

CHAPTER 137

PROTOTYPE MEASUREMENTS OF WAVE PRESSURES ON A WAVE SCREEN: COMPARISON TO PHYSICAL AND ANALYTICAL MODELS

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ABSTRACT

The results of a field campaign of measurements of pressures on a prototype wave screen are analyzed and compared to the results of laboratory tests and to the calculation methods proposed by Jensen, 1984, Günbak et al., 1984, and Martin et al. 1995. Both the field campaign data and the lab tests results seem to fit quantitatively better to the method of Martin et al. than to other proposed formulae. Moreover, the proposed modelization and description of the process employed to develop the method of Martin et al., 1995, are consistent to the measured pressure profiles and time-pressure series.

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, Martin et al. 1995. However, others 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 Engineering methods.

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The main objectives of the present paper are (1) to describe the instrumentation of the Gijón breakwater wave screen and (2) to compare the prototype measurements to model test and to several formulae being used in the engineering practice.

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 analyzing the measurements and finally a quantitative comparison of results.

PROTOTYPE INSTRUMENTATION AND MEASUREMENTS

Gijón is located at the Cantabrian Sea, in the north of Spain (fig. 1). It is exposed to sea states 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.

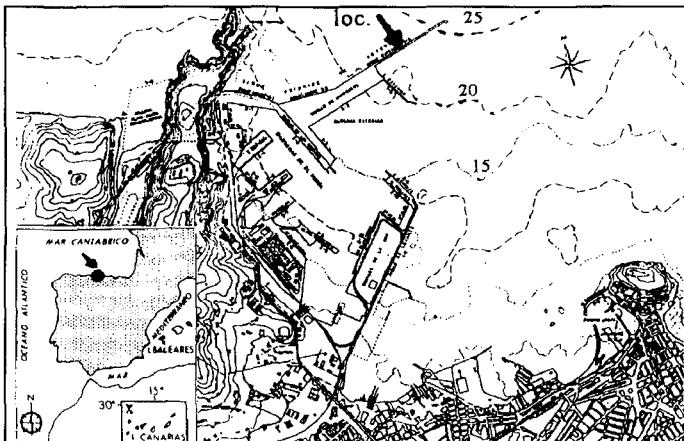


Fig. 1. Prototype location.

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. Five specially designed pressure gages were placed in the wall front (P1-P5), while three pressure cells were drilled across the wall basement to record the uplift pressures (S1-S3) (fig. 2). 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.

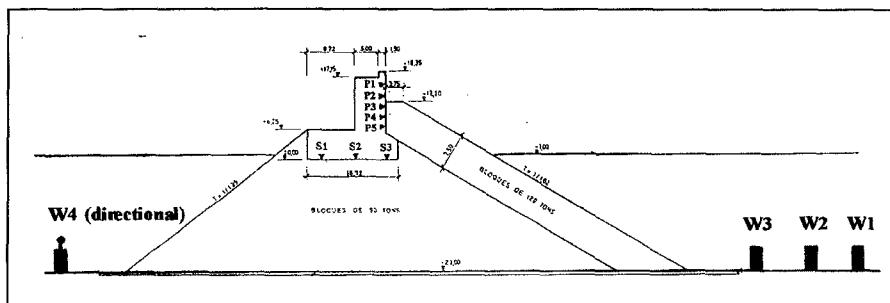


Fig. 2. Instrumentation setup.

The sampling rate of pressures in the wave screen is 20 Hz, which is enough to record the main characteristics of interest of the time-pressure series being research. The system is continuously logging data in 45-min. bursts. After the burst is finished, the system checks if a given minimum level of pressures is exceeded. If so, the data is stored and if not, it is deleted.

The sampling rate for the wave recorders is 0.5 Hz. Notice that, for a 16-second wave period this data rate gives 8 data/wave period which is enough to define the wave shape. Waves shorter than 8 seconds will be underdefined, but their forces on the wave screen are not expected to be noticeable. Moreover, the wave recorders are fixed in 25 m water depth, and the information of short period waves in this depth will be strongly affected by the difficult-to-define pressure/free surface oscillation transfer function.

