TSUNAMI RISK ASSESSMENT OF ESENKÖY FISHERY HARBOR BREAKWATER

A THESIS SUBMITTED TO

THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

OF

THE MIDDLE EAST TECHNICAL UNIVERSITY

BY

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

IN

THE DEPARTMENT OF CIVIL ENGINEERING

JULY 2003

Approval of the Graduate School of Natural and Applied Sciences

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ABSTRACT

TSUNAMI RISK ASSESSMENT OF ESENKÖY FISHERY HARBOR BREAKWATER

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August 2003, 50 pages

Within the scope of this thesis, a reliability based risk assessment, based on Monte Carlo simulation was used to analyse the safety levels of Esenköy Fishery Harbor main breakwater, Sea of Marmara, Turkey. In the past, in reliability-based risk assessment methodology in Turkey, the design conditions were only wave characteristics, tidal range, storm surge, wave set-up and the structural system parameters. However in this study, the **tsunami risk** which was considered as a major design parameter is included in the computations. In this study, development of a structural stability criterion in coastal engineering was suggested to achieve a common definition of reliability including the tsunami risk. The model introduced in this study is a practical technique in the reliability-based risk assessment of breakwaters subject to tsunami risk. In order to determine the occurrence probability of design condition, which is a function of storm waves, tidal range, storm surge and tsunami height, the Monte Carlo simulation, was applied. From the reliability-based risk assessment model applied to Esenköy Fishery Harbor 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 reflected. For Esenköy Fishery Harbor main breakwater, tsunami was not the key design parameter when compared to storm waves. However, in regions with great seismic activity, tsunami risk may be very noteworthy depending on the frequency and the magnitude of the tsunami.

Keywords: Tsunami, earthquake, reliability, risk, damage, simulation

ÖZ

ESENKÖY BALIKÇI BARINAĞI DALGAKIRANI TSUNAMI RISK DEĞERLENDİRMESİ

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Ağustos 2003, 50 sayfa

Bu tez kapsamında, Marmara Denizi'nde yer alan Esenköy Balıkçı Barınağı ana dalgakıranının güvenilirliğe dayalı risk değerlendirilmesi yapılmıştır. Yöntem olarak Monte Carlo Simülasyonu kullanılmıştır. Geçmişte, güvenilirliğe dayalı risk değerlendirmesi uygulamalarında tasarım öğeleri yalnızca dalga karakteristiği, gelgit seviyesi, fırtına kabarması ve dalga tırmanması ve yapısal parametrelerdi. Bu çalışmada ise, **tsunami riski** hesaplamalarda ana tasarım parametresi olarak alındı. Bu çalışmada, tsunami riskini de içeren, kıyı mühendisliğindeki geliştirilmiş yapısal sağlamlık kriteri güvenilirliğin ortak bir tanımını elde etmek için önerildi. Bu çalışmada tanıtılan model, tsunami riski altındaki dalgakıranlar için güvenilirliğe dayalı risk değerlendirilmesinde kullanışlı bir tekniktir. Tasarım koşulunun oluşma olasılığının belirlenmesinde Monte Carlo Simülasyonu kullanılmıştır. Esenköy Balıkçı Barınağı'na uygulanan pilot çalışmada tsunami riskini katmak yapının başarısız olma olasılığını artırmaktadır. Yapı ömrü arttıkça da başarısızlık koşulunda tsunami riskinin etkisi daha çok görülmektedir. Esenköy Balıkçı Barınağı ana dalgakıranında tsunami, fırtına dalgalarına nazaran, ana tasarım parametresi değildi. Ancak sismik aktivitenin yoğun olduğu bölgelerde tsunami riski, büyüklük ve sıklığına göre önem kazanabilir.

Anahtar Kelimeler: Tsunami, deprem, güvenilirlik, risk, hasar, simülasyon.

ACKNOWLEDGMENTS

I would like to thank to my supervisor Prof. Dr. Ayşen Ergin and my co-supervisor Assoc. Prof. Dr. Can Elmar Balas for their endless support, encourage and forbearance.

I am very indebted to my mother for her endless love and patience.

It was impossible to finish this research without direct or indirect contribution of my family, friends and instructors.

LIST OF SYMBOLS

θ	:	Angle of structural slope,
D_{N50}	:	Nominal diameter of the armor rock,
H_s	:	Design wave height at the toe of the structure,
Δ	:	Relative mass density of the stone,
ρ_r	:	Mass density of the armor stone,
ρ_w	:	Mass density of water,
K_D	:	Dimensionless stability coefficient,
H_d^L	:	Maximum value of the design wave height within a year,
$P(H_d^L)$	:	Non-exceedance cumulative probability of the extreme wave height,
γ, η	:	Parameters of the Gumbel distribution,
σ_T	:	Standard deviation of sample data,
$\overline{T}$	:	Average of sample data,
g_i	:	Failure functions; $g \leq 0$: failure, $g > 0$: safe,
Y_1	:	Uncertainty variable,
$H_{(1/3)}$	:	Deep water significant wave height,
T	:	Wave period,
Kr	:	Coefficient of refraction,
Ks	:	Coefficient of shoaling,
H_s’	:	Wave height acting on structure, which is found by $Hs' = H_{(1/3)} Kr\ Ks$
Pf	:	Failure probability of structure in 1 year,
P_f10	:	Failure probability of structure in 10 years,
P_f25	:	Failure probability of structure in 25 years.

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CHAPTER 1

INTRODUCTION

1.1 General Descriptions

A coastal project is carried out under some special environmental conditions, which is a unique collection of multiple work tasks, aiming to provide protection from the impacts of the sea. Since this is an interdisciplinary work of unique activities, a sea-land interacted structure is made by various interconnected organizations. Each coastal project is characterized by an exclusive objective. It requires great amount of capital investment and a considerable construction period facing various environmental conditions.

Coastal structures are used in coastal defense schemes with the objective of sheltering of harbor basins and harbor entrances against waves. Other objectives include preventing shoreline erosion and flooding of the hinterland, stabilization of navigation channels at inlets, and protection of water intakes and outfalls.

The main risk factors taken into consideration before this study, are the reliability of wave climate data and the reliability of design and construction methodologies as well as the low stability of incomplete structures during the construction stage in the coastal projects. In this study, tsunami risk is included in the reliability based risk assessment of coastal structures.

Breakwaters are built to reduce wave action in an area in the lee of the structure. Wave action is reduced through a combination of reflection and dissipation of incoming wave energy. When used for harbors, breakwaters are constructed to create sufficiently calm waters for safe mooring and loading operations, handling of ships, and protection of harbor facilities. Breakwaters are also built to improve maneuvering conditions at harbor entrances and to help regulate sedimentation by directing currents and by creating areas with differing levels of wave disturbance. Rubble-mound breakwaters are the most commonly applied type of breakwaters in Turkey.

Reliability is the best quantitative measure of the integrity of a designed part, component, product, or system. Reliability is the probability that a system will perform designed-for functions without failure in specified environments for desired periods at a given confidence level (Kececioglu, 1991).

1.2 Statement of the Problem

Turkey is a well-known earthquake area. There are faults in Marmara Sea as well as faults inland. Faults in Marmara Sea and İzmit Bay are active and cause earthquakes in the area. If the neotectonic features around the gulf are examined, two different sets of faults were determined: an earlier and now mostly inactive set of faults that are responsible for the formation of the large pull-apart depression in which the Gulf of İzmit is located and a younger, second set of active faults in the gulf that pass through the former set. İzmit Bay is E–W trending pull-apart basin having a surface area of about 300 km^2 along the North Anatolian Fault Zone (NAF), in the eastern extension of the Sea of Marmara.

The excessive seismicity of this particular region can be explained by current geophysical knowledge of its structural development. The North Anatolian fault is a major fracture that transverses the Northern part of Asia Minor and marks the boundary between the Anatolian tectonic plate and the larger Eurasian continental block. The area is considered as one of the most seismically active zones of the world, because of this unstable tectonic system.

