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A Computational Risk Assessment Model for Breakwaters

CAN E. BALAS
Civil Engineering Department
Gazi University
Celal Bayar Bulvarı, 06570 Maltepe
ANKARA

RIFAT TUR
Civil Engineering Department
Gazi University
Celal Bayar Bulvarı, 06570 Maltepe
ANKARA

Abstract: In the reliability-risk assessment, the second order reliability index method and the Conditional Expectation Monte Carlo (CEMC) simulation were interrelated as a new Level III approach in order to analyse the safety level of the vertical wall breakwaters. The failure probabilities of sliding and overturning failure modes of the Minikin method for breaking wave forces were forecasted by approximating the failure surface with a second-degree polynomial having an equal curvature at the design point. In the new approach, for each randomly generated load and tide combination, the joint failure probability reflected both the occurrence probability of loading condition and the structural failure risk at the limit state. The approach can be applied for the risk assessment of vertical wall breakwaters in short CPU durations of portable computers.

Key-Words: Risk Assessment, Breakwaters

1 Introduction

In the structural design of vertical wall breakwaters, two methods have been widely applied in European countries. The first method is the First Order Mean Value Approach (FMA) [1], and the second one is the Hasofer-Lind second order reliability (HL) index. The partial coefficient system utilizes the former and the latter has been employed to compare risk levels of rubble mound and vertical wall structures [2]. Goda and Tagaki [3] suggested a reliability design criteria in which the Monte Carlo simulation of expected sliding distance was carried out for caisson breakwaters.

The reliability-risk assessment of Ereğli harbor main breakwater involves the second order reliability index (β_{II}) method interrelated with CEMC simulation as a Level III method. In this technique, uncertainties that affected most of the variables in the design were incorporated throughout the lifetime of structures by the use of the simulation of design conditions, i.e. the water level change due to tidal action and the random wave action. The proposed Level III method was compared with the individual application of β_{II} (Level II) method.

2 Computational Risk Model

The safety of vertical wall breakwater was evaluated by modelling random resistance and load variables with common probability distributions at their limit-state. The primary variable vector **z** in the normalized space indicates these random variables. The functional form of the basic variables consistent with the limit state is the failure function denoted by: $g(\mathbf{z})=(z_1,z_2,...,z_n)$. The safety of the structure can be assured by designating an admissible value of the probability of achieving the limit state defined by: $g(\mathbf{z})=0$.

In the reliability-based study, the second-order reliability index method was utilized, in which the failure surface was approximated by a rotational parabolic surface. The parabolic limit state surface in standard normal space, $g(\mathbf{z})$ [4] was taken in the model as follows:

$$g(\mathbf{z}) \approx a_0 + \sum_{i=1}^{n} b_i z_i + \sum_{i=1}^{n} c_i z_i^2 \tag{1}$$

where a_0,b_i, and c_i are the regression coefficients of the second-order polynomials; z_i are the standardized normal random variables and n is the number of random variables. Regression coefficients were obtained by using the response surface approach in standard normal space [5]. The positive sum of the principle curvatures of limit state surface at the design point ($\mathbf{z}^*$) was expressed as:

$$K_s = \frac{2}{|\nabla g|} \sum_{i=1}^{n} c_i \left[1 - \frac{1}{|\nabla g|^2} \left(b_i + 2 c_i z_i^* \right)^2 \right] \tag{2}$$

$$|\nabla g| = \sqrt{\sum_{i=1}^{n} \left(b_i + 2 c_i z_i^* \right)^2} \tag{3}$$

$$\beta_{II} = -\phi^{-1} \left[\phi(-\beta_I) \left(1 + \frac{\varphi(\beta_I)}{R\phi(-\beta_I)} \right)^{-\frac{n-1}{2}\left(1 + \frac{2K_s}{10(1+2\beta_I)}\right)} \right] \tag{4}$$

