

WAVE FORCES ON SOLID AND PERFORATED CAISSON BREAKWATERS: COMPARISON OF FIELD AND LABORATORY MEASUREMENTS

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Abstract

Following an old Italian tradition of prototype measurements of wave pressures at vertical breakwaters (Franco, 1994), a new twin recording station was set up and operated in 1992-1994 at Porto Torres (Sardinia, Italy) industrial harbour breakwater. In the framework of the MAST3-PROVERBS project an extensive analysis of the available data has been carried out.

Within the same project 2D model tests have also been performed in order to investigate the relation between pressure distribution and overall forces on and under both plain and perforated (multichamber) caissons. The results have been compared with the available design formulae like Goda's (1985). Statistical distributions of the horizontal and uplift forces for both structure types have also been derived. The scope of the study has been to assess the reliability of the present design methods and to outline a more physically based approach especially for the perforated structure type.

Introduction

The prototype structure is a vertical composite breakwater subjected to nonbreaking wave conditions. Two caissons (20.5x13.9 m), one with plain (solid) wall and one perforated, 62 m apart, based at -15 m on rubble footing in 20 m water depth, were instrumented with ultrasonic wave gauges and pressure sensors along the caisson's base, on the vertical face and also in the internal chambers as described by De Girolamo et al. (1995). The operating time of the instrumentation was 1992-1994, in which 10 significant storms ($2 \leq H_s \leq 3.5$ m, $6 \leq T_p \leq 9.2$ s) have been recorded. A directional wave gauge has been installed 700 m away from the breakwater at the same depth (20 m). Water levels at the wall, front and uplift pressures have been recorded with a sampling frequency of 20 Hz, except for deep sensors (2 Hz).

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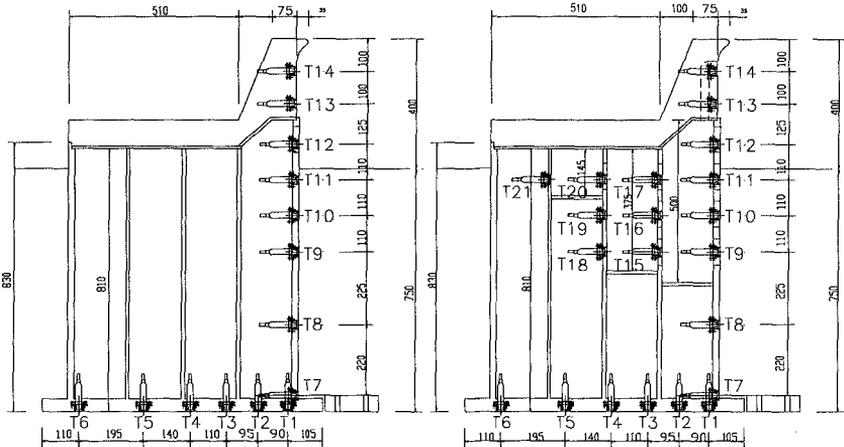


Fig. 1 Cross section of the instrumented model caissons of P.to Torres (plain wall at left and perforated at right, dimensions in mm)

Experimental conditions identical to prototype have been reproduced in the ENEL random wave flume (but just 2D homogeneous wave field) where the model caisson was equipped with the same number of pressure transducers as in the prototype (Fig. 1) and also with a dynamometer for simultaneous recording of global forces and pressures. The same wave frequency spectra were reproduced in the lab according to the prototype water levels recorded at the wall. Some synthetic Jonswap spectra have been simulated as well.

Prototype data

Parameters describing the loading signal shape of the most severe events of all recorded storms, together with all the other data relative to wave measurements, are collected in a database compiled according to MAST3/PROVERBS notations and confirming the single-peak shape for the pulsating (non-breaking) wave conditions. A further statistical analysis of wave loads proved a good fitting with 2-parameters Weibull statistical distribution for both front and uplift forces on plain caisson and for uplift forces on perforated caisson. With regards to perforated caisson, the trapezoidal integration method cannot be applied, because of geometrical complexity, to calculate horizontal forces from pressure measurements: the Kriebel (1992) method was used instead, and it has proven to give approximation of +20% in laboratory, not much bigger than values obtained from statistical analysis of front and back pressures.