The westward propagation of the seismic ruptures along the North Anatolian Fault (NAF) during the last century has increased the probability for a next rupture located in the Marmara Sea, in the prolongation of the 1999 İzmit earthquake faulting. Historical tsunamis have already been evidenced in the Marmara Sea, between 358 and 1999 that strong earthquakes that broke submarine parts of the NAF, in the vicinity of İstanbul. In this area, the transition between a pure strike slip deformation pattern, in the east, and the Aegean extensional regime, in the west, occurs. Recent geological and geophysical data acquired in the Marmara Sea have allowed improving the location and characteristics of the emerged NAF. In this study, it is proposed to include tsunami hazard in the coastal structures in the Marmara Sea through reliability based risk assessment of rubble mound breakwaters.

To examine a system's risk of failure, the load and resistance parameters of the system and the relationship between them should be identified. Since the predictions of maximum load and actual resistance of a structure are subject to uncertainties, obtaining zero risk is impossible. Therefore, a probabilistic approach indicating the likelihood that the available resistance will adequately withstand the maximum load over the lifetime of the structure, must be utilized.

The reliability-risk assessment of Esenköy Fishery Harbor main breakwater was carried out by a Level III risk assessment technique named Monte Carlo 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.

Generally applied design practice for coastal structures is deterministic in nature and is based on the concept of predetermined load and safety margin without consideration of the involved uncertainties. The safety margin is the difference between the strength of the system and the load on the system. However, this approach does not allow the determination of the reliability of the design, and consequently it is nearly impossible to optimize or to avoid over-design of a structure. In order to overcome this problem, upper levels of sophistication where uncertainties of the involved loading and strength variables are considered, is utilized.

1.3 Scope of the Thesis

In this study, Level III reliability method is used for the risk assessment of rubble mound breakwaters for stability criteria under the effect of storm waves and tsunami. The results obtained from analyses not including tsunami risk are compared with the results including tsunami risk.

For the determination of failure or success on the Esenköy rubble mound breakwater under storm waves and tsunami, Hudson's method is used.

The main objectives are:

To determine the failure probability of Esenköy Fishery Harbor rubble mound breakwater under the action of storm waves and tsunami at the limit state by using the failure mode functions generated from Hudson's method;

To compare the results of the analysis including tsunami by the results obtained from analysis with no tsunami.

1.4 Literature survey

Conventional design practice for coastal structures is deterministic in nature and is based on the concept of a design load which should not exceed the resistance (carrying capacity) of the structure. The design load is usually defined on a probabilistic basis as a characteristic value of the load, which consists of wave characteristics, tidal range, storm surge, wave set-up and the structural system parameters. However, this selection is often made without consideration of the involved uncertainties and tsunami. In regions where the load is defined as above may lead to unexpected risk and low reliability. This is because most of the available design formulae only give the relationship between storm wave characteristics and some structural response. Examples for such studies are: Hudson and W. van der Meer rubble mound breakwater design formulae.

Reliability based studies started after the World War II triggered by low reliability and poor maintainability of the equipment used in war. Reliability based studies in coastal structures is relatively new approach.

There are studies on reliability based risk assessment of structures during construction process including wave characteristics, tidal range, storm surge, wave set-up and the structural system parameters. An example of such study is:

Pişkin (2000), investigated the construction network alternatives by considering uncertainties inherent in the construction stage and the common bias factors special to Turkey and the damage risk of the coastal structures to obtain the optimum construction durations of network alternatives and coastal project.

Also there are studies on reliability based risk assessment of structures of vertical wall breakwaters using a specific load condition. An example of such study is:

İçmeli (2001), investigated the level of reliability of vertical wall breakwaters subjected to breaking waves by considering the uncertainties in the design parameters and compared the results of reliability methods of Level I and Level II with Level III.

In addition, there are studies on modeling and simulation of tsunamis in various locations. Examples of such study are:

In thesis of Özbay (2000), the tsunami generation, propogation and coastal amplifications are investigated by applying the simulation model "two-layer", which solves the sets of non-linear long wave equations simultaneously within two interfacing layers with necessary boundary conditions at the sea bed, interface and water surface.

Haboğlu (2002), investigated the coastal amplification of tsunamis near southwestern Anatolia, in the light of the historical earthquakes and associated tsunamis obtained from catalogues. Tsunamis are modeled and simulated by “Tunami-N2” and “Two-Layer.”

Yalçıner, Alpar, Altınok, Özbay and Imamura F., (2002), investigated past documented tsunamis in Marmara Region and how they were triggered. The main purpose of this study is to determine the slope failure potential as a possible tsunamigenic source in the Sea of Marmara by utilising multibeam bathymetry, shallow and deep seismic reflection data. Scenarios were tested by using tsunami simulation model “Two-Layer”.

Also there are studies on the effects of recently occurred tsunamis. One of such study is:

Yüksel, Alpar, Yalçıner, Çevik, Özgüven and Çelikoğlu (2002), documented and analysed the effects of 17 August 1999 İzmit earthquake effects on existing marine structures and coastal areas on the basis of field observations. The tectonic setting and geotechnical properties were analysed, and soil-structure interaction problems are discussed. In order to identify the distribution of damage and serviceability of marine structures, the scales of damage and serviceability levels are determined, tabulated and discussed.

None of such studies includes tsunami risk on rubble mound breakwaters, because of the time scales of storm waves and tsunami. This study is one of the pioneering reliability based risk assessment of a rubble mound breakwater including tsunami hazard.

CHAPTER 2

RELIABILITY-BASED RISK ANALYSIS

2.1 Description of Risk and Reliability Analysis

In the assessment of coastal engineering systems, the capability of a designed system must be evaluated to respond to project requirements or to meet users' demands. However, coastal engineering systems cannot be considered without unpredictable environmental conditions and with uncertain structural behavior. A system can fail to perform its intended function for one or more reasons, such as exceedance of design wave conditions or lower performance of the structure than predicted.

Coastal engineers are interested in evaluating the chance that a system is successful over its expected lifetime. Thus, reliability is defined as the probability that, under given operating conditions, a system performs adequately over a specified period of time, and a failure is said to occur when the system is incapable of performing its intended function. The risk that a system is incapable of meeting the demand is defined as the probability of failure over the specified system lifetime under specified operating conditions. System reliability is the complementary probability of non-failure, risk = 1 - reliability of the structure.

The conventional design method in coastal engineering is to set a return period of loading events, to select the design load with a given return period, and to design a structure, with a certain margin of safety. This is the deterministic design method. Uncertainties in the magnitudes of loading on and resistance of the structure are supposed to be covered by the safety margin. Since coastal engineering systems involve multivariate formulations, often with several random variables, one must use analytical methods that provide information for functions of random variables. Probability distributions can be assigned and statements made with respect to the reliability of the system. The probability distribution given a function of a number of random variables must be determined.

2.2 Reliability Method

The selected reliability method for this study is Monte Carlo simulation. It is a Level III reliability design method, in which all the load and resistance factors are described with the respective probability density functions. The probability of failures is calculated without assumptions of normal distributions. Level III methods use full joint probabilistic descriptions of the random variables. By this way, this method handles the uncertainties in the variables like tsunami, better, compared to Level II and Level I techniques.

2.3 Monte Carlo Simulation

The term "Monte Carlo" was introduced by von Neumann and Ulam during World War II, as a code word for the secret work at Los Alamos; it was suggested by the gambling casinos at the city of Monte Carlo in Monaco. Monte Carlo method is now the most powerful and commonly used technique for analyzing complex problems (Rubinstein 1981).

When the system structure is very complex, analytical derivation of the system reliability function becomes cumbersome. A numerical evolution of the system reliability function may be performed in such cases by a simulated experiment. The method consists of generating possible states of the components according to their reliability functions, and evaluating the system state of each combination of individual states.

Monte Carlo simulation is a powerful engineering tool that can be used for the statistical analysis of uncertainty in engineering problems. It is particularly useful for complex problems in which several random variables are related through nonlinear equations. The Monte Carlo analysis can be considered as an experiment performed on a computer rather than performed in an engineering laboratory. If a system parameter is known to follow certain probability distribution, the performance of the system is studied by considering several possible values of the parameter, each following the specified probability distribution (Rao, 1992).

