RISK ANALYSIS OF THE VERTICAL OFFSHORE BREAKWATER AT GELA, ITALY

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ABSTRACT

A reliability method (FOSM) has been used to assess the probability of sliding failure of the 33 years-old island caisson breakwater sheltering an oil terminal at Gela (Sicily).

The vertically composite structure is placed in just 13 m depth upon a weak silty seabed. It has experienced settlements of about 1 m, toe scour around 2 m and degradation of the slender reinforced superstructure, especially damaged after a severe storm on 24 November 1991.

On the basis of new detailed wave climate studies and field investigations a modern probabilistic analysis has been performed to verify the actual risk of failure, particularly against sliding over the rubble foundation due to wave loads.

A new formulation of the failure function as been derived introducing new expressions for the pressure coefficients in the Goda formula. A detailed sensitivity analysis of the failure probability with the caisson geometry has also been performed.

KEY WORDS: Vertical Breakwater, Stability, Reliability, FOSM.

INTRODUCTION

Conventional breakwater design in coastal engineering is mostly based on a deterministic approach, i.e. the design load is compared with the resistance of the structure. The assessment of the probability of failure is only performed by studying the probability distribution of the most important loading variable, the wave height (Franco et al., 1986; Lamberti, 1992; Burcharth, 1992; Van der Meer et al., 1995). On the contrary, the real behaviour of the structure is much more complex, mainly because other uncertainties are also due to those random variables which express the mechanical characteristics of the structure, hence its reliability. Therefore both a PIANC working group (PTC II-n. 28) and a EU-funded research project (MAST III Proverbs)are now being devoted to the probabilistic analysis of vertical breakwaters.

A first attempt was recently done by Christiani et al. (1996) assessing the reliability of a vertical breakwater as a series system of failures even if the geometrical parametrisation has been limited to the caisson width.

The possibility of an accurate assessment of the failure risk of a structure at sea is very difficult to be performed because all the joint probability density functions are generally unknown. Only for some of them the marginal probability density functions are known. When the random variables can be considered as independent (non correlated), the failure probability is obviously given by a *n*-fold integral, where *n* is the number of the independent variables. In practice, the evaluation of the failure probability via a direct multi-dimensional integration implies a huge amount of computations even for relatively small numbers *n*. In this respect an intermediate approach may be the so called FOSM (First Order Second Moment) methods, in which all the variables do exhibit a similar stochastic behaviour, i.e. reduced to the Gaussian distribution. This hypothesis is more reasonable for vertical type (caisson) breakwaters, at least when just considering the critical safety against sliding. The real occurrence of this characteristic failure case has been reported by Franco and Passoni (1994).

In the following section a brief description of the main probabilistic methods is given. Then a FOSM algorithm is applied to a specific real breakwater case which appears to survive with relatively small safety margin. Finally the numerical procedure has been iterated to explore the influence of the main geometric parameters on the risk of failure.

PROBABILISTIC APPROACH OF THE RELIABILITY ASSESSMENT

The evaluation of the safety of a structure, or its reliability, is accomplished by defining the failure modes and the corresponding "failure functions". Many failure events in marine engineering are characterised by different failure modes, but the attention can often been focused to the most important one, thus reducing to a single failure function. In the following the various approximations for the estimate of the failure probability are briefly summarised.

Level III Methods

Level III methods are based on the knowledge of the joint probability density function (jpdf) of the random variables which have to obey to the failure function $g(\underline{X})=\underline{R}-\underline{S}$. Denoting $f(\underline{X})$ the jpdf of a random variables vector $\underline{X}=(X_1, X_2, \ldots, X_n)$ the failure probability P_f is then given by the integral of the $f(\underline{X})$, over the subdomain in which the resistance R is lower than the load S, i.e.

$$P_f = \int_{R<S} f(x)dx \qquad (1)$$

For the simplest case of two independent random variables R and S, the previous integral (1) is reduced to

$$P_f = \iint_{R<S} f(r,s)drds \qquad (2)$$

In this case it is easier to indicate in a 3D plot the design point, as the most probable value of the variable $\underline{X}(\underline{R},\underline{S})$ (see fig. a).