where, β_{II} is the second-order reliability index, R is the average principal curvature radius expressed as $R=(n-1)/K_s$, β_I is the first order reliability index $\beta_I=\boldsymbol{\alpha}^T\ \mathbf{z}^*$; Φ is the standard normal distribution function, φ is the standard normal probability density function, $\boldsymbol{\alpha}$ is the directional vector at the design point. The structural performance of the breakwater under the affect of wave loading was investigated by utilizing the Conditional Expectation Monte Carlo (CEMC) simulation. The exceedance probability (P_f) of failure damage level was

obtained by utilizing the control random variable vector of $\mathbf{z}_i = (z_{i1}, z_{i2}, \ldots, z_{ik})$ as follows:

$$P_f = E_{z_j:j=1,2,\ldots,n \text{ and } j \neq i} \left[P_f(\mathbf{z}_i) \right] \tag{5}$$

where, E[.] is the conditional expectation (mean) and P_f ($\mathbf{z}_i$) is the failure probability evaluated for $z_{i1}, z_{i2}, \ldots, z_{ik}$, by satisfying the conditional term in eqn (6) for the last control variable as follows:

$$P_f(z_{ik}) = \Pr\left[z_{ik} < g_{ik}\left(z_j : j = 1,2,\ldots,n \,\&\, j \neq i \right) \right] \tag{6}$$

where, k is the number of control variables in the simulation. A computer program was developed for the simulations that repetitively reproduced breakwater performance at the limit state condition until the specified standard mean error of convergence (ε) was satisfied. The limit state equations for breaking wave forces acting on the vertical wall breakwaters were derived in this study from the Minikin's method [6] as illustrated in Fig.1. For sliding failure mode, the limit state equation utilized in the model was [7]:

$$g = \mu_f \left[B h_S \gamma_C - B d_S \gamma_o - \frac{1}{4} \gamma_o B H_b \right] - \frac{101}{3} \gamma_o \frac{d_s (d_s + d)}{d L_d} H_b^2 - \frac{1}{2} \gamma_o d_s H_b - \frac{1}{8} \gamma_o H_b^2 = 0 \tag{7}$$

The limit state equation for the overturning failure mode was obtained as:

$$g = \left(\frac{h_S B^2}{2} \right) \gamma_C - \left[\left(\frac{1}{4} d_S^2 \gamma_O + \frac{1}{6} \gamma_O B^2 \right) H_b + \left(\frac{101}{3} \frac{\gamma_O}{L_d d} d_S^3 + \frac{101}{3} \frac{\gamma_O}{L_d} d_S^2 + \frac{1}{8} \gamma_o d_s \right) H_b^2 + \left(\frac{1}{48} \gamma_o \right) H_b^3 + \left(\frac{1}{2} d_s B^2 \gamma_o \right) \right] = 0 \tag{8}$$

In eqns (7) and (8), d_S is the depth from still water level, h_s is the height of vertical wall breakwater, B is the width of wall, μ_f is the coefficient of friction, γ_o is the weight per unit volume of seawater, γ_c is the weight per unit volume of concrete, H_b is the breaking wave height, h_C is the breaker crest taken as $H_b/2$, P_m is the maximum pressure acting at the SWL, d is the depth at a distance one wavelength seaward of the structure, L_d is the wavelength at the water of depth d.

In the application of the suggested Level III method, the offshore wave height was randomly generated and a linear wave transformation was carried out to obtain the design load of the structure. Then, the reliability of the structure was investigated (on average 30,000 times) by the β_{II} method at the limit state. As a result, the joint failure risk reflected the occurrence probabilities of wave loading and the limit state for each random load combination generated in the simulation. Then, the β_{II} method was applied individually to the case study as a Level II approach and the results obtained from these methods were compared with each other.

