WAVE IMPACTS ON VERTICAL SEAWALLS AND CAISSON BREAKWATERS

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SUMMARY

In most developed coastal areas, seawalls protect towns, road, rail and rural infrastructure against wave overtopping. Similar structures protect port installations worldwide, and may be used for cliff protection. When a large tidal excursion and severe environmental conditions concur to expose seawalls and vertical face breakwaters to wave impact loading, impulsive loads from breaking waves can be very large.

Despite their magnitude, wave impact loads are seldom included in structural analysis of coastal structures and dynamic analysis is rare, leading to designers ignoring short-duration wave loads, perhaps contributing to damage to a range of breakwaters, seawalls and suspended decks.

Over the last 10 years, improved awareness of wave-impact induced failures of breakwaters in Europe and Japan has focussed attention on the need to include wave impact loads in the loading assessment, and to conduct dynamic analysis when designing coastal structures.

Recent experimental work has focused more strongly on recording and analyzing violent wave impacts. These new data are however only useful if methodologies are available to evaluate dynamic responses of maritime structures to short-duration loads. Improvements in these predictions require the development of more complete wave load models, based on new measurements and experiments.

Moving from a brief review of documented structural failures of caisson breakwaters and existing design methods for wave impact loads, this paper reports advances in knowledge of impulsive wave loads on vertical and steeply battered walls, based on physical model tests in the large wave flume at Barcelona under the VOWS project (Violent Overtopping of Waves at Seawalls). These data are used to support a revised simple prediction formula for wave impact forces on vertical walls.

The paper also discusses dynamic characteristics of linear single degree of freedom

systems to non-stationary excitation. Responses are derived to pulse excitation similar to those induced by wave impacts. Response to short pulses is shown to be dominated by the ratio of impact rise time t_r to the natural period of the structure T_n. A functional relation between impact maxima and rise-times is given for non-exceedance joint probability levels. The relation is integrated in a simplified method for the evaluation of the static-equivalent design load and the potential cumulative sliding distance of caisson breakwaters.

1. WAVE LOADS AT SEAWALLS

Wave forces on coastal structures strongly depend on the kinematics of the wave reaching the structure and on the geometry and porosity of the foreshore as well as on the dynamic characteristics of the structure itself. A sketch of the wave loads usually determined in the design of seawalls is represented in Figure 1.

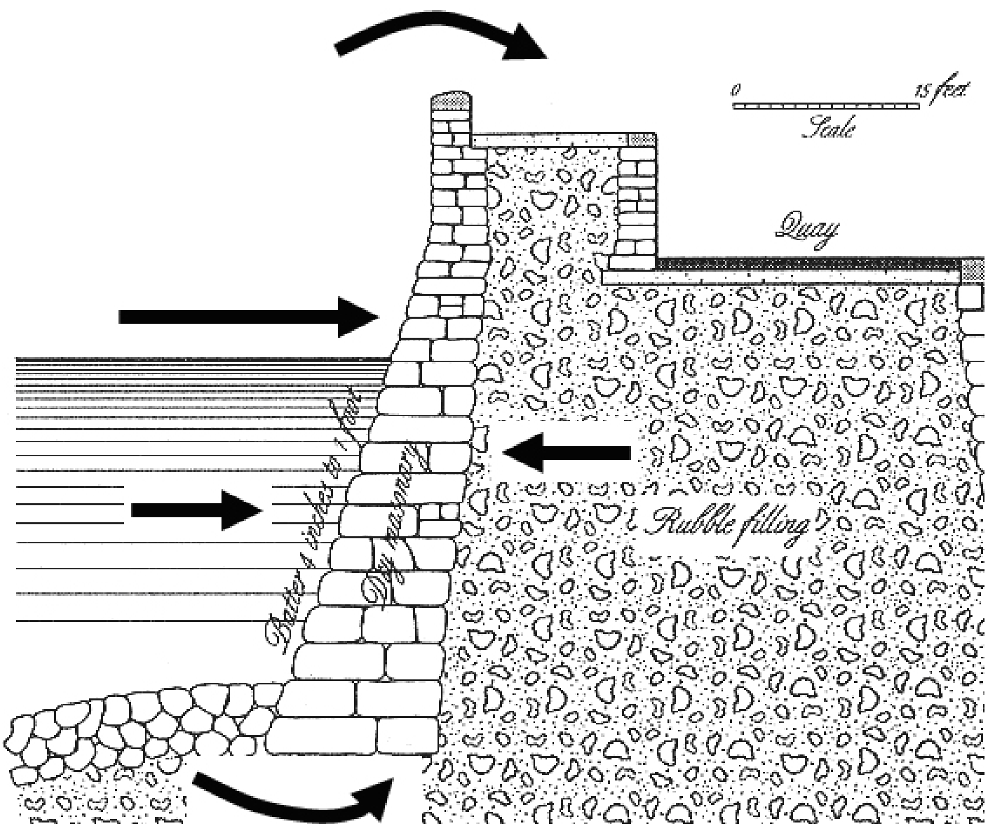

Fig. 1 Wave loads at seawalls (courtesy of N. W. H. Allsop)

They can be summarised as follows:

- shoreward loads on the front face of the breakwater;
- seaward (suction) loads on the front face of the breakwater;

- uplift loads at the base of the wall;
- downward loads due to overtopping green water;
- seaward loads induced by large wave overtopping.

In the design practice, it is common to distinguish three different types of wave attacks, namely:

- non-breaking waves;
- breaking waves;
- broken waves.

While well-established and reliable methods are available for the assessment of wave loads exerted by both non-breaking and broken waves (Sainflou, 1928; Goda, 2000), the assessment of hydraulic loads to be used in design of seawalls, vertical breakwaters and crown walls subject to breaking waves still represents an open issue and impulsive wave loads are often ignored despite their magnitude: "Due to the extremely stochastic nature of wave impacts there are no reliable formulae for prediction of impulsive pressures caused by breaking waves. [...] Impulsive loads from breaking waves can be very large, and the risk of extreme load values increases with the number of loads. Therefore, conditions resulting in frequent wave breaking at vertical structures should be avoided." (Coastal Engineering Manual, 2002 - CEM hereinafter). Vertical breakwaters have been designed in Japan to resist breaking wave loads since the beginning of the 20th century, when a tentative formula for wave impact pressure was firstly introduced by Hiroi (1919). Since then, the need for the realisation of wave barriers in deep water has required a continuous effort towards the development of prediction methods for impact wave loads, along with innovative construction technologies for the realisation of titanic structures (Goda, 2000).

When, as it (not rarely) happens along the North European coasts, a large tidal excursion and severe environmental conditions concur to expose vertical face breakwaters to wave impact loading, designers in "Western countries" also rely on the guidelines drawn within the framework of the PROVERBS (Probabilistic design tools for Vertical Breakwaters) research project (Oumeraci et al. 2001) that represents the most recent and significant European effort towards the understanding and assessment of wave forces on seawalls. An extensive review of state-of-the art design methods for both pulsating and impulsive wave loads on coastal structures is given in Cuomo (2005).