Out of the ten storms recorded in field (Franco, 1996) the three largest ones in terms of significant wave height were chosen. A transfer function between the incident spectral densities and the ones measured at the wall was applied and spectra reproduction was good enough to allow a consistent comparison of pressure

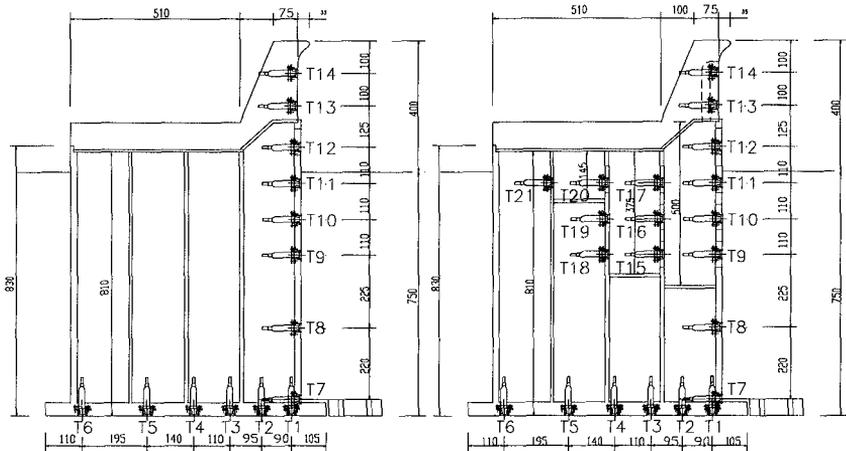


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measurements. Details on the sea storms parameters are given in de Gerloni (1997 et al.).

Uplift pressure model and scale effects

Both plain and perforated caissons and their foundation were accurately reproduced in the lab with a geometric reduction scale of 1/20. The tests were carried out in a 43 m long wave flume equipped with a wedge type generator and with a system of porous walls and lateral channels intended to absorb reflected waves. The rubble foundation grain size simulation was studied in a particular set of tests with a simplified superstructure. In order to ensure the test repeatability, the rubble base set up in the flume was accurately controlled and the rubble foundation material was deposited by means of the so called "pluvial deposition" method, which ensures the maximum density of the deposited material (Pedroni et al., 1992). A 5 mm thick sheet of tender and waterproof rubber mousse, with holes for six pressure transducers, was applied on the caisson bottom in order to avoid the water to find preferential paths between the caisson bottom and the top layer of the rubble foundation (see Fig. 2).

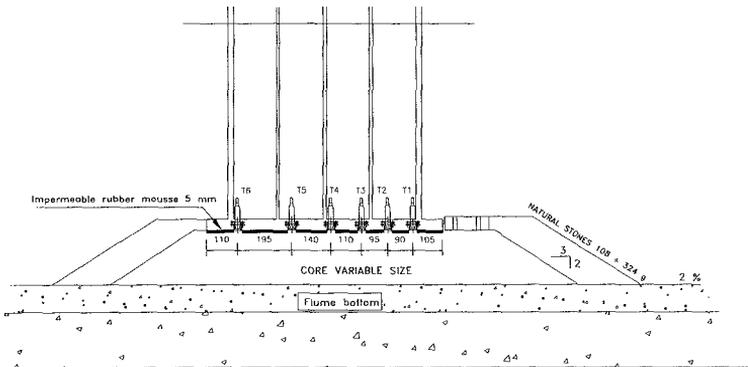


Fig. 2 Model set-up for rubble foundation material tests.