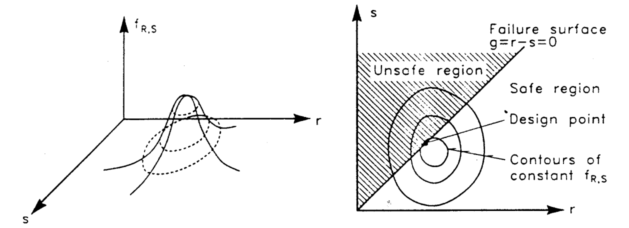


Fig. A: Illustration of the two-dimensional *jpdf* for loading and strength (Burcharth, 1992)

Without sake of generality, let's consider a 2D case where the two random variables are independent, the previous integral may be casted in the form:

$$P_f = \iint_{R<S} f_R(r)\cdot f_S(s)drds \qquad (3)$$

which, after integration by parts, may be reduced to a 1D integral:

$$P_f = \int_0^\infty F_R(x)\cdot f_S(x)dx \qquad (4)$$

in which $F_R(x)$ is the cumulative distribution of R.

All the methods which fall into this class do express the reliability in terms of failure probability, which can be assessed by analytical solution for some simple cases, or numerical integration. Some examples are the First Order Reliability Methods (FORM) in which the failure hypersurface is approximated by the tangent hyperplane at a specific point, and Second Order Reliability Methods (SORM) in which a quadratic approximation of the failure hypersurface is adopted (Madsen et al., 1986).

Another simple way to compute the jpdf of the dependent variable g is the Monte Carlo method in which this goal is reached via the simulation of a large number of realisations of the stochastic process through a pseudo-random procedure applied to the random variables $\underline{X}=(X_1, X_2, \ldots\ldots, X_n)$, as reported by Ang and Tang (1984), Augusti et al. (1984), Casciati, Natale (1992), Lamberti (1992), Ditlevsen and Madsen (1996) and among others.

Level II Methods

In the engineering practice it is difficult to know the pdf of the vector $\underline{X}$, due to the lack of data, and furthermore the expression (2) is not trivial so that it can be integrated only by numerical quadrature. The most frequently known parameters of the distributions are the 1^{st} (mean) and 2^{nd} (standard deviation) order moments of the random variables, only. The methods which are based on this statistical information are the so called level II methods. Specifically they are referred to as reliability index methods. One such method is the First Order Second Moment method (FOSM) which has been used in this work.

Considering, for simplicity, a 2D case, and assuming as "failure" (or "performance") function the *safety margin* $g=R-S$, the limit state ($g=0$) between the safe ($g>0$) and the failure ($g<0$) regions is given by (see fig. b):

$R'\sigma_r - S'\sigma_s + \mu_r - \mu_s = 0$

where the primes indicate the normalised variables and σ, μ stand for standard deviation and mean values respectively.

In the normalised variables space the minimum distance β from the origin is a measure of the probability of failure (Cornell, 1969); in fact increasing β the area under the function f (r,s) will be larger, i.e. the probability P_f will be smaller.

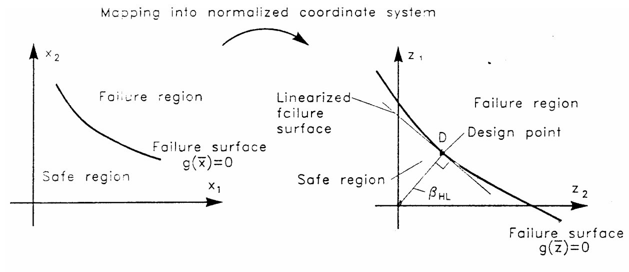


Fig. B: Definition of the Hasofer and Lind reliability index β_{HL} (Burcharth, 1992)

From simple geometric considerations it's easy to verify that

$\beta = (\mu_r - \mu_s)/(\sigma_r^2 + \sigma_s^2)^{1/2}$

i.e. the ratio between the mean value of the function g and its standard deviation, assuming that g is a linear function of random independent variables. In this case the failure probability is given by:

$$P_f = \text{prob}(g \le 0) = \int_0^\infty f_g(x)dx = \\ = \Phi\left(\frac{0-\mu_g}{\sigma_g}\right) = \Phi(-\beta) \qquad (5)$$

