

Prototype Measurements of the Dynamic Response of Caisson Breakwaters

Alberto LAMBERTI¹ and Luca MARTINELLI¹

Abstract

Prototype tests, aiming to describe the dynamics of caisson breakwater oscillations, were carried out at Genoa Voltri and at Brindisi Punta Riso. The rigid body natural frequencies and modes of oscillation of the caissons were evaluated as well as the rate of damping. The effect of the longitudinal array structure, never tested in physical models, was found extremely relevant; the Mass Spring Dash-pot model for an isolated caisson of Oumeraci & Kortenhaus (1994) was then conveniently updated in order to represent an array structure, showing finally good agreement with the recorded signals.

1. Introduction

When vertical breakwaters are subject to breaking waves, the plane front wall is hit by impulsive pressure loading, which, provided the wall resists the load, will accelerate the caisson causing its oscillation. In these conditions only part of the applied load will be transferred to the foundation and may cause failure, normally by displacing the caisson (Franco, 1994).

Several models were proposed in literature in order to simulate the caisson dynamics: Petrashen (1956), Loginov (1958), Hayashi (1965), Benassai (1975), Smirnov & Moroz (1983), Marinski & Oumeraci (1992), Goda (1994), Oumeraci & Kortenhaus (1994). The last two provide calibrations based on physical models that do not consider the effects of the longitudinal structure of the breakwater. Prototype measurements were carried out only by Muraki (1966), who identified a single system eigenfrequency (≈ 0.2 Hz) but apparently the identified oscillations were not coherent with the wave force that was generating them.

Given the overall lack of knowledge of the response of a vertical caisson subjected to impulsive waves, the EU has financed a project, PROVERBS, with the aim of providing information and tools to allow the design of vertical breakwaters with a desired probability of failure, accounting for the mentioned dynamic behaviour. University of Bologna, within the project, was charged of exciting artificially some prototype caissons and measuring their dynamic response, in order to verify existing models of caisson dynamics and to check errors in the estimate of soil parameters.

In this paper the prototype tests and analysis will be briefly described (chapter 2); the analysis of the movements of the excited caisson showed the presence of natural modes of oscillation that could not be interpreted through the simple models present in literature (chapter 3), based on the dynamic description of an isolated caisson. The analysis of movements of the caissons adjacent to the excited ones suggested that these dynamic models should be adapted in order to represent the caisson array structure (chapter 4). In particular the model by Oumeraci & Kortenhaus (1994) was updated and calibrated, resulting suitable for the simulation of all the observed modes of oscillation (chapter 5).

¹ Università di Bologna, DISTART Idraulica, viale Risorgimento 2, I-40136 Bologna, Italy
E-mail: lamberti@idraulica.ing.unibo.it



Fig. 1 2 tons sand sac used for caisson excitation.



Fig. 2 100 tons tug-boat hitting the caisson

2. Description of the tests

The prototype tests were performed in Genoa Voltri and Brindisi, during 1997. The tests were carried out hitting a vertical breakwater with a sac and/or with a tug-boat in order to produce a significant dynamic response.

The sac, half filled with sand and weighting 2 tons, see Fig. 1, could fall completely free from a height of 5 m (or partially slowed down from a greater height) and it hit the caisson in proximity of the harbour edge, i.e. with a strong eccentricity. One accelerometer was installed on a "buoy" right into the sac on the sand surface, in order to give information on the applied force.

Fig. 2 shows the excitation due to a tug boat displacing 100 tons and hitting a caisson in Genoa Voltri (a 500 tons tug boat was used in Brindisi). The speed before the contact was close to 0.3 m/sec, varying on the occasions, and the impact force lasting about 0.5 sec for the smaller tug (1.0 sec for the bigger one) was evaluated by one accelerometer fixed on the tug-boat in the longitudinal direction.

The accelerations of the excited caisson and of the two adjacent ones were measured by 15 accelerometers placed as shown in Fig. 3. 9 accelerometers are placed on the central caissons, 1 describing the movements in the longitudinal direction and 8 describing the movements in the perpendicular plane. The adjacent caissons were monitored with 3 accelerometers each. In total, seven groups of three adjacent caissons were monitored, belonging to three vertical breakwaters (main breakwater in Voltri, western lee breakwater in Voltri and main breakwater in Brindisi, see Fig. 4) similar in shape but different in size (3 10³ kg, 1 10³ kg and 2 10³ kg respectively). The joints between adjacent caissons are wide (5-10 cm) only for the case of the main breakwater in Genoa, but in all cases the superstructure has joints at the ends of each caissons. The rubble mound height vary from 2 m to 15 m for the different cases, and the foundation has a first layer of clayey-silty sand over a more fine material and rock below. A detailed description of the hydrodynamic conditions, caisson geometry, structural and foundation aspects are given in Lamberti et al (1998) and Lamberti & Archetti (1998).

In order to reduce the high frequency vibrations produced by sharp impacts, and thus avoid amplifier saturation, the accelerometers of the central caisson were fixed on a concrete cubes acting as mechanical filters. Since vibrations are greatly reduced passing from the central excited caissons to the adjacent ones, only the instruments placed on the central caissons needed to be mechanically filtered.

The filter (See Fig. 5) is formed by a concrete cube (side of 20 cm) glued to the superstructure through 4 round rubber disks. The response of this filter to an impulse is shown in Fig. 6. Note that the natural frequency of oscillation (≈ 20 Hz) is out of the range of interest ($[1+8]$ Hz) and that the damping is rather strong.