A sensitivity analysis has been conducted on the scale effect of the grain size distribution of the rubble mound foundation in terms of uplift pressures. Three grain size curves of the rubble mound foundation with different density and gradation have been tested (cores A, B, C). Limestone rock was used with diameter D_{10} varying from 1.8 to 20 mm and diameter D_{50} varying from 3 to 23 mm. The average grain size has been scaled with respect with the Froude number. Further details are given in de Gerloni et al. (1997). Prototype and model data were analysed in terms of time-sequences values and statistical parameters associated to pressure waves, obtained after zero-down-crossing analysis. A first comparison with prototype data was done by looking at comparable runup time sequences in the model and prototype (Fig. 3).

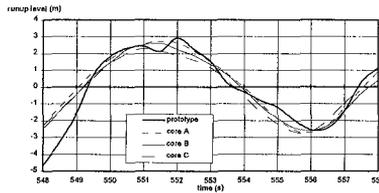


Fig. 3 Storm 1 - Prototype and model runup levels at the vertical wall

Such comparison doesn't take into account the influence of runup history previous to the selected sequences, which has been proven to be negligible. A strong difference between prototype and model wave uplift pressure on the caisson base (the latter being twice as large) is apparent especially at the seaward edge (Fig. 4 right), but when approaching the harbour edge wave effects get smaller and they cannot be distinguished from the "noise". It is also shown that the grain size has little influence.

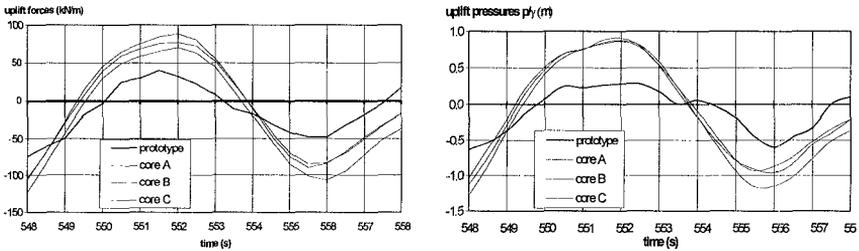


Fig. 4 Storm 1-Prototype and model uplift forces (left) and pressure (right) at 2.1m back from seaward edge

Also the triangular pressure diagram recorded in the model at the moment of maximum uplift force (Fig. 5) appears not to be affected by the model grain size, or conversely the prototype is characterized by a non uniform transversal transmissivity which may be explained by cyclic tilting loading at the edges, as reported by Van Hoven (1997). In fact, neither Le Mehauté (1965) theory (suggesting a grain size scale from 1/16 to 1/11, close to core C scale 1/10) nor Jensen & Klitting's (1983) studies (suggesting a scale 1/18 close to core B scale 1/20) seem able to justify the differences found. Similar uniform diagram shapes are observed in the troughs of pressure waves, too. Alike conditions had also been noticed in previous prototype measurements at Molo Cornigliano of Genoa Harbour, in 1975 (possibly with the foundation silted up on the harbour side), in small and large-scale model tests (Marchi et al., 1975; Kortenhuis et al., 1994) and in prototype measurements at Dieppe caisson (ULH Group, 1998).

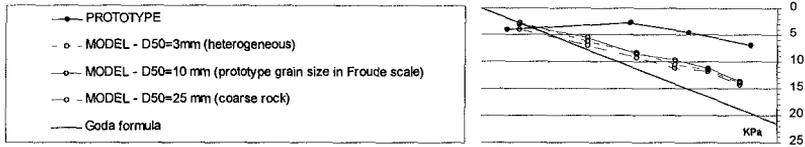


Fig. 5 Prototype, model and design formula for underpressures at max. uplift force. (storm1)

In order to verify directly the actual conditions of the caisson foundation at Porto Torres a scuba diving inspection was carried out by the first author in October 1997. Observation was made along some 150 m of breakwater at the instrumented sections in the transition trunk between plain and perforated caissons on both sides. The outer apron slabs at toe were found to be placed very carefully/regularly one to each other with no more than 2 cm distance from caisson toe. No sand or fine material was observed at toe either sides. Gravel and small stones were found beneath the armour slabs. Info from construction divers confirms that the rubble foundation top was levelled with some 20 cm of gravel before accurate and smooth caisson placement.