Generally g is a function $g(\underline{X}) = g(X_1, X_2, \ldots, X_n)$ of several random variables, and $g(\underline{X})=0$ defines the limit of the safe state. If $\underline{X}=(X_1, X_2, \ldots, X_n)$ is a vector of n uncorrelated variables, after the normalisation in the n-space ($Z_i=(X_i-\mu_{Xi})/\sigma_{Xi}$) the reliability index becomes:

$$\beta_{HL} = min_{g(z)=0}\left(\sum_{i=1}^{n} z_i^2\right)^{1/2} \qquad (6)$$

i.e. the minimum distance of the limit state surface $g(\underline{X})=0$ from the origin of the normal space. The problem of finding the minimum distance of the function, subject to the constraint $g(\underline{X})=0$, has been approached by the method of Lagrange's multipliers (Harr, 1987; Ang and Tang, 1984) which, after some algebra, yields:

$$\beta = \frac{-\sum_i \left(z_i^*\right)\left(\frac{\partial g}{\partial Z_i}\right)_*}{\left(\sum_{i=1}^{n}\left(\frac{\partial g}{\partial Z_i}\right)_*^2\right)^{1/2}} \qquad (7)$$

where * stands for the point around which the function is approximated.

The same result may be obtained when looking at the first order term in the Taylor's series (8) of the function g where the mean value is given by the expression (9) and the approximation of the variance is given by expression (10) (Ang and Tang, 1984):

$$g(X_1, X_2, \ldots, X_n) \cong \sum_{i=1}^{n} (z_i^*)\left(\frac{\partial g}{\partial Z_i}\right)_* \qquad (8)$$

$$\mu_g \cong -\sum_{i=1}^{n}(z_i^*)\left(\frac{\partial g}{\partial Z_i}\right)_* \qquad (9)$$

$$\sigma_g^2 \cong \sum_{i=1}^{n}\left(\frac{\partial g}{\partial Z_i}\right)_*^2 \qquad (10)$$

and the ratio $\mu_g/\sigma_g = \beta_{HL}$ is the distance from the tangent plane of the failure surface at z_* to the origin of the normalised variates. The sensitivity of the reliability index with respect to the performance function (especially for non-linear functions g) has been overcome by first order approximations evaluated at a certain point (design point) on the failure surface (Hasofer and Lind, 1974). A similar procedure, may be applied also to correlated variates by mapping them into an uncorrelated set, after a coordinate transformation (Ang and Tang, 1984).

If some of the random variables are not normally distributed, the reliability can be assessed via the β_{HL} index, provided that a local equivalent Gaussian distribution can be found (Rackwitz, 1976). Such equivalent normal distribution can be also obtained using the Rosenblatt transform (Ang and Tang, 1984).

The numerical implementation of the procedure can be organised in different steps (Burcharth, 1992):

1) set some trial coordinates for the design point

$$Z^d = (z_1^d, z_2^d, \ldots, z_n^d)$$

2) computation of the derivatives at the design point $\alpha_i = \dfrac{\partial g}{\partial z_i}$

3) better estimate of the unknown vector

$$z_i^d = a_i \frac{\sum_{i=1}^{n}(a_i z_i^d) - g(Z^d)}{\sum_{i=1}^{n}(a_i)^2}$$

4) iteration of steps 2) and 3) until convergence in the coordinates is achieved.

5) estimation of the reliability index by the definition

$$\beta_{HL} = \left(\sum_{i=1}^{n} z_i^2\right)^{1/2}$$, as given in expression (6).

In order to avoid the convergence of the algorithm towards local minima, different sets of initial guesses for the vector Z^d have to be considered.

Level I Methods

The basic assumption in the Level I methods is the reduction of the system to a single random variable dependency. These are often quoted as deterministic/quasi probabilistic methods. With respect to maritime engineering application the wave height distribution is therefore taken as a direct measure of the probability of failure.

THE CASE OF GELA OFFSHORE BREAKWATER

The offshore breakwater of Gela industrial harbour is located inside the homonymous gulf on the Southwest coast of Sicily in the middle of the Mediterranean Sea (fig. c). It shelters a 2800 m long jetty and at the same time it is used as mooring berth for oil tankers. The design was carried out in 1962 and construction was completed in 1964.

This "island harbour" solution was selected due to the very shallow sandy seabed slope (0.4%). The water depth at and off the breakwater varies between 11 and 14 m MSL for a long distance offshore: a scour trench 150 m wide on average and up to 2 m deep now exists at the structure toe.