The system was set up in February 95 and is still working. During the February-April 1995 period, two storms of $H_s = 5.9$ m and 4.5 m respectively were recorded. In the second period (December-April, 1996) two more storms were measured corresponding to 6.0 m and 5.2 m significant wave height. Due to the tidal level at the instant of maximum wave height only three of these four storms produced significant pressures on the wave screen. The selected storms are described in Table 1. The tidal level is defined above the zero datum (minimum low tide level)

Date	Signif. wave height	Peak period	Tidal level
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

As the system is exposed to the natural and port labour actions, some measuring problems appeared in different instants of the field campaign: some electrical noise in the signal and power outages in the initial months, lightning which hit the amplifiers and affected almost all of the system, etc. In figure 3, an abstract of the field work development and incidences is represented.

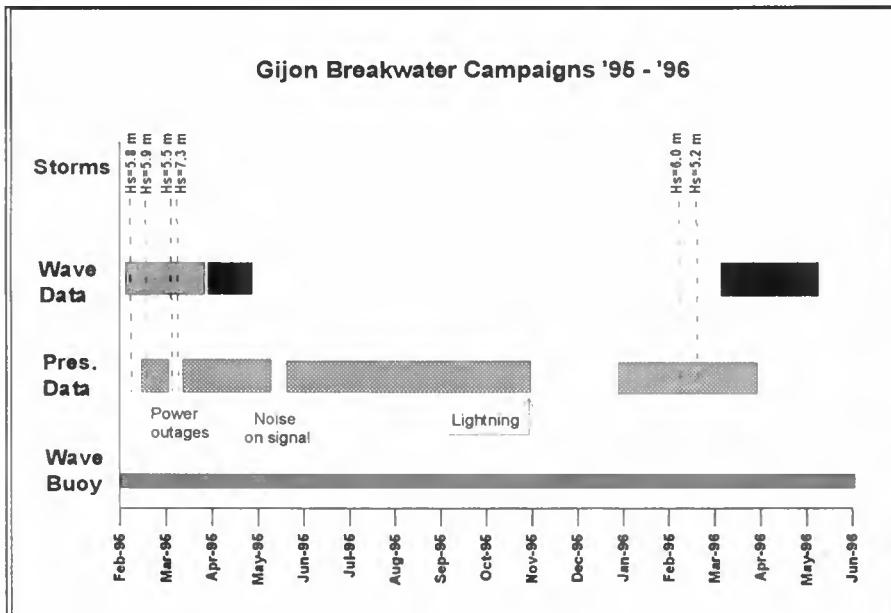


Figure 3. Field campaign schedule and incidences

LABORATORY EXPERIMENTAL SETUP

Model scale lab tests were conducted in the 70 m long, 2 m wide, 2 m high wave flume at the Oceanographical 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 Port of Gijón (Spain) shown in figure 2. This breakwater has a wave screen based at low tide level (0.0) and crowned 18.3 m above. The main layer is built of 120-Ton parallelepipedical blocks, and the core is built of 90-Ton blocks. The water depth was set to correspond to high tide level in the prototype. Regular cnoidal waves were generated by a piston-type wavemaker. 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 irregular wave characteristics are represented in Table II.

Table II. Model scale 1/90. Irregular wave series.

Significant wave height	4 m	6 m	8 m	10 m	12 m
Peak period	12 s	14 s	16 s	18 s	20 s

Free surface in front of the structure was measured by three resistance gages and a reflection analysis of the free surface time series was done. By using this technique it is possible to obtain the incident and the reflected wave height. The transmitted wave height was measured by one free surface gage located 1 m from the lee side toe of the breakwater. Four strain-gage type pressure gages were installed in the wave screen basement while eight gages were fixed to the structure front.

One of the main targets of the tests was to identify and quantify the effect of the berm length on the resulting pressures. 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.

ENGINEERING ANALYTICAL METHODS

There are few methods available for the calculation of forces on wave screens, some are mainly analytical: Iribarren et al., 1964, which largely overpredicts the resulting pressures, and Gündak et al., 1984; some are mainly experimental: Jensen, 1984, revisited by Bradbury et al., 1988, and Pedersen et al., 1992, which makes a probabilistic approach to the horizontal forces (not pressure distributions or uplift forces). The parametrization selected in Jensen, 1984, generates a relatively large dispersion in the results, and Bradbury et al., 1988, found that the influence of wave period on the resulting forces is not represented adequately.