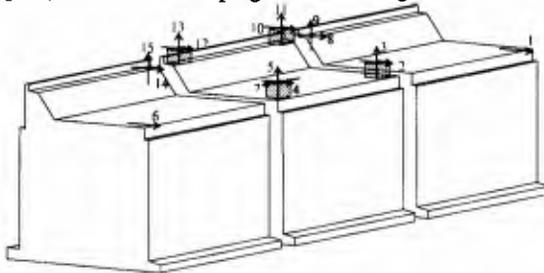


Fig. 3 Position of accelerometers.



Test sites
 V1, V2, V3 3rd -5th Jun 1997
 B1, B2 30th Sep-1 Oct 1997
 W1, W2 19th 20th Nov 1997

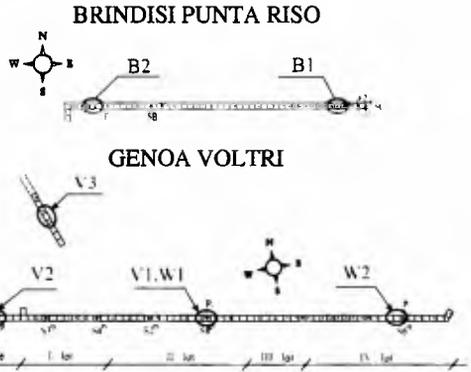


Fig. 4 Position of tested caisson in tested breakwaters. Symbols starting with letters S, P or F are relative to several geotechnical drillings, analysed in Lamberti & Archetti (1998) and Lamberti et al. (1998)

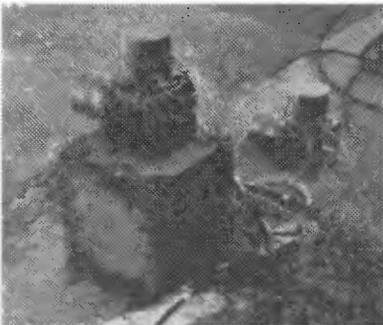


Fig. 5 The accelerometers of the central caissons were fixed on a concrete cube

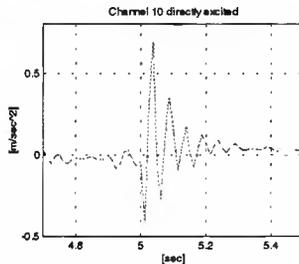


Fig. 6 The concrete block was excited by a hammer, and this is the response of one of the accelerometers fixed on the block. The eigenfrequency of the mechanical filter is about 20 Hz and the damping is high: amplitude is halved in one cycle.

3. Analysis of tests results relative to the hit caisson

The analysis started with the quantitative definition of the original applied force signal. For the sand sac exciting case the accelerometer direction was controlled fixing the accelerometer to a round and flat disk inserted into the sac, that could 'float' over the sand. The sac is not rigid and the average acceleration could not be measured; the signal was used for the evaluation of the length of the impact and for the force-response synchronisation. The force history was then defined assuming an anelastic behaviour, knowing the sac weight, the velocity before the impact and its duration. For the tug-boat excitation, the acceleration during the contact is actually proportional to the real applied force (a 10% hydraulic added mass was considered).

All the 16 channel registrations of the same type (same caisson, same kind of excitation) were cut 2 seconds before the impact and 8 seconds after it, analysed in the frequency domain and bandpass filtered (in Tab. 1 the range for the different tests is given), and pasted into a unique *sequence* in order to have data in a single file. Furthermore, *phase averaged signals* were created averaging each channel for the same kind of excitation. The records were synchronised maximising the cross correlation.

The interpretation scheme at the base of the analysis is that caissons can be considered as rigid bodies. Oumeraci & Kortenhaus (1994) described a Mass, Spring and Dash-pot (MSD) model where contributions to mass, stiffness and damping are due to rubble mound and foundation and to seawater. Such a system has three natural oscillation modes (see Fig. 7):

1. the *sway mode* (m_1): an almost horizontal translation (or rotation around a low centre);
2. the *roll mode* (m_2): a rotation around a higher centre;
3. the *heave mode* (m_3): an almost vertical translation.

In order to describe caisson movements a reference system was defined, the origin being in proximity of the instrumentation in order to reduce the overall error; components are:

1. *sway*, or harbour directed translation (ξ_o),
2. *heave*, or vertical translation (η_o),
3. *roll*, or rotation around a longitudinal axis (θ).

The choice of the pole 'O' (points of which displacements ξ_o and η_o are given) is arbitrary and the relation between any other pole position $\{x_o^c, y_o^c\}$ and its displacement is trivial (see Fig. 8):

$$\begin{aligned} \xi_c &= \xi_o + \theta_o y_o^c \\ \eta_c &= \eta_o - \theta_o x_o^c \\ \theta_c &= \theta_o \end{aligned} \tag{1}$$

(index is then usually omitted for rotations)

The rigid body movements of the central caisson were extracted combining the eight horizontal and vertical accelerations of the central caissons with a best fit procedure: the sway signal is approximately obtained by averaging the four original horizontal signals; the heave is approximately the average of the vertical signals and the roll is similar to the difference between the upper and lower horizontal and vertical accelerations divided by the respective distance of the instruments.

A similar procedure was applied to the adjacent caissons, whose rigid movements have been averaged over the two caissons.

The following analysis consisted in assessing the Power Spectral Density of the responses and the Transfer Functions between force and response, both for phase averaged signal and sequence of tests. The

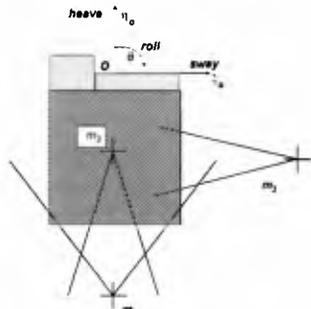


Fig. 7 Scheme of the theoretical oscillation modes of an isolated caisson and reference origin adopted for system identification.