The actual non-triangular shape of the uplift pressure diagram can be also explained by the obliquity and shortcrestedness of the real incident waves (recorded storm waves showed a mean attack angle up to 30° to the normal to the breakwater axis). As shown by Franco et al.(1996) after a systematic 3D model study on plain and perforated caissons, the uplift pressure diagrams under 3D waves can have a concavity that shifts the resultant towards the seaward toe (increasing the overturning moment) and the total uplift force is generally overestimated by Goda. This is also consistent with storm 1 records where the Goda design formula describes the triangular model diagram but overestimates uplift pressure values especially near the seaward edge.

Horizontal loads on plain walls

With the same method as for uplift pressures, the horizontal pressure time-series on the vertical wall have been compared between prototype and model for specific time intervals in which runup levels were in satisfactory agreement. In Fig. 7 (left) two different prototype time intervals having the same peak elevation and one model time interval in good agreement with them are plotted; the corresponding model and prototype (seq. a) time sequences of front pressure at -2.90 m M.S.L. during storm 1 are shown in Fig. 6 (right) and all the pressure data recorded at each sampling time in the model are compared with those in the prototype in Fig. 7. They show that with similar runup sequences, model pressures are on average larger than the prototype ones ($\approx +50\%$ in wave troughs, $\approx +20\%$ in wave crests).

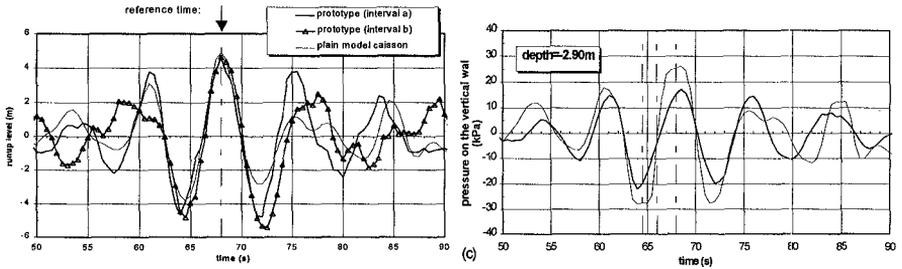


Fig. 6 Model compared with two prototype water levels (left) and model compared with prototype pressure time series (right).

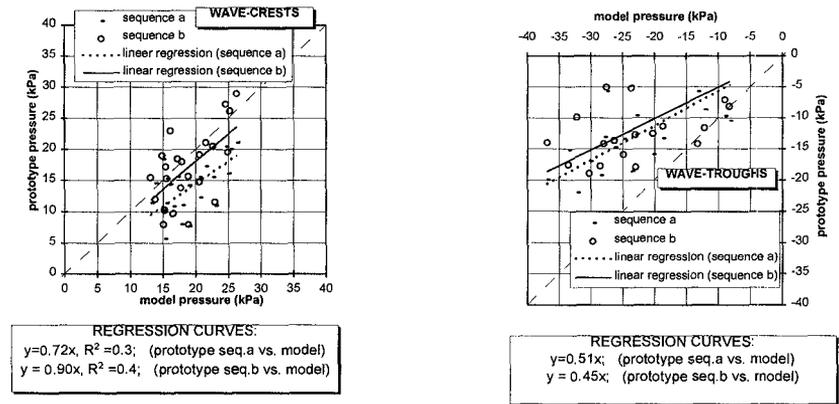


Fig. 7 Model and prototype pressure measurements under similar wave runup crests (left) and troughs (right)

The overall actions on the model and prototype caissons at comparable runup sequences are plotted in Fig 8: differences are evident in both pressure gradients (uplift pressures and around the peak of the front pressure diagram) and in absolute values; under wave crests the differences are reduced.

