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Reliability-Based Risk Assessment of Rubble Mound Breakwaters Under Tsunami Attack

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ABSTRACT

In the reliability-based risk assessment, the second order reliability index (βII) method and the Conditional Expectation Monte Carlo (CEMC) simulation were utilised in order to analyse the safety levels of Haydarpasa Port main breakwater, Sea of Marmara, Turkey. The failure probability was forecasted in the βII method by approximating the Hudson performance surface with a second-degree polynomial having an equal curvature at the design point where the design and structural parameters were taken into consideration. In the past, applications of reliability-based risk assessment methodology in Turkey for design conditions, only wave characteristics, tidal range, storm surge, wave set-up and the structural system parameters were included. Tsunami risk was not considered a major design parameter and was not included in the computations.

In this study, a structural stability criterion in coastal engineering is suggested to achieve a common definition of reliability including the tsunami risk. The model introduced in this study is a practical technique for reliability-based risk assessment of breakwaters subject to tsunami risk. In order to determine the occurrence probability under design conditions, which is a function of storm waves, tidal range, storm surge and tsunami height, the CEMC simulation was interrelated with the βII method. From the reliability-based risk assessment model applied to Haydarpasa port as a pilot study in Turkey it was found that, inclusion of the tsunami risk increases the failure risk of the structure, and as lifetime of the structure increases, the impact of tsunami risk on the failure mechanism is more important. For Haydarpasa port main breakwater, tsunami was not the key design parameter when compared to storm wave, however, in places with great seismic activity, tsunami risk may be very significant depending on the occurrence probability and the magnitude of the tsunami.

ADDITIONALINDEX WORDS: *harbour, Turkey, Sea of Marmara.*

INTRODUCTION

In the reliability-based risk assessment of coastal structures, two methods have been widely applied in European countries. The first method is the First Order Mean Value Approach (FMA) (BURCHARTH, 1992), and the second is the Hasofer-Lind second order reliability (HL) index (BURCHARTH and SORENSEN, 1998). The former utilizes the partial safety factors instead of overall safety factor and the latter has been employed to compare risk of damage levels and; hence safety of the structure under design loads (BURCHARTH and SORENSEN, 1999).

The reliability-risk assessment of Haydarpasa breakwater was carried out by an improved risk assessment technique developed in this paper. The technique involves the second order reliability index (β_{II}) method interrelated with CEMC simulation. 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.

RELIABILITY BASED RISK ASSESSMENT

The safety of breakwaters was evaluated by modelling random load and resistance variables with probability distributions at their limit-state. The primary variable vector z in the normalized space indicates these random variables. The serviceability limit-state was implemented for the safety evaluations, as the exceedance of the failure damage level which may not result in complete breakdown of the structure, but may cause an interruption in the achievement of its functions. The functional form of the basic variables used in the limit state is defined as failure function by: $g(z)=(z_1,z_2,...,z_n)$. The safety of the structure can be assured by designating an admissible value of the probability of achieving the limit state defined by: $g(z)=0$.

In the reliability-based assessment, the second-order reliability index method was utilized, in which the failure surface was approximated by a rotational parabolic surface. For the rubble mound breakwater the parabolic limit state surface in standard normal space expressed by the structural performance function of g(z) (BALAS and ERG_N, 2000) was:

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

where a0,bi, and ci are the regression coefficients of the second-order polynomials; zi are the standardized normal random variables and n is the number of random variables. Geometrically, the structural performance function is the failure surface that separates the failure region (g<0) from the safe region (g≥0). Regression coefficients were obtained by using the response surface approach in standard normal space (HONG, 1999). The positive sum of principle curvatures of limit state surface at the design point (z*) was expressed as:

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

where,

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

The second-order reliability index (_II) was empirically determined as follows:

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

where, R is the average principal curvature radius expressed as R=(n-1)/K_s, φ is the standard normal distribution function, φ is the standard normal probability density function, β_I is the first order reliability index $\beta_I = \alpha T$ z* , α is the directional vector at the design point, z*.

