HAZARD ANALYSIS OF DYNAMICALLY LOADED CAISSON BREAKWATERS

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Abstract

Probabilistic analysis, based on the final outcomes of the European research project PROVERBS, Oumeraci et al. 2000a, is applied to two example cases: Genoa Voltri main breakwater (Italy) and Easchel breakwater, a fictitious case combining Eastern Scheldt Storm Surge Barrier (the Netherlands) for the subsoil and Gela breakwater (Italy) for the rest.

Sliding induced by non-breaking, impact or broken waves is the discussed failure mechanism. A calibrated 1 degree of freedom dynamic model, easy to use in a probability based design process, is adopted for the evaluation of the force transferred to the foundation by impacts.

For both example cases, realistic results were obtained also thanks to some attention on the choice of congruous statistics.

1 Introduction

Probabilistic methods have been applied during the last years to vertical breakwaters and are now considered (Oumeraci et. al., 2000b) to be potentially an important support to the designer.

One of the main problems in the design of vertical breakwaters is the identifications and quantification of the effects of breaking waves on the stability of the structure. Generally, the forces caused by breaking waves (impacts) are so high that they appear to govern the design completely. For this reason an estimate of the

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magnitude and the probability of occurrence of impacts is necessary if a probabilistic analysis of the design is to be successful.

A breakwater caisson is usually designed for an equivalent static force, being a static force providing the same effect on the foundation as the dynamic load occurring in reality. The equivalent static load is evaluated either empirically or by a dynamic model on the basis of the load signal (force time history) and the property of the caisson and its foundation (mass, stiffness, damping).

In a deterministic analysis, static equivalent forces are handled with relatively confidence by means of the consolidated empirical Goda-type formulae for the 'maximum' force induced by the waves, see for instance Takahashi (1996).

On the contrary for a probabilistic analysis, where much attention is given to uncertainties, the 'dynamic model' analysis seems a more adequate approach: it is indeed easier to associate the uncertainty to physically based models than to empirical methods, since the scatter of the empirical formulae (Van der Meer, 1994) must be dependent on too many parameters, like for instance the incident wave distribution, foundation stiffness, etc.

At the present state of the art, beside uncertainty on design wave and foundation conditions, the models describing wave forces at the wall or the dynamic behaviour are not sufficiently consolidated. In this note a choice of existing models, tested on two example cases, is suggested.

The tools needed to carry out a hazard analysis of a vertical breakwaters are the statistical description of wave forces applied to caisson vertical wall (loading) and of soil parameters (resistance) plus an essentially deterministic description of dynamic behaviour and of failure modes.

The first step for a description of applied loads is the identification of incident wave distribution, that was thoroughly investigated when applied to beaches (the most famous approaches are described in Dally et al., 1984, Goda, 1985, and Battjes & Janssen, 1978) but there is no consolidated application to the case of a vertically composite breakwater, where impacts are partly induced by the underlying berm. A method based on indications by Calabrese (Calabrese & Vicinanza, 2000) appeared to the authors the most reliable.

The force induced by breaking waves on the wall has been studied since a long time (Stevenson, 1874, Bagnold, 1939, Minikin, 1950, Hayashi & Hattori, 1958, Nagai, 1973) but the magnitude of impulses is still very uncertain.

The dynamics of the vertically composite breakwaters should be studied in order to evaluate the force transferred to the foundation; in general a simple model based on rigid body mechanics is proposed: with small differences in the approach it is described, among the others, in Petrashen (1956), Loginov (1958), Hayashi (1965), Smirnov & Moroz (1983), Marinski & Oumeraci (1992), quoted in Oumeraci and Kortenhaus (1994), Benassai (1975), Goda (1994), Shimosako et al.(1994), Voortman et al. (1999). The 2 Degrees of Freedom (DoF) model used in this note is presented in detail in Lamberti & Martinelli (2000), calibrated against prototype measurements

(Lamberti & Martinelli, 1998). A 1 DoF model is also suggested, easy to use in a design process and slightly conservative (equivalent in case of longcrested waves) compared to the 2DOF model.

No probabilistic tools are discussed in this note; see Kottegoda & Rosso (1997) for reference on these latter aspects.

2 Contents

In the following paragraphs the steps used to perform a hazard analysis in presence of impacts will be presented and applied to two example cases, described in Martinelli et al. (1999): a real case, Genoa Voltri main breakwater, Italy, and a realistic case, Easchel breakwater, the subsoil being the sand which was thoroughly investigated during design of Eastern Scheldt Storm Surge Barrier in the Netherlands while the geometry and hydraulic conditions are relative to Gela breakwater in Italy.

Section 3 presents the hydraulic models assumed in this note: subsections 3.1 and 3.2 describe the models for incident wave distribution and for the impulse time history. Section 4 quotes the assumed dynamic model and subsection 4.1 presents the resulting horizontal static equivalent forces. Section 5 describes the failure mode. Section 6 presents the variable statistics and the modified Monte Carlo simulation. Final results and conclusions are given in Sections 7 and 8.