2. STRUCTURAL FAILURE OF CAISSON BREAKWATERS DUE TO WAVE LOADS

Oumeraci (1994) gave a review of analysed failure cases for both vertical and composite breakwaters. 17 failure cases were reported for vertical breakwaters and 5 for composite or armoured vertical breakwaters. The reasons which had lead to the failure of such structures were subdivided into:

- reasons inherent to the structure itself;
- reasons inherent to the hydraulic conditions and loads;
- reasons inherent to the foundation and seabed morphology.

Among the reasons due to the hydraulic influencing factors and loads, the author listed the exceedance of design wave conditions, the focusing of wave action at certain location along the breakwater and the wave breaking. According to Oumeraci, wave breaking and breaking clapotis represent the most frequent damage source of the disasters experienced by vertical breakwaters, by means of sliding, shear failure of the foundation and (rarely) overturning.

Franco (1994) summarised the Italian experience in design and construction of vertical breakwaters. The author gave a historical review of the structural evolution in the last century and critically described the major documented failures (Catania, 1933; Genova, 1955; Ventotene, 1966; Bari, 1974; Palermo, 1983; Bagnara, 1985; Naples, 1987 and Gela, 1991). According to Franco, in all cases the collapse was due to unexpected high wave impact loading, resulting from the underestimation of the design conditions and the wave breaking on the limited depth at the toe of the structure.

Seaward displacement also represents a significant failure mode of vertical breakwaters. Minikin (1963) provided a description of the seaward collapse of the Mustapha breakwater in Algeria in 1934. According to the author this failure was due to a combination of "suction" forces caused by the wave trough and structural dynamic effects. Other cases of lesser seaward tilting have been reported by Oumeraci (1994).

Our knowledge on failure mode of vertical breakwaters has been recently widened by the large experience inherited in recent years from observation made all through last decades in Japan. Among the others, Hitachi (1994) described the damage of Mutsu Ogawara Port (1991), Takahashi et al. (1994) discussed the failures occurred at Sakata (1973-1974), and

Hacinohe. More recently, Takahashi et al. (2000) described typical failures of composite breakwaters, they distinguished the following failure modes:

- meandering sliding (Sendai Port) due to local amplification of non-breaking waves for refraction at the structure;
- structural failure due to impulsive wave pressure (Minamino-hama Port) due to impulsive wave pressure acting on a caisson installed on a steep seabed slope;
- scattering of armor for rubble foundation (Sendai Port) due to strong wave-induced current acting around the breakwater head;
- scouring of rubble stones and seabed sand due to oblique waves;
- erosion of front seabed;
- seabed through-wash;
- rubble foundation failure;

Fig. 2 Caisson failure due to sliding during a storm in the northern part of Japan (courtesy of S. Takahashi)

The authors analysed 33 major failures occurred between 1983 and 1991, more then 80% of them were caused by storm waves larger then the ones used in the design. More then 50% suffered from the application of unexpected wave-induced loads while only 20% were due to the scour of the foundation.

Goda and Takagi (2000) summarised the failure modes of vertical caisson breakwaters observed in Japan over several tens of years, listed below in order of importance:

- sliding of caissons;
- displacement of concrete blocks and large rubble stones armoring a rubble foundation mound;
- breakage and displacement of armor units in the energy-dissipating mound in front of a caisson;
- rupture of front walls and other damage on concrete sections of a caisson;
- failure in the foundation and subsoil.

The authors confirm that ruptures of caisson walls are usually reported as occurred under exceptionally severe wave conditions while the generation of impulsive breaking wave forces is cited as the major cause of caisson damage together with the wave concentration at a corner formed by two arms of breakwater.

3. EXISTING PREDICTION METHODS FOR WAVE IMPACT LOADS ON VERTICAL WALLS

Based on pioneering work by Bagnold (1939), Minikin (1963) developed a prediction method for the estimation of local wave impact pressures caused by waves breaking directly onto a vertical breakwater or seawall. The method was calibrated with pressure measurements by Rouville (1938). Minikin's formula for wave impact forces on vertical walls reads:

$$F_{H,imp} = \frac{101}{3} \cdot \rho g H_D^2 \frac{d}{L_D D} \cdot (d + D) \qquad (1)$$

Where H_D is the design wave height, L_D is the design wave length, D is the water depth at distance L_D from the structure, d is the water depth at the toe of the structure and $101 = 32\pi$ is a conversion factor from American units. Although more recent studies (Allsop et al. 1996) demonstrated Minikin's formula to be obsolete and theoretically incorrect (F_{imp} in Equation 1 decreases with increasing incident wave length L_D), such model is commonly used in the design practice (especially in the United States of America) and is still recommended in the last version of the Coastal Engineering Manual (CEM).

Moving from previous observations by Ito, Goda (1974) developed a new set of wave pressure formulae for wave loads on vertical breakwaters based on a broad set of laboratory data and theoretical considerations. Predictions of wave forces on vertical walls by Minikin's and Goda's methods have been compared by many authors (see, among the others, Chu 1989 and Ergin and Abdalla 1993). Further work by Tanimoto et al. (1976), Takahashi et al. (1993) and Takahashi and Hosoyamada (1994) extended the original method by Goda allowing to account for the effect of the presence of a berm, sloping tops, wave breaking and incident wave angle. Prediction method by Goda (2000) represents a landmark in the evolution of more developed approach to the assessment of wave loads at walls, and is well established and adopted in many national standards (i.e. Japan, Italy, Great Britain) because of its notoriety, the model is not further discussed here.

Blackmore and Hewson (1984) carried out full scale measurements of wave impacts on sea walls in the South of West England using modern measuring and recording equipments. Comparison of new data-sets with previous experiments and prediction formulae proved that impact pressures in the field are generally lower then those measured during laboratory tests, mainly due to the high percentage of air entrained. The following prediction formula, related to the percentage of air entrainment (expressed in terms of an aeration factor λ), was developed:

$$F_{H,imp} = \lambda \cdot \rho \cdot c_s^2 \cdot T \cdot H_b \tag{2}$$

where c_s is the shallow water wave celerity. British standard code of practice for marine structures (BS 6349) suggests evaluating wave impact pressures on sea-walls by means of Equation 2, values for λ range between 0.3 for rough and rocky foreshores and 0.5 for more regular beaches.

Within the framework of PROVERBS research project, an extended set of physical model tests at large and small scale were run respectively in the Large Wave Flume (GWK) of Hannover, Germany and in the Deep Wave Flume (DWF) at the Hydraulic Research Wallingford (HRW), Wallingford, UK. The analysis of pressures and forces recorded during the model tests led to the development of a new prediction method for wave impact forces on vertical breakwaters (Allsop et al. 1996 and Allsop and Vicinanza, 1996). The method is recommended in Oumeraci et al. (2001) and the British Standards (BS6349-1 and BS6349-2, 2000) and is expressed by the following relation:

$$F_{H,imp} = 15 \cdot \rho g d^2 \cdot \left(H_{si}/d\right)^{3.134} \tag{3}$$

Where H_{si} is the (design) significant wave height and d is the water depth.