The Conditional Expectation Monte Carlo (CEMC) simulation was utilized to investigate the structural performance of the breakwaters under the design conditions. The failure probability (Pf) is obtained by utilizing the control random variable vector of zi=(zi1, zi_2,…,z_{ik}) as follows:

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

where, E[.] is the conditional expectation (mean) and P_f (z_i) is evaluated for z_{i1}, z_{i2},…, z_{ik}, by satisfying the conditional term in Equation (6) for the last control variable as follows:

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

where, k is the number of control random variables in the simulation. The control random variable in the structural performance was selected as the significant design wave height that follows the Gumbel extreme value distribution. By selecting the design wave as a control random variable effects the results significantly, the variance of the estimated quantity was reduced by increasing the convergence rate and decreasing the simulation cycles by circa 1/10, when compared to the direct Monte Carlo method, where design wave is taken as another random variable. Computer simulations repetitively reproduced breakwater performance at the limit state condition until the specified standard mean error of convergence (ε), as 0.1% was satisfied.

In the applications for a selected design wave, tidal level, storm surge and tsunami set-up were randomly generated (on average 30,000 times) from the probability distributions to obtain the design load of the breakwater. Its reliability was investigated (again on average 30,000 times) by the β_{II} method at the limit state. Consequently for each randomly generated load combination of the computer, the joint damage probability reflected both the occurrence probability of loading conditions and the exceedance probability of the limit state which is the damage level of the Haydarpasa breakwater.

RISK ASSESSMENT OF HAYDARPASA PORT MAIN BREAKWATER

The Haydarpasa port situated on the Anatolian side of the Bosphorus (latitude 41 00 N and longitude 29 01) has a wide hinterland connected to inland by means of highways and railroads which are the shortest route connecting Europe to Middle East countries. The port has two breakwaters of 1700 m. and 600 m. long protecting the sea area of 62 hectares. The main breakwater was designed at a depth of 15 m. with armor weights of 4 tonnes on a slope of 1:2.5 (Table 1). The significant design wave height and period were H_s= 4m. and T_s=6 sec, respectively, obtained from extreme value statistics (ERGIN and ÖZHAN, 1986).

The reliability risk assessment model was applied in order to evaluate the structural safety of the breakwater by modelling random design and structural variables, i.e. wave height, tidal range, storm surge, wave set-up and the structural system parameters with probability distributions (Table 2). The wave set-up and surf beat were taken as 5% and 10% of the significant wave height, respectively (GODA, 2000).

Since for centuries inhabitants of coastal areas in Turkey have suffered from the effects of tsunamis, it is important to include the tsunami in the reliability-based structural risk assessment models. Since the breakwater was constructed at a depth of 15m tsunami height is taken as the design parameter as defined in Figure 1 (FARRERAS, 2000). The available information concerning tsunamis associated with

Table 1. Structural and load characteristics of Haydarpasa breakwater.

Design Variable X_I	Value
Nominal diameter D_n (m.)	1.15
Weight W_{50} (tones)	4.0
Design wave height H_s (m.)	4.0
Tidal range R_T (m.)	±0.2
Surf Beat (m.)	0.4
Storm surge S_S (m.)	0.2
Relative density Δ_ρ	1.63
Height of structure (m.)	10
Structure slope Cotθ	2.5

Table 2. Distribution parameters of basic variables used in reliability-based risk assessment.

X_i	Distribution	Parameters
Y	Beta	a=3 ; b=2 c=1.5
D_n	Beta	a=3.0 b=1.7 c=1.6
H_s	Gumbel (FTT-I)	a=1.88 b=0.78
Tide	Triangular	min=-0.2 max=0.2
Tsunami height	Log-normal	a=2.3 b=8.3
Δ_ρ	Normal	μ_{xi}=1.63 σ_{xi}=0.15
Cotθ	Beta	a=8 ; b=1.9 c=2.7

Figure 1. Maximum horizontal extension of the inundation produced by the tsunami, adopted from FARRERAS (2000).

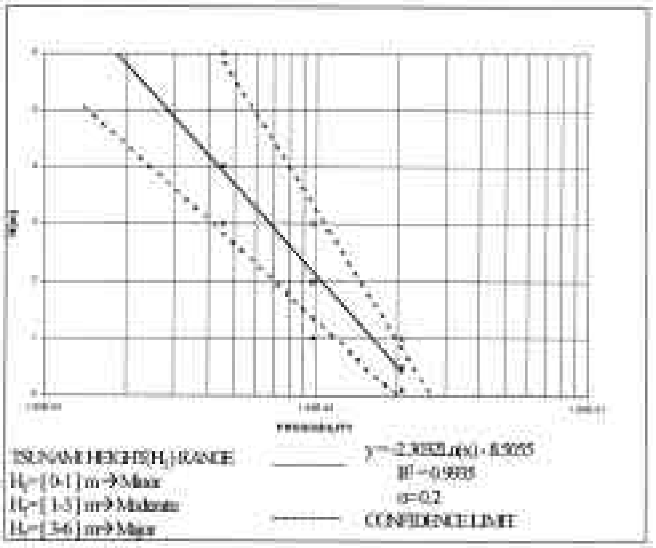

Figure 2. Probability distribution of tsunami heights in Istanbul and the adjacent coasts of Marmara in terms of probability of occurrences of the mean values.