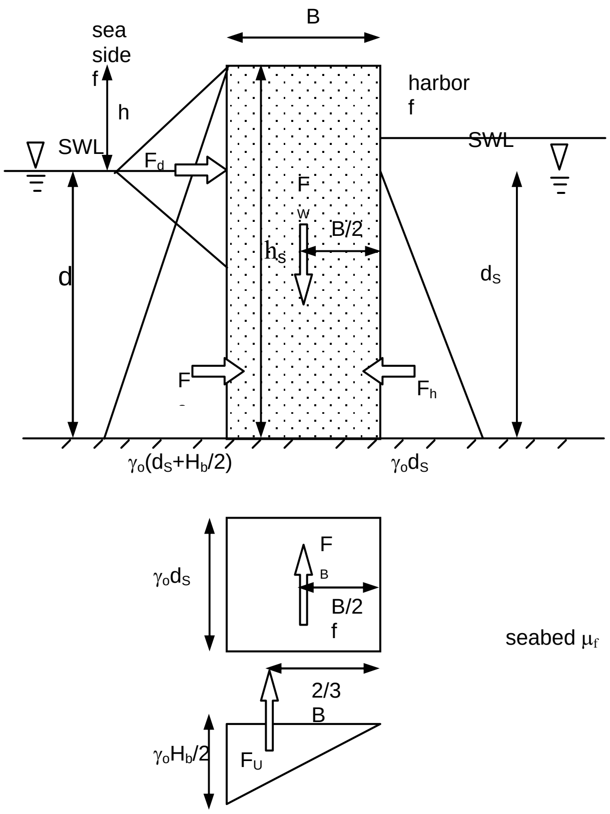

Fig. 1 Breaking wave forces.

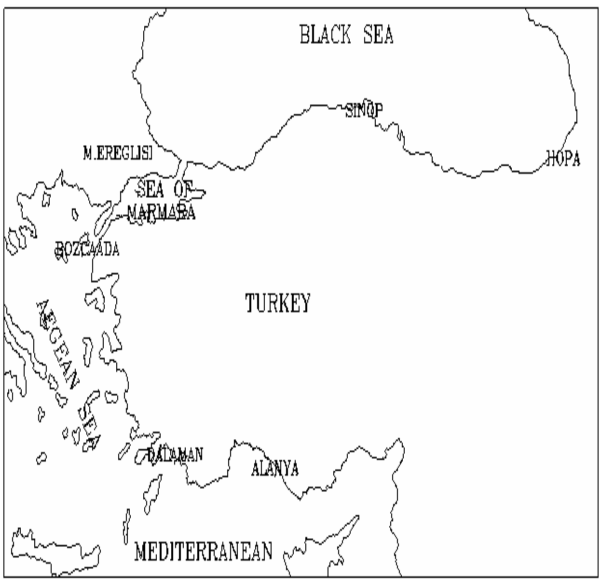

Fig. 2 Locations of recently planned harbours in Turkey.

3 Model Application

A commercial harbor will be constructed in Marmara Ereğlisi on the inland Sea of Marmara of Turkey (Figures 2 and 3). The basic parameters in the design are listed in Table 1 with the mean (μ) and standard deviation (σ) of normally distributed random variables. The wave height was modeled by a joint Weibull-Rayleigh probability distribution with the scale (α) and shape (β) parameters listed in Table 1 by using the wave characteristics listed in Table 2 [8]. In Table 2, H_b is the breaker height at construction depth, H_o' is the unrefracted deep-water wave height, M is the plotting position, Kr is the linear refraction coefficient, Hs and Ts are the annual maximum significant deep-water wave height and period, respectively. The new Level III reliability approach, in which the second order reliability index (β_{II}) and the Conditional Expectation Monte Carlo (CEMC) simulation were interrelated, was suggested to handle the uncertainties inherent in wave data and design methodology [9]. Wave characteristics of the site were randomly generated by simulation. Afterwards, the failure mode probability was predicted by the parabolic limit state surface having the identical curvature at the design point with the higher degree failure surface. The mean (μ) and the standard deviation (σ) of the wave height distribution were μ_H= 4.77m and σ_H = 0.95m, respectively. Probabilities of failure of the Ereğli vertical wall breakwater in 50 years of lifetime obtained by the suggested Level III approach for both sliding and overturning failure criteria are given in Fig. 4 in which sliding criterion governs the design. The sensitivity study carried out by using a rank correlation method reveals that the overturning failure function is sensitive to the wave height with a correlation coefficient of Rc=-0.84 (load variable) and to the weight per unit volume of concrete with Rc=0.46 (resistance variable). The sliding failure function is sensitive to the wave height with Rc=-0.73 (loading variable), to the weight per unit volume of concrete Rc=0.22 (resistance variable), and to the coefficient of friction with Rc=0.57.