The advances in knowledge and prediction of wave loadings on vertical breakwaters achieved within the framework of the PROVERBS research project led to the development of a new procedure for the assessment of wave impact loads on sea walls. The new methodology is the first to quantitatively account for uncertainties and variability in the loading process and therefore represented a step forward towards the development of a more rational and reliable design tool. Moving from the identification of the main geometric and wave parameter, the method proceeds trough 12 steps to the evaluation of the wave forces (landward, up-lift and seaward) expected to act on the structure, together with the corresponding impact rise time and pressure distribution up the wall. The new design method is described in details in Oumeraci et al. (2001), Klammer et al. (1996) and Allsop et al. (1999).

4. WAVE IMPACT TIME-HISTORY LOADS

Due to the dynamic nature of wave impacts, the evaluation of the effective load to be used in design needs accounting for the dynamic response of the structure to pulse excitation (Cuomo et al., 2003). This requires the parameterisation of wave-induced time-histories loads as well as the definition of simplified time-history loads for use in the dynamic analysis (Cuomo and Allsop, 2004a; Cuomo et al., 2004b).

4.1. WAVE IMPULSE, IMPACT MAXIMA AND RISE TIME

An example idealised load-history is superimposed on an original signal in Figure 3, the triangular spike is characterized by the maximum reached by the signal during loading (P_{max}), the time taken to get to P_{max} from 0 (rise time, t_r) and back (duration time, t_d). This is usually followed by a slowly varying (pulsating) force of lower magnitude (P_{qs+}) but longer duration. The shaded area in Figure 3 represents momentum transfer to the structure during the impact, the impulse. As the impulse represents a finite quantity, more violent impacts will correspond to shorter rise times and vice versa.

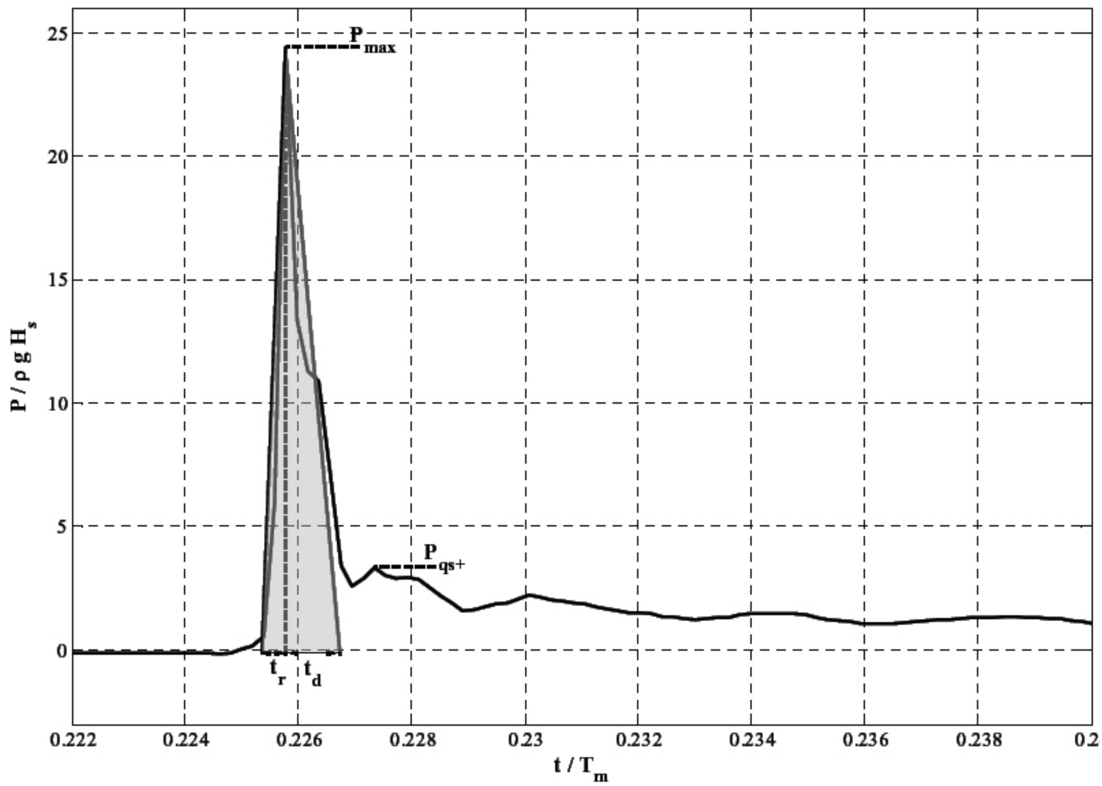

Fig. 3 Wave-impact time-history load recorded during physical model tests

The consistency of wave pressure impulse can be expressed by the following relationship between the maximum impact pressure P_{max} and the impact rise time, t_r (Weggel and Maxwell, 1970):

$$P_{\max} = a \cdot t_r^{\,b} \tag{4}$$

where P_{max}[Pa] and t_r[s] and *a* and *b* are dimensionless empirical coefficients.

Coefficient *b* being negative, the shape of the function defined by Equation 4 is always hyperbolic. For wave impact pressures on walls, values of coefficients *a* and *b* available in literature are summarised in Table 1.

Within the framework of the PROVERBS research project a modified version of Equation 4 was proposed by Oumeraci et al. (2001) to account for the relative influence of the geometry of the foreshore in the proximity of the wall on impact dynamics by expressing parameter *a* as a function of the effective water depth in front of the structure. Parameter *b* was taken as -1.00. The total impact durations (t_d) were also analysed leading to the following relation between t_d and t_r:

$$t_d = -\frac{c_d}{\ln t_r} \tag{5}$$

where empirical parameter c_d is normally distributed with $\mu = 2.17$ and $\sigma = 1.08$.

Table 1. Values of coefficients *a* and *b* for enveloping curves of impact maxima versus rise-time (from previous measurements on seawalls)

Researchers	**Scale of experiments**	**a**	**b**
Weggel & Maxwell, 1970	Small	232	-1.00
Blackmore & Hewson, 1984	Full	3100	-1.00
Kirkgoz, 1990	Small	250	-0.90
Witte, 1990	Small	261	-0.65
Hattori et al., 1994	Small	400	-0.75
Bullock et al., 2001	Full	31000	-1.00

4.2. SIMPLIFIED TIME-HISTORY LOADS

Simplified time-history loads for use in dynamic analysis of caisson breakwaters have been suggested, among the others, by Lundgreen (1969), Goda (1994) and Oumeraci and Kortenhaus (1994). Based on original work by Goda, Shimoshako et al. (1994) proposed a time-history load for use in the evaluation of caisson breakwater displacement. The model assumes a triangular time-history of wave thrust variation with a short duration, which simplifies the pattern of breaking wave pressures.

$$P(t) = \begin{cases} \dfrac{2t}{\tau_0} \cdot P_{\max} & 0 \le t \le \dfrac{\tau_0}{2} \\ 2\left(1 - \dfrac{t}{\tau_0}\right) \cdot P_{\max} & \dfrac{\tau_0}{2} \le t \le \tau_0 \\ 0 & \tau_0 \le t \end{cases} \tag{6}$$

The model has been more recently extended (Shimoshako and Takahashi, 1999) to include the contribution of the quasi-static component, nevertheless, as the peak force is mainly responsible for the sliding of the superstructure, use of model given in Equation 6 is more efficient when the sliding distance of the caisson has to be evaluated (Goda and Takagi, 2000).