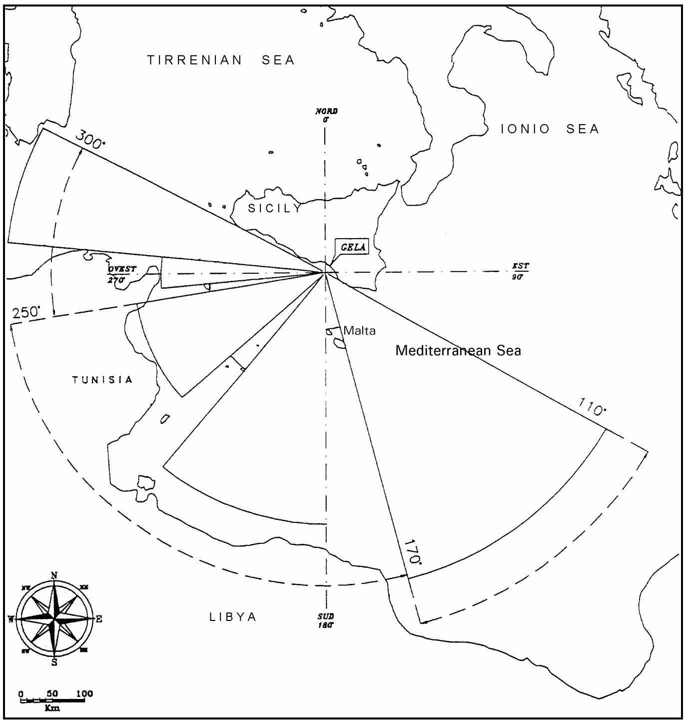


Fig. C: Location map and fetches of Gela breakwater (AQUATER-Noli, 1996)

The breakwater planshape shows three differently aligned portions with a total length of 1160 m (fig. d). The breakwater is composed by 60 r.c. caissons sitting upon a thin (2 m) rubble foundation of 0.5-1.0 kN rock. The dimensions of each caisson are: length 20.4 m; width 14 m (b=17 m with the lateral bottom slab expansions); height hi=12 m (hi+Rc=17.5 m, including superstructure and parapet wall) (fig. e).

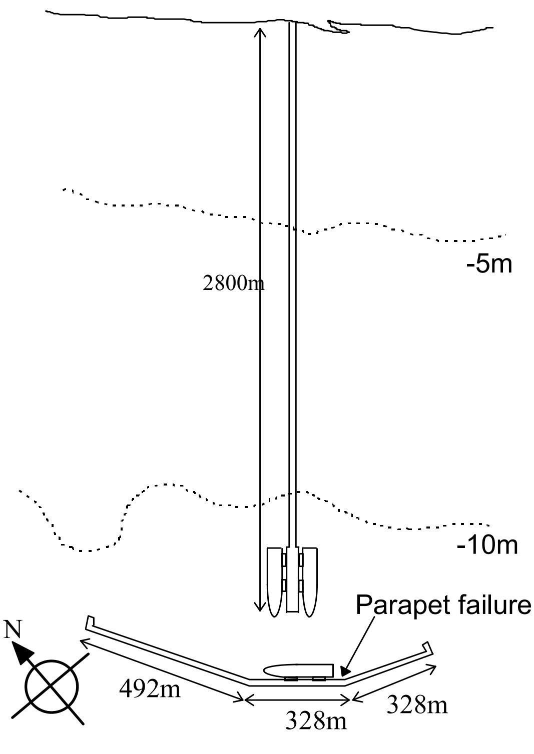


Fig. D: Layout of Gela offshore breakwater

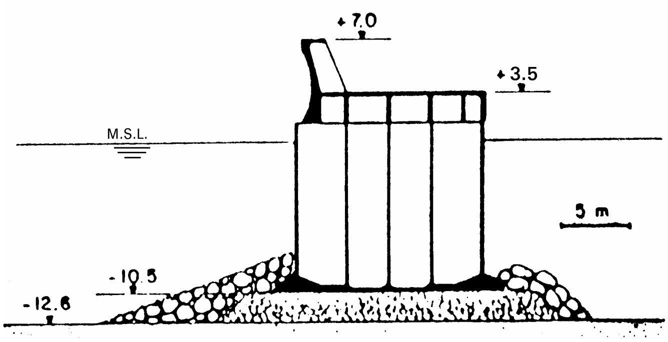


Fig. E: Design cross section of Gela breakwater (Franco, 1994)

The caissons are divided in 24 cells, that are filled with a mixture of water and sand dredged from the seabed (unit weight of 19 kN/m^3). The thickness of the internal walls is 0.2 m, while the external ones are 0.3 m thick. Even the partly precast r.c. crown slab was made cellular and almost completely filled with sand, in order to reduce pressures on the seabed. The total volume per unit length is about 200 m^3/m. The total dry weighted averaged density results to be 20.2 kN/m^3 (2.06 t/m^3). Special attention was paid to the design of the parapet shape and elevation (originally +7.0 m MSL), in order to minimise wave overtopping and allow the loading and unloading operations on the rear quay. A thin curved r.c. crownwall is stiffened by light buttresses, again to reduce loads on the foundation.

