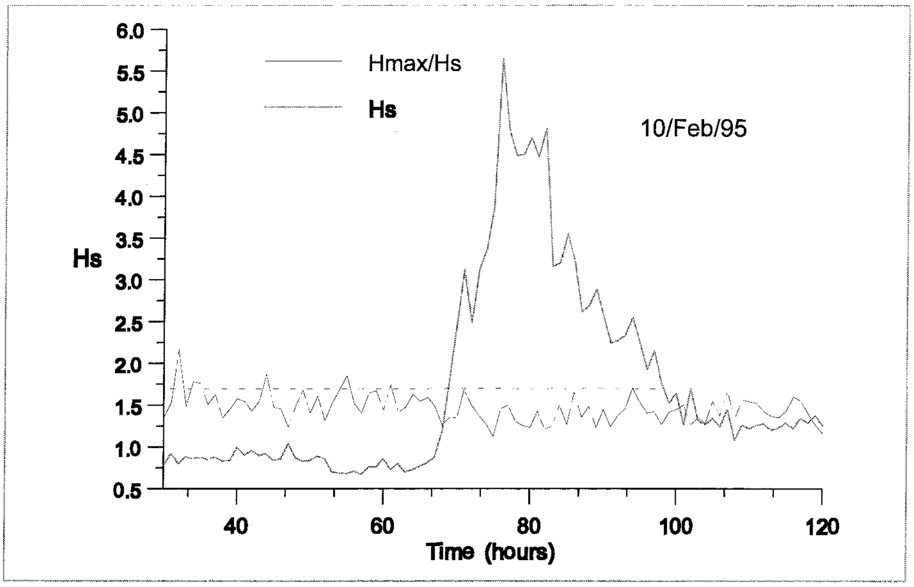


Figure 11 - Hmax/Hs ratio under storm conditions

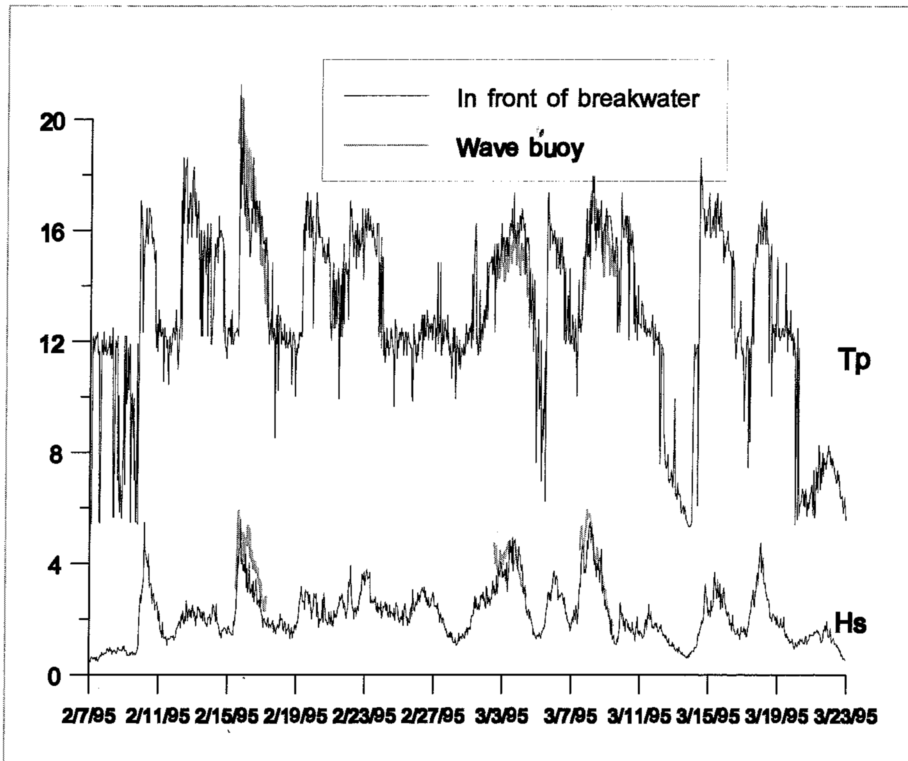


Figure 12 - Comparison of buoy data to wave recorder data

The comparison between measurements is done in two ways: a deterministic comparison of pressure profiles due to single waves and a statistical analysis of forces onto the crown wall.