the Istanbul and the eastern Marmara earthquakes have been used as the tsunami data (ALTINOK and ERSOY, 2000). Data documented for Istanbul between the years of 358-1999 (1641 years) gives the number of major tsunamis in Istanbul and on the coasts of Marmara Sea as N=32, where the tsunami height exceeds 0.5 m, as descriptively listed in Table 3.

There are several valuable magnitude and intensity definitions, classifications and statistical approaches for the occurrence probabilities of tsunami. Efforts towards the quantification of tsunami in terms of intensity scale started with the pioneering work of SIEBERG (1927). Since then several investigators put a great effort to grade the tsunami in terms of intensity (AMBRASEYS, 1962) and magnitude scales (IMAMURA, 1942; 1949; IIADA, 1956; 1970; ABE, 1979; SHUTO and MATSUTOMI, 1995). Based on these studies Table 4 was prepared for structural risk assessment, with the possible tsunami height ranges judged by the intensity scale and the descriptions related to the major earthquakes in the Marmara region tabulated in Table 3 which is adopted from ALTINOK and ERSOY(2000). With the limited and inaccurate data, the tsunami height ranges were identified based on the tsunami information in Table 3. The lack of tsunami intensity and magnitude scale with detail description in the data utilized, forced the investigators to make a decision on the tsunami height range by using Modified Sieberg Seismic Sea-Wave Intensity Scale (i) (AMBRASEYS, 1962) in Table 3. In this table, HTm is the maximum tsunami wave height (m), D is the distance that the water penetrated inland (m) and NTI designates that there is no tsunami information.

As a pioneering work, a simple statistical analyses of the classified tsunami heights in Istanbul and the adjacent coasts of Marmara is analysed in terms of probability of occurrences of the mean values of the ranges as given in Figure 2. In this figure, the regression line, presenting the statistical characteristic of the offshore mean values, was provided with a certain confidence limit indicating the

Table 3. Major tsunamis in Istanbul and Marmara region, adopted from ALTINOK and ERSOY(2000). NTI = no data.

Date	**Place**	**Tsunami Information**
24.08.358	Izmit Gulf, Iznik, Istanbul	NTI
11.10.368	Iznik and its surroundings	NTI
01.04.407	Istanbul	NTI
08.11.447	Marmara Sea, Istanbul, Izmit Gulf, Marmara Islands	i=3
26.01.450	Marmara Sea, Istanbul	i=3
26.09.488	Izmit Gulf	NTI
Winter 529	Thracian coasts of Marmara	NTI
Winter 542	West coast of Thracia	i=4
06.09.543	Kapidag Peninsula, Erdek Bandirma	NTI
15.08.553	Istanbul, Izmit Gulf	D=3000 m.
15/16.08.555	Istanbul, Izmit Gulf	NTI
14.12.557	Istanbul, Izmit Gulf	D=5000 m.
715	Istanbul, Izmit Gulf	NTI
26.10.740	Marmara Sea, Istanbul, Iznik Lake	i=3/i=4
26.19.975	Istanbul, Thracian coast of Marmara	i=3
989	Istanbul, Marmara coast	NTI
990	Istanbul, Marmara coast	NTI
02.02.1039	Istanbul, Marmara coast	NTI
23.09.1064	Iznik, Bandirma, Murefte, Istanbul	NTI
12.02.1332	Marmara Sea, Istanbul	i=3
14.10.1344	Istanbul, Marmara coast, Thracian coast, Gelibolu	i=4
10.09.1509	Istanbul, Marmara coast	i=3 ; HTm >6m.
17.07.1577	Istanbul	NTI
05.04.1646	Istanbul	i=3/i=4
15.08.1551	Istanbul	NTI
02.09.1554	Izmit Gulf, Istanbul	NTI
22.05.1766	Istanbul, Marmara coast	i=2
23.05.1829	Istanbul, Gelibolu	i=2
19.04.1878	Izmit, Istanbul, Marmara coast	i=3
10.05.1878	Izmit, Istanbul	40 people killed by tsunami
09.02.1894	Istanbul	i=3 ; HTm <6 m.
18.09.1963	Eastern Marmara, Yalova, Gemlik Gulf	HTm =1m.
17.09.1999	Izmit Gulf	i=3