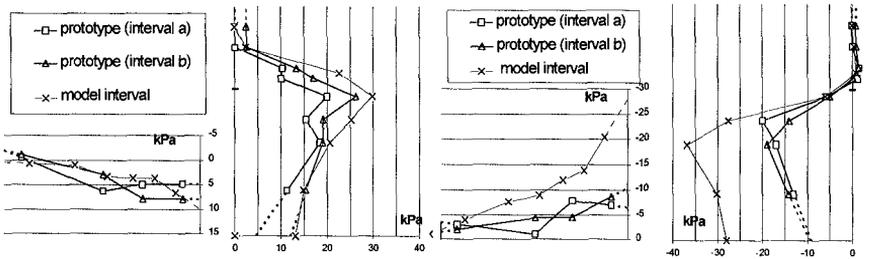


Fig. 8 Uplift and horizontal pressure diagrams under wave crests (left) and wave troughs (right) as in Fig. 6 for the plain wall caisson.

The pressure diagrams proposed by Goda formula are compared in Fig. 9 with the corresponding model and field measurements at the different instants of maximum horizontal and uplift force: Goda method is further “conservative” because it assumes simultaneously the max values of horizontal and uplift forces.

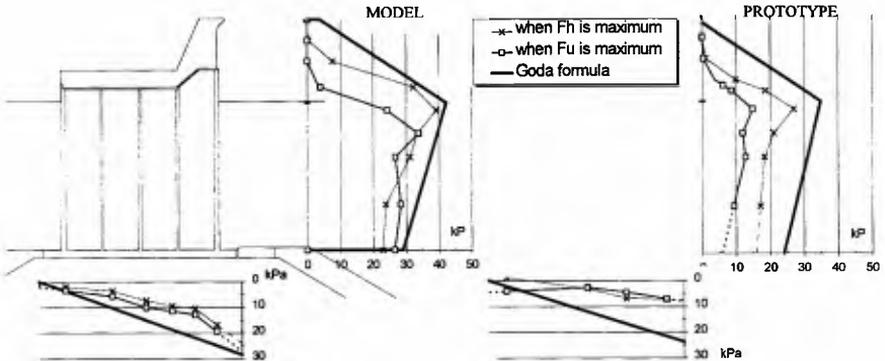


Fig. 9 Maximum forces diagram compared with Goda prediction (left: model tests; right: prototype measurements).

Both model and prototype pressure data were integrated with linear interpolation over the vertical face and along the bottom; crests extracted with the zero-down crossing method are well fitted by the 2-parameters Weibull distribution (Fig. 10). Prototype shape parameter (α) relative to horizontal forces F_h is well reproduced in the model but the corresponding scale parameter (β) is in the average 72% smaller, which means that the F_h distribution is the same but shifted towards higher values in the model; F_u distributions have both different shape and scale parameters.

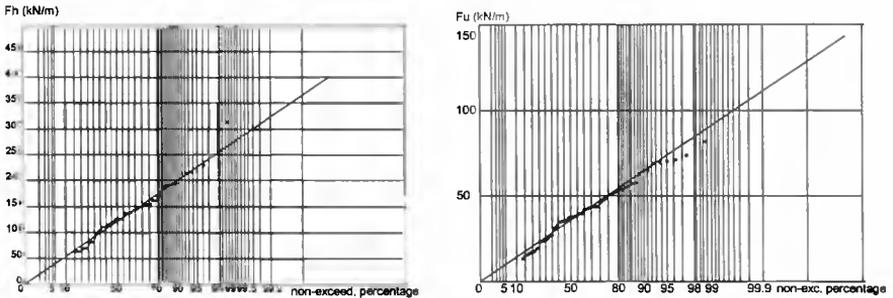


Fig. 10 Prototype F_h (left) and F_u (right): Weibull distribution of upper 85% (Storm 1)

Statistical estimates of horizontal and uplift forces ($P=95\%$, 98% , 99% , 99.6% , 99.85% , 99.9%) have been compared with the corresponding model ones and also with the extreme estimates ($P=99.85\%$) according to the Goda method. Forces calculated with Goda formula are on average higher than the corresponding estimated ones, especially prototype F_h (Fig. 11). The lab force data are, on the average, 35%

and 40% larger than those of the prototype respectively for F_h and F_u as shown in Fig. 12.