4.3. THE DYNAMIC RESPONSE OF THE STRUCTURE

Structurally relatively simple, the dynamic behaviour of caisson breakwater is usually driven by the dynamic characteristics of the foundation soil. Simple models for the dynamic response of caisson breakwaters to impulsive wave loading have been presented, among the others, by Oumeraci and Kortenhaus (1994), Goda (1994), Pedersen (1997). The interpretation of the dynamic response of the foundation soil subject to transient loading is a complex matter that lies outside the aims of this work, a comprehensive review of the state of the art of foundation design of caisson breakwaters is given in de Groot et al. (1996), further development can be found in Oumeraci et al. (2001).

In the following, the relative importance of the impact rise-time on the evaluation of the effective load to be used in design of caisson breakwaters is discussed briefly, based on the analogy with a single degree of freedom (SDOF) linear system.

4.3.1. DYNAMIC RESPONSE TO PULSE EXCITATION

For a linear SDOF system of known mass (*M*), stiffness (*K*) and viscous damping (*C*), subject to a force *f(t)* arbitrarily varying in time, the solution to the equation of motion at time *t* can be expressed as the sum of the responses up to that time by the convolution integral:

$$u(t) = \frac{1}{M\omega_D}\int_0^t f(\tau)\cdot e^{-\xi\omega n(t-\tau)}\cdot\sin[\omega_D(t-\tau)]d\tau \tag{7}$$

Where $\omega_n = \sqrt{K/M}$ and $\xi = C/2M\omega_n$ is the damping ratio and $\omega_D = \omega_n\sqrt{1-\xi^2}$. Equation 7 is known as Duhamel's integral and, together with the assigned initial conditions, provides a general tool for evaluating the response of a linear SDOF system subject to arbitrary time-varying force (Chopra 2001). Equation 7 can be integrated numerically to give the maximum displacement of the system in time $u(t)_{max}$, it is then possible to define a dynamic amplification factor (Φ) as the ratio of $u(t)_{max}$ and the displacement u_0 of the same system due to the static application of the maximum force F_{imp}:

$$\Phi = \frac{u(t)_{\max}}{u_0} \tag{8}$$

4.3.2. RELATIVE IMPORTANCE OF PULSE SHAPE AND DURATION

For a SDOF system of given damping ratio, subject to pulse excitation, the deformation of the system in time *u(t)*, and therefore Φ, only depend on the pulse shape and on the ratio between the pulse rise-time (t_r) and the period of vibration of the system ($T_n = 2\pi/\omega_n$) (Chopra, 2001). For a given shape of the exciting pulse, Φ can therefore be regarded as a function of the ratio t_r/T_n only. The variation of the Φ with T_n (or a related parameter) is named "response spectrum", when the excitation consists of a single pulse, the term "shock

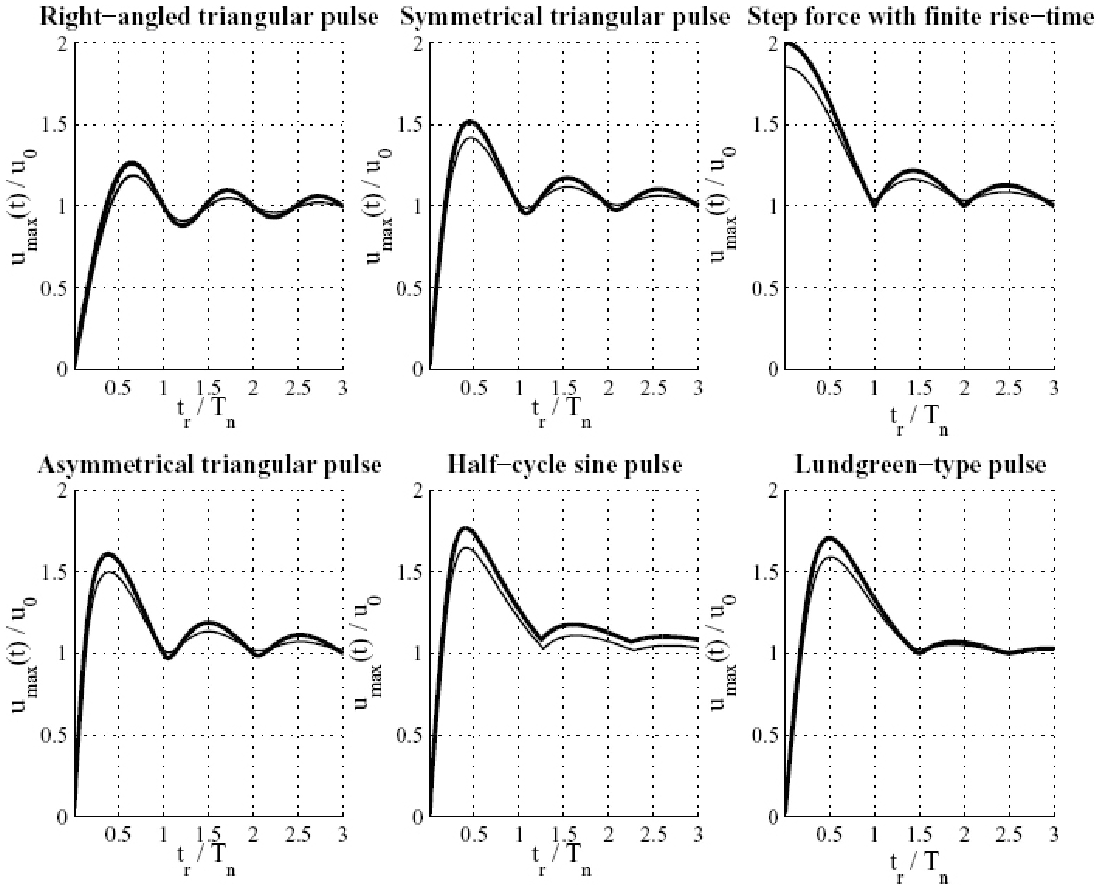

Fig. 4 Dynamic amplification factor (ϕ) of a un-damped (bold line) and damped (ξ = 0.05, thin line) SDOF systems subject to pulse excitation

spectrum" is also used. Cuomo (2005) used the procedure described above to investigate the dynamic response of damped and un-damped SDOF systems to a number of simplified time-history loads. Example shock spectra are given in Figure 4 for different pulse shapes. The effective pulse shape depends on both the incoming wave kinematics and the dynamic characteristics of the structure, moving from previous (Schmidt et al. 1992, Oumeraci et al. 1993 and Hattori et al. 1994) and new observations, an association between breaking wave types and shock spectra in Figure 4 have been suggested in Cuomo (2005). When no further information is available, a symmetric triangular pulse represents a reasonable choice.