The presented tools are a refinement of the final outcomes of the European project PROVERBS (PRObabilistic design tools for VERtical BreakwaterS), Oumeraci et al. (2000a).

3 The loads

3.1 Wave classification and breaking criteria

Waves can be classified as unbroken, breaking at the wall or broken. They are considered to break at the wall when the curl or the vertical front hits the wall, causing a violent peak in pressure against the vertical caisson. They are classified as broken if the curl falls in water before the wall.

Distance run by the breaker during its breaking process is assumed, following Goda, 5 H_s. If the breaker is formed at a greater distance from the wall (normally on the foreshore slope), waves will arrive broken and induce smaller pressure.

We need actually two breaking criteria, one for the slope and one for the berm. Only incident waves that fall within the two criteria result breaking at the wall: higher waves will indeed be considered as broken and smaller waves as unbroken.

Authors generally suppose that off-shore waves are Rayleigh distributed. We follow this recommendation and suppose that such wave distribution is modified in front of the structure as suggested by Goda (1985).

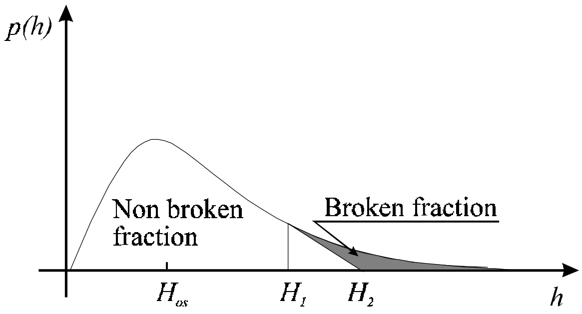
Goda describes a truncated incident wave distribution and identifies the fraction of waves that break before reaching the structure; he also suggests that such waves are not statistically distinguishable from the 'non breaking waves', i.e. the fraction of broken waves is proportionally redistributed among the remaining part.

Others authors (e.g. Battjes & Janssen, 1978, Dally et al., 1985) suggest on the contrary that broken waves follow a totally different distribution more concentrated near to breaking limit.

The limiting condition of the assumed 'broken criterion' (Goda, 1985), is:

$$H = A\, L_{os}\ [1 - e^{-1.5\,\pi\, h_s\, k_s / L_{os}}], \quad (1)$$

where A= 0.17 for a deterministic approach or 0.12 and 0.18 for separate evaluation of H_1 and H_2 (see Fig. 1);
L_{os} is the off-shore significant wave length, h_s the depth in front of structure, and



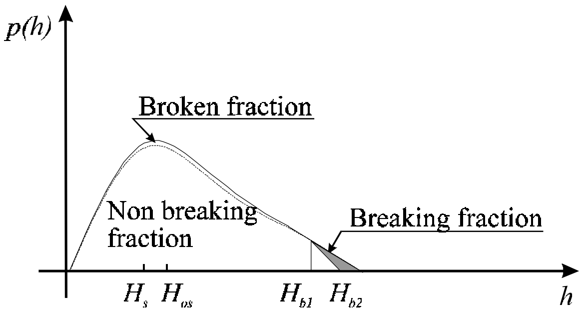


Fig. 1

$k_s = (1+15 \tan^{4/3}\theta)$ accounts for the foreshore slope θ, slightly increasing the depth.

The assumed 'breaking criterion' is based on indications synthesised in Calabrese & Vicinanza (2000), who performed or analysed model tests of several structures with differently shaped berms, all of relevant height. The main result of the work is the berm effect on the breaking wave height H_b: when such wave height is reached, breaking is supposed to take place. It represents the $H_{99.6\%}$ of the measured incident wave field for the tests where the percentage of breaking is 0.4% (at least 2 impacts out of 500 waves) and therefore no assumption is made on the incident wave distribution, although a Rayleigh incident wave distribution is suggested in the text.

The breaking condition of Calabrese is:

$H_b = L_p\ 0.1242\ k_r \tanh(2\,\pi\ k_b\, h_s / L_p)$ where,

$k_r = 1 - 0.348\ Cr/(1+Cr)$, ($k_r = 1 \div 0.826$) accounts for reflection in front of the breakwater, Cr being the reflection coefficient.

$k_b = 0.0076\, (B_{eq}/d)^2 - 0.1402\, (B_{eq}/d) + 1$, ($k_b = 1 \div 0.358$) accounts for berm effects, B_{eq} being the berm width at mid berm height, d being the depth on top of berm, $0 < B_{eq}/d < 10$

In conclusion, Calabrese applies some correction factors to a Miche-type formula in order to account for the berm effects and for the reflection induced by the structure.

This formula is checked only for berms of significant heights, approximately 30% of depth in front of structure (h_s), like for Genoa Voltri. For the case of Easchel geometry (where the berm is only 20% of h_s) the berm effect may be overestimated.