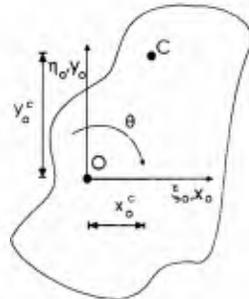


Fig. 8

power peaks coherent with the force for each signal were then evaluated, see Fig. 9 for instance. Since the force showed a rather flat spectrum in correspondence of peaks of the signals, their frequencies were interpreted as the natural frequencies of oscillation.

Signals were then band-pass filtered around the natural frequency.

Vibrations are only partially filtered out from the rigid body signal, they are finally recognised comparing the single and fitted rigid body signals in the appropriate frequency range.

The damping of the natural oscillations was assessed measuring, for each mode, the rate of amplitude decrease of the *phase averaged signals* after the impact. Similarly, comparing for each mode the acceleration amplitude and phase of the central and adjacent caissons, the amount of energy that travels along the breakwater was evaluated.

The upper graph of Fig. 10 shows the acceleration at quay level (precisely the phase averaged sway signal) induced in Brindisi by the tug-boat. It is possible to note immediately the presence of two different harmonics. The harmonics are presented in the lower graph of the same figure, and they are obtained bandpass filtering the signal around [0.5-1.8 Hz] and [1.8-6 Hz] (e.g. around the frequency peaks observed in the roll and sway signals, see Tab. 1).

The sum of this two harmonics reproduces almost exactly the original signal (almost undistinguished dotted line in upper graph). The same two harmonics are present in the 'roll signal' describing two rigid body rotations, i.e. two modes.

The rotation centre can be assessed band-pass filtering the *average signal* around the rigid mode frequencies and evaluating the ratio between sway and roll, and/or, for the central caissons, evaluating the acceleration directions at the four corners.

The power of the horizontal acceleration at any point can be easily evaluated combining sway and roll signals: the power of the horizontal acceleration is minimum if the pole is placed at the height of the rotation centre (Eq.1 governs the affect of changing the reference system).

Fig. 11 shows the relative power of the horizontal acceleration at different heights for the two different frequency bands: it is clear that the minimum of the curve, i.e. the position of the rotation centre, is placed below the caisson base.

Similar results are found for the cases of Voltri: this means that two modes have two low rotation centres, in total disagreement with the assumed mathematical model described in Fig. 7, according to which the higher frequency mode (present in the sway

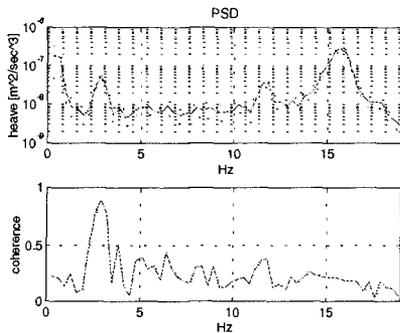


Fig. 9 PSD (with confidence interval 95%) of the fitted vertical acceleration of the central caisson and coherence with the force in case of sand sac impact (the recorded signal saturated, the reconstructed force signal is not well reliable at high frequencies, and this explains the low coherence of the two peaks above 10 Hz).

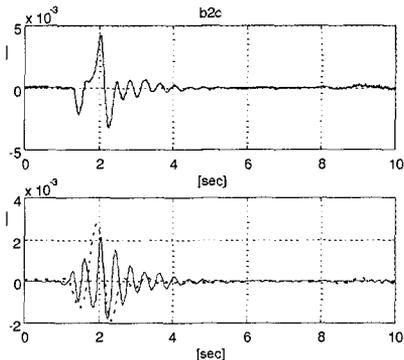


Fig. 10 The sway acceleration induced in Brindisi by the tug-boat (upper graph), is the sum of two harmonics, presented in the lower graph, and they are obtained bandpass filtering the signal around [0.5-1.8 Hz], dotted line, and [1.8-6 Hz], full line). The sum of this two harmonics reproduces almost exactly the original sway signal (dotted line in upper graph).

and roll signal) should have a high rotation centre.

Fig. 12 represents, maybe more clearly, the same effect: within the frequency range of the 2nd observed frequency, the corners of the caissons move in the directions shown by the two graphs, obtained plotting the horizontal vs vertical acceleration. The rotation centre is located at the intersection of the two radii orthogonal to the displacement or acceleration: evidently a rotation around a very low centre is taking place.

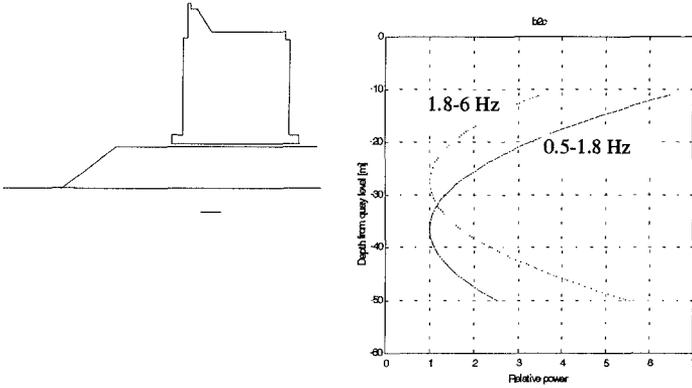


Fig. 11 The horizontal acceleration power (divided by its minimum value) in the two frequency bands (see Fig. 10) is presented as function of the distance from quay level. The minimum value of the acceleration power is considered as a good estimation of the height of the rotation centre. The rotation centre is placed well below the caisson base (caisson height = 21.7 m).