4.3.3. RELATIVE IMPORTANCE OF DAMPING

When a system is subject to an harmonic excitation at or near resonance, the energy dissipated by damping is significant. On the contrary, when the system is excited by a single pulse, the energy dissipated by damping is much smaller and the relative importance of damping on maximum displacement decreases. This is confirmed in Figure 4, where the shock spectra of a damped SDOF system (ξ = 0.05) is superimposed to the one corresponding to the equivalent un-damped system. Nevertheless, for maritime structures, damping can be much larger then for other civil structures (i.e. $\xi \gg 0.05$), due to the high dissipative role played by both water and soil foundation (Pedersen, 1997). Although being generally safe, not taking into account the effect of damping when assessing effective design load might result in a significant overestimation of wave-induced loads.

4.4. DYNAMIC CHARACTERISTICS OF TYPICAL PROTOTYPE STRUCTURES

Prototype measurements of the dynamic characteristics of caisson breakwaters have been assessed by Muraki (1966), Ming et al. (1988), Schmidt et al. (1992) and Lamberti and Martinelli (1998). The estimates given by the authors are summarised in Table 2.

Table 2. Dynamic characteristics of typical prototype caisson breakwaters

Researcher	**Period of vibration (s)**
Muraki, 1966	0.20 ÷ 0.40
Ming et al., 1988	0.26
Schmidt et al., 1992	0.15 ÷ 0.60
Lamberti and Martinelli, 1998	0.15 ÷ 2.00

4.5.SLIDING

The risk of sliding of caisson breakwaters subject to impact loadings has firstly been proven by Nagai (1966) who stated: "It was proven by 1/20 and 1/10 scale model experiments that, at the instant when the resultant of the maximum simultaneous shock pressures just exceeds the resisting force, the vertical wall slides". Based on sliding block concept (Newman, 1965), Ling et al. (1999) and Shimoshako and Takahashi (1999) performed numerical experiments to evaluate the permanent displacement of composite breakwaters under extreme wave loading.

The method has been included in the performance-based design method for caisson breakwaters allowing for sliding proposed by Goda and Takagi (2000). Under the assumption of a rigid body motion, the authors adopted the following expression for the sliding distance:

$$S = \frac{\tau_0 \cdot (F_S - \mu W_e)^3 \cdot (F_S + \mu W_e)}{8\mu \cdot M_c W_e F_S^2} \qquad (9)$$

where F_S is the sum of the horizontal and uplift force, μ is the friction coefficient between the caisson and the soil foundation and $W_e = g\,(M_c - M_w)$ is the effective weight of the caisson in water. Parameter τ_0 in Equation 9 is given as a function of the incident wave period.

5. THE EXPERIMENTAL SETUP

Large-scale experiments were carried out at the CIEM / LIM wave flume at Universitat Politecnica de Catalunya, Spain. The LIM wave flume is 100*m* long, 3*m* wide along its full length, and has an operating depth of up to 4*m* at the absorbing-wedge paddle. For these experiments, a 1:13 concrete foreshore was constructed up to the test structure shown in Figure 5. Pressures up the wall were measured by mean of a vertical array of 8 pressure transducers; logging at a frequency of 2000Hz, distance between two successive transducers was equal to 0.20*m*. Each test consisted of approximately 1000 irregular waves to a JONSWAP spectrum with $\gamma = 3.3$.

Five different water depths *d* were used ranging between 0.53 and 1.28m. The test matrix of about 40 different conditions is summarized in Table 3, together with information relative

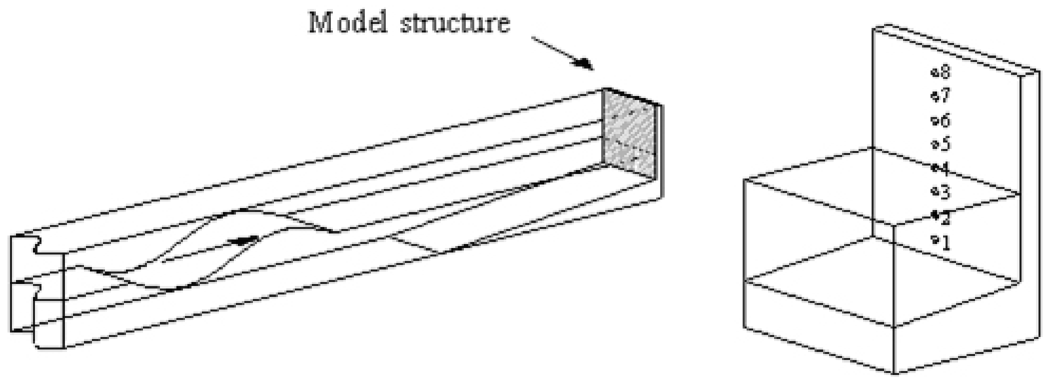

Fig. 5 Experimental set-up: aerial view with pressure transducers

to the whole set of experiments. A snapshot from the physical model tests is shown in Figure 6.

Fig. 6 Large scale tests at LIM-UPC, snapshot of a wave impact during physical model

Three structural configurations were tested, respectively:

– 1:10 Battered wall

– Vertical wall

– Vertical wall with recurve

Table 3. Summary of test conditions

Test Series	**Configuration**	**Nominal wave period Tm [s]**	**Nominal wave height His [m]**
1A & 1B	Rc = 1.16m / 1.40m d = 0.83m	2.56 3.12 3.29 3.64 1.98	0.48, 0.45, 0.37 0.60, 0.56, 0.39 0.67 0.60 0.25
1C	Rc = 1.46m d = 0.53m	1.98 2.56 3.12 3.29 3.64	0.25, 0.22 0.48, 0.45, 0.37, 0.23 0.63, 0.60, 0.56, 0.39 0.67 0.60
1D & 1E	Rc = 0.71m / 0.95m d = 1.28m	1.97 2.54 3.12 3.65	0.26, 0.23 0.44, 0.35, 0.23 0.58, 0.50, 0.34 0.55
1F & 1I	Rc = 1.38m / 1.42m d = 0.82m	2.60 3.15 3.40 3.80	0.46 0.59, 0.51 0.59 0.51
1G & 1H	Rc = 0.98m / 1.02 m d = 1.22m	3.15 3.40 3.80	0.59 0.59 0.51

More detailed descriptions of the experimental setup are given in Cuomo (2005) and Pearson et al. (2002).

6. SUGGESTED PREDICITON METHOD FOR IMPACT WAVE LOADS

6.1. COMPARISON WITH EXISTING PREDICITON METHODS

Impact horizontal (shoreward) forces as measured over the vertical face of the wall during the physical model test have been compared with a range of methods, including those suggested in the Coastal Engineering Manual (CEM), British Standards BS-6349, and the guidelines from PROVERBS. Wave impact loads at exceedance level $F_{1/250}$ (i.e. the average of the highest four waves out of a 1000-wave test) are compared with predictions by Hiroi (1919), Minikin (1963), Blackmore and Hewson (1984), Goda (1994), Allsop and Vicinanza (1996), and Oumeraci et al. (2001) in Figure 7. Within the range of measured forces the scatter is large for all the prediction methods used. Points falling above the 1:1 line represent un-safe predictions.