Generally speaking, the response of the built wave screens reveals that 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 conceptualization of the process and the study of the procedures of momentum transfer between a bore and a vertical surface. Finally, Martin et al. (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.

From the results of the experimental study conducted at the Oceanographical Eng. lab at University of Cantabria (Losada et al. 1995, Martin, 1995) it can be concluded that two maxima of force occur due to each single wave; the former peak is generated during the abrupt change of direction of the bore front due to the wave screen (horizontal decelerations), while the latter maximum occurs after the instant of maximum run-up and is related to the vertical acceleration of the water mass piling in front of the wave screen (early Run-down movement).

The distinct nature of these force peaks is well revealed in the vertical distribution of pressures. In figure 4 an interval of the time-force curve obtained from lab tests is shown, and the two force maxima (A and B) are pointed out. The vertical distribution of pressures at the wall in the instants A and B are shown in the same figure. The pressures due to the first peak (A), hereafter denoted as Shock pressures (P_s), present an almost vertically uniform profile, where two zones are well distinguished: the upper zone, not protected by the rubble-mound layer, and the lower zone, protected by the rubble-mound layer. The pressure profile due to the second peak (B), hereafter denoted as Reflecting pressure (P_r), grows vertically with an almost constant increasing factor, always equal to or smaller than ρg . Martin et al., 1995, proposed a method to calculate the pressure profiles in the two instants of maximum force (P_s, P_r).

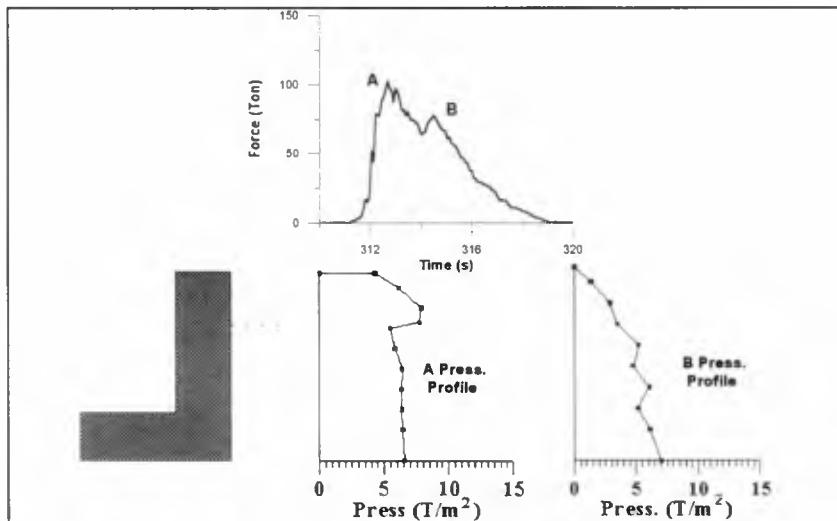


Figure 4. Force-time series and pressure profiles in instants A and B.

In this paper, the results from the lab and field campaigns are compared to the methods proposed by Jensen, 1984, Gümback et al., 1984 and Martin et al., 1995. The methods of Martin et al., 1995, and Gümback 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.

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 et al., 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 5, 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 (Ac) while the latter almost reached the level Ac.