The seabed stratigraphy shows up to 3-5 m sandy-silty soil; from 5 m to 40-45 m silty-clayey soil and below -45 m sandy-silty soil again (friction coefficient of 10° and shear resistance of 10 to 30 kN/m^2).

The geotechnical study made in 1961 gave a soil pressure from the structure of 170 kN/m^2. A load breaking-point of 145 kN/m^2 was obtained and a soil consolidation then suggested. A pretty uniform settlement of 0.7 - 1.0 m has recently been measured along the structure.

The original wave climate study and the model tests were performed by the French SOGREAH in 1963. Model tests with regular waves (T=7-11s) were conducted in 2-D (scale 1:50) and 3-D (scale 1:40) to check stability, overtopping and toe scour around the head.

The local spring tidal range is only 0.4 m. The wind setup is also small due to the narrow continental shelf. In storm conditions with strong persistent onshore winds and low atmospheric pressure a total water level setup of +0.5-0.6 m MSL can be assumed at the structure.

On 24 Nov. 1991 the breakwater superstructure (wave wall and pipelines behind) was damaged by a violent storm from SSW, when large overtopping occurred, although no caisson displacement has been observed. The estimated hindcasted deepwater significant wave height was around 7 m with a return period of about 50 years. The RON directional buoy off Mazara (some 150 km westward) recorded a maximum H_s = 6.15 m.

Some urgent repair work was carried out and a general rehabilitation project started up. A new detailed study of the local wave statistics has been recently performed (AQUATER-Noli, 1996) based on wave hindcasting models calibrated against buoy and satellite measurements. Wind input was gained from UKMO-Bracknell (10 years) and from Malta island meteorological station (28 years). Then the extreme wave height analysis has been carried out with the POT method (Goda, 1988) on the 28 years wave data set to produce directional and omnidirectional deepwater wave statistics: the Gumbel estimates obtained for a few selected return periods are listed in tab.1, together with the corresponding values of the standard deviation.

An advanced wave propagation model has been used to account for wave energy dissipation in the extended shallow water area (the isobath -50.0 m is about 10 km offshore) due to refraction, shoaling, friction and breaking and for energy spreading in frequency and direction. Computations were performed for three main offshore wave directions and three inshore water depths to derive wave height attenuation of 20-50%, mainly due to breaking dissipation.

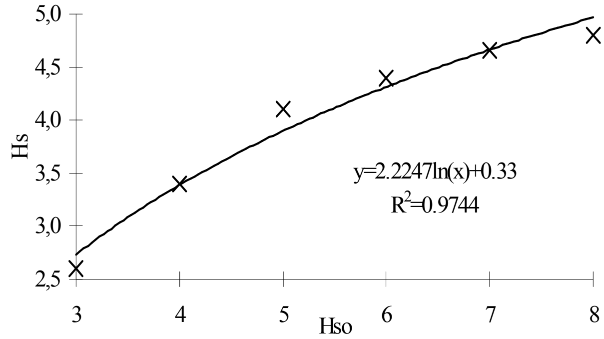


Fig. F: Correlation between offshore and inshore significant wave height (in m) at 12m water depth

The resulting "transfer function" at a depth of -12 m is plotted in fig. f and the actual values of Hs at the structure for the same return periods are also listed in tab.1, together with the corresponding standard deviations (reduced with the same ratio). Finally the corresponding mean values of the local Hmax (and their std. devs.) are obtained as 1.65 Hs, according to Goda, to account for a non Raileigh distribution of wave heights in shallow water (table a).

Tr	Hso	σHso	Hs	σHs	Hmax	σHmax
2	5.6	0.2	4.2	0.1	6.9	0.2
10	5.9	0.2	4.3	0.1	7.1	0.2
50	6.8	0.3	4.6	0.2	7.6	0.3
100	7.2	0.3	4.7	0.2	7.8	0.3

Table A Offshore (Hso) and inshore (Hs) extreme wave heights at Gela (all values in m except Tr in years)

FAILURE ANALYSIS AND RELIABILITY ASSESSMENT

A First Order Second Moment method has been used to assess the probability of failure of the vertical structure against horizontal sliding over the foundation. Other failure modes (e.g. overturning, slipping, settlement, scour etc.), which are usually less critical, are here neglected for sake of simplicity, even if they should be considered together their complex interactions in a full risk analysis of the breakwater.