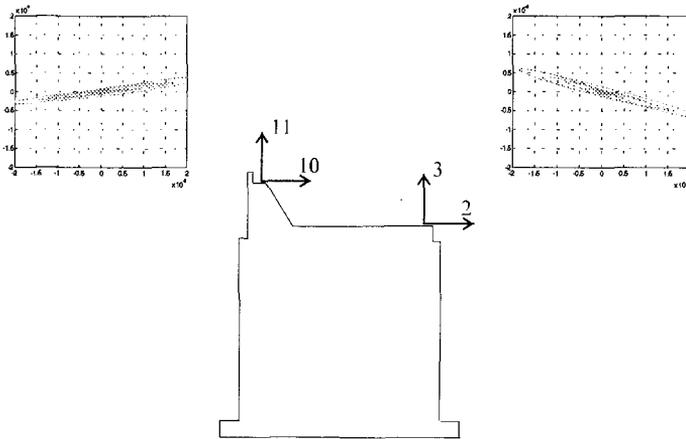


Fig. 12 The accelerations of the corners of the caissons describe a rotation around a very low centre. The graphs are obtained plotting the horizontal vs vertical acceleration in the higher frequency band [1.8-6 Hz], which according to the single caisson model is instead associated to a mode with a high rotation centre.

Tab. 1 The natural periods of oscillation were assessed as well as the associated rigid movements: for the almost horizontal oscillation, which are rotations around low centres, the actual depth below the base of rotation centre (R.C.) could be identified comparing the sway and the roll signal. Superstructure vibrations (v) are present only in case of sharp impacts and if the band of frequency, given in the rightmost column, is large enough. Non accurate data are given in square brackets.

| Test Type | 1 st Frequency Sway/Roll signal [Hz] | R.C. below base [m] | 2 nd Frequency Sway/Roll signal [Hz] | R.C. below base [m] | Frequency present in Heave signal [Hz] | Vibrations [Hz] | Band of frequencies considered [Hz] |
|-----------|---|---------------------|---|---------------------|--|-----------------|---------------------------------------|
| V1A | not evident | | not evident | | not evident | | 0.5-9 |
| V1B | [2.3] | | [3.2] | | not evident | | 0.5-9 |
| V1C | 1.4 | | [2.3] | | not excited | | 0.5-9 |
| V2A | [1.2] | | 2.5 | | 3.0 | | 0.5-9 |
| V2B | 1.4 | | 2.7 | | not evident | | 0.5-9 |
| V2C | 1.4 | | 2.5 | | not excited | | 0.5-9 |
| V3A | 1.8 | | 3.6 | | 4.3 | | 0.5-9 |
| V3C | 1.8 | | 3.6 | | not excited | | 0.5-9 |
| B1A | | - | 2.4 | - | 2.5 | 10 | 0.2-19 |
| B1C | 1.4 | 10 | 2.4 | 7 | not excited | not excited | 0.2-19 |
| B2A | | | 2.5 | [0] | 2.7 | 10 | 0.2-19 |
| B2C | 1.4 | 17 | 2.4 | 8 | not excited | not excited | 0.2-19 |
| W1A | | | 2.7 | [0] | 2.5 | 15 | 0.2-19 |
| W2A | | | 2.7 | [0] | 2.7 | 15 | 0.2-19 |
| W2C | 1.3 | 10 | 2.7 | 6 | not excited | not excited | 0.2-19 |

Tab 1 summarises the results of the analysis: the identified frequencies and modes. Two natural frequencies of oscillation were identified in the sway and roll signals plus one in the heave signal (see Fig. 9). The two modes recognised in the roll and sway signals represent rotations around low centres.

Even disregarding the position of the rotation centres, a 3 DOF model for isolated caisson could not be calibrated interpreting the observed oscillations as modes m_1 and m_2 ; the ratio between the measured eigenfrequencies was lower than foreseen by the model for any value of rotational and horizontal stiffness. Also using the model of the foundation suggested by Goda (1994), it was impossible to obtain a calibration without assuming an unrealistic anisotropy of the foundation.

In conclusion the test analysis pointed out three main modes, two of which are rotations around low centres (two m_1 modes, in disagreement with the system described in Fig. 7) and one is a vertical displacement (m_3).

4. Effect of adjacent caissons and interpretation of the identified modes of oscillation

The oscillation amplitudes of the caissons adjacent to the central one were almost one third compared the central one and significantly delayed; a large amount of energy is then subtracted from the central caisson by waves propagating along the breakwater. The effect of this wave can not be completely described by the considered single caisson model.

Comparing the horizontal oscillation of the central and adjacent caissons in a single frequency band (see Fig. 13), it was possible to observe that the oscillations were almost in phase in the frequency band [0.5-1.8 Hz] (around 1st observed frequency) and almost in opposition of phase in the frequency band [1.8-6 Hz] (around 2nd observed frequency).

A possible interpretation of the two identified sway modes involves then the longitudinal dimension: the 1st observed mode is given by a movement of all the caisson in phase, the 2nd observed mode is given by an alternate movement of these.

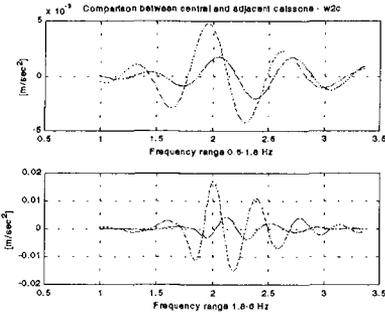


Fig. 13 Horizontal accelerations induced by the tug at Brindisi on the central and adjacent caissons: the oscillations are almost in phase in the frequency band around the 1st observed frequency (upper graph) and nearly in opposition of phase in the frequency band around the 2nd observed frequency (lower graph).