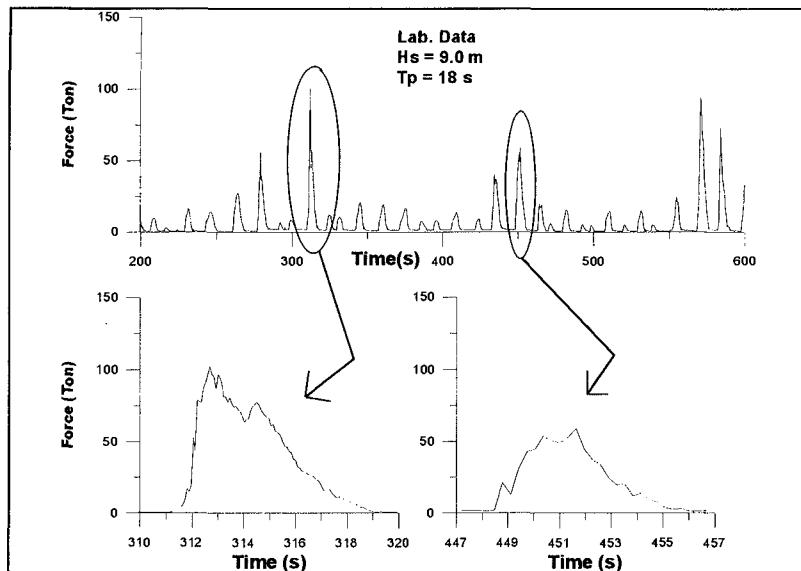


Figure 5. Experimental time-force series.

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 overcome 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 pressures 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 analyzed 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.

In the case of Gijón Breakwater, A_c 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 $0.7 \times 8.0 = 5.60$ m. In figure 6, a pressure-time series corresponding 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. 6, the tide level was 3.9 m.

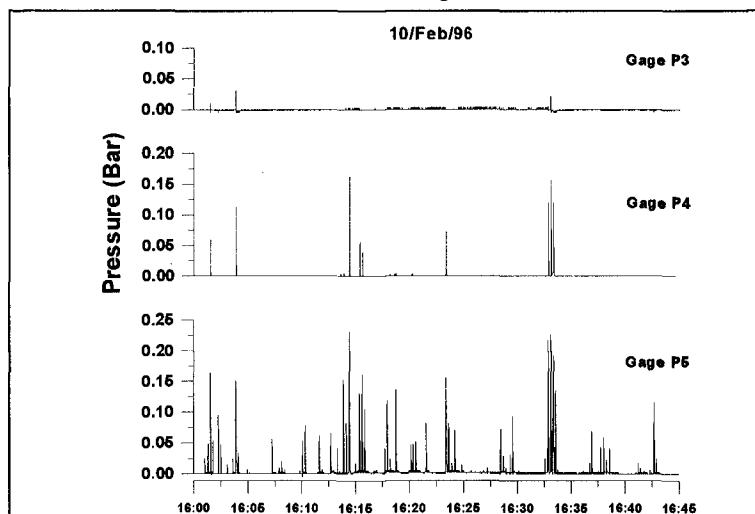


Figure 6. Pressure-time series measured in the prototype front face.

Figure 6 shows that only three waves reached gage P3 (0.4 m below Ac 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 overcome Ac level.

One of the main hypotheses introduced in Martín et al., is the assumption that the basic run-up tongue characteristics (thickness, bore front velocity, etc...) on breakwaters with wave screens are similar than 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 $R_u \approx H$, 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 lead in maximum waves about 8-9 meters. These waves would run-up 8-9 meters above the SWL (4.0 m tidal level) and merely reach the Ac level. This is consistent with the prototype measurements. Although this comparison is rough, it can be used as a 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 7, a stretching of the previous fig. 6 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/Ac in this storm is around the 0.7 limit. Once again, this is consistent with the lab results and the modeling of the double peak effect done in Martín et al., 1995.

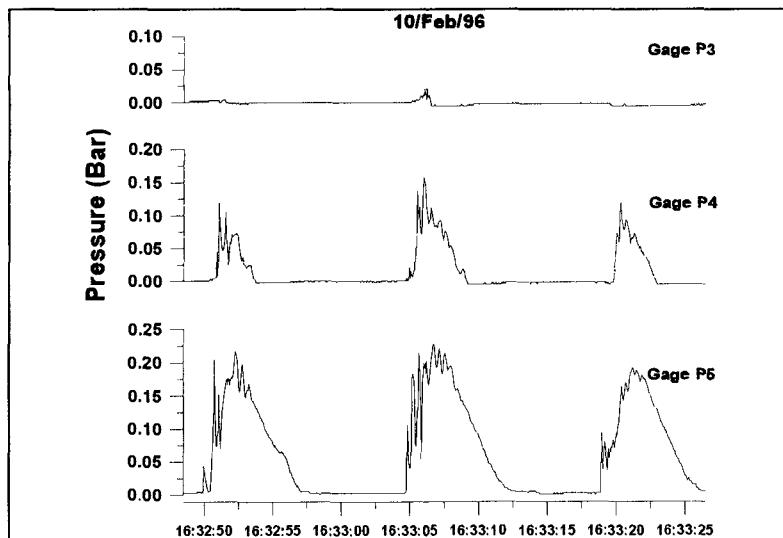


Figure 7. Pressure-time series in prototype.