The sliding performance function has been casted in the form:

$$g = f\left(\gamma_c V_{dry} - \gamma_w V_{wet} - UF_u\left(H_{max}\right)\right) - UF_h\left(H_{max}\right) \quad (11)$$

where:

f friction coefficient between caisson and rubble mound foundation (Gaussian variate)

γ_c specific weight of the caisson (Gaussian variate)

U uncertainty coefficient for the Goda wave pressure formula (Gaussian variate)

H_{max} maximum wave height (Gumbel variate)

are the four random variables, while F_u and F_h are expressions given by the complex formulae proposed by Goda (1985) for the vertical (i.e. uplift) and horizontal wave loads respectively and the remaining factors are known deterministically. The 1st and 2nd moments of the probability distributions are reported in the following table b:

random variables	μ	σ
***f* [#]**	0.636	0.10
γ_c [kN/m^3]	21.00	1.10
***U* [#]**	1.00	0.20
H_{max} [m]	5.67	0.78

Table B Selected random variables for Gela caisson breakwater

The formal expressions of the functions F_u and F_h can include terms dependent on the wave length L, which is not known a priori. A linear approximation with respect to the wave height H_{max} is accurate enough (Rigoni, 1995), at least within a limited interval of practical interest (H_{max}=5-10 m, R^2>0.95) and the failure function becomes:

$$g = f\left(A\gamma_c - B\gamma_w - U\left(CH_{max} + DH_{max}^2 + EH_{max}^3\right)\right) + \qquad (12)$$
$$- U\left(FH_{max} + GH_{max}^2 + IH_{max}^3 + JH_{max}^4 + L\right)$$

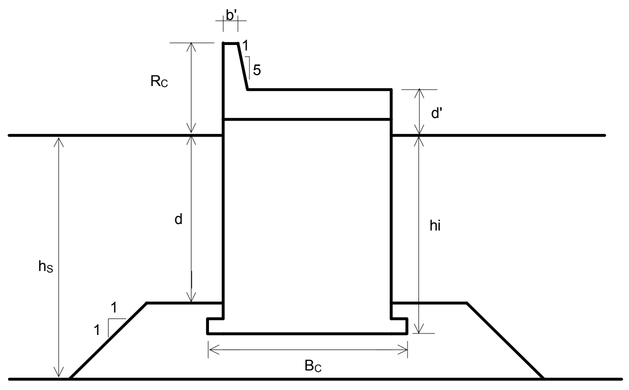
where the coefficients *A*, *B*, *C*, *D*, *E*, *F*, *G*, *J*, *I*, *L* depend also on the geometrical parameters. The dependency on H_{max} is of the third order, with respect to the uplift force, and of the fourth order for the horizontal contribution. Different expressions are reported in the recent literature (Burcharth et al., 1994) mainly because in structural geometry and in the parametrisation of the Goda's pressure formulae were introduced (e.g. constant wave steepness).

The convergence of the numerical procedure, described in section 2, has been carefully tested with several trial values of the random variables, even if this convergence is not always achieved nor theoretically proved. Finally a threshold error for the g function was fixed of the order $|10^{-4}|$.

Some comparisons have been performed with other authors' results (Sorensen, private communication) and also with other numerical solvers for constrained non-linear optimisation problems (Microsoft-Excel 5.0, 1993; Wolfram, 1991).

The numerical procedure has been first used to determine the probability of failure for the present geometry of the Gela caisson breakwater (considering a storm water level setup of 0.6 m and a settlement of 0.7 m). The result of this Level II computations give *Pf* = 0.037 (1/year), which corresponds to a risk of failure *R*=84 % in a 50 years lifetime *Tv* (31% in 10 years, 71% in 33 years) since $R(Tv)=1-(1-Pf)^{Tv}$.

The same analysis has also been performed with a traditional Level I (deterministic) approach, but in this case no failure has been predicted because only a very high wave height (probably depth limited) could determine g=0 for the failure function.



R_C=5.7m B_C=17m d=9.7m h_S=13.1m
hi=11.8m d'=2.2m b'=2.0m

Fig. G: Cross section of the schematised caisson breakwater.