For sliding failure criterion, the probability of failure determined from Level II reliability method is lesser than the probability of failure obtained from Level III reliability method with a root mean square error of 0.25 and a bias of –0.17 (Fig. 5). For overturning failure criterion, the probability of failure obtained from Level III reliability method is greater than the probability of failure obtained from Level II method with a root mean square error of 0.32 and a bias of –0.17.

4 Disscussion

Forecasts of neural networks trained by the CG algorithm showed a better performance and had higher values of determination coefficient than other training algorithms for same testing sets. Therefore, decreasing the training epoch may provide flexibility in predictions of neural networks. Elman type neural networks can demonstrate a better performance when compared to other neural networks. For example, they improved predictions of H_s and T by 142% and 98%, respectively, when the data in the fifth testing set were utilized.

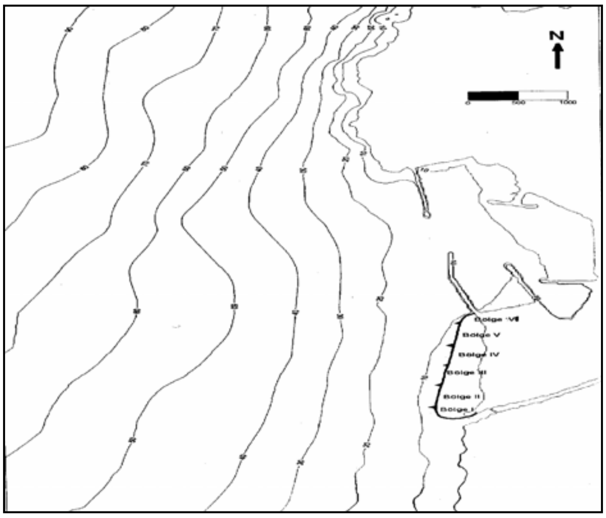

Fig. 3 Ereğli (Marmara) harbour.

Table 1: Design parameters for the breakwater.

Height of the structure	h_s = 17 m	
Depth at the toe of the wall	d_s = 6.5 m	
Depth at distance one wave length seaward	d = 8 m	
Wave length at water depth d	L_d = 76 m	
Weight per unit volume of sea water	γ_o = 1.02 t/m^3	
Weight per unit volume of concrete (t/ m^3)	μ=2.4 t/m^3	σ=0.1 t/m^3
Coefficient of friction (normally distributed)	μ= 0.64	σ= 0.1
Wave height (Weibull-Rayleigh) H (m)	α =2.97	β =1.55

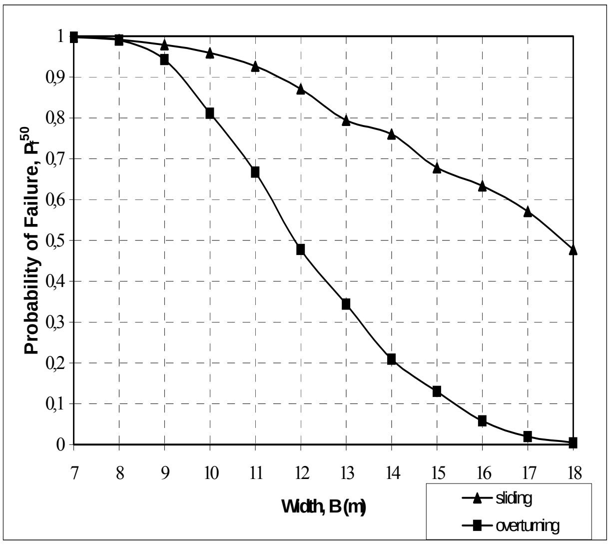

Fig. 4 Failure risk of the Ereğli vertical wall breakwater in lifetime.