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 et al., 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 the homogeneous vertical distribution proposed for shock pressures in Martin et al., 1995, quite well. 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 et al., when $R_u < A_c$, it is assumed that the shock forces are always smaller than the reflecting forces and can be neglected.

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 pressure profile is quite similar. Notice that this comparison is done in qualitative terms. It is easy to understand the difficulty of make 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 8 show the proposed reflecting pressure by Günbak et al., 1984 and Martín et al., 1995. The overall trend is well simulated by both methods, but the profile proposed by Martin et al., 1995, fits better the actual quantitative values measured.

As a result of the quantitative comparison, it can be concluded that there are not large and noticeable qualitative scale effects between lab and prototype results, and that Martin et al., 1995, method adequately represents the main characteristics of the process.

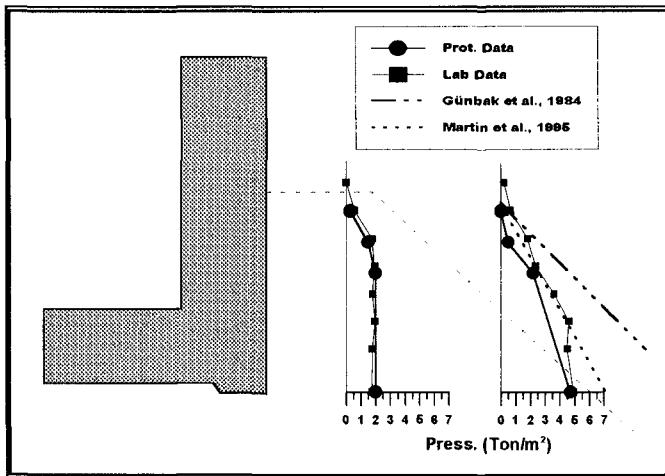


Figure 8. Pressure profiles in prototype, lab tests and analytical methods

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 9 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 storms measured up to now in the prototype are given.

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 9.

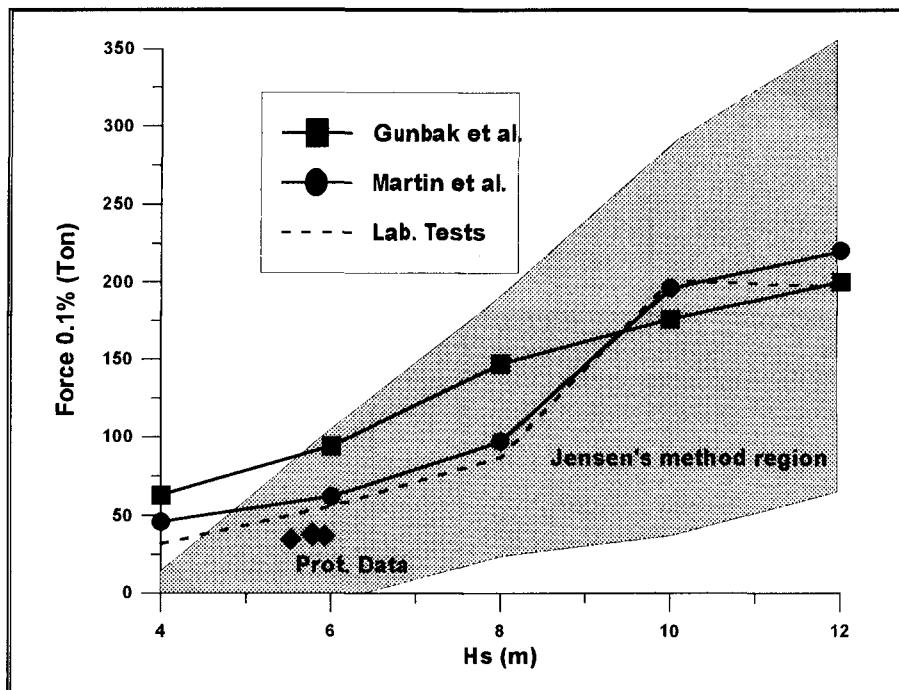


Figure 9. Quantitative comparison of results.