Since the impact probability is in general small, we must consider only formulas with a similar structure, otherwise we may mistake some different behaviours of the formulas for actual breakings. This necessity allows for some small recalibrations of the used models.

In order to have a consistent scheme, the breaking wave given by Calabrese for a null berm (k_b = 1) and in absence of structure (k_r = 1) should be equal to the broken wave condition given by Goda for flat foreshore, since percentage of breaking should tend to zero as the berm vanishes. We obtained this consistency applying the same coefficients to the formula given by Goda:

$$H_b = A\, k_r\, L_{os}\ [1 - e^{-1.5\, \pi\, h_s\, k_b\, k_s / L_{os}}] \qquad (2)$$

that we may evaluate with k_r=1 and k_b=1 when we consider the 'broken criterion' far from the structure (thus being equivalent to eq. 1), and with k_s relative to θ = 1:20÷1:70 (range of foreshore slope for Calabrese tests) when we consider the conditions against the vertical wall H_b ('breaking criterion').

The deterministic approach, A=0.17, is used in the following for simplicity.

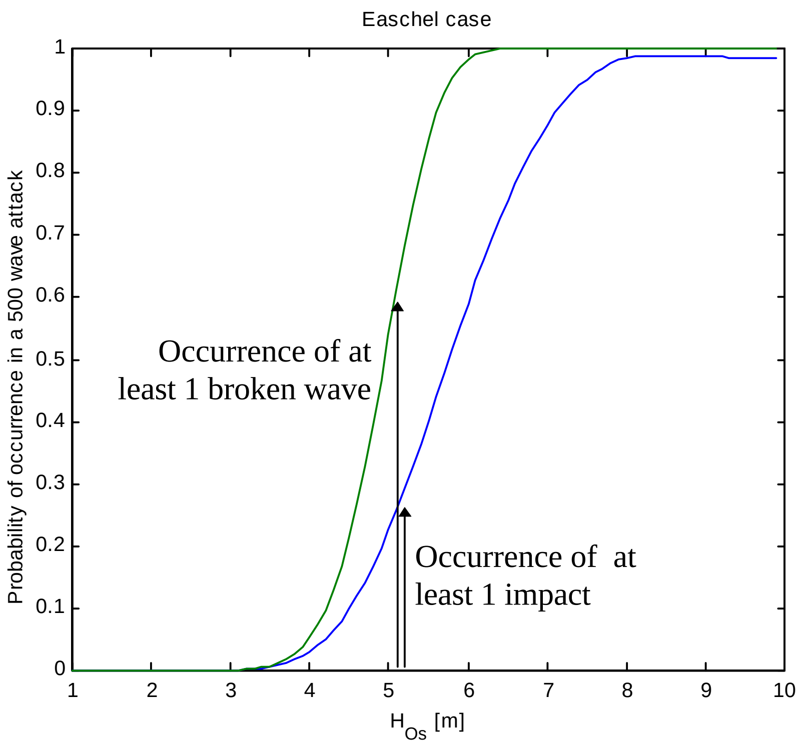


Fig. 2 Distribution of waves among breaking categories conditioned by a given significant wave height for the Easchel case. Since impact probability is very small, probability of at least 1 impact and 1 broken wave in a 500 wave attack is shown.

For the case of Easchel, the probability of occurrence of an impact at the structure is much more frequent if broken waves are not recognised. Fig 2 shows two curves representing the probability of occurrence of a breaking and a broken wave in a storm. For the hypothetical but realistic 1 year return period storm (indicated by an arrow), the evaluation of the broken fraction is essential in the hazard analysis, since many waves arrive to the structure already broken.

Fig. 3 shows the distribution of waves in front of Genoa Voltri main breakwater. In the storm with 50 years return period (H_{Os} = 6.2 m) the probability of impacts is remote and separation of broken waves is not critical.

The two analysed cases are very different: Easchel, characterised by a small berm (k_b=0.99), shallow water and flat foreshore, presents a higher probability of broken waves than of impacts for any significant wave height H_{Os} higher than 3.2 m; on the contrary for Genoa Voltri, placed in deep water and with a significant berm (k_b=0.85), the occurrence impacts is really remote but more probable than that of broken waves (for reasonable wave conditions).