4.2.1. Qualitative comparison of results

This comparison is done in order to achieve two main targets: 1) to make a qualitative check of some of the hypothesis in which the methods of Günbak et al., 1984 and Martín, 1995 are based and; 2) to identify any possible qualitative scale effects between lab and prototype results. To do that, comparison of force-time series and pressure profiles measured in the field campaign, in the lab and proposed by the methods, are done.

In Figure 13, a brief interval of force-time series measured in the lab is presented. The wave train characteristics are H_s = 9.0 m and T_p = 18 s and the tests were done with a tidal elevation of 4.0 m above the zero datum. In this figure, two impinging wave forces are pointed up. The former (time 310-320 s) shows a double peak pattern while the latter (447-457 s) shows a single peak pattern. The only difference between the two impinging waves was the Run-up height. The former wave, slightly larger wave height and period, produced a Run-up tongue which overcame the main layer berm level (A_c) while the latter almost reached the level A_c.

If the main layer units can stand the rush-up wave action, most of the bore front horizontal momentum is transmitted to these units, in the region below A_c. If the bore does not exceed the A_c level, the former peak of force (shock pressures) is smoothed, with only more or less noticeable pressure oscillations appearing, depending on the berm length and main layer porosity. The second peak (reflecting pressure) always occurs because it is generated by the water mass piled by the wall developing a pseudohydrostatic pressure profile.

In Günbak et al., 1984, only one maximum force situation obtained as the sum of the shock and reflecting pressures is defined. These two pressure maxima occur at different instants in the evolution of the bore and are due to different processes that must be analysed separately.

From lab tests over regular shaped breakwaters (uniform slopes 1:1.5-1:2, main layer porosity ranging 0.3-0.4) it can be estimated that shock pressure maximum of force is expected to appear in the cases when $H_s/A_c \geq 0.7$. Of course, this value heavily depends on the run-up and thus, on the breakwater characteristics.