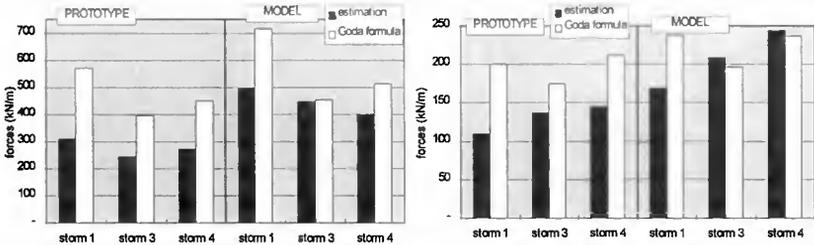


Fig. 11 Comparison of estimates of F_h (left) and F_u (right) forces from 99.85 % Weibull estimates and Goda formula.

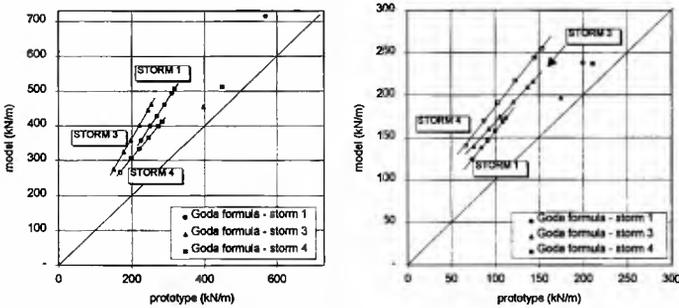


Fig. 12 Extreme horizontal (left) and uplift (right) forces: model and prototype.

Horizontal loads on perforated wall

In the same way as for the plain caisson, the perforated wall caisson was instrumented with pressure transducers on the bottom slab and on each vertical wall, and also with a dynamometer for global horizontal forces measurements. In order to investigate the pressures on the internal walls, the tests were repeated by turning backwards the pressure transducers placed on the perforated walls (Fig. 13).

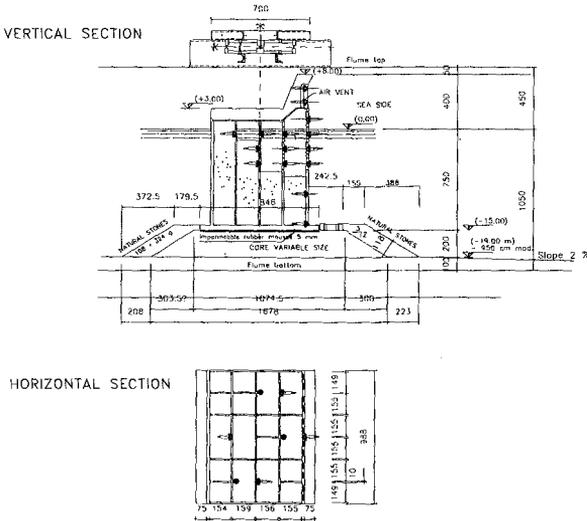


Fig. 13 Setup of perforated caisson model with pressure transducers turned backwards

The model pressure data, recorded in the external and internal walls, have been integrated over the vertical face using the Kriebel (1992) and the Canel (1995) method. Both results exhibit non negligible discrepancies when compared with the lab dynamometer data (Fig. 14). This may be explained both by the intrinsic overestimate of the Goda distribution also on plain walls (up to 20%) and by the lack of knowledge about the true pressure distribution on the perforated walls.