lowest and the highest tsunami height ranges. In the simulation, the generated tsunami heights were transformed to the construction depth of the main breakwater by considering near-shore attenuation. Since the transformation by the linear wave theory is not valid for the relative depth of $d/L_0<0.1$ (KAMPHUIS, 1991), the cnoidal theory was utilized. IWAGAKI *et al.* (1982) developed a simple approximate expression that replaces the relatively complex calculations associated with exact cnoidal theory, as follows:

$$\frac{H}{H_0} = \left(1 + \frac{2kd}{\sinh kd}\right)\tanh kd + 0.0015\left(\frac{d}{L_0}\right)^{-2.8}\left(\frac{H_0}{L_0}\right)^{1.2} \qquad (7)$$

in which k is the wave number, H_0 and L_0 are the offshore wave height and length, respectively; d is the local (breakwater construction) depth, H and Lare the local wave height and length, respectively. The simplified cnoidal wave theory was used to find the local wave heights in front of the breakwater. The purpose of this attempt was to introduce a simple conceptual statistical model of tsunami occurrence and the probability distribution of tsunami height for Istanbul. Based on the number of occurrences of tsunamis in 1641 years, the return period of tsunami heights (for the mean values of ranges) were estimated as given in Table 5.

Climatic and geomorphologic changes in the future may alter the statistical characteristics of tsunamis in the Marmara region. Hence it is well known that modeling of tsunami in general depends on the number and accuracy of

Table 4. Tsunami height range defined for the structural risk assessment.

Intensity	Tsunami height range	Description
i = very light ii =light	H_T=0.1-1 m	Minor
iii = rather strong iv = strong	H_T=1-3 m	Moderate
v= very strong vi =disastrous	H_T=3-6 m	Major

Table 5. Return periods based on the number of occurrences of tsunamis in 1641 years.

Tsunami Height (m.)	Return Period (years)
H_T=0.5	R_p=50
H_T=2.0	R_p=100
H_T=4.0	R_p=200

Table 6. Simulation characteristics of failure function with 30 000 trials for 100 years.

{Private} Cases	Tsunami not included	Tsunami risk included
Fitted distribution of g	Gumbel	Gumbel
Distribution parameters	Mode: 1.73 Scale: 0.79	Mode: 1.74 Scale: 0.82
Average (μg m)	1.08	1.09
Variation coefficient (δ%)	366%	402%
Minimum of range	-43.82	-44.32
Maximum of range	3.62	3.65

data, time period studied and the statistical analysis techniques utilized. Although this study is based on the limited descriptive data available, results were regarded as representative for the population.

In the structural risk assessment model, the offshore wave heights and the offshore tsunami height were generated by the CEMC simulation from the extreme value probability distribution of FTT-I and the probability distribution given in Figure 2. Then the stability of the breakwater was investigated by the β_{II} method. The structural performance function $g(z_i)$ for the rubble mound breakwater (Equation 1) was simulated under design conditions using the probability distributions of the load and strength parameters for the following two cases: Case I: Tsunami risk not included and Case II: Tsunami risk included. The scatter range of the randomly generated values was between g_{min}= –43.82 m and g_{max}=3.62 m for Case I; g_{min}= –44.32 m and g_{max}=3.65 m, for Case II. This signifies the effect of uncertainties inherent in the design parameters of the limit-state functions having the simulated mean values of μg=1.08 m (Case I) and μg=1.09 m (Case II) (Table 6).

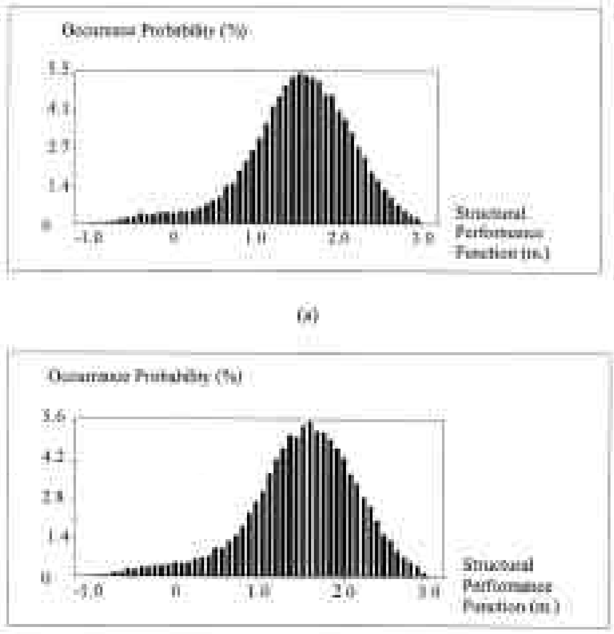

Figure 3. Frequency simulation of the structural performance function at the limit state for the duration of 100 years. (a) Case I: Tsunami risk not considered (b) Case II: The effect of tsunami risk.