Table 2: Annual maximum significant wave characteristics of the site [8].

M	Hs	Ts	L_o	Kr	H_o'	H_b/H_o'	H_b
1	3,170	8,072	101,641	1,000	3,170	1,11	3,519
2	3,200	8,096	102,256	0,974	3,116	1,11	3,459
3	3,430	8,278	106,901	1,000	3,430	1,1	3,773
4	3,780	8,539	113,76	1,000	3,780	1,09	4,120
5	3,820	8,568	114,528	1,000	3,820	1,09	4,164
6	3,900	8,625	116,058	0,973	3,793	1,09	4,134
7	4,540	9,055	127,911	0,972	4,411	1,08	4,764
8	4,880	9,267	133,962	0,984	4,802	1,08	5,186
9	4,900	9,279	134,313	0,984	4,822	1,08	5,207
10	5,070	9,381	137,277	0,984	4,989	1,08	5,388
11	5,520	9,640	144,955	0,984	5,432	1,07	5,812
12	6,020	9,911	153,227	0,984	5,924	1,06	6,279
13	6,650	10,231	163,305	0,984	6,544	1,05	6,871

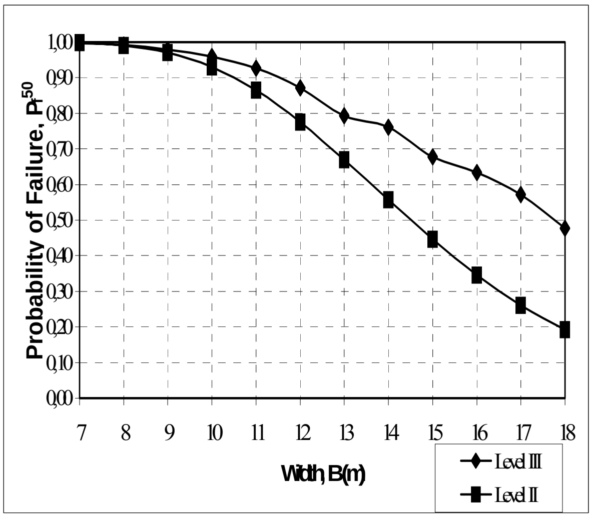

Fig. 5 Comparison of the failure probability of the vertical wall breakwater in a lifetime of 50 years, obtained by Level II and Level III methods for sliding criterion under breaking wave forces.

Van der Meer failure surface in the two-dimensional standardized coordinate system of random variables for Mersin yacht harbor main breakwater is shown in Fig. 6. Exceedance probabilities of the damage level S=2, as a function of nominal rock diameter D_{n50} was obtained by Van der Meer failure function of plunging. These design curves are illustrated in Fig. 7. Figures 6 and 7 permits the rubble mound structure to be designed for several damage levels and failure consequences, and for design wave height and lifetime alternatives. Hence, the economic consequences of structural failure can be reflected in the model, by choosing a reliability level, which is defined as the non-exceedance probability of a damage condition during a specified reference period and under given environmental conditions. High reliability levels are selected, when the economic consequences of failure are serious, so that the public is not exposed to a risk, such as loss of property or life.

5 Conclusions

For the case study at Ereğli, results obtained from Level II method deviated from results obtained by simulation. Therefore, structural reliability evaluated by using Level II method was considered as approximate, when compared to Level III methods. The type of distribution (Normal or extreme value) in design parameters of the failure function also effected the reliability evaluations irrespective of the design level. As a result, the reliability of vertical wall structures was highly variable and depended upon the unpredictable nature of coastal storms, the reliability method and distributions utilized in the design.

However, the reliability methods had advantages when compared to the deterministic practice, since the random behaviour of structural performance in lifetime could be estimated at the planning stages. The reliability approach applied in this paper within few minutes of CPU time in a portable computer was recommended for the risk assessment of vertical wall breakwaters.

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