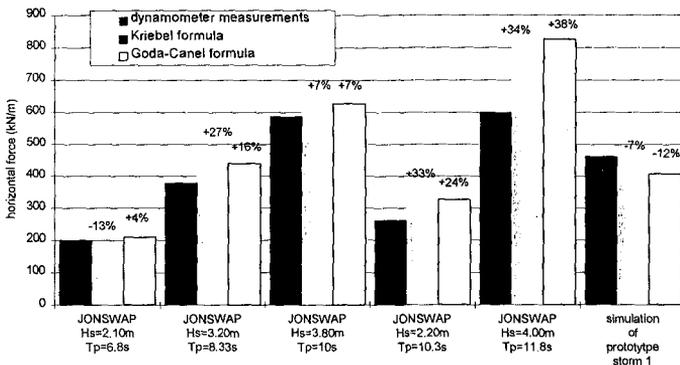


Fig. 14 Comparison between lab data, Goda and Kriebel estimates.

The model test results, in terms of statistical distributions, also indicate that the perforated caissons can be subjected to larger horizontal loads than the plain ones when extreme waves attack the structure (Fig. 15 left). This appears different when

looking at the negative forces (Fig. 15 right) for which the perforated caisson behaves efficiently and more consistently in the load reduction.

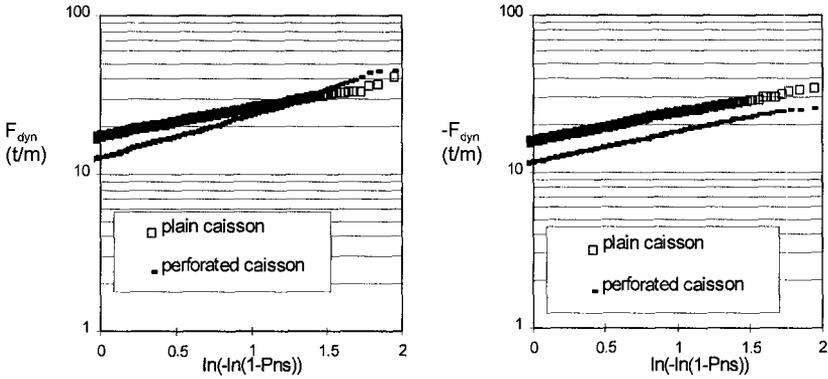


Fig. 15 Force Pdfs at wave crests (left), troughs (right).

A simple formula was looked for to calculate wave forces on perforated caissons as an improvement of Canel's (1995) one that is a modified version of Goda formula itself. By Canel method the hydrodynamic load due to an incoming wave of height H on a partially reflecting structure is estimated by the Goda model with a suitably corrected design wave

$$\left\{ \left(\frac{1 + Cr}{2} \right) * H \right\} \tag{1}$$

in which Cr is the expected reflection coefficient. This approach was suggested by the observation of Goda model that establishes a linear relation between the hydrodynamic effects of waves and their height in most hydraulic situations. Values of Cr around 0.5 were measured in the flume for the specific caisson with wall porosity of 0.31.

In order to give an input as general as possible to that formula, a simple modification of the design wave height involving only the dimensionless parameter B/L (where B is the chamber width and L the wavelength), instead of the reflection coefficient Cr was found. The total horizontal force F_h by Goda model was worked out and compared with the corresponding measured values for each available perforated structure data set, coming from different experimental set-ups (University of Le Havre ULH, Leichtweiss Inst. Braunschweig LWI, as reported in MASTIII-PROVERBS 2nd Overall Project Workshop). Fig. 16 shows the comparison of the experimental data with the corresponding Goda values; as expected all the data are below the matching line.

The difference in percentage

$$\left\{ \frac{(F_{h\text{GODA}} - F_{h\text{MEAS}})}{F_{h\text{GODA}}} \right\} \tag{2}$$

with respect to the Goda calculation resulted on average equal to 30 % for ENEL and PM/DH 3D random wave tests (Franco et al., 1996) and 24 % for ULH regular wave tests.