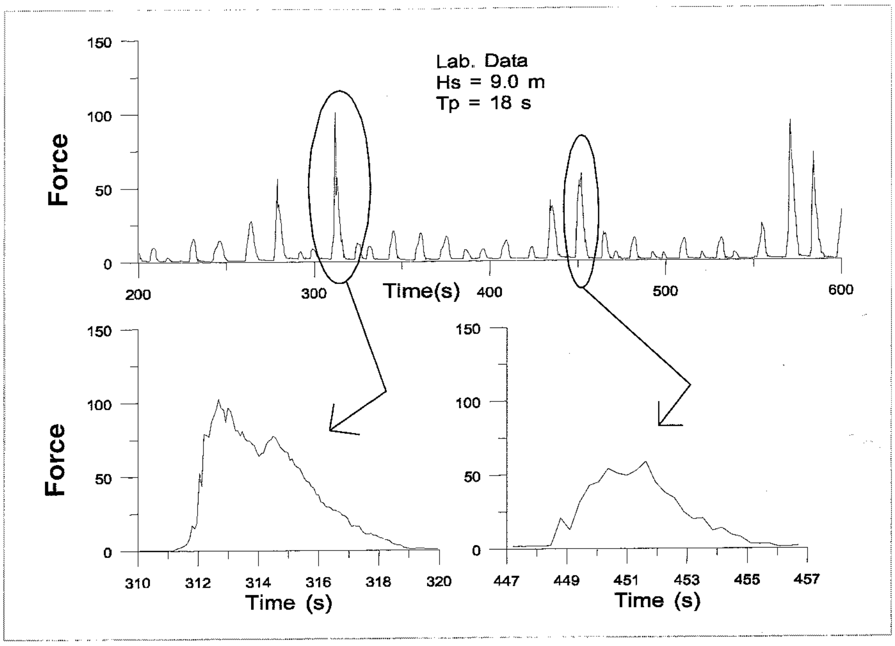


Figure 13 - Experimental force-time series

In the case of Gijón Breakwater, the Ac level is 12 meters above the zero datum. In high tide situations (+4.0 m), the berm freeboard is 8.0 m. Thus, shock pressures are expected to occur for significant wave heights above 0.7 x 8.0 = 5.60 m. In figure 14, a pressure-time series corresponding

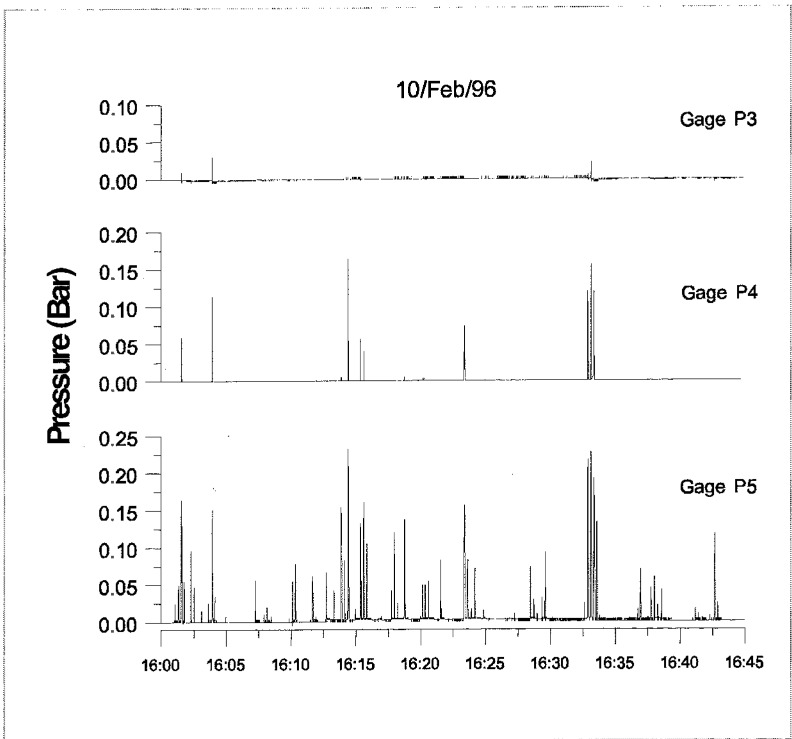


Figure 14 - Pressure-time series measured in the prototype front face

to gages P3, P4 and P5 in the prototype front are represented. These series were measured during the storm on February 10, 1996. Recalling Table I, the characteristics of this storm were H_s = 6.0 m and T_p = 19 s. In the instant of the measurements shown in Fig. 14, the tide level was 3.9 m.

Figure 14 shows that only three waves reached gage P3 (0.4 m below A_c level) in 45 minutes. This is consistent with the test results in 1/90 scale model under H_s = 6.0 m, where bores do not exceed the A_c level.

One of the main hypotheses introduced in Martín, 1995, is the assumption that the basic run-up tongue characteristics (thickness, bore front velocity, etc...) on breakwaters with wave screens are similar to those in bores running up on infinite slopes and, thus, the effect induced by the presence of the wave screen can be neglected. Under this assumption, the classical run-up formulae can be employed. As an example, estimating Ru ≈ H, the maximum run-up in a given sea state can be calculated. A sea state of H_s = 6.0 m and 150 waves (45 minutes of storm on 10/2/96) will produce maximum waves of about 8-9 meters. These waves would run up 8-9 meters above the SWL (4.0 m tidal level) and merely reach the A_c level. This is consistent with the prototype measurements. Although this comparison is rough, it can be used as an engineering check of the hypothesis. The run-up on rough permeable slopes is a process with high experimental variability, and all engineering formulae for run-up are "best fit" methods. The hypothesis included in Martin et al., can not be experimentally distinguished from the experimental "noise".