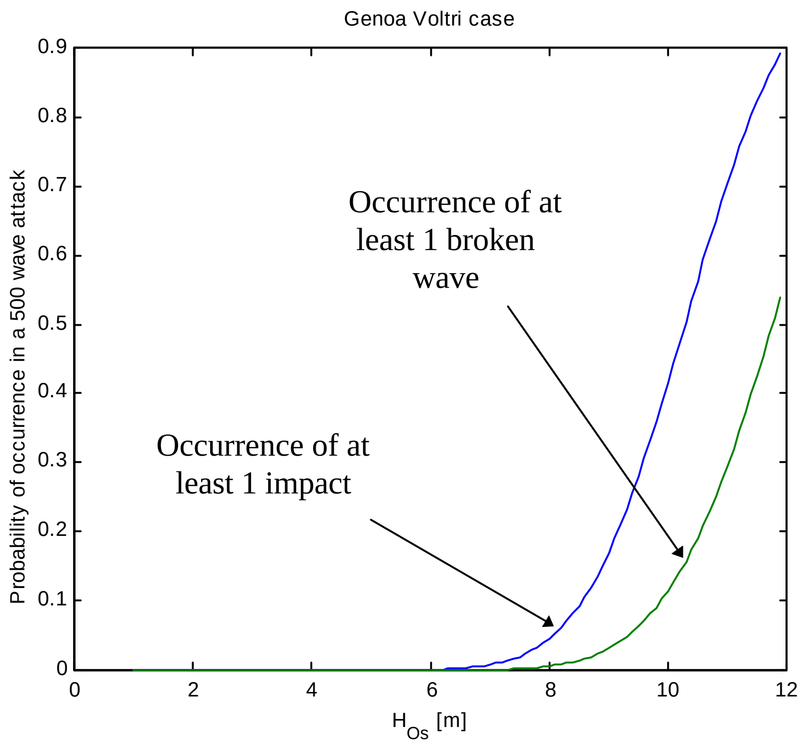


Fig. 3 Distribution of waves among breaking categories conditioned by a given significant wave height for the Genoa Voltri case. Since impact probability is very small, probability of at least 1 impact and 1 broken wave in a 500 wave attack is shown.

3.2 The loading induced by impacts

Breaking wave forces are computed applying a scheme based on Proverbs final outcomes (Oumeraci, 2000a). The impact characteristics are determined assuming a symmetric triangular impulse having a random non dimensional intensity k_I, not depending on any other variable than the scaling factor ($\rho_w\ g\ H_b^2\ L_c\ \sqrt{h_s/g}$). The impulse form and intensity is then considered independent from the actual wave conditions.

$k_I = F_{max}^*\ t_r^*$ = Stochastic variable (Log-Normal mean=0.086 and std=0.084),

where , F_{max}^* and t_r^* are shown in Fig. 4, and

$$F_{max}^*(P_f) = F_{max}/\rho_w\, g\, H_b^2\, L_c = e^{1/\gamma[-\log(1-P_f)]^{1/\alpha}+\beta} \qquad \text{(Genoa)}$$

$$F_{max}^*(P_f) = F_{max}/\rho_w\, g\, H_b^2\, L_c = \lambda - (\mu/\nu)\,(1-(\ln(1/P_f))^{-\nu}) \qquad \text{(Easchel)}$$

$$t_r^* = t_r/\sqrt{h_s/g}$$

where H_b is the actual breaking wave height given by eq. 2, L_c is the caisson length, P_f the non exceeding probability of the force, h_s the depth in front of the structure where braking presumably starts, the parameters $\alpha=6.45$, $\beta=2.49$, $\gamma=-0.14$, λ=6.45, μ=2.49, ν= 0.14 are estimated in Oumeraci et al. (2000a) where different distributions were proposed accounting for the different form factors relative to the two example breakwaters.

Proverbs indications on impulse are relative to the rise part only. Statistics on impact duration is sometimes associated to the total impulse time history, but neither the resulting total momentum nor the shape was ever calibrated.

If total duration is assumed equal to twice the rise time, k_I describes the total impulse by its definition. A reasonable non-dimensional maximum of k_I, (relative to 3 standard deviations) is 0.34; this is only 30% smaller than indicated by Goda (1994) who compares the maximum impulse to the momentum (M_{max}) of a semicircular cylinder advancing with the speed of the wave celerity:

$$M_{max} = \rho_w H_b^2\, L_c\, \pi/8 \sqrt{g\,(h_s+H_b)}$$

Non-dimensionalising according to the expressions given above:

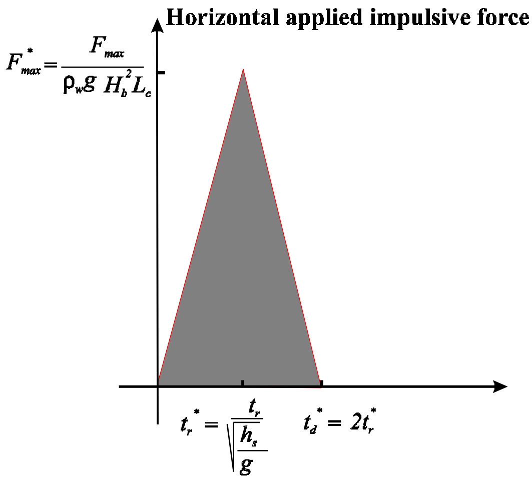


Fig. 4 Definition sketch of impact parameters

$$M_{max}^* = M_{max} / (\rho_w\, g\, H_b^2\, L_c \sqrt{h_s/g}\,) = 0.39\sqrt{1 + H_b/h_s} = (\text{for } H_b/h_s = ½) = 0.48$$

Total agreement between Goda suggestion and impulse description would be obtained for t_d=2.8 t_r, which is a reasonable time history also according to the test results shown in Archetti et al. (2000).