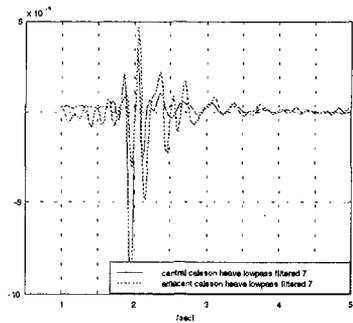


Fig. 14 7 Hz lowpass filtered heave for central and adjacent caissons. The frequency peak in the heave signal describes in phase movements of the central and adjacent caisson. Any other combination of vertical modes was not identified, probably due to the high environmental noise.

The MSD model was then modified considering an array of caissons, formed by several MSD modules connected one another by spring and dash-pot elements. The added stiffness was estimated as a fraction of the stiffness between caisson and foundation, and the proportionality coefficient was calibrated so that the adjacent caisson accelerations do fit the actually measured ones. Damping coefficients were not calibrated directly, since the model describes damping effects directly on the uncoupled modes.

Since the system is symmetric and it was excited symmetrically, only the symmetric movements were considered in the model.

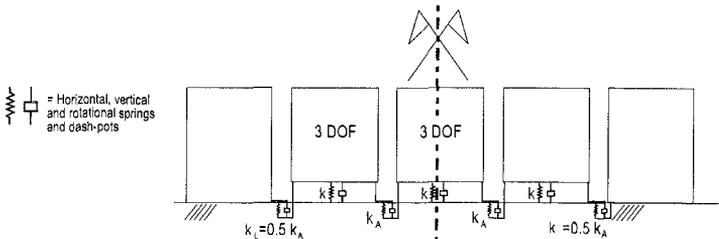


Fig. 15 6 DOF model considering the 3 DOF of the central caisson and 3 symmetrical DOF of the adjacent caisson on the plane perpendicular to the longitudinal direction of the breakwater.

A boundary condition must be inserted if the represented array of caissons is shorter than the actual one. If it is assumed that the last caisson is fixed and the stiffness between the end caissons and the adjacent ones (K_L) is equal to the stiffness between any two other caissons (K_A), the system results more stiff than it actually is; the opposite case is found if the last caissons are supposed to be free to move, i.e. $K_L=0$. The assumed lateral condition was then a compromise, consisting of a reduction of the last stiffness coefficient to 50 % of the others ($K_L/K_A=0.5$). The effect of this assumption is reduced if

more caissons are considered, and for an array of 7 caissons the two limit cases do not affect significantly the acceleration of the central caisson.

Let's consider, for simplicity, a model of just three caissons: the output is not totally satisfactory, but it is possible to highlight the most important characteristics. The system has 6 DOF (three movements of the central caisson and three symmetrical movements of the two adjacent ones) and thus 6 eigenmodes. Fig. 16 shows that two of these eigenmodes are actually formed by rotations around low centres: one is due to all the caisson moving together and the other to a movement in opposition of phase. The observed natural oscillations (Tab. 1) can be interpreted along this line.

When the adjacent caissons move in phase with the central caisson (superscript +), the geodynamic added mass is reasonably bigger than in case of a movement in opposition of phase (superscript -), since in the second case part of the foundation between adjacent caissons is resting (not moving). This effect explains different heights of the 'm₁' rotation centres (see Fig. 11). Such difference is less important for the case of Voltri, where the longitudinal dimension of the caisson is bigger (the caisson length is 30.1 m in Voltri, 21.0 m, in Brindisi). A small mixed term in the geodynamic added mass matrix was considered (only 1/3 of the geodynamic added mass is considered for an opposition of phase) in all the simulations.

Kinematic of the system with 6 DOF

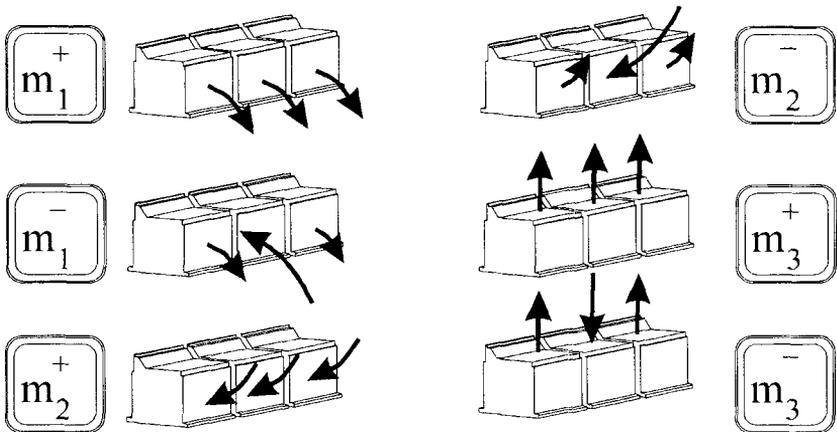


Fig. 16 If the two adjacent caissons are considered in the dynamic system, each eigenmode described in Fig. 7 appears twice: the mode due to all the caisson moving together (superscript +) and the mode due to a movement of the caissons in opposition of phase (superscript -).

In heave oscillations just one frequency peak was clearly identifiable, which was apparently associated to an 'in phase' motions of the caissons (see Fig. 14) and it was thus interpreted as mode m₃⁺. Mode m₃⁻ and of type 'm₂' were not identified, probably due to the low signal/noise. In the next chapter Tab. 2 shows that modes 'm₂' are almost not excited by the tug, while for the falling sac excitation case in Fig. 22 a comparison between simulated and measured PSD of the roll signals is presented, showing that computed 'm₂' frequency peaks in the range 3.9-7.9 Hz could be hidden by the environmental noise.