In Figure 15, a stretching of the previous Fig. 14 is done in order to show three wave actions. It can be noticed that the shock pressure peak is not clear and only some pressure oscillations appear. These measurements are as expected because the ratio H_s/A_c in this storm is around the 0.7 limit. Once again, this is consistent with the lab results and the modelling of the double peak effect done in Martín, 1995.

Finally, the vertical profiles of pressure due to a selected single wave of 8.3 m height and 16.2 s period measured in the prototype, measured in the lab and proposed by Martin, 1995 were compared. The results are shown in Figure 8. It can be noted that the shock pressures profile is quite similar in the prototype and in the lab, and fits quite well the homogeneous vertical distribution proposed for shock pressures in Martín, 1995. Generally speaking, the total force produced by the shock oscillations in the cases when $R_u < A_c$ are low and smaller than the reflecting force. In Martín, 1995, for cases when $R_u < A_c$, it is assumed that the shock forces are always smaller than the reflecting forces and can be neglected.

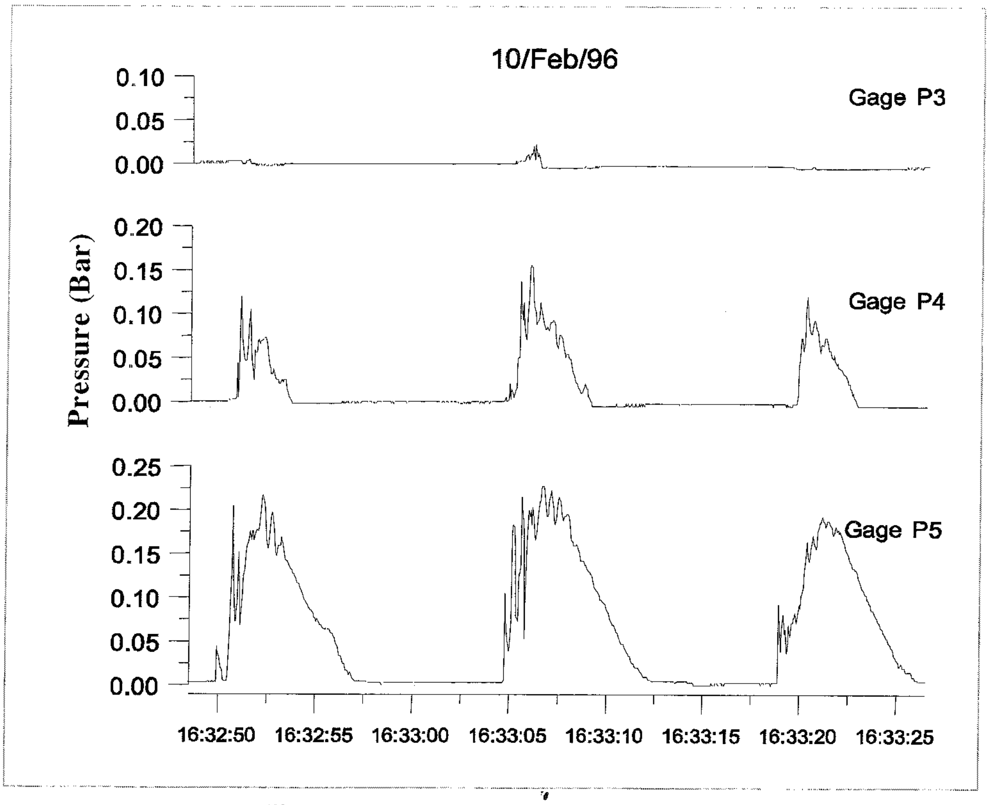


Figure 15 - Pressure-time series in prototype

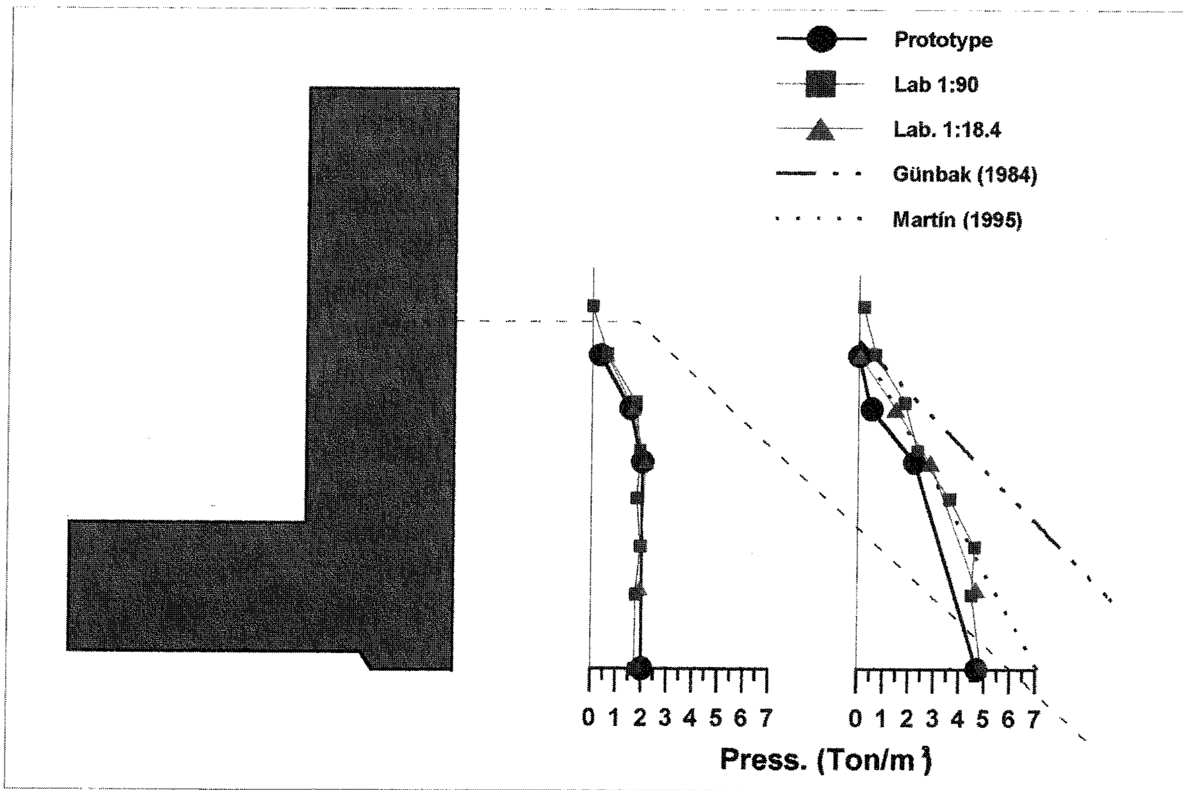


Figure 16 - Pressure profile in prototype, lab tests and analytical methods

pressure profile is quite similar. Notice that this comparison is done in qualitative terms. It is easy to understand the difficulty of making a deterministic comparison between the results in the prototype and the lab, trying to simulate exactly the same wave height, period, tidal condition, etc...

The dashed lines in Figure 16 show the proposed reflecting pressure by Günbak et al., 1984 and Martín, 1995. The overall trend is well simulated by both methods, but the profile proposed by Martín, 1995, fits better the actual quantitative values measured.