SENSITIVITY ANALYSIS OF *Pf* ON THE BREAKWATER GEOMETRY

A sensitivity analysis has then been performed in the space of the geometrical parameters in order to ascertain the influence of some possible design changes on the failure probability. For this purpose the structure has been schematised as indicated in fig. g with the following set of non dimensional variables:

$$Rc* = \frac{Rc - d'}{hs},\ Bc* = \frac{Bc}{hs},\ d* = \frac{d}{hs},\ d'* = \frac{d'}{hs},\ hi* = \frac{hi}{hs}$$

The intervals of the parameters have been chosen in order to include the actual geometry of Gel caisson breakwater, whose values are *Rc**=0.27, *Bc**=1.30, *d**=0.74, *hs*=13.10 m, *d'**=0.17, *hi**=0.90. The ranges are: 0.15<*Rc**<0.46, 1.15<*Bc**<1.45, while all the parameters within brackets have been set constant in order to detect the effect of the parapet height alone. Different sets of variables have been investigated.

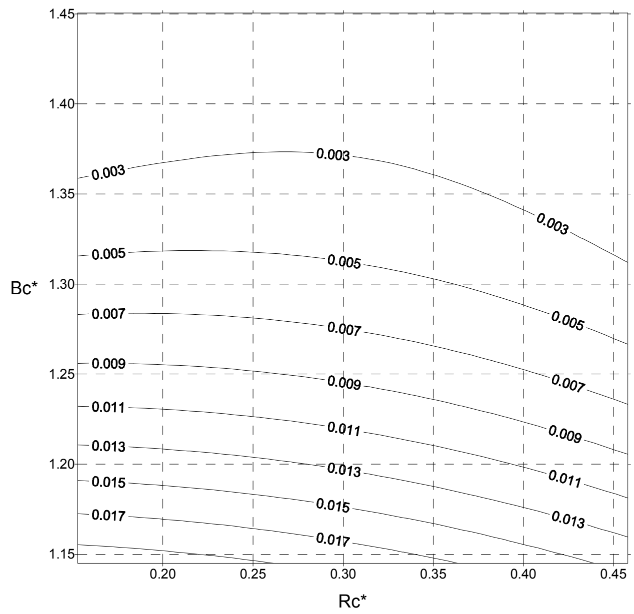


Fig. H: Contour plot of P_f vs. *Rc** and *Bc** (*d**=0.74, *d'**=0.33, h_i*=0.74)

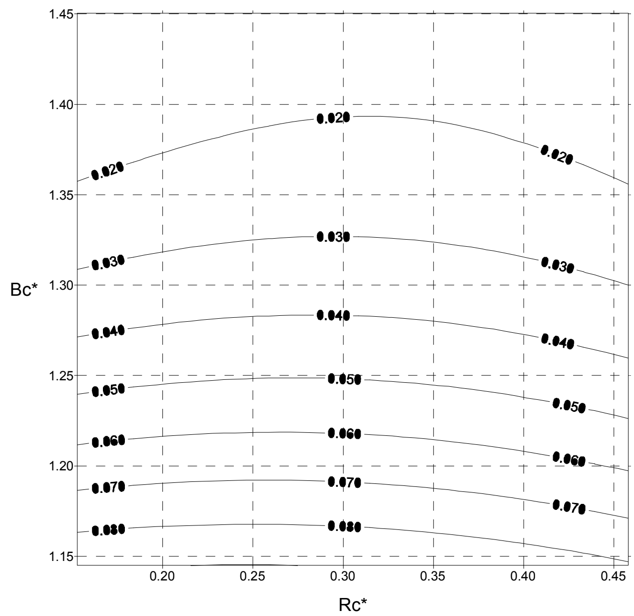


Fig. I: Contour plot of P_f vs. Rc^* and Bc^* (d^*=0.74, d'^*=0.17, h_i^*=0.9)

First the failure probability has been computed for different values of Rc^* and Bc^* while the remaining ones at some intermediate values have been set constant (d^*, d'^* and h_i^*) for the case of the same caisson simply placed ($d^*=h_i^*$) over the rubble mound foundation (fig. h) or corresponding to the actual ones (d^*=0.74, h_i^*=0.17) of Gela breakwater (fig. i). The behaviour of the curves exhibits a trend of slightly increasing *Pf* for lower Rc^* and smoothly decreasing for larger Rc^*. Anyway the gradient of *Pf* exhibits a much larger component in the Bc^* direction, as obviously explained by the net weight increase with the caisson width. The negligible variability in the Rc^* direction is due to the slight increase of the horizontal load, which is counterbalanced by the increase of the total weight for larger Rc^*, thus determining a final reduction of *Pf*. The results presented in Fig. 9 refer to the actual overall geometry, i.e. a caisson whose bottom is lower than the berm elevation, while the other geometrical factors remain the same. The trend of the curves is similar, but the *Pf* values are much higher in this case which can be also explained by the increase of the buoyancy.