5. Numerical simulations and calibration of the final model

The final simulations were performed considering a 12 DOF model presenting an array of 7 caissons.

The horizontal, vertical and rotational stiffness between caisson and foundation depend on the shear modulus G , and, secondarily, on the Poisson coefficient ($=0.4$). G was evaluated assuming $E_o=320$ MPa (Young modulus for average pressure = 100 kPa);

$$E \propto \sqrt{\sigma_v \frac{1+2K_c}{3}}, \text{ vertical pressure } \sigma_v=350 \text{ kPa, coefficient of lateral confinement } K_c=1.$$

The horizontal and rotational stiffness between central and adjacent caisson (K_A) was calibrated (in a preliminary way) as 40 % of the stiffness between caissons and foundation (K_C) given by the elastic homogeneous half space theory. For vertical oscillations, the stiffness of the link with the adjacent caissons was calibrated as 60 % of the stiffness with the foundation.

Fig. 17 shows the simulation of the horizontal acceleration at quay level for the case of Voltri (W2). The exciting force due to the tug boat is presented in the upper graph. The simulation is compared with phase averaged sway signal of the central caisson. Also the recorded rotational oscillations were found to be well simulated.

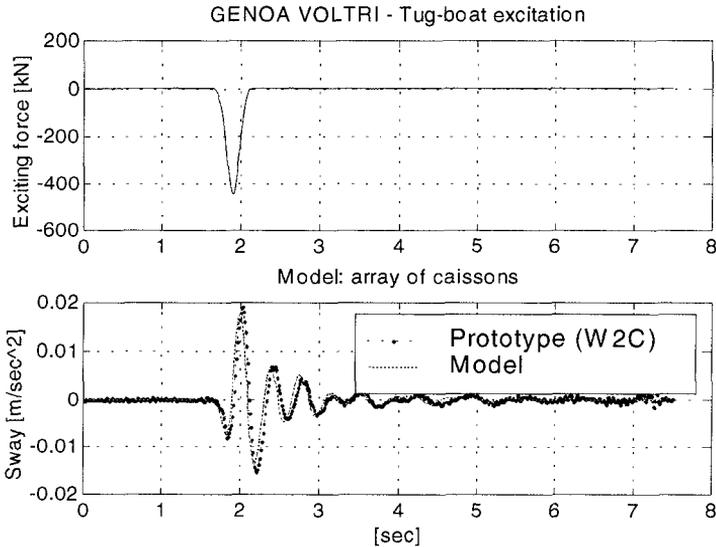


Fig. 17 Simulation of horizontal acceleration at quay level for the case of Voltri, induced by the tug-boat excitation.

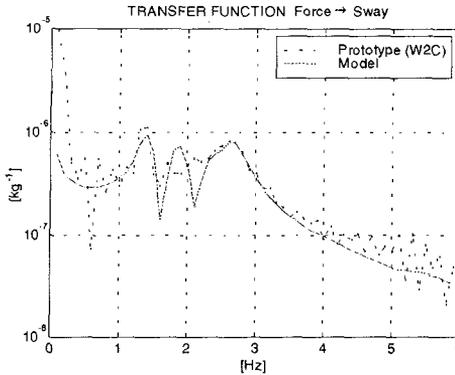


Fig. 18 Transfer Function between exciting force and accelerations shown in Fig. 17

Comparison between the model and experimental transfer function between such acceleration and the applied force, presented in Fig. 18, is extremely relevant. The first peak of the TF is placed at 1.4 Hz and it is relative to a sway mode of all the caissons in phase (m_1^{+++}). The last peak is placed at 2.7 Hz and is relative to a sway mode of the caissons in opposition of phase (m_1^+). Since the model represent 7 caissons (the central one plus 3 couples) there are in total 4 sway modes, which are modes m_1^{+++} , m_1^{++} , and the two modes m_1^+ and m_1^{-+} with intermediate eigenfrequency (1.9 Hz and 2.4 Hz, in the simulation). The horizontal force applied by the tug does not excite the four heave modes, and also the four rocking ones are weakly excited. The tug-boat impact excited only the rotations around low centres mainly for two reasons:

1. the point where the force is applied is very close to the rocking centre and
2. the length of the impact is not short compared to all ' m_2 ' eigenfrequencies.

Tab. 2 shows the relative power of the excited modes (in terms of rotations).

Tab. 2 Computed eigenmodes for Voltri main breakwater, excited by the tug boat. The damping coefficients (reduction of the oscillation equal to 40% per cycle) were globally calibrated on the basis of the transfer function. The vertical positions of the computed rotation centres, R.C., are given.

| Mode | Eigen-frequency [Hz] | Relative power [%] | Computed R.C. [m above base] | Mode | Eigen-frequency [Hz] | Relative power [%] | Computed R.C. [m above base] |
|-------------|----------------------|--------------------|------------------------------|-------------|----------------------|--------------------|------------------------------|
| m_1^{+++} | 1.4 | 45.1 | -7.5 | m_2^{+++} | 3.9 | 0.3 | 17.6 |
| m_1^{++} | 1.9 | 26.9 | -7.7 | m_2^{++} | 5.2 | 0.1 | 16.9 |
| m_1^+ | 2.4 | 10.5 | -7.8 | m_2^{-+} | 6.8 | 0.1 | 16.2 |
| m_1^{-+} | 2.7 | 17.2 | -8.0 | m_2^+ | 7.9 | 0.1 | 15.8 |

In prototype, the breakwater is formed by many caissons and thus many modes are placed between mode m_1^{all+} and $m_1^{alternate+}$, i.e. a continuum spectrum of modes is present.