In the reflecting pressures there are some differences between the quantitative values of the measuring points in the lab and in the prototype, but the overall trend of the

As a result of the quantitative comparison, it can be concluded that there are no large and noticeable qualitative scale effects between lab and prototype results, and that the method of Martín (1995), adequately represents the main characteristics of the process.

4.2.2. Quantitative comparison of results

As the method proposed by Jensen, 1984, provides the 0.1% probability force, this force has been selected as a comparison parameter. In figure 17 the net 0.1% horizontal force given by the lab tests, the methods from Jensen, Günbak et al. and Martín et al., and the three main storms measured at the prototype are given.

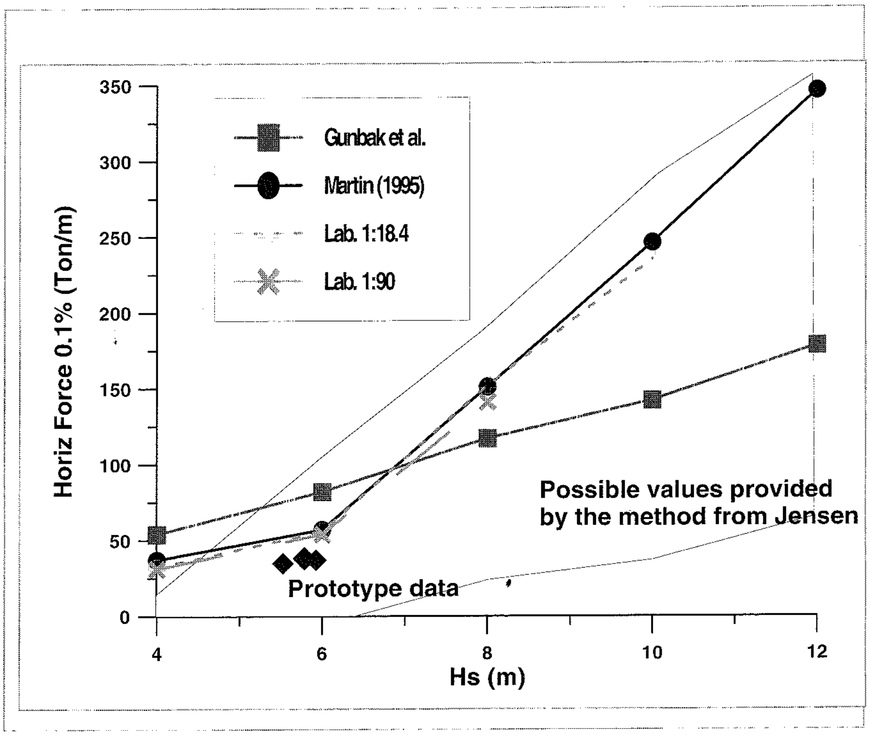


Figure 17 -Quantitative comparison of results

The method from Jensen is basically empirical and must be applied using some experimental parameters. In this case, it is applied using the experimental data collected by Pedersen and Burcharth, 1992. This data shows a wide spreading that makes it difficult for the engineer to define design values of the parameters. In this case, an upper and lower value of the parameters are selected and, thus, an upper and lower 0.1 % force is given for each wave height. These two lines define a region assigned as Jensen's results region in Figure 17.

The 0.1 % force produced by the three storms are obtained by extrapolating the probability curve of forces in 45-min. bursts, because in the next 45-min. burst the tidal range is different and the "test conditions" are not homogeneous. The maximum forces are measured in the high tide situation (about 4.0 m in the three storms) and is equal to the tide level employed in the lab tests. Notice that in 45-min. bursts, an average of 150-200 waves are measured and the 0.1% force requires 1,000 waves (about 6 hours).

The prototype results are 20% smaller than those of the lab results. Regarding the qualitative comparison done, it is clear that all the forces measured in the prototype under such storms are due to reflecting pressures. In the comparison of vertical pressure profiles, it was noticeable that the pressures measured in the lab were slightly larger than those measured in the prototype. This can be explained regarding the breakwater core. In the lab the core was built by small scale 90-T blocks, which can simulate the same porosity but not the same permeability. As the reflecting pressures are due to the water mass piled by the wall, larger wave transmission across the breakwater will produce less water accumulation by the wave screen.