4 The dynamic model

Prototype tests were carried out at caisson breakwaters of Genoa Voltri and of Brindisi Punta Riso, see Lamberti & Martinelli (1998). The interpretation scheme at the base of the tests and of stability checks is that caissons can be considered as rigid bodies linked with springs and dash-pots, representing the foundation, to the deep and fixed soil. A Mass, Spring and Dash-pot (MSDA) model for an Array structure can be set up; contributions to mass, stiffness and damping are due to every component connected with the caissons, i.e. the rubble mound, the foundation, the seawater. The link between adjacent caissons through the foundations are represented as an added stiffness term.

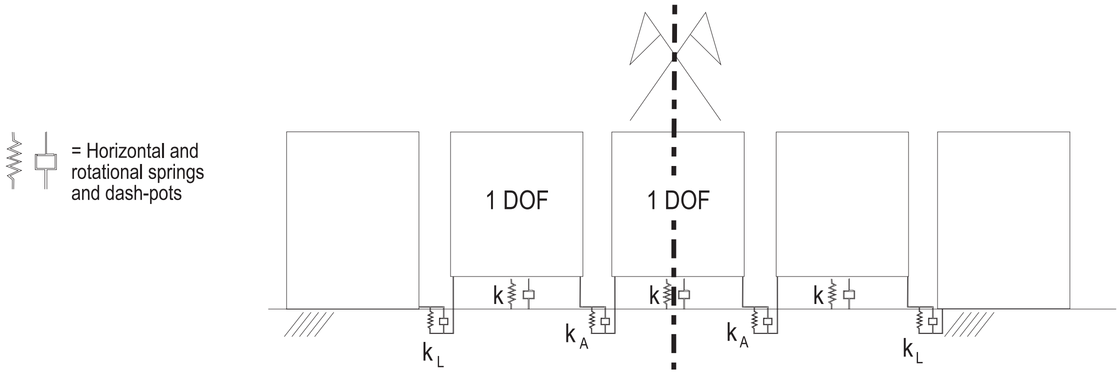


Fig. 5 Dynamic model.

Details of the 2 DoF model can be seen in Lamberti and Martinelli (2000). For a longcrested sea, such model is reduced to a single degree of freedom.

For the 2 DoF model the stiffness between caissons and foundation is computed with theory of strip foundation. The limitation of the number of caissons required a calibration of the link between side caisson to the fixed ones (K_L in the picture) as 20% of the stiffness with the foundation (K in the picture). Such side link gives an extra stiffness to the system, and this agrees with the stiffness assessed for the 1 DoF model, computed with theory of the rectangular foundation.

4.1 Force distribution

The adopted classification of waves into three groups (smaller waves that do never break and are just reflected by the wall, higher waves that break on the berm or in proximity of the structure hitting the wall, and the highest or steepest waves that break at a greater distance from the structure but not against it) is useful for the description of the load at the wall.

The distribution function of wave loads exerted at the breakwater is written as:

$$P(F<f) = P_n\, P(F<f \mid \text{no breaking}) + (1-P_n - P_b) \cdot P(F<f \mid \text{impact}) + \\ + P_b \cdot P(F<f \mid \text{broken}) \qquad (3)$$

where F = wave load modelled as a stochastic variable; P_n = probability of occurrence of pulsating loads; $P(F<f \mid \text{no breaking})$ = distribution function for pulsating wave loads, conditioned by no breaking, obtained by the model of Goda (1985); $P(F<f \mid \text{impact})$ = distribution function for impact loads, described in subsection 3.1; P_b = probability of occurrence of broken waves; $P(F<f \mid \text{broken})$ = distribution function for broken wave loads, obtained by the model of Goda (1985).

This continuous distribution can be used directly as the design load in a level II analysis.

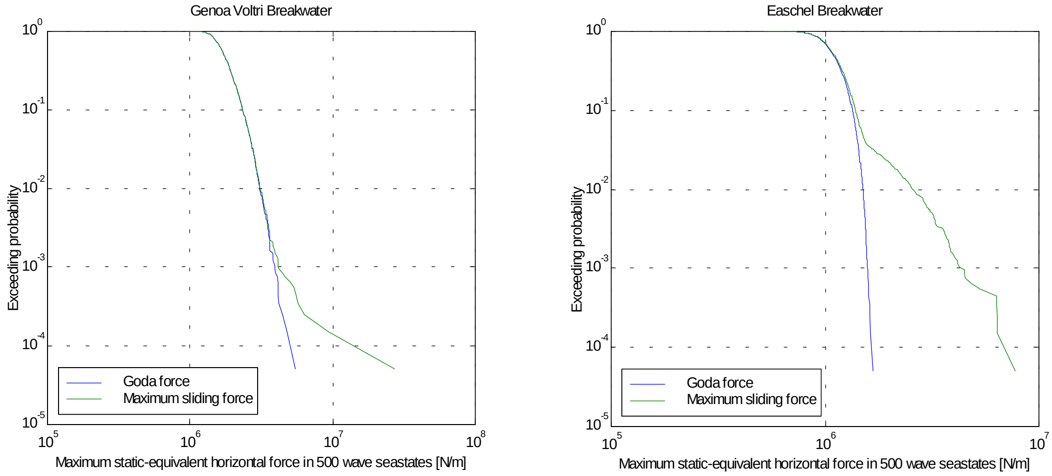


Fig. 6 Full distribution of design forces using the PROVERBS model with identification of broken waves, compared to Goda model, for a specific case (Voltri on the left, Easchel on the right).