The essential feature of the Monte Carlo computations is that each random variable should be substituted with a corresponding set of numbers having the statistical properties of that random variable. The numbers that are substituted are called random numbers, on the grounds that they could well have been produced by chance by a suitable random process. The basic steps of this method are (Perry and Hayes, 1985):

- Assessing the range for the variables being considered, and determine the probability distribution most suited to that variable,

- Selecting a value for each variable within its specific range; this should be randomly chosen and must take into account of the probability distribution for the occurrence of the variable. This is usually achieved by generating the cumulative frequency curve for the variable and choosing a value from a random number table,

- Repeating a number of times to obtain the probability distribution of the result. The number of iterations required depends on the number of variables and the degree of confidence.

CHAPTER 3

METHODOLOGY

3.1 Stability Model for the Rubble Mound Breakwaters

In Turkey, because of economical reasons, the breakwaters are type Rubble Mound Breakwater. So, the breakwaters that exist in the vicinity of İzmit Gulf, in which the North Anatolian Fault takes place, are also rubble mound breakwaters.

The generally accepted design method of rubble mound breakwaters is Hudson’s Method (Hudson, 1953). This is why the author selected Hudson’s method for the risk analysis.

Another method that could also be used was Van der Meer Method (Van der Meer, 1988). In this method, the wave steepness is taken into consideration. When tsunami variables are inserted in the Van der Meer formula, because of the limitations of the formula, the result is always “safe.” So this method is not suitable for this type of study.

3.2 Hudson Equation

Hudson equation is an empirically derived equation. Hydraulic model studies are conducted under the attack of regular waves that generate the same wave forces on the structure for each single wave. Armor layer rocks respond this resonance type repeated wave forces fast. Hudson equation is valid for structures having the crest elevation higher than the wave run-up level, and with the armor layer slope having $\cot\alpha$ values greater than 1.5 because of the models that the experiments were done.

(Hudson, 1953) In spite of these disadvantages, Hudson equation has been verified in large and small scale model tests conducted under the attack of both regular and irregular waves. In the model tests executed by irregular waves, the significant wave height (H_s or $H_{1/3}$) which is defined as the average of highest one third of all waves, has generally been accepted as the design wave height in Hudson equation (Shore Protection Manual, 1973).

The well-known Hudson formula is:

$$H_s / \Delta D_{n50} = (K_D \cot\alpha)^{1/3} \tag{3.1}$$

where,

α	:	Angle of structural slope,
D_{N50}	:	Nominal diameter of the armor rock,
H_s	:	Design wave height at the toe of the structure,
Δ	:	Relative mass density of the stone defined by $\Delta = \frac{\rho_r}{\rho_w} - 1$
ρ_r	:	Mass density of the armor stone,
ρ_w	:	Mass density of water,
K_D	:	Dimensionless stability coefficient which signifies the degree of damage, as discussed below.

Stability of the armor layer unit depends on shape of armor units, number of units comprising the thickness of the armor layer, manner of placing armor units, surface roughness and sharpness of edges of the armor units (degree of interlocking of the armor units), type of wave attacking structure (breaking or non-breaking), part of the structure being attacked (trunk or head), and angle of incident wave attack. These characteristics are reflected in Hudson equation by the stability coefficient K_D, obtained by hydraulic model studies.

The values of the stability coefficient (K_D) are listed in Table (3.1) for the design wave height parameters of H_s and $H_{1/10}$, recommended by the 1973 and 1984 editions of Shore Protection Manual (SPM) respectively. In Table (3.l), the stability coefficients are presented by categorizing according to the type of the section and state of the wave in front of the structure.

The stability coefficients presented in Table (3.1) are valid for an armor layer having a thickness of two units of randomly placed, rough and angular quarry stones, and an impervious filter layer. The coefficient values are specified for minor overtopping of waves and a damage value less than 5%. This description is defined as "no-damage criteria." The percent damage is defined as the percentage of the armor stone volume displaced from the original cross section of the whole layer by waves.

It is obvious from Table (3.l) that, the values of stability coefficient for breaking waves attacking the trunk section has been decreased in the 1984 edition of SPM, at the same time for non-breaking waves, it remained same. For breaking condition, these modifications increase the rock weight of armor stone for the trunk section significantly. Consequently, the weight of the armor stone obtained by using the 1984 edition of SPM is considered as a conservative value in the design. In the widely used practice, the design wave height for the rubble mound structures is selected as the significant wave height.

In Table (3.2) the results of damage tests where $H_s/H_{s(D=0)}$ is a function of the percent damage D for various armor units. H_s is the wave height corresponding to 0 to 5 percent, defined as no-damage criteria.

Table (3.l) Comparison of stability coefficients (K_D) for the design wave height parameters H_s and $H_{1/10}$ (Shore Protection Manual, 1973 and 1984).

Section Type	Slope	Breaking		Non-breaking	
		H_s	$H_{1/10}$	H_s	$h_{1/10}$
Trunk	1.5-3	3.5	2.0	4.0	4.0
Head	1.5	2.9	1.9	3.2	3.2
	2.0	2.5	1.6	2.8	2.8
	3.0	2.0	1.3	2.3	2.3

Table (3.2) $H_s/H_{s(D=0)}$ as a function of cover-layer damage

	Damage (D) in Percent						
	0 to 5	5 to 10	10 to 15	15 to 20	20 to 30	30 to 40	40 to 50
$H_s/H_{s(D=0)}$	1.00	1.08	1.19	1.27	1.37	1.47	1.56

3.3 Determining The Parameters

In Level III reliability analysis, every parameter is considered to be variable and has a probability distribution.

1) Storm Wave Height: The main parameter in a design of rubble mound breakwater is storm wave height. It is a time varying quantity which is best modeled as a stochastic process. Distinction is made between short-term and long-term statistics of the wave heights. The first one deals with the distribution of the wave height H during a stationary sequence of a storm, i.e. during a period of constant H_s (or any other characteristic wave height). The short term wave height distribution

follows the Rayleigh distribution in case of deep-water waves and some truncated distribution in case of shallow-water waves.

The long term statistics deals with the distribution of the storms which are then characterized by the maximum value of H_s occurring in each storm. The storm history is given as the example (H_{s1}, H_{s2}, ... , H_{sn}) covering a period of observation. For the extreme value statistics, Gumbel (used in this study) distribution is then fitted to the sample data (Ergin and Özhan, 1986).

The long term variation of H_s within a period of L years (e.g. lifetime of the structure) can be described by assuming statistically independent storm events and using the extreme type probability distribution of Fisher-Tippett Type I (Gumbel) as:

$$P(H_d^L) = \exp\left(-\exp\left(-\frac{H_d^L - \gamma}{\eta}\right)\right) \tag{3.2}$$

where,

H_d^L : Maximum value of the design wave height within L years,

$P(H_d^L)$: Non-exceedance cumulative probability of the extreme wave height within L years,

γ, η : Parameters of the distribution, where $\eta > 0$.

Estimation of the parameters of the Gumbel distribution can be done by matching moments technique. In this technique, the distributions mean and standard deviation is equal to their corresponding sample mean and sample standard deviation.

$$\eta = 0{,}7797\ \sigma_T \tag{3.3}$$

where

σ_T : Standard deviation of sample data.

$$\gamma = \overline{T} - 0{,}577\ \eta \tag{3.4}$$

where

$\overline{T}$: Average of sample data.

2) Tsunami Elevation: Because of the fact that, 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. The available information concerning tsunamis associated with Esenköy and the eastern Marmara earthquakes have been used as the tsunami data (Altınok and Ersoy, 2000). If the documented tsunami data for İstanbul and Esenköy between the years of 358-1999 (1641 years) is examined, the number of major tsunamis is found as N=32, where the tsunami elevation exceeds 0.5 m, as descriptively listed in Table (3.3).

There are several valuable magnitude and intensity definitions, classifications and statistical approaches for the occurrence probabilities of tsunami. The first effort on the quantification of tsunami in terms of intensity scale was done by Sieberg (1927). Then a few investigators continued to put 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 (3.4) 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 (3.3), which is adopted from Altınok and Ersoy (2000). With the limited and inaccurate data, the tsunami elevation ranges were identified based on the tsunami information in Table (3.3). 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 elevation range by using Modified Sieberg Seismic Sea-Wave Intensity Scale (i) (Ambraseys, 1962) in Table (3.3). In this table, H_{Tm} is the maximum tsunami elevation (m), D is the distance that the water penetrated inland (m) and NTI designates that no tsunami information is available.

Table (3.3). Major tsunamis in Esenköy and Marmara region (Altınok and Ersoy, 2000). NTI = no tsunami information.

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

As a pioneering study, a simple statistical analyses of the classified tsunami elevations in Esenköy and the adjacent coasts of Marmara is analysed in terms of probability of occurrences of the mean values of the ranges.

The purpose of this attempt was to introduce a simple conceptual statistical model of tsunami occurrence and the probability distribution of tsunami elevation for Esenköy. Based on the number of occurrences of tsunamis in 1641 years, the return period of tsunami elevations (for the mean values of ranges) were estimated as given in Table (3.5).

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 data, time period studied and the statistical analysis techniques utilized. This study is based on the limited descriptive data available; however, results can be regarded as representative for the population.

Table (3.4) Tsunami elevation range defined for the structural risk assessment.

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

Because tsunami has very long period relative to storm waves, it causes an apparent variation in water depth over a long distance. Storm waves riding on top of the tsunami will have a wave celerity corresponding to the depth (including tsunami elevation) at any particular point. If two storm waves are otherwise equivalent (e.g., the same period and wave height), and one is at the crest of the tsunami while the other is at the leading edge, the storm wave at the tsunami crest will have a higher celerity. Therefore, the tsunami can cause one storm wave to overtake and superimpose itself on another storm wave, producing higher waves at the shoreline.

Table (3.5) Return periods based on the number of occurrences of tsunamis in 1641 years.

Tsunami Elevation (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

As a first approximation, the tsunami run-up on a structure will have a run-up height (vertical rise) equal to the wave elevation at the shoreline. This assumption is based on the idea that a tsunami will act like a rapidly rising tide. This assumption cannot always be used with accuracy. The effects of ground slope, wave period, and the possible convergence or divergence of the run-up must be considered. Since the breakwater was constructed at a depth of 6 m, tsunami elevation is taken as defined in Figure (3.1) (Farreras, 2000).

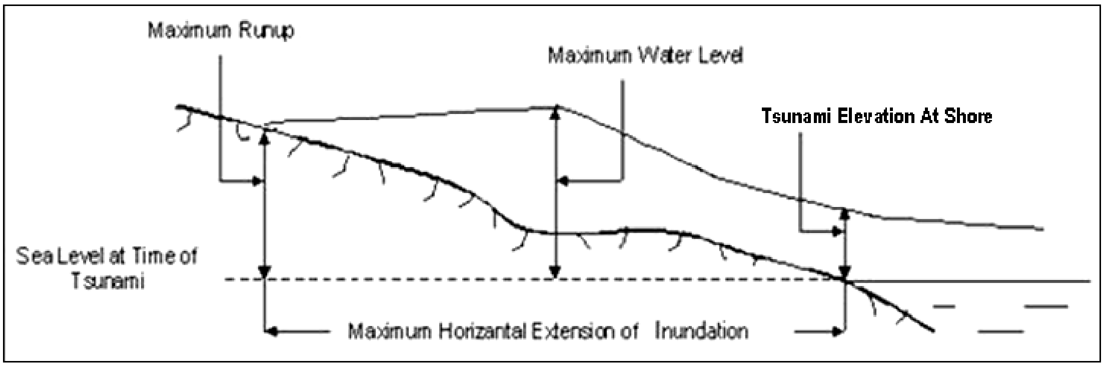

Figure (3.1). Maximum horizontal extension of the inundation produced by the tsunami (Farreras, 2000).

Tsunamis at a shoreline could be categorized into three types of waves: non-breaking waves (i.e., a tsunami which acts as a rapidly rising tide); waves which break far from the shoreline and become fully developed bores before reaching the shoreline; and waves which break near the shoreline and act as partially developed bores which are not uniform in height. In addition, there are some cases where reflected waves become bores after reflecting from a shoreline. Except in rare cases, most tsunamis appear as non-breaking waves. For the non-breaking wave, the assumption that the run-up height equals the wave elevation (crest amplitude) at the structure is reasonable. For our case study, because of the fact that the distance between potential tsunami generating earthquake epicenter and structure is short compared to tsunami wavelength, this situation is applied.

To analyze the run-up of breaking waves and fully developed bores, where maximum run-up heights have been observed to be much higher than the wave or bore elevation at the shoreline, it is necessary to consider the actual form of the run-up. Several experimental studies have been performed for flat, uniform slopes with no convergence of the wave crest. In general, the experiments show that for flatter slopes (less than 8 deg) the run-up height appears equal to or less than the wave elevation at the shoreline. For steeper slopes, the run-up height increases as the slope increases, and the ratio of run-up height to wave elevation at the shoreline appears to reach a maximum value for vertical walls. However, the higher run-up on the steeper slopes appears to have a relatively shallow depth.

3.4 Limit-State Equation For Armor Layer

The failure function can be obtained from Hudson equation (Equation (3.1) in Section 3.2). In the presented reliability model, Hudson failure function as a function of its basic variables: $g_1=f(H_d,Y_1,D_{n50},K_D,\Delta,\theta)$ is obtained as follows:

$$G_1 = Y_1\, \Delta\, D_{n50}\, (K_D \cot(\theta))^{1/3} - H_d \qquad (3.4)$$

where, in general representation:

g_1 : Failure functions; $g \leq 0$: failure, $g > 0$: safe.

Y_1 : Uncertainty variables signifying the uncertainty of the equation.

CHAPTER 4

APPLICATION OF RELIABILITY ANALYSIS

4.1 Application of Monte Carlo Simulation

The reliability analysis presented in previous chapters is applied to a case study in Esenköy Fishery Harbor. The harbor is located at the entrance of İzmit Bay, Sea of Marmara, Turkey. The layout of the area concerned is given in Figure (4.2).

The breakwater of Esenköy is a rubble mound breakwater. The length of the main breakwater is 538 m. The enclosed area is 2.05 hectares. In this chapter, the probable damage of this breakwater for two cases: Case 1: No tsunami risk, Case 2: Tsunami risk included, are investigated. The methodology presented in Chapter 3 is used. The location of Esenköy Fishery Harbor and fetch directions are shown on Figure (4.1). Layout of coastal structures along the coastline of the İzmit Bay is shown on Figure (4.3).

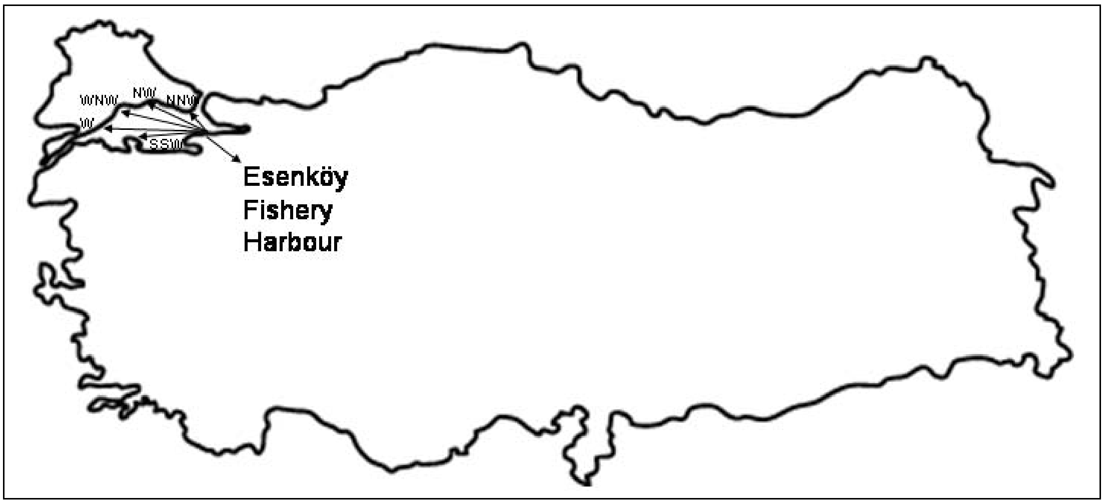

Figure (4.1) Location and fetch directions of Esenköy Fishery Harbor.

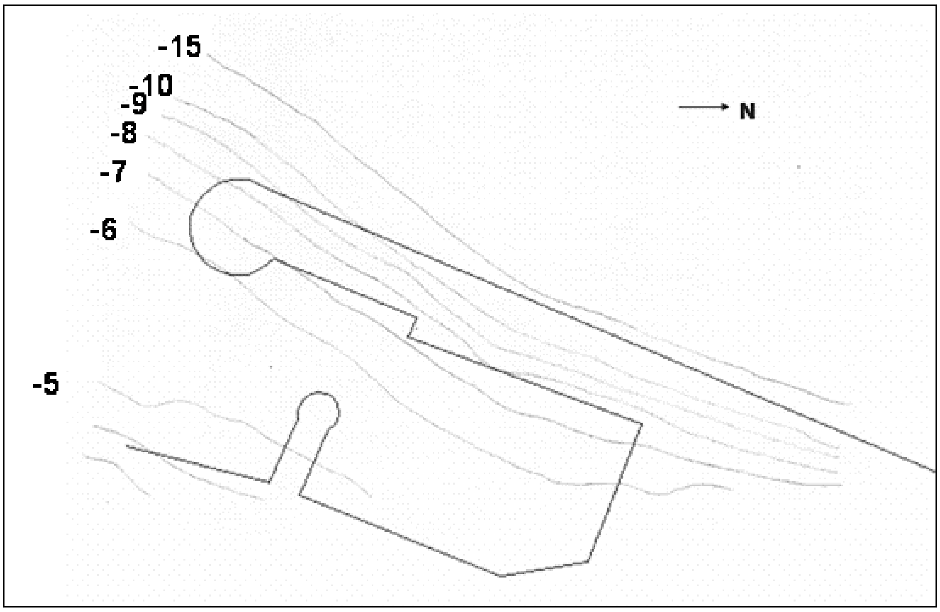

Figure (4.2) Layout of Esenköy Fishery Harbor (Not in scale).

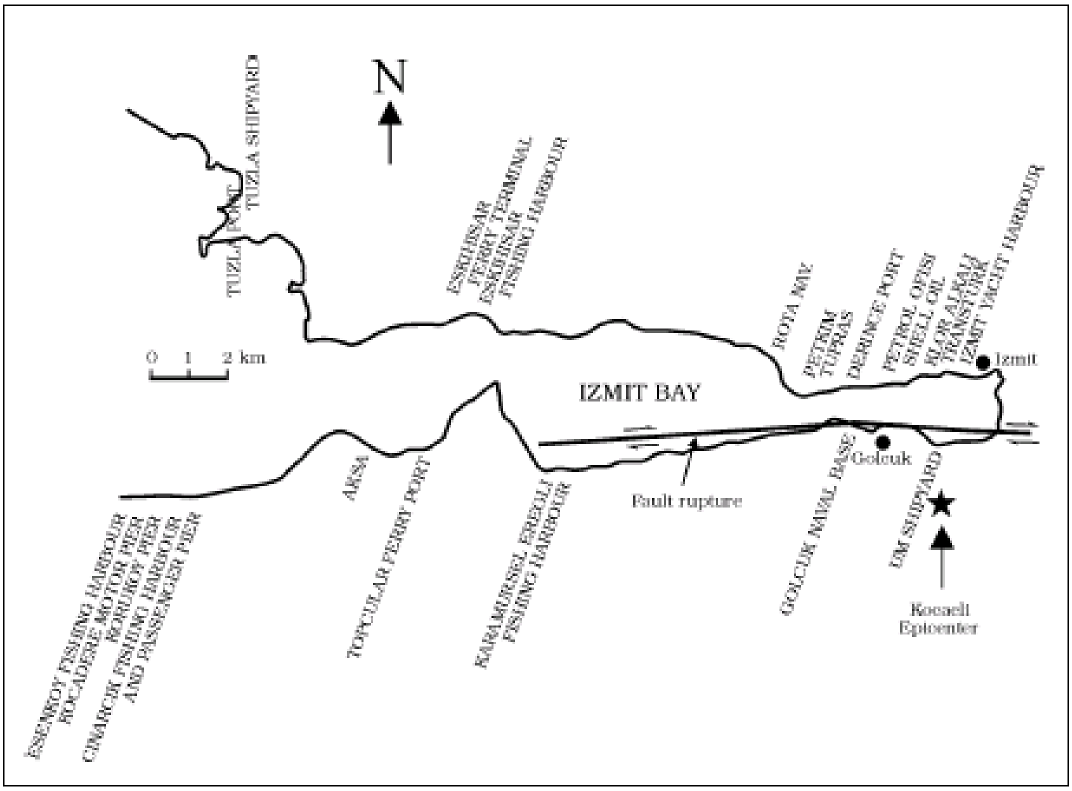

Figure (4.3) Layout of coastal structures along the coastline of the İzmit Bay.

4.2 Reliability Analysis For Both Cases: No Tsunami Risk And Tsunami Risk Included

The rubble mound breakwater is constructed at a water depth of 7 meters under non-breaking wave attack. The values used in determination of coefficient of refraction, Kr and coefficient of shoaling, Ks are tabulated in Table (4.1). The wave characteristics of Esenköy are used (Ergin and Özhan, 1986). These wave characteristics are tabulated in Table (4.2).

Table (4.1) Variables used for determination of coefficient of refraction and shoaling

Angle between the direction of wave approach and a line normal to breakwater	22,5°
m, sea bottom slope	1/30
d, breakwater depth	6 m

Table (4.2) Deep water extreme value statistics (Ergin and Özhan, 1986)

M	$H_{(1/3)}$ (m)	T (sec)	Direction	KrKs	Hs' (m)
1	1,37	4,26	SW	0,89	1,22
2	1,51	4,47	SW	0,89	1,34
3	1,78	4,85	SW	0,90	1,60
4	1,78	4,85	SW	0,90	1,60
5	1,78	4,85	SW	0,90	1,60
6	1,84	4,93	SW	0,90	1,66
7	2,14	5,32	WSW	0,91	1,95
8	2,19	5,38	WSW	0,91	1,99
9	2,26	5,47	SW	0,91	2,06

Where,

$H_{(1/3)}$: deep water significant wave height,

T : wave period,

Kr : coefficient of refraction,

Ks : coefficient of shoaling,

Hs' : wave height acting on structure, which are found by $Hs' = H_{(1/3)}\,Kr\,Ks$

Using the values given in Table (4.2), the extreme value statistics parameters are found from Equations (3.3) and (3.4) as:

$\overline{T}$ = 1,67 m,

σ_T = 0,296 ,

γ = 1,54 ,

η = 0,223.

Then, the probability density function of storm waves is obtained as follows,

$$P(H_d^L) = \exp\left(-\exp\left(-\frac{H_s - 1{,}54}{0{,}223}\right)\right) \tag{4.1}$$

The cross-section of head and trunk sections are shown in Figure (4.4) and (4.5) respectively. The input parameters used for the analysis are tabulated in Table (4.3) and Table (4.4)

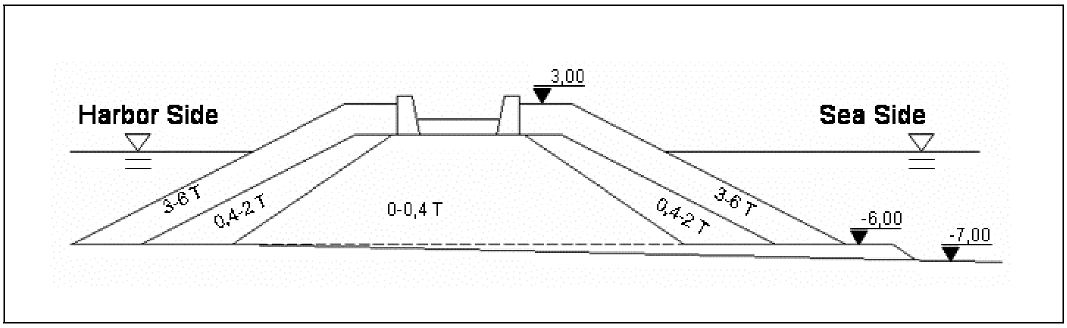

Figure (4.4) Cross-section of the head section.

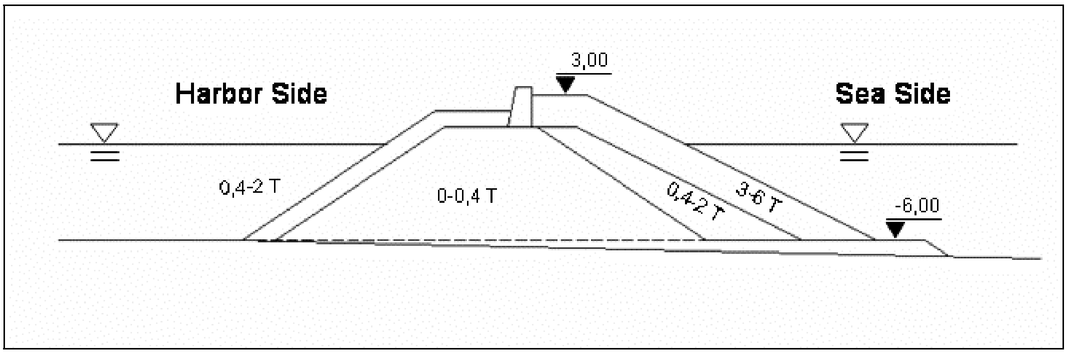

Figure (4.5) Cross-section of the trunk section.

Table (4.3) Representative values for Monte Carlo Simulation.

Surf Beat (m)	0,1
Wave Setup (m)	0,05
K_D	2,8
Δ	1,6
D_{N50} (m)	1,2
H_s (m)	1,67
COTθ	2
Y_1	1
Tide (m)	0
Tsunami Wave Elevation (m)	1

Table (4.4) Random variables for Monte Carlo analysis.

Random Variable	Distribution Type	Mean	Standard Deviation	Mode	Alpha	Beta	Scale	Range
Δ	Normal	1,63	0,15	-	-	-	-	1,53-1,73
D_{N50}	Beta	-	-	-	3,00	1,70	1,60	1,00-1,6
H_s	Gumbel	-	-	1,54	-	-	0,22	1,1-3
Cotθ	Beta	-	-	-	6,00	1,60	2,30	1,63-2,83
Y_1	Beta	-	-	-	3,00	2,00	1,50	0,8-1,30
Tide	Uniform	-	-	-	-	-	-	-0,2-0,2
Tsunami	Non-continuous	-	0,2	-	-	-	-	0-4
Wave Setup	Gumbel	-	-	0,05	-	-	0,30	0,00-0,10
Surf Beat	Gumbel	-	-	0,10	-	-	0,30	0,00-0,15

The type and parameters of distributions are taken from (Ergin and Balas, 2002) to be consistent with the series of studies on this subject at Marmara Sea.

The results of analysis of tsunami information yielded the results given in Figure (4.7). In this figure, the regression line, presenting the statistical characteristic of the offshore mean values, was provided with a certain confidence limit indicating the lowest and the highest tsunami elevation ranges.

Computer simulations repetitively reproduced breakwater performance at the limit state condition until the specified standard mean error of convergence of 0.1%

was satisfied. The simulation is executed for 30.000 iterations and a probability for the limit state is obtained. It can be observed from Figure (4.6) that number of iterations of 30.000 is optimum.

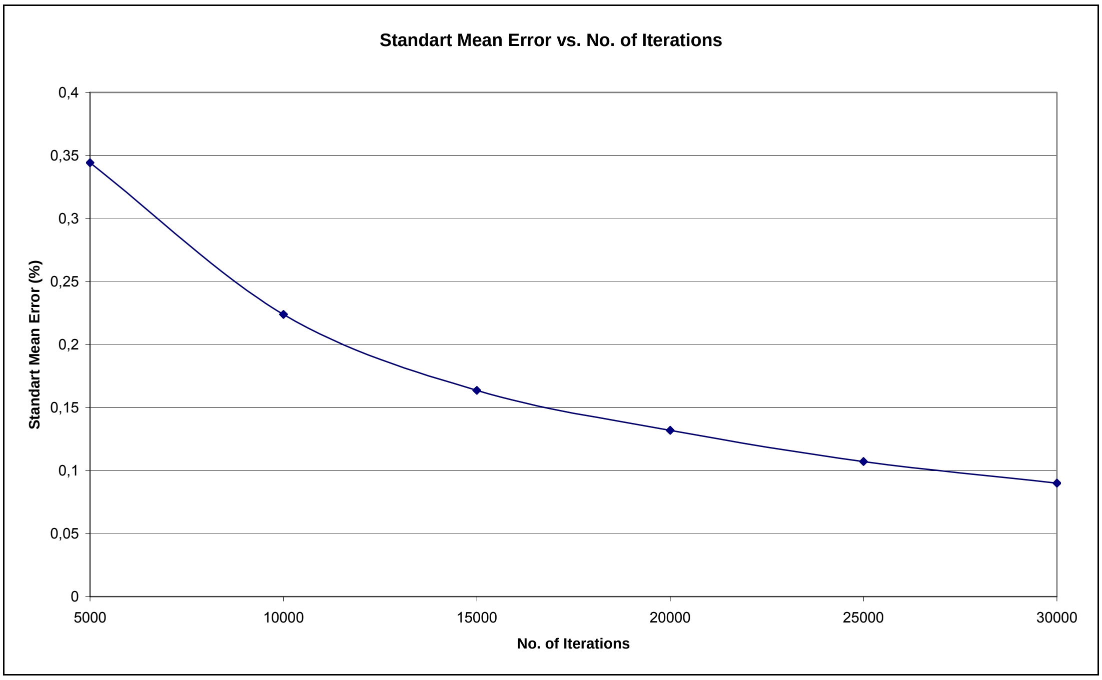

Figure (4.6) Standart mean error of convergence versus number of iterations.

Each iteration represents the structures 1 year performance. This process is repeated for different damage levels and two cases; Case 1: No tsunami, Case 2: Tsunami risk included. In Appendix A, a simulation input and output of a run is given as an example.

The results of the Monte Carlo simulation for Case 1: No tsunami, are shown in the Figures (4.8) – (4.11).

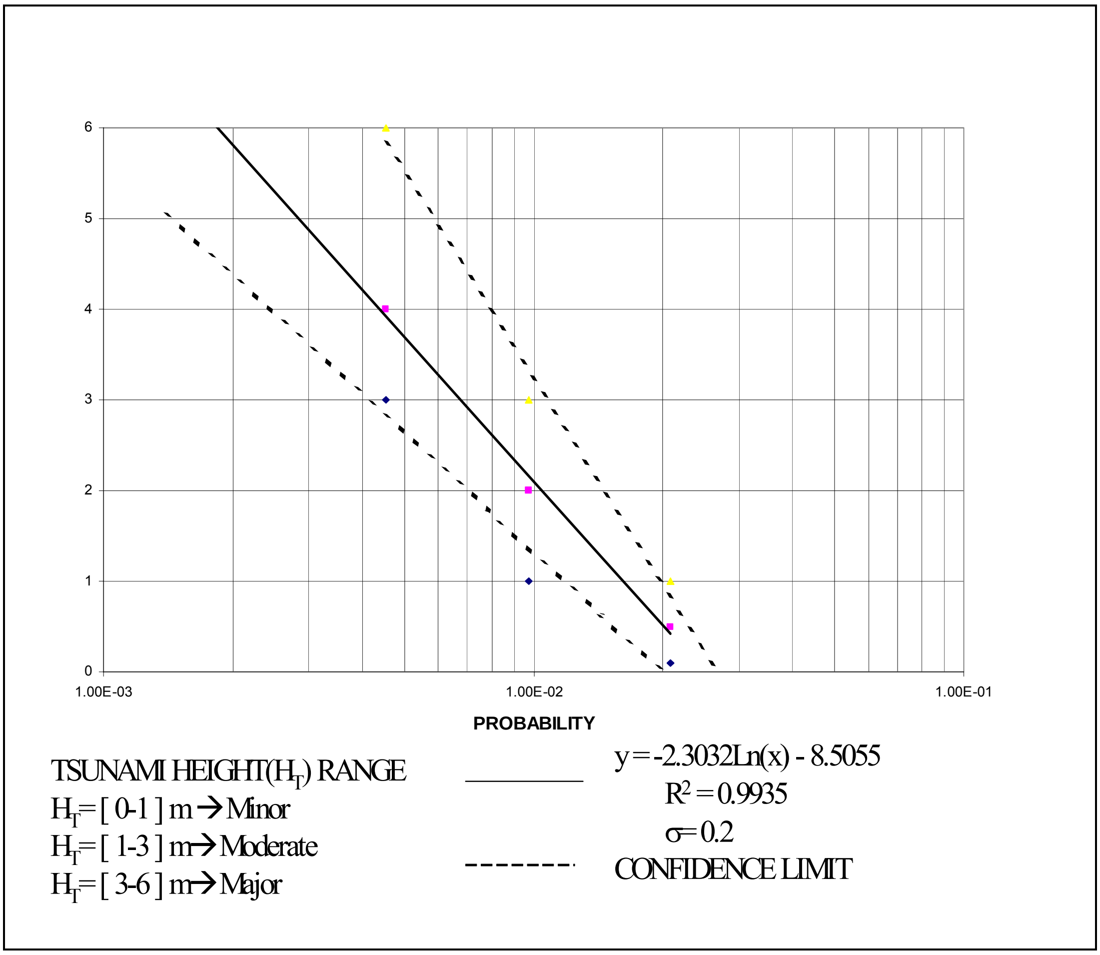

Figure (4.7). Probability distribution of tsunami elevations in Esenköy and the adjacent coasts of Marmara in terms of probability of occurrences of the mean values (Ergin and Balas, 2002).

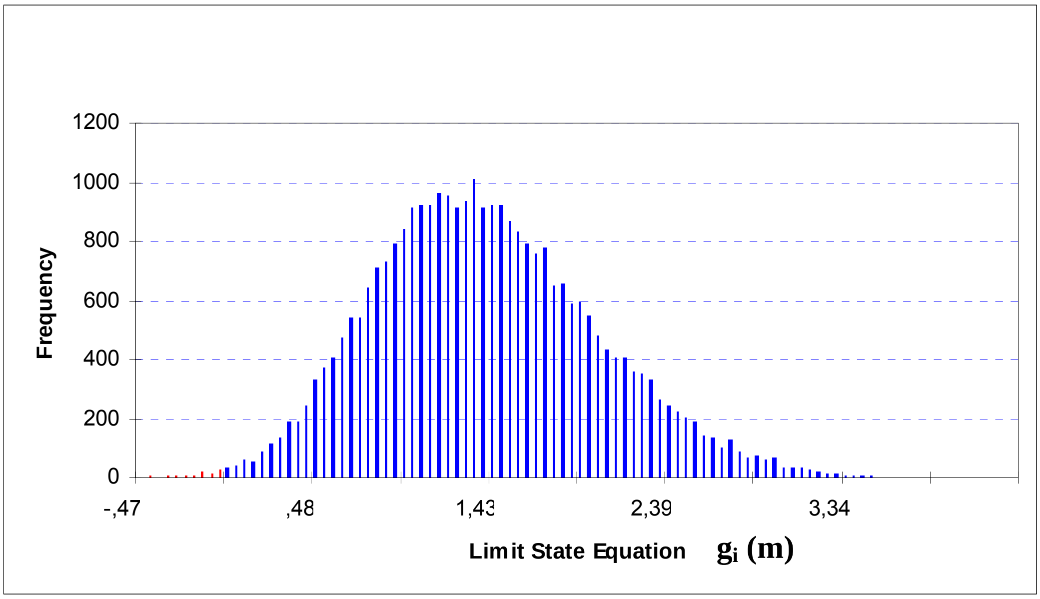

Figure (4.8) Frequency distribution for Case 1: No tsunami, Damage Level 0-5%

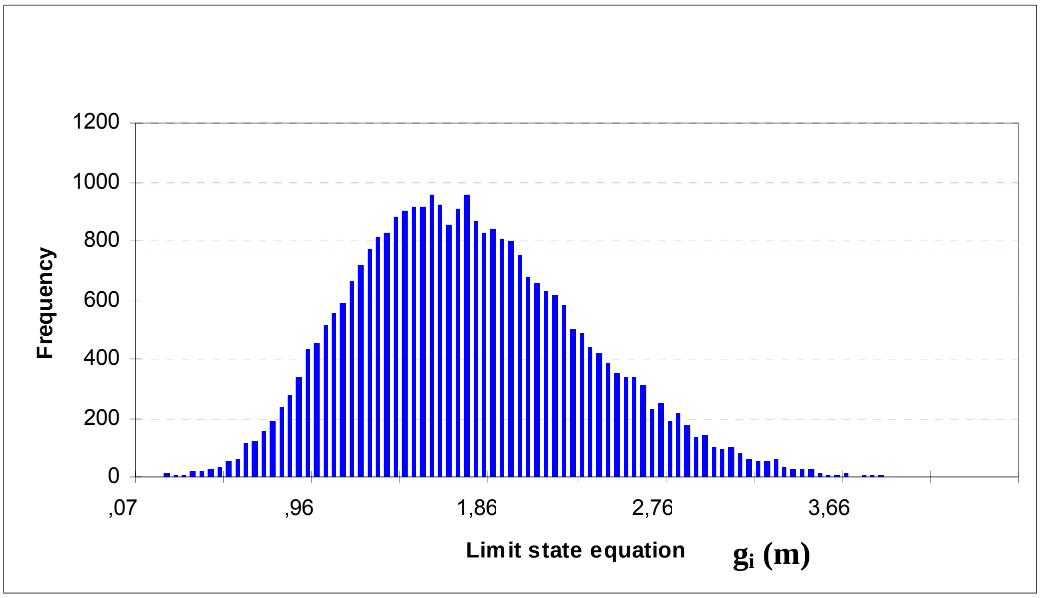

Figure (4.9) Frequency distribution for Case 1: No tsunami, Damage Level 20-30%

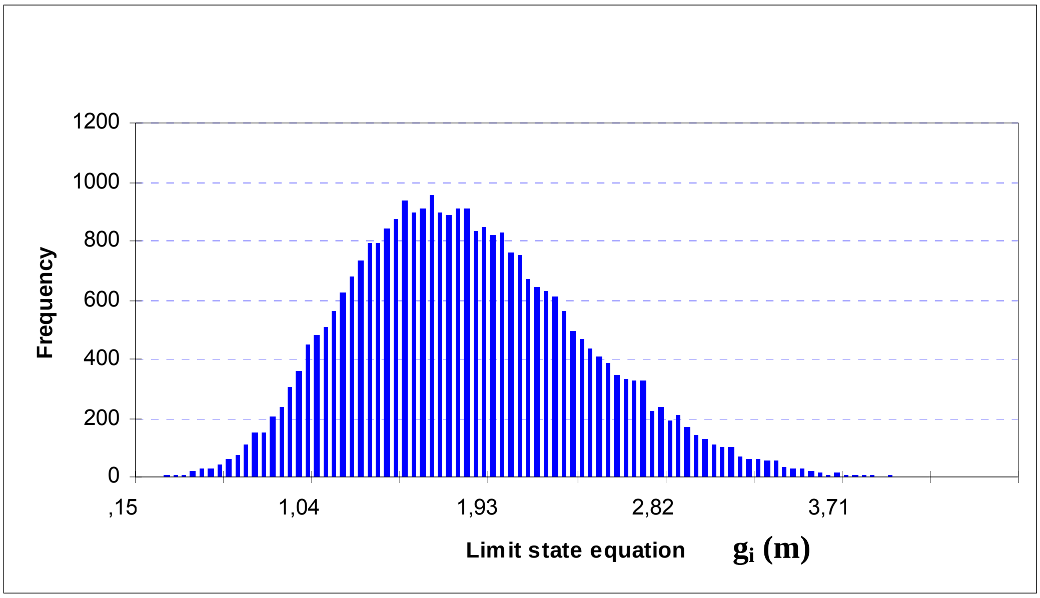

Figure (4.10) Frequency distribution for Case 1: No tsunami, Damage Level 30-40%

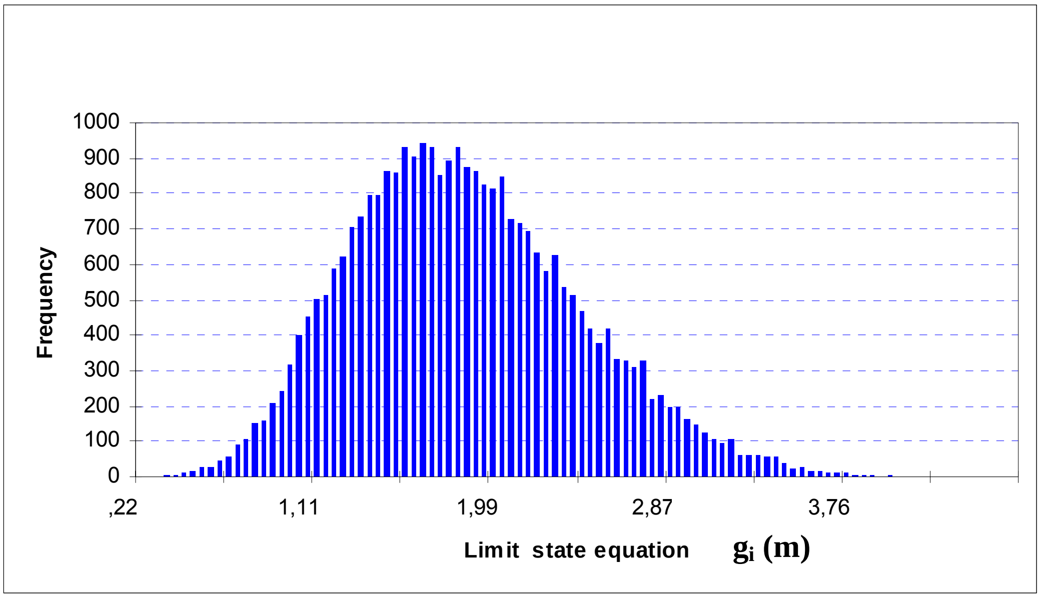

Figure (4.11) Frequency distribution for Case 1: No tsunami, Damage Level 40-50%

The results of the Monte Carlo simulation for Case 2: Tsunami risk included are shown in the Figures (4.12) – (4.15)

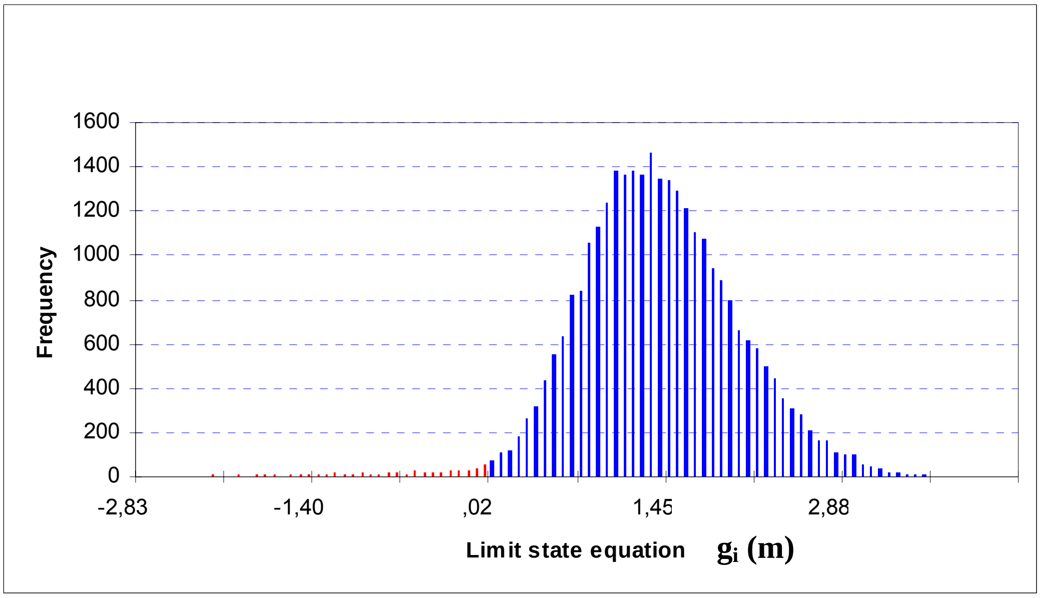

Figure (4.12) Frequency distribution for Case 2: Tsunami risk included, Damage Level 0-5%

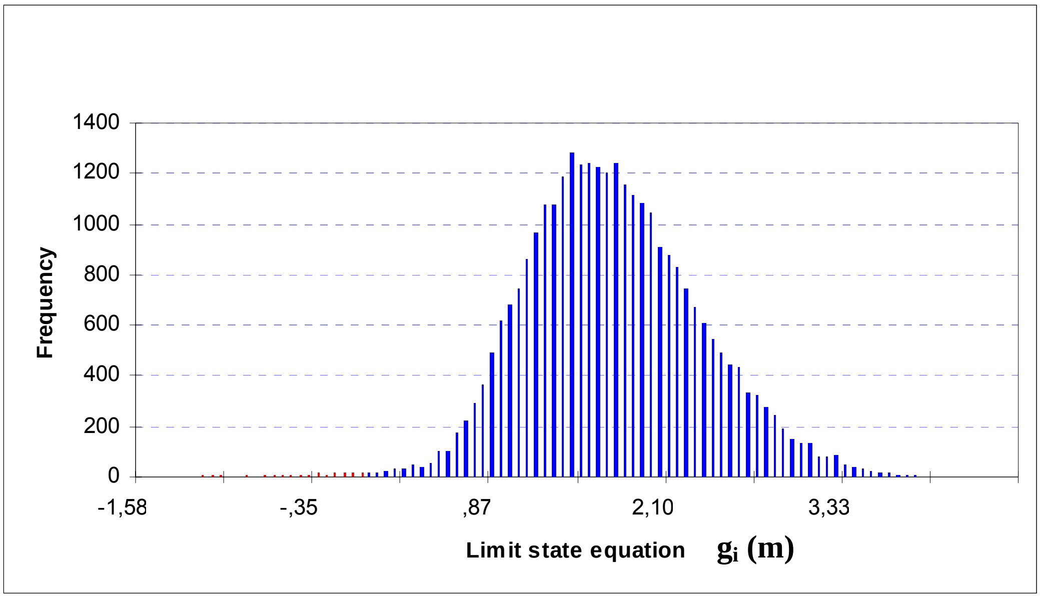

Figure (4.13) Frequency distribution for Case 2: Tsunami risk included, Damage Level 20-30%

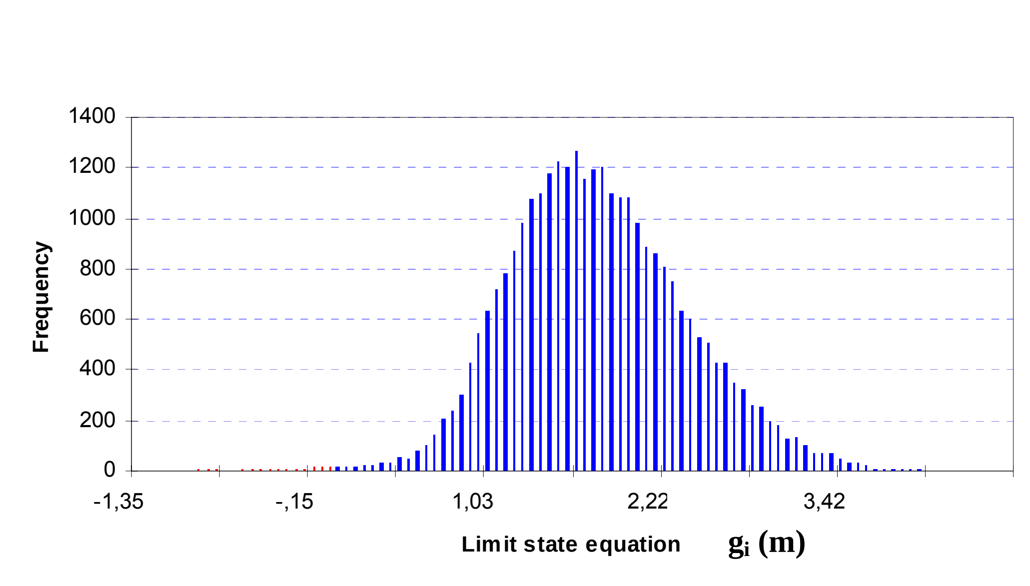

Figure (4.14) Frequency distribution for Case 2: Tsunami risk included, Damage Level 30-40%