Martin et al., 1995 and Günbak et al., 1984 methods are developed to be applied wave to wave. In this case the hypothesis of equivalence (Saville, 1962) is assumed and the methods are applied to a series of 3,000 synthetic simulated individual waves that represent a TMA spectrum. The fitting of Martín, 1995 to the lab results is very good. The small differences in the 10 m wave height is due to the breaking criteria implemented in the numerical simulation (Hb/d = 0.8).

The results of Günbak et al., 1984, overpredicts the experimental data (100% respect prototype, 60 %, respect lab results) for smaller wave heights while for larger wave heights the calculations are smaller than the measurements. Perhaps the most important characteristic to point out is the very different trend shown by Günbak et al. results and Martín, 1995. It is clear that the slope of the wave height-force line (almost linear for $H_s >$ 6 m) is very much different between the methods, showing major differences.

As an example of the scale effects between the lab and the prototype structures, Figure 18 represents the probability distribution of forces measured in the prototype on 19/2/96 (H_s = 5.5 m, T_p = 16 s) and the results measured at the lab (scale 1/18.4) under the same simulated sea state.

Small scale effects are apparent by comparing the two probability curves. As no shock pressures occur for these wave heights these effects are not expected to be related to the water compressibility or aeration. Moreover, as Reynolds number is above 10,000 frictional forces should be well scaled by Froude. Due to the high porosity of the breakwater core (built of 90-ton blocks), the differences in the resulting forces could be due to an incorrect simulation of the core permeability in the scale model (the porosity is well simulated). Moreover another source of errors could be the differences in the wave trains between the lab and the prototype. Nowadays further studies are been carried out on this topic.

CONCLUSIONS

- A field campaign is being developed as well as intensive lab tests over the 1/90 and 1/18.4 scale models of the Príncipe de Asturias breakwater. The results of the field campaign and the lab tests are used to check the validity of some analytical methods employed in engineering practice to design wave screens.

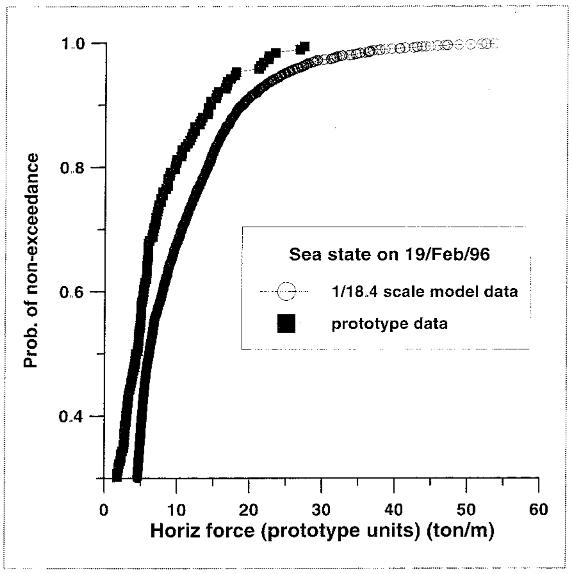


Figure 18 - Force probability distributions

- The lab test results and the analytical methods seem to slightly overpredict the forces measured in the prototype.

- Maximum forces measured in the prototype are due to reflecting pressures, where Froude scaling works properly, and the discrepancies must be explained by other modelling effects (core permeability, wave modelling, etc).

- No severe scale effects between lab results and prototype results are identified in a qualitative analysis.

- The method proposed by Martín, 1995, and Martín et al., 1999, produces results that fit the lab test results well and are 60% more accurate than Günbak et al. for small wave heights. For large wave heights, the differences between lab data and Günbak et al., 1984, are larger. Jensen's method is difficult to apply for design purposes.

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