The vertical oscillations induced by the sand sac impacts are not perfectly simulated (see Fig. 19) probably because the applied force, as explained in chapter 3, was reconstructed and since the impact is not really impulsive (the impact lasts about .15 sec), the time history has some importance. The simulated response could reproduce only the first cycles of the recorded acceleration oscillation. Since the impact strongly excites superstructure vibrations, identified at 10-15 Hz, the rigid body oscillations in Fig. 19 were assessed by filtering below 10 Hz the recorded signal.

In the records of the vertical accelerometers 9 and 15 placed in the adjacent caissons (the position is shown in Fig. 3) there is much less noise. Since the main

breakwater caissons in Voltri have large longitudinal joints, the superstructure vibrations excited in the central caissons do not pass to the adjacent ones. The low-pass filtering at 10 Hz is then not necessary for these signals (presented in Fig. 20); they are similar between them for symmetry reasons and they describe the vertical acceleration of the seaward corner of the adjacent caissons. Such acceleration was simulated and presented in Fig. 21.

Comparing Fig. 20 and Fig. 21, it can be seen that the first three oscillations are very well defined with regard to maximum values and wave lengths, while a 4 Hz component seems to be unsufficiently damped in the model (a constant damping was applied to every vertical mode, resulting in a reduction of the oscillations of 47% per cycle) Tab. 3 shows how the sand sac impact excite the various simulated modes in terms of displacements. In general the 'all in phase' and the 'alternate' motions are the most excited modes in terms of accelerations, the former more than the latter. Note that all the modes are excited, even the 'm₂' modes that were not identified.

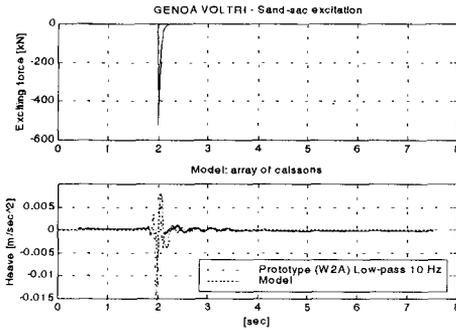


Fig. 19 Comparison between simulated and measured vertical accelerations induced by the falling sand-sac. The vibrations induced by the sharp impact are filtered out (lowpass 10 Hz) for a better comparison.

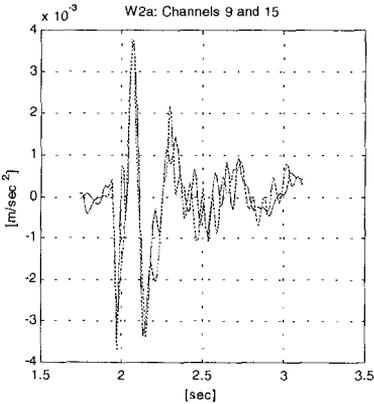


Fig. 20 Mean acceleration averaged over the W2A tests. Channels n° 9 and n° 15 (placed on the adjacent caissons, see Fig. 4)

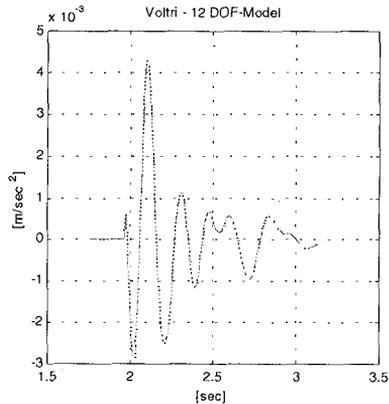


Fig. 21 Simulation of the vertical accelerations of the seaward corner of Voltri main breakwater caisson, induced by the sand sac falling in the adjacent caisson.

Fig. 22 presents PSD of the simulated roll induced by the sand sac excitation, compared to the recorded case: at 2.5 Hz an experimental peak is present, much higher than the simulated one, which might be effect of the vertical oscillation motion having almost the same frequency that was not perfectly identified by the combination of signals that described the rigid body 'roll', or it could be induced by resonance between the two modes with same frequency. Above 10 Hz a lot of energy is present, relative to

superstructure vibration. It is evident that in between the electronic noise can hide the peaks of the simulated signal.

Tab. 3 The falling sand sac eccentric impact excites not only the heave modes but also and the rocking and sway modes. The relative power of each mode is given below (for a meaningful comparison, the rotational oscillations were multiplied by the lever arm of the force, i.e. the eccentricity of the falling sac). The applied reduction per cycle was globally calibrated as 47% for the vertical modes, 40% for the rotational modes.

| Mode | Eigen-frequency [Hz] | Relative power [%] | Mode | Eigen-frequency [Hz] | Relative power [%] | Mode | Eigen-frequency [Hz] | Relative power [%] |
|-------------|----------------------|--------------------|-------------|----------------------|--------------------|-------------|----------------------|--------------------|
| m_1^{+++} | 1.4 | 12.7 | m_2^{+++} | 3.9 | 21.1 | m_3^{+++} | 2.5 | 9.8 |
| m_1^{+-} | 1.9 | 9.3 | m_2^{+-} | 5.2 | 13.0 | m_3^{+-} | 3.8 | 3.9 |
| m_1^{+} | 2.4 | 4.6 | m_2^{+} | 6.8 | 5.8 | m_3^{+} | 5.0 | 1.4 |
| m_1^{-} | 2.7 | 9.0 | m_2^{-} | 7.9 | 7.5 | m_3^{-} | 5.7 | 2.1 |