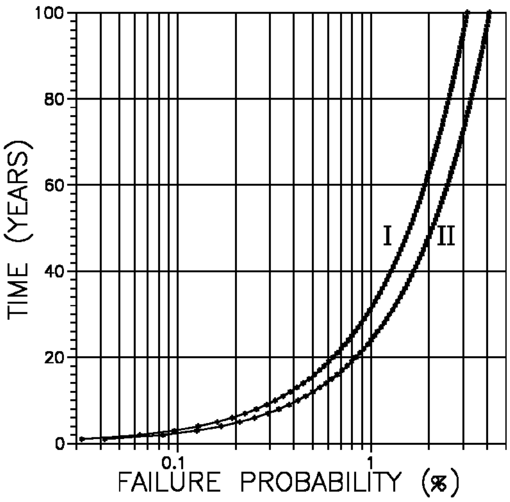

Figure 4. Failure probability of the breakwater as a function of time. I: No tsunami II: Tsunami risk included.

The CEMC simulation was carried out for 30000 times with an average CPU (Central Processing Unit) time of 2 minutes and 03 seconds (Case I) and 2 minutes and 12 seconds (Case II) with a standard mean error of _=1% by using a portable computer with an AMD K6-2+ (3-D) processor. The CPU times are relatively short for the large number of simulations performed on a portable computer. This enables the efficient utilization of the methodology in design practice.

DISCUSSION OF RESULTS

Simulation results for Case I and Case II are illustrated in Figure 3a and 3b, respectively, where the occurrence probability of structural performance function in 100 years is given. From Figure 3a, where the tsunami risk was not included (Case I), structural performance function g(zi) was safe with an annual probability of 99.97%, signifying that the failure risks will be 1.7% and 3.3% in 50 years and 100 years, respectively.

As for the Case II (Figure 3b) where the tsunami risk was included, it is seen that, the structural performance function is safe with an annual probability of 99.95% signifying the failure risks as 2.1% and 4.1% in 50 years and 100 years, respectively. It is clearly seen that including the tsunami risk increases the failure risk of the breakwater. For comparison, the failure probabilities for Cases I and II were plotted in Figure 4 as failure probability versus time in years, from which it is observed that:

1. The failure probability is relatively higher in Case II (tsunami included) than Case I (not included). Generally, inclusion of tsunami will significantly increase the risk of failure due to the near-shore attenuation. On the other hand, this affect cannot be directly observed for this case study, since the location of Haydarpasa port in Bosphorus is sheltered from effects of inland sea of Marmara (TINTI *et al*., 2000).
2. Longer the duration in years, the failure probability and the impact of tsunami risk on the failure mechanism are increased.
3. For Haydarpasa port main breakwater, tsunami was not the key design parameter when compared to storm waves therefore the difference in the failure risk was not very significant.
4. However, in places with great seismic activity, tsunami risk may be very significant depending on the occurrence probability and the magnitude of the tsunami.

CONCLUSIONS

The reliability-based structural risk assessment model provides a valuable tool in the design of coastal structures, which are characterised by large failure consequences and substantial capital expenditures. The new reliability approach suggested in this study, where the second order reliability index (β_{II}) and the Conditional Expectation Monte Carlo (CEMC) simulation were interrelated, is recommended to handle the uncertainties inherent in tsunami, storm surge and wave data. The tidal range of sites and the storm surge 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 reliability method had advantages when compared to the deterministic practice, since the random behaviour of structural performance could be estimated at the planning stages. Therefore the new reliability approach, which can be applied within few minutes of CPU time in portable computers, is recommended for the design of breakwaters. Tsunami risk should be included in the reliability-based model, since it increased the failure risk as a risk parameter in this case study. Especially in places with great seismic activity, where the magnitude of the tsunami and its occurrence probability is high, tsunami gains vital importance. Such a structural risk assessment, carried out by using reliability-based models, may be used as a successful tool in emergency preparedness and response to natural hazards, as an early risk mitigation mechanism for important coastal projects, such as nuclear power plants.

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