Fig 6 shows the statistic of horizontal force obtained with Monte Carlo method, for two example cases. For the case of Genoa Voltri (on the left), the force statistic given by static and dynamic approach does not differs too much.

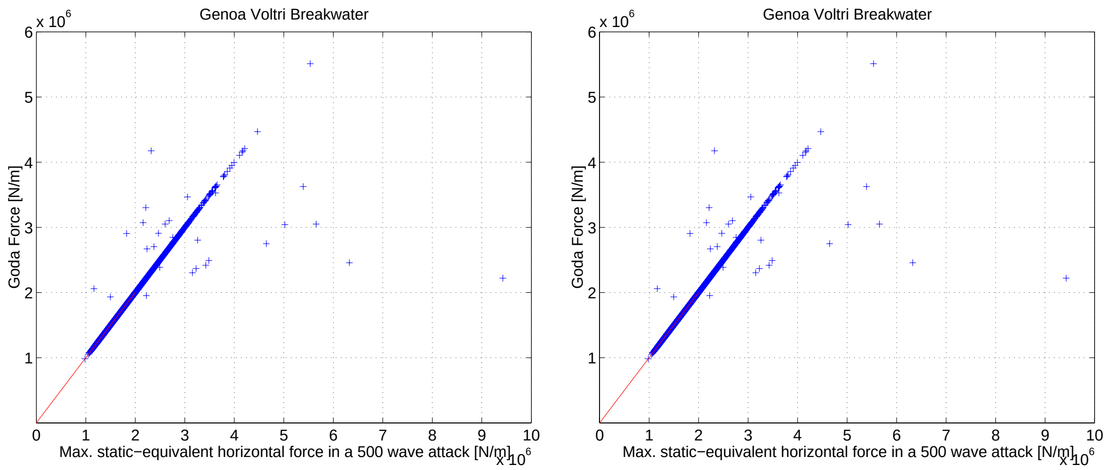


Fig. 7 Goda Force compared to computed Static equivalent force

Fig. 7 shows the computed static equivalent forces versus Goda forces, and it should be noted the high scatter of the values. When breaking occurs, the maximum force applied to the foundation is generally higher than foreseen by Goda formula.

4.2 The vertical component

Available statistics of uplift impulsive force (intensity and duration) is much more uncertain than horizontal one and apparently not congruous; the dynamic simulation along the vertical direction did not produce accurate results.

The horizontal and vertical translation modes have quite different eigenperiods and a separate dynamic analysis is, in a first approximation, significant. Actually vertical modes are about three times shorter than the horizontal ones, and rather damped (much more than the horizontal one, according to Barkan, 1962). We may then assume that for very short impacts the vertical reaction is almost static when the horizontal oscillation is at its maximum. The Goda equivalent static uplift force induced by the highest non-breaking wave is then assumed.

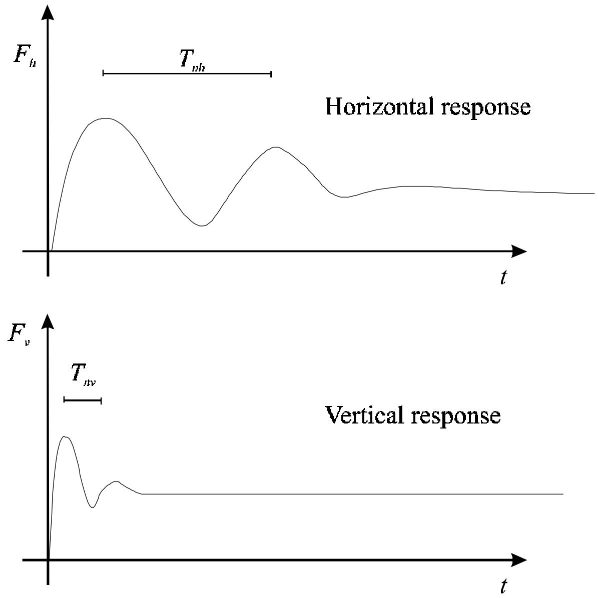


Fig. 8 Maximum horizontal elastic response takes place when the vertical response is approximately static.

The idea that dynamic vertical movements are not critical is implicit in many dynamic approaches: the vertical movements are usually not considered in many dynamic models (Goda, Kortenhaus, etc.) and the impact force parameters of Takayama are only applied to the horizontal force.