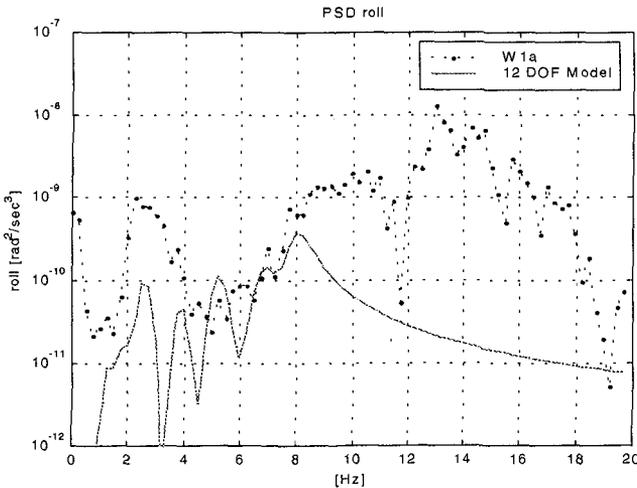


Fig. 22 PSD of computed roll, solid line, and PSD of roll derived from tests W1a: the simulated modes 'm₂ type' (frequencies in [3.9-7.9 Hz], solid line) might be present but hidden by the environmental noise

6. Conclusions

Due to the interaction of all the breakwater caissons, the eigenmodes of caisson system are many. The most excited modes in case of a horizontal force applied at the sea-level (as for the tug-boat or a breaker hitting the breakwater) are all sway oscillations (rotational oscillations around centres placed below the caissons base). The relative eigenfrequencies were measured for the cases of both Genoa Voltri and Brindisi breakwaters and shown in Tab. 1: they are in the range 1.3-3.6 Hz

The heave and rocking oscillations (vertical oscillations and rotation around high centres) were excited only by the falling sand sac load, which induced also high vibration of the superstructure; consequently much more noise was present in the records. The most excited heave mode (apparently the 'all in phase' mode) had frequency around 2.5 Hz.

Unfortunately the rocking modes were not identified with certainty, probably due to the high noise present: Fig. 22 shows that the noise is higher than the model simulated signal.

The superstructure vibrations of the analysed caissons were assessed and found in the range 10-15 Hz.

A MSD model combined to an elastic half space foundation model (Oumeraci and Kortenhaus) can describe the system dynamics only if the movements of at least three adjacent caissons are represented. A very good description (see Fig. 17) of the system dynamics was obtained simulating the movements of an array of 7 caissons. The calibrated parameters were 5: the foundation stiffness for isolated caissons, the stiffness between central and adjacent caisson (40% of the stiffness with foundation for modes 'm1' and 60% for modes 'm3') and the damping of the 'm1' and 'm3' modes (reduction per cycle of 40% and 47%, respectively).

The calibration of the foundation stiffness was obtained considering the relation between the Young modulus and the confining stress (the assumed Poisson coefficient being 0.4 and not yet better investigated). The calibration for both Brindisi and Genoa Voltri suggested a Young modulus of 320 MPa relative to a nominal average pressure of 100 kPa.

Acknowledgements

The partial support of European Community within project "Probabilistic design tools for vertical breakwaters (PROVERBS)" under contract MAS3-CT95-0041 is gratefully acknowledged.

References

- E. Benassai (1975). *The stability against sliding of breakwaters under action of breaking waves*. PIANC, Vol II, n°21
- L. Franco (1994) *Analysis of the dynamic response of caisson breakwaters*. Coastal Engineering, vol. 22, nos 1-2, pp. 31-55.
- Y. Goda (1994). Coastal Engineering, vol. 22, nos 1-2, pp. 31-55.
- T. Hayashi (1965) *Virtual mass and the damping factor of the breakwater during rocking, and the modifications by their effect on the expression of the thrusts exerted upon breakwater by the action of breaking waves*. Coastal Eng. Jpn., 8, pp. 105-117.
- A. Lamberti & L. Martinelli (1997) *Detailed design of prototype tests and analysis*. PROVERBS 2nd Project Workshop, Las Palmas, 18-22 February 1997
- A. Lamberti, L. De Angelis & L. Martinelli (1998) *The Port of Genoa Voltri*. PROVERBS 3rd Project Workshop, Napoli, 25-27 February 1998.
- A. Lamberti & R. Archetti (1998) *The Port of Brindisi*. PROVERBS 3rd Project Workshop, Napoli, 25-27 February 1998.
- V.N. Loginov (1958): *Dynamic analysis of the stability of harbour protective structures under the action of wave impacts*. Tr. SNIIMF, vol. 19, pp. 58-68 (in Russian)
- J.G. Marinski & H. Oumeraci (1992): *Dynamic response of vertical structures to breaking wave forces - review of the CIS design experience*. Proceedings International Conference Coastal Engineering (ICCE), ASCE, vol. 23, Part 2, pp. 1357-1370.
- Y. Muraki (1966): *Field observation of wave pressure, wave run-up, and oscillation of breakwater*. Proceedings International Conference Coastal Engineering (ICCE), ASCE, Tokyo, Japan, vol. 10, pp. 302-321.
- H. Oumeraci & A. Kortenhaus (1994) *Analysis of the dynamic response of caisson breakwaters*. Coastal Engineering, vol. 22, nos 1-2, pp. 159-183.
- V.I. Petrashen (1956) *Action of breaking waves on vertical structures*. Sbornik trudov VNIIGs, 7, pp. 75-110 (In russian)
- G.N. Smirnov & L.R. Moroz (1983) *Oscillation of gravity protective structures of a vertical wall type*. IAHR, Proc. 20th Congress, Vol. 7, pp. 216-219