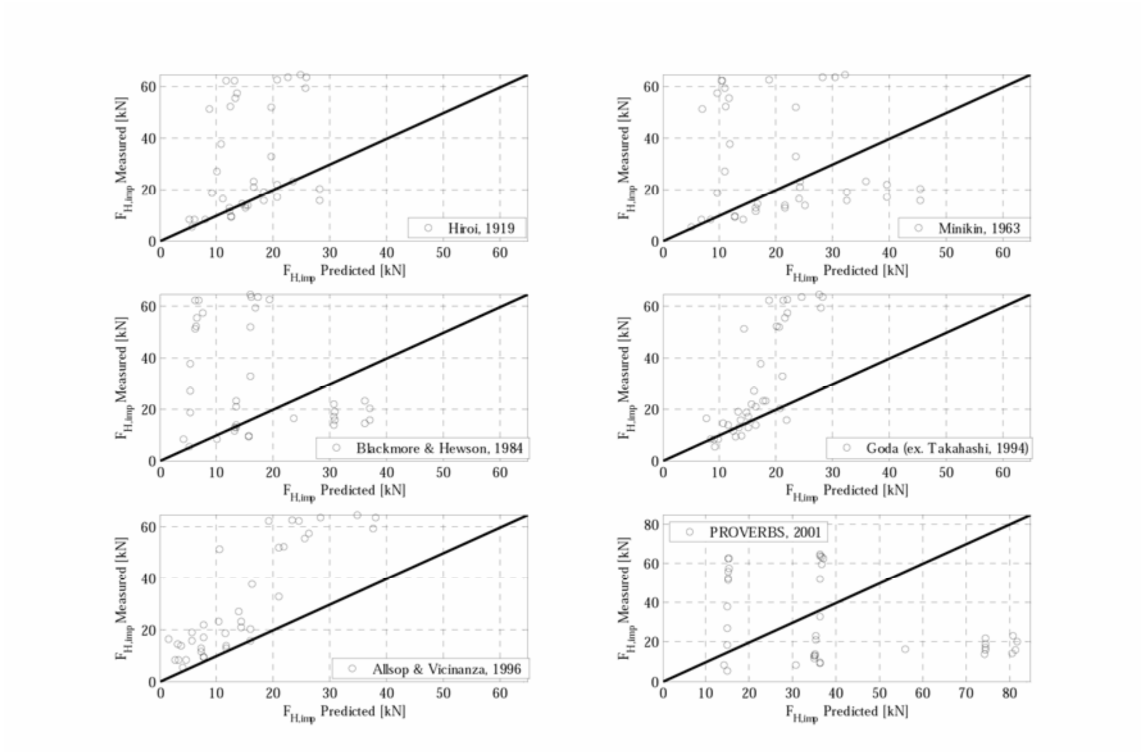

Fig. 7 Comparison of measured impact loads with existing prediction methods

6.2. PREDICITON FORMULA FOR WAVE IMPACT LOADS

The relative importance of the incident wave height and wave length on horizontal wave impacts has already been discussed, combining the two contributions, the following formula is proposed for the prediction of wave impact forces on seawalls:

$$F_{H,imp,1/250} = \alpha \cdot \rho g \cdot H_S \cdot L_0 \cdot \left(1 - \frac{|d_b - d|}{d}\right) \tag{10}$$

Where α = 0.842 is an empirical coefficient fitted on the new experimental data. The term in brackets in Equation 10 represents the difference between the water depth d at the structure and the water depth at breaking (d_b) and to a certain degree accounts for the severity of the breaking at the structure. Here, d_b is evaluated by inverting breaking criteria by Miche (1951) assuming H_b=H_S:

$$d_b = \frac{1}{k}\operatorname{arctanh}\left(\frac{H_S}{0.14 \cdot L_0}\right) \tag{11}$$

Where $k = 2\pi/L_0$ and L_0 is the deep water wave length for T=T_m.

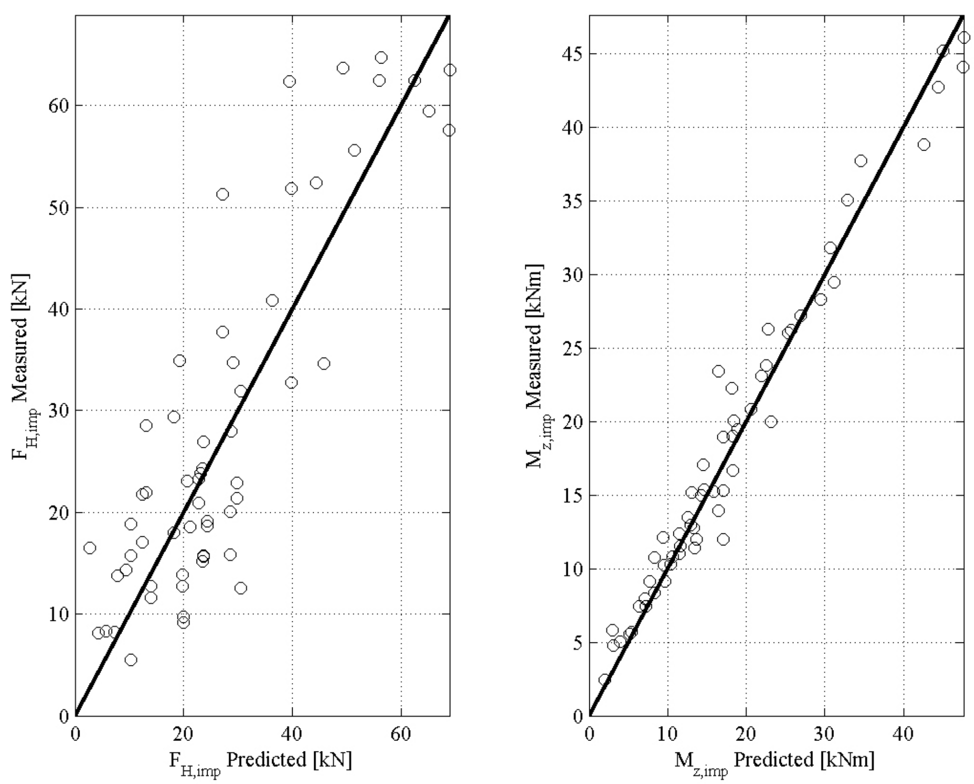

Fig. 8 Comparison between measured and predicted impact loads

Predictions by Equation 10 compare satisfactorily well with wave impact forces measured during the physical model tests (at exceedance level $F_{1/250}$) on the left hand side of Figure 8.

The following expression is suggested for the level arm:

$$l_{F_H} = d \cdot \left(0.781 \cdot H^* + 0.336\right) \tag{12}$$

Where l_F is given in meters [m] and $H^* = H_S/d$ accounts for the attitude to break of incident waves. Measured overturning moments are compared with predictions by Equation 12 on the right hand side of Figure 8.