The 0.1 % force produced by the three storms are obtained by extrapolating the probability curve of forces in 45-min. burst, 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. burst, an average of 150 waves are measured and the 0.1% force requires 1,000 waves (about 6 hours).

The prototype results are 15% 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 et al., 1995 to the lab results is not surprising as the parameters of the method were adjusted to this breakwater from the experimental results. The difference in the 12 m wave height is due to the breaking of waves in the wave flume, which occurs for breaking parameters (H_b/d , breaking wave height over depth) smaller than in the nature.

The results of Günbak et al., 1984, overpredicts the results (100% respect prototype, 60 %, respect experimental results) for smaller wave heights while for larger wave heights the results are smaller than the experimental. Perhaps the most important characteristic to point out is the different trend shown by Günbak et al. results (quasi linear) and Martín et al. results (quasi parabolic). It is clear that the shock forces on the wave screen front face depend on the pressure on the wall (horizontal momentum related to the water mass and the bore celerity) and on the area of wall exposed to the pressure (Run-up). Both of them are related to the wave height and, thus, the wave height must have an effect on the resulting forces at least in a quadratic form. It is clear that once the wall is overtopped ($H_s > 10$ m in fig. 9) the quadratic trend disappears.

Figure 10 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 lab tests results for $H_s = 6$ m and $T_p = 14$ s which are the most similar cases available nowadays. The differences for smaller waves and the better fit for larger waves can be noticed.

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 and perhaps more related to the core permeability

in the scale model or the differences in the wave trains between the lab and the prototype.

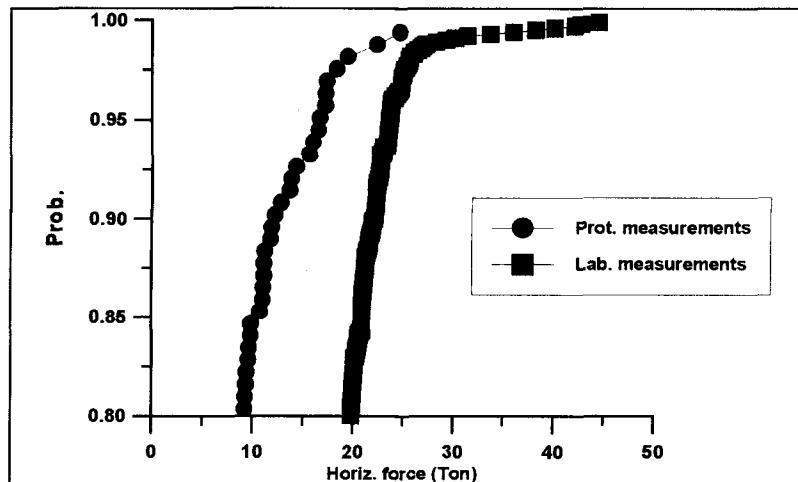


Figure 10. Force probability distributions.

CONCLUSIONS

- A field campaign is being developed as well as intensive lab tests over the 1/90 scale model of 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. Moreover, the comparison of the results between the prototype and the 1/90 model allows to analyze the scale effects.
- The lab test results and the analytical methods seem to slightly overpredict the forces measured in the prototype.
- Maximum forces measured in the prototype up to now are due to reflecting pressures, where Froude scaling works properly, and the discrepancies must be explained by other modelling effects (core permeability, wave modeling, etc).
- No severe scale effects between lab results and prototype results are identified in a qualitative analysis.
- The method proposed by Martín et al., 1995 produces results which fit the lab test results well and 60% more accurately than Günbak et al., method. Jensen's method is difficult to apply by the designing engineers.

- The field campaign will provide more fruitful results when $H_s > 7$ m occur and shock pressures are measured.

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