A reduction coefficient was calculated for each experimental datum in order to make the measured and calculated force coincident. Fig. 17 shows the coefficient trend as a function of the dimensionless parameter B/L. A linear model to describe the relationship between the reduction coefficient of the Goda's H_{max} and B/L parameter was found. The equation of the fitted model is:

$$\text{reduction coefficient} = 1 + a * (B/L) \tag{3}$$

where:

$$a = -1.43$$

$$\sigma_a = 0.08$$

$$R^2 = 0.52$$

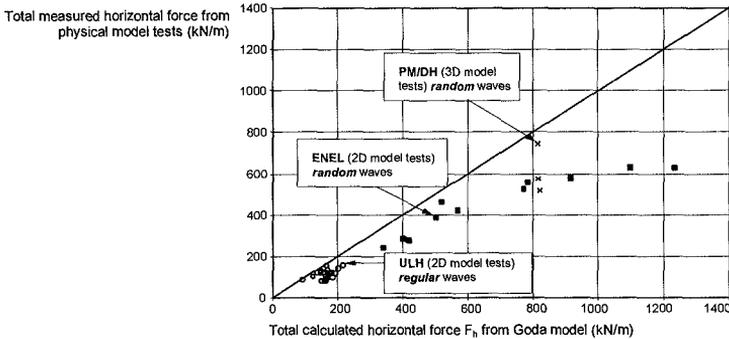


Fig. 16 Comparison between measured vs calculated forces

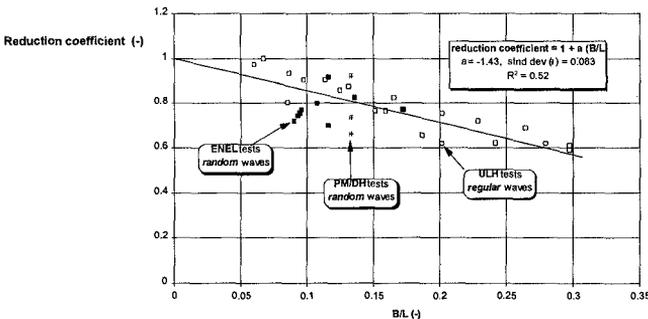


Fig. 17 Reduction coefficient trend as a function of the dimensionless B/L parameter

The reflection (C_r) and the reduction coefficient $[(1+Cr)/2]$ trends are shown in Fig. 18 as a function of the dimensionless parameter B/L . The linear model adopted for the reduction coefficient is therefore conservative for a total chamber width B less than about a quarter of wave length, while it gives lower wave heights (and then forces) for greater values (though the more B/L increases, taking B constant, the less becomes the force).

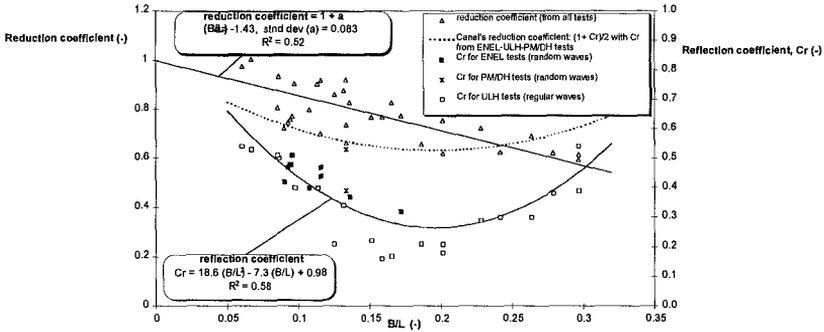


Fig. 18 Force reduction and reflection coefficient as a function of B/L

For such analysis both field and lab data were used, but in the prototype caisson the actual total horizontal force is unknown while in the lab the total horizontal force have been measured.

Conclusions

The behaviour of caisson type breakwaters is generally less severe than predicted by the Goda method both with respect to the horizontal and vertical loads. The real uplift pressure distribution is not triangular as conventionally assumed, but a trapezoidal diagram is more reasonable. With respect to the perforated caissons, the global forces are not well described by the standard design methods. The overall performance of this structure types is effective in the reduction of water levels and negative forces, but they can show an opposite tendency for the horizontal forces at the highest wave crests. A new simplified formula for the preliminary design of multichamber perforated caissons is also proposed.

Acknowledgements

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