Once $F_{H,imp}$ has been evaluated according to Equation 10, pressure distribution over the caisson can be evaluated according to Oumeraci et al. (2001). In particular, the uplift force can be estimated as follows:

$$F_{U,imp,1/250} = F_{H,imp,1/250} \cdot \frac{0.27 \cdot B_c}{0.4 \cdot H_b + 0.7 \cdot (d + d_c)} \tag{13}$$

where B_c is the caisson width, and d_c is the length for which the caisson is imbedded in the rubble mound; the corresponding level arm is equal to $l_{F_U} = 0.62 \cdot B_c$.

6.3. JOINT PROBABILITY OF IMPACT MAXIMA AND RISE TIME

Most recent standards are oriented toward a probabilistic approach to design of civil structures and new tools are therefore needed to account for the uncertainties due to the variability of the loading process when assessing hydraulic loads for design purposes. Furthermore, as impact maxima and rise times are strictly bounded to each other by physical reasons, assuming these two parameters to be independent is obviously wrong and necessary results in a large overestimation of impact impulses for design purposes.

In order to reduce scatter in the wave impact maxima as recorded during the testing, the dimensionless impact force $F^*_{imp} = F_{imp}/F_{qs+,1/250}$ and rise-time $t^*_r = t_r/T_m$ have been introduced. With this assumption Equation 4 can then be re-written in dimensionless form as:

$$\frac{F_{max}}{F_{qs+,1/250}} = a \cdot \left(\frac{t_r}{T_m}\right)^b \tag{14}$$

The joint probability of dimensionless wave impact maxima and rise-times has been evaluate by means of the kernel density estimation (KDE) method (Athanassoulis and Belibassakis, 2002) with the aim of associating a non-exceeding probability level to coefficients in Equation 14 and therefore to the dynamic characteristics of the impact load to be used in design.

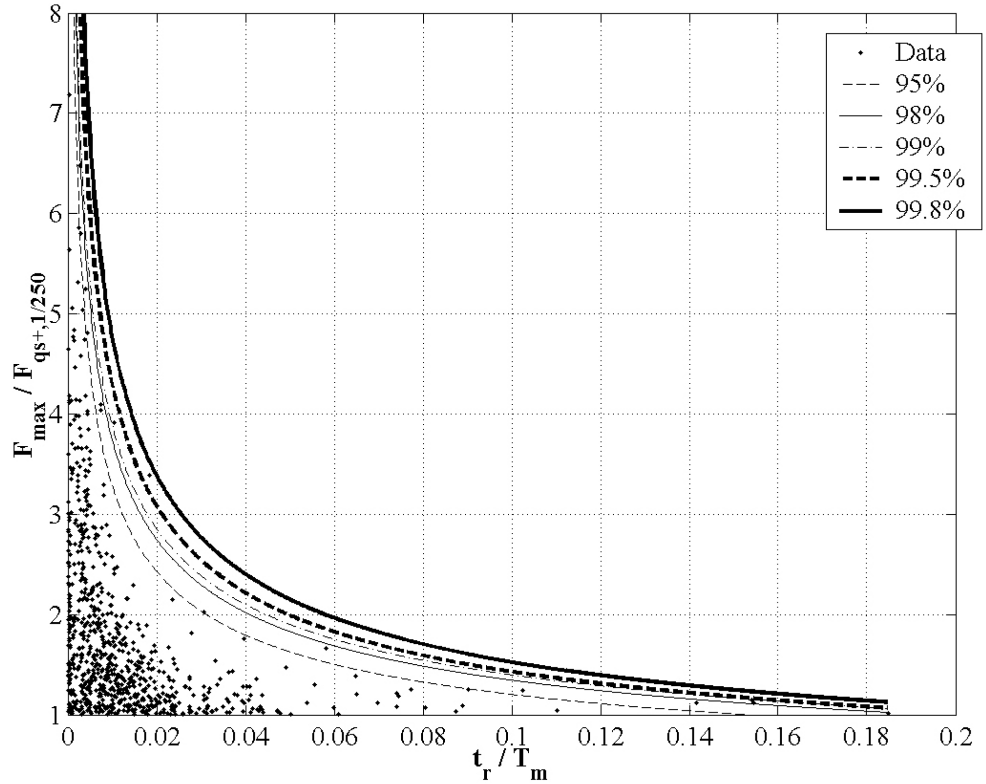

Fig. 9 Dimensionless impact maxima versus rise times

Impact maxima and rise-time on walls are superimposed to their corresponding joint probability contour in Figure 9. Envelope lines in Figure 9 obey Equation 14 and have been fitted to the iso-probability contour at $P(F^*_{imp}; t^*_r)$ = 95% to 99.8%. For increasing non-exceedance levels between 95% and 99.8%, empirical coefficients *a* and *b* in Equation 14 are given in Table 4.

Table 4. Coefficients *a* and *b* for enveloping curves of impact maxima versus rise-time on seawalls for increasing non-exceedance joint-probability levels

$P(F^*_{imp}; t^*_r)$[%]	**a**	**b**
95	0.441	-0.436
98	0.484	-0.444
99	0.503	-0.444
99.5	0.477	-0.477
99.8	0.488	-0.495

6.4. EVALUATION OF STATICALLY EQUIVALENT DESIGN LOAD

The following procedure is therefore suggested for the evaluation of the (statically equivalent) load to be adopted in the design of impact wave forces on vertical walls:

1) Evaluate the impact load (F_{imp}) according to Equation 10;
2) Compute the corresponding quasi-static load according to Goda (1974) that is, assuming α_2, $\alpha_I = 0$ in the expressions given in Goda (2000) and Takahashi et al. (1994);
3) Enter graph in Figure 9 or use Equation 14 with coefficient in Table 4 to evaluate the value of t_r/T_m corresponding to F_{imp}/F_{qs+} at a given non-exceedance probability level;
4) Enter graph in Figure 4 to evaluate the dynamic amplification factor s a function of t_r/T_n;
5) Evaluate the design load as: $F_{eq} = F_{imp}\ \Phi$.

Results from an example calculation are shown in Figure 10 where the statically

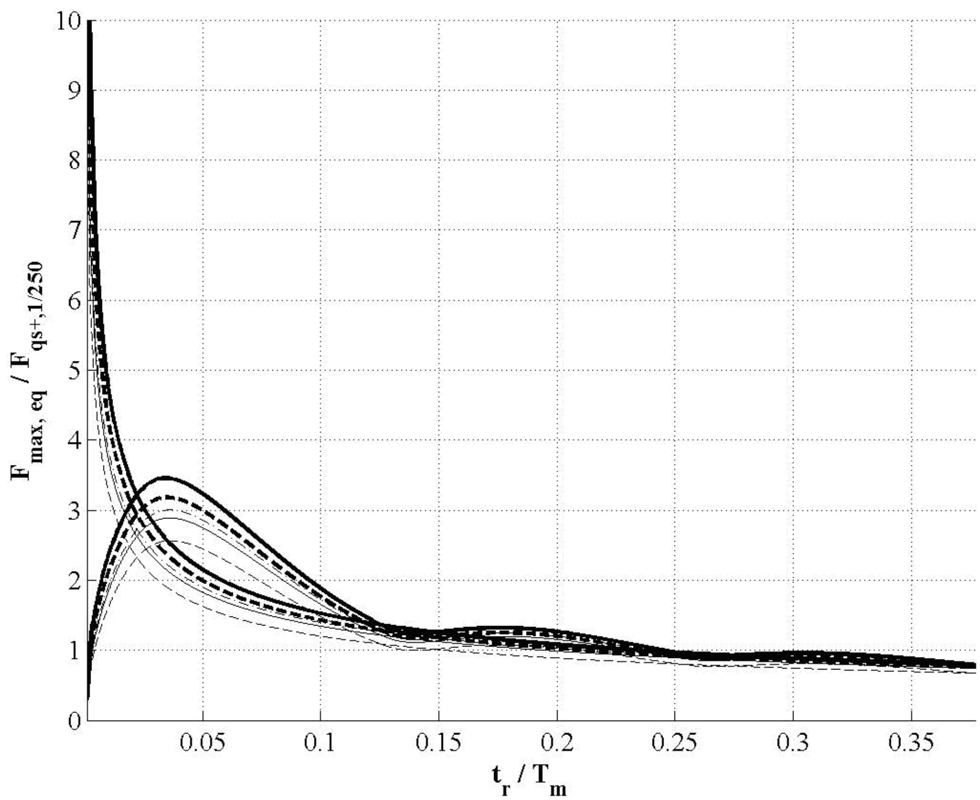

Fig. 10 Evaluation of the static-equivalent design loads

equivalent load F_{eq} has been evaluated for $0 < t_r/T_m < 0.35$ at non-exceedance levels ranging between 95 and 99.8%.

6.5. EVALUATION OF SLIDING DISTANCE

The same methodology also applies to the evaluation of the potential sliding of caisson breakwater. In this case the following procedure is suggested:

1) Compute F_{qs+} according to Goda (1974) that is, assuming $\alpha_2, \alpha_I = 0$ in the expressions given in Goda (2000) and Takahashi et al. (1994);
2) For a given F_{imp}, enter graph in Figure 9 or use Equation 14 to evaluate the value of t_r/T_m corresponding to F_{imp}/F_{qs+} at a given non-exceedance probability level;
3) Enter the graph in Figure 4 to evaluate the dynamic amplification factor s a function of t_r/T_n corresponding to the more appropriate shape pulse expected to act on the structure;
4) Evaluate the design load as: $F_{S,eq} = F_{imp}\,\Phi$;
5) For each couple of values $F_{S,eq}$ and τ_0, evaluate the sliding distance by means of Equation 9;
6) Repeat steps 2 to 5 for different values of F_{imp};
7) Evaluate the sliding distance due to a single wave as: $S = \max\left\{S(F_{imp})\right\}$;
8) Evaluate the percentage of breaking waves P_b by means of the following Equation (Oumeraci et al. 2001):

$$P_b = \exp\left[-2\left(\frac{H_{bc}}{H_{si}}\right)^2\right] - \exp\left[-2\left(\frac{H_{bs}}{H_{si}}\right)^2\right] \tag{15}$$

where H_{bc} is the wave height at breaking and H_{bs} is the "transition" wave height from impact to broken waves, respectively:

$$H_{bc} = 0.1025 \cdot L_{pi} \cdot \tanh\left(\frac{2\pi}{L_{pi}} \cdot k_b \cdot d\right) \tag{16}$$

$$H_{bs} = 0.1242 \cdot L_{pi} \cdot \tanh\left(\frac{2\pi}{L_{pi}} \cdot d\right) \tag{17}$$

where L_{pi} is the wave length at the local water depth d for $T = T_p$ and k_b is an empirical coefficient given as a function of the ration of the berm width to the local water depth (Oumeraci et al. 2001);

9) Evaluate the cumulate sliding distance as:

$$S_{tot} = N_z \cdot P_b \cdot \max\left\{S(F_{imp})\right\} \tag{18}$$

For the sake of simplicity, the methodology proposed herein assumes the sliding distance due to each breaking wave to be equal to that corresponding to the severest combination of impact force and rise time and therefore generally leads to an overestimation of the sliding distance. When a more precise evaluation of the sliding distance is needed, a more realistic prediction can be obtained by assuming an adequate wave distribution at the structure (Cuomo 2005) and generating a statistically representative number of random waves. The total sliding distance will then result from the sum of the contribution of each wave as evaluated in steps 2 to 5.

7. CONCLUSIONS

Despite their magnitude, very little guidance is available for assessing wave loads when designing seawalls and caisson breakwaters subject to breaking waves. Within the VOWS project, a series of large scale physical model tests have been carried out at the UPC in Barcelona with the aim of extending our knowledge on wave impact loads and overtopping induced by breaking waves on seawalls.

New measurements have been compared with predictions from a range of existing methods among those suggested by most widely applied international code of standards, showing large scatter in the predictions and significant underestimation of severest wave impact loads.

A new prediction formula has been introduced for the evaluation of wave impact loads on seawalls and vertical faces of caisson breakwaters. When compared to measurements from physical model tests, the agreement between measurements and predictions is very good for both wave impact force and level arm.

Due to the dynamic nature of wave impact loads, the duration of wave-induced loads has to be taken into account when assessing wave loads to be used in design.

Based on the joint probability distribution of wave impact maxima and rise times, a model for the prediction of impact loads suitable for probabilistic design and dynamic response of structures has been developed.

The new methodologies have been integrated with existing design methods for the evaluation of the effective wave loads and sliding distances of seawalls and caisson breakwaters, leading to the development of improved procedures to account for the dynamic response of the structure when assessing wave loads to be used in design.

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9. NOTATION

α	dimensionless empirical coefficient
a, b	dimensionless empirical coefficients
B_c	caisson width
C	viscous damping
c_d	empirical parameter
c_s	shallow water wave celerity
d	water depth at the toe of the structure
d_b	water depth at breaking
d_c	length for which the caisson is imbedded in the rubble mound
D	water depth at distance L_D from the structure
Φ	dynamic amplification factor
F_{eq}	statically equivalent design force
F_H	horizontal force
F_U	uplift force
F_{qs+}	pulsating force
F_{imp}	impact force
F^*_{imp}	dimensionless impact force

F_S	total wave force
g	gravitational acceleration
H_{si}	design significant wave height
H_b	wave height at breaking
H_D	design wave height
H_S	significant wave height
k	wave number
K	stiffness
λ	aeration factor
l_F	level arm
L_0	deep water wave length
L_D	design wave length
L_{pi}	wave length at local water depth for $T = T_p$
M	mass
M_c	weight of caisson
M_w	buoyancy
N_z	number of waves in a storm
μ	friction coefficient
P	pressure
P_b	percentage of breaking waves
P_{max}	impact pressure peak value
ρ	water density
S	permanent displacement
T	wave period
T_m	mean wave period
T_n	$2\pi/\omega_n$ natural period of vibration of the system
T_m	peak wave period
τ_0	total impact duration
t_r	impact rise-time
t^*_r	dimensionless impact rise-time
t_d	impact duration time
u	displacement

u_0 F_{imp}/K static equivalent displacement

ω_n natural frequency of vibration of the system

ω_D natural frequency of vibration of a damped system

W_e effective weight of caisson in water

ξ damping ratio

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