5 The failure mode

Shimosako and Takahashi (2000), Takayama et al.(2000), found that in some conditions the sliding distance is small or very small and can be tolerated.

When friction is exceeded in part of the contact surface, very small sliding may be a frequent tolerable phenomenon. Assuming, on the contrary, caisson and rubble mound as rigid bodies, tolerable sliding was considered a remote possibility (at least for Genoa Voltri for which case specific numerical simulations were run). In this paper, for the sake of simplicity, we assumed breakwater failure each time that a caisson slides.

The statistical description of friction factor is based on indications from Takayama & Ikeda (1992) and Takayama et al. (2000).

6 The hazard analysis

The hazard can be evaluated with a level II or a level III analysis.

A level II hazard analysis must take into account that the load function is discontinuous where breaking wave conditions are possible. It is a general property of approximating methods that they are unable to deal with discontinuous and/or highly non-linear failure functions. A possible approach requires a separation of the domains from the beginning, treating quasi static (non breaking or broken) and impact loads separately (by level II or III) and deriving the total distribution using eq. 3.

Tab. 1 Variable statistics

Variable		**Distribution**	***Mean***	***Standard deviation***
H_{Os} (Genoa)	Significant off-shore wave height	Gumbel	*6.20 m*	*0.75*
s_{0m} (Genoa)	Deep water wave steepness H_{0s}/L_{om}	Gaussian	*0.035*	*10%*
H_{Os} (Easchel)	Significant off-shore wave height	Gumbel	*5.05 m*	*0.63*
s_{0m} (Easchel)	Deep water wave steepness H_{0s}/L_{om}	Gaussian	*0.038*	*25%*
k_I	Parameter of GEV distribution of impulse	LogNormal	*0.086*	*0.084*
U_{fh}	Goda model uncertainty for Horizontal forces	Gaussian	*0.90*	*0.20*
U_{fu}	Goda model uncertainty for Uplift forces	Gaussian	*0.77*	*0.20*
h_w	Water level variation with respect to MSL	Gaussian	*0.10*	*0.20 m*
S_{thickn}	Superstructure thickness (depends on construction settlement)	LogNormal	*2.20 m*	*0.25 m*
μ	Friction Coefficient for SlidTak (Sliding/Takayama)	LogNormal	*0.64*	*10%*
$Z_{SlidTak}$	Model uncertainty SlidTak	Gaussian	*1.0*	*1 %*
$Z^d_{SlidTak}$	Model uncertainty SlidTak in dynamic conditions	Gaussian	*1.0*	*1 %*
Tp	Test variable for breaking (YES if < $P_{breaking}$)	Uniform [0,1[		
P_F	Force intensity	Uniform [0,1[		

Otherwise a level III analysis (Monte Carlo simulation) should be chosen in conditions where impact loads might occur. The Monte Carlo computation assumed in this note is based on the generation of single waves derived from a Rayleigh off-shore distribution based on a Gumbel distributed significant wave height.

Since this approach is rather time consuming, particular attention is given to reduce the actual computations. Fig 9 shows a flow chart describing in detail the hazard analysis procedure. The load applied to the vertical caisson by all waves belonging to the non breaking fraction is computed through the consolidated Goda (1985) formulae. The dynamic computation takes place only if at least 1 impact is identified comparing a uniform 'Test variable' to the breaking probability. On the basis of geometry (where the only non-deterministic parameter is the superstructure thickness, dependent on the settlement during construction) and sea level, the mass and stiffness of the system, and therefore the natural periods, are computed. The impulsive response is then evaluated. Assuming that breaking is a rare event, probability of occurrence of two impacts in the same storm is considered insignificant.

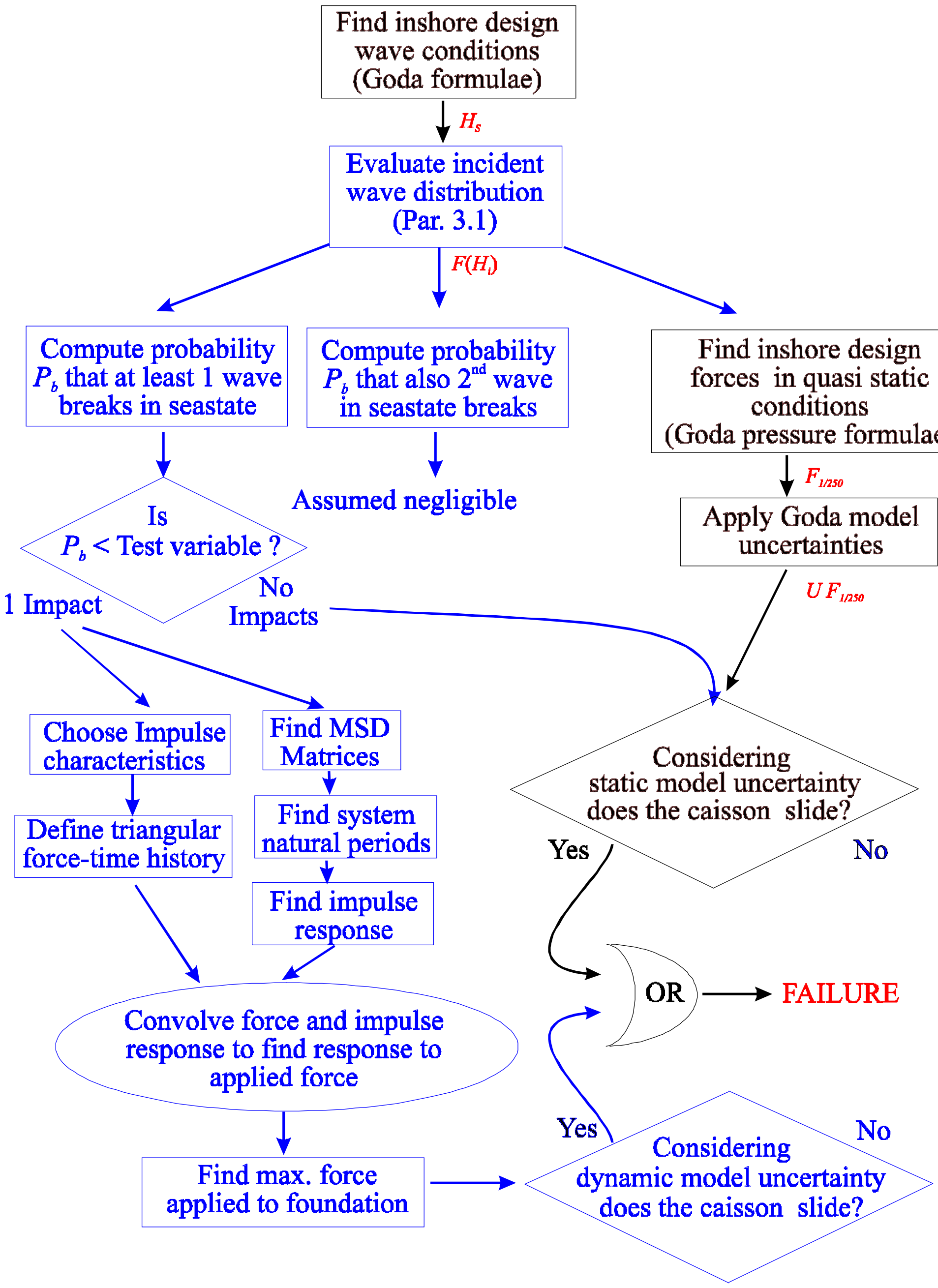


Fig. 9 Monte Carlo simulation

A correct evaluation of the lifetime failure probability should consider the dependency of the wave and resistance distribution on time, for the expected life of the breakwater (in general hydraulic processes are easier to handle being stationary and ergodic). The geotechnical parameters should be calibrated on the most weak spot of the breakwater, since failure should be related to the whole breakwater in order to have a meaningful engineering purpose.

Tab. 1 shows the assumed variable statistics.

For Genoa Voltri, storms characterised by smaller waves are definitely less dangerous, the second storm during lifetime being definitely much less critical and the relative probability of failure insignificant. Differently from the Easchel case, the hazard for Genoa Voltri main breakwater can then be obtained considering only the probability of failure for the most critical event.

7 Results

For both analysed example cases, the distribution of Goda forces diverges and is less conservative than distribution of forces computed according to the dynamic model. The probability level for which the two curves diverges (Fig. 6) is well below 0.1% for the case of Genoa Voltri, and about 5% for Easchel breakwater. If design force is smaller than this probability level, no difference is expected in the reliability analysis between Goda and more sophisticated models. On the contrary, if design force is higher than the above cited probability level, we should expect a significant difference on the results of the analysis, obtained using Goda or a dynamic model.

Lifetime failure probability due to sliding for Genoa Voltri main breakwater, where breaking events are unlikely, was found 2.4% with dynamic analysis (only slightly lower with static analysis). Lifetime breaking probability resulted to be 0.3%.

Failure probability due to sliding for Easchel fictitious breakwater, where breaking events are possibly induced by the berm, was found for the 1-year return period conditions, approx. 3.5% with dynamic analysis and 2.2% with static analysis. The computed probability of having at least 1 breaking event per year is 23%.

8 Conclusions

A procedure for deriving the probability of impacts was tested, accounting for the existence of broken waves. The assumed criteria for evaluation of the fraction of unbroken/breaking/broken waves limits the high values on the tail of the force distribution.

The analysis shows that PROVERBS impact model can be applied in a full probabilistic approach and that full dynamic model, calibrated against prototype tests, can reasonably evaluate the risk in a probabilistic framework, provided that:

- the formulas describing the two breaking criteria are concordant;
- all statistics, and in particular the combination of impulse, maximum force and duration are reliable and congruous.

Acknowledgements

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

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