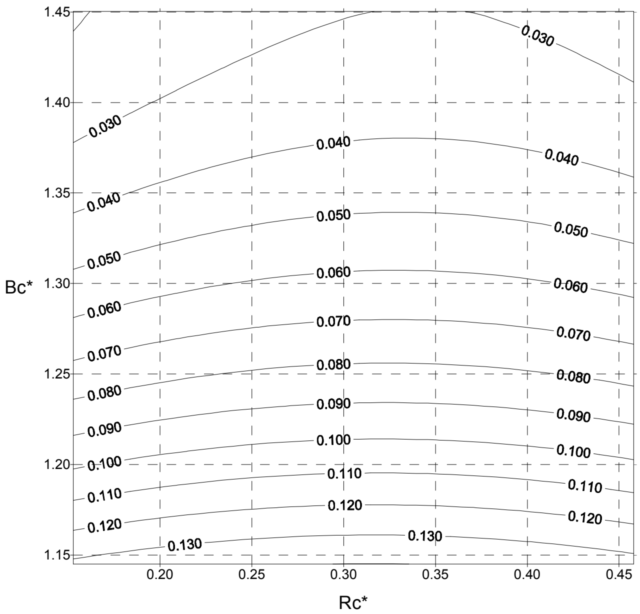


Fig. J: Contour plot of P_f vs. Rc^* and Bc^* (d^*=1.0, d'^*=0.07, h_i^*=1.0)

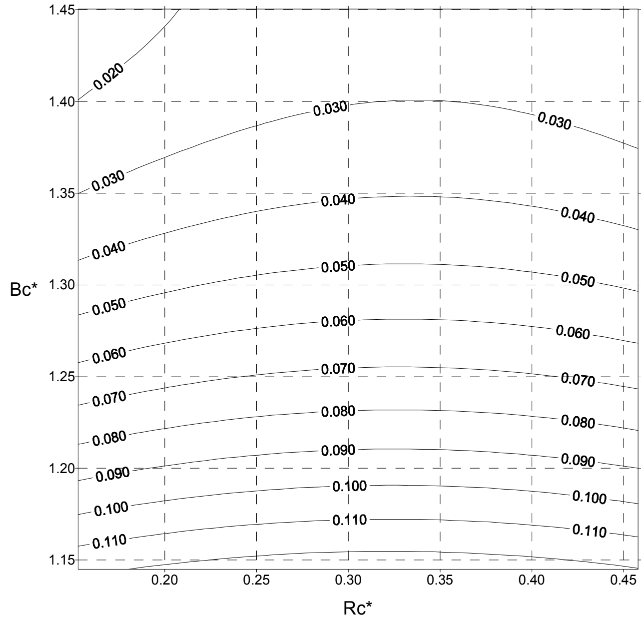


Fig. K: Contour plot of P_f vs. Rc^* and Bc^* (d^*=1.0, d'^*=0.07, h_i^*=1.0)

The horizontal loads are larger because of the additional contribution of the dynamic pressure acting in the zone between the berm crest and the caisson's bottom. Even larger *Pf* values result when the caisson is placed directly over the sea bottom without any rubble mound foundation, as indicated in fig. j where d^*=1.0 and h_i^*=1.0.

The last geometry has also been tested using different expressions for the pressure coefficients (α_1 α_2 α_3) in the Goda's formula. Namely in fig. j, as also in the previous ones, the coefficients have been expressed as linear function of the design wave height as quoted in Rigoni (1995). The results in fig. k, on the contrary, have been obtained under the assumption of constant Goda's pressure coefficients for varying wave height and the difference of the results due to this approximation appears not negligible.

CONCLUSIONS

The application of a FOSM probabilistic computation for the stability against sliding of an existing 33-year old caisson breakwater off Gela (Sicily) has produced a quite large risk of failure (55% in 50 year lifetime).

The influence of some main geometrical parameters on the probability of failure has been studied with a parametric analysis. The elevation of the superstructure has a negligible influence on the probability of failure against sliding, under the particular environmental conditions considered in this case. This means that the safety against overtopping is not in contradiction with the global stability of the structure.

Other parameters, like the berm geometry, should be considered in a more general stability analysis of a caisson type breakwater together with the extension of the model to other failure modes. The dependency of the algorithm on the order of the failure function has also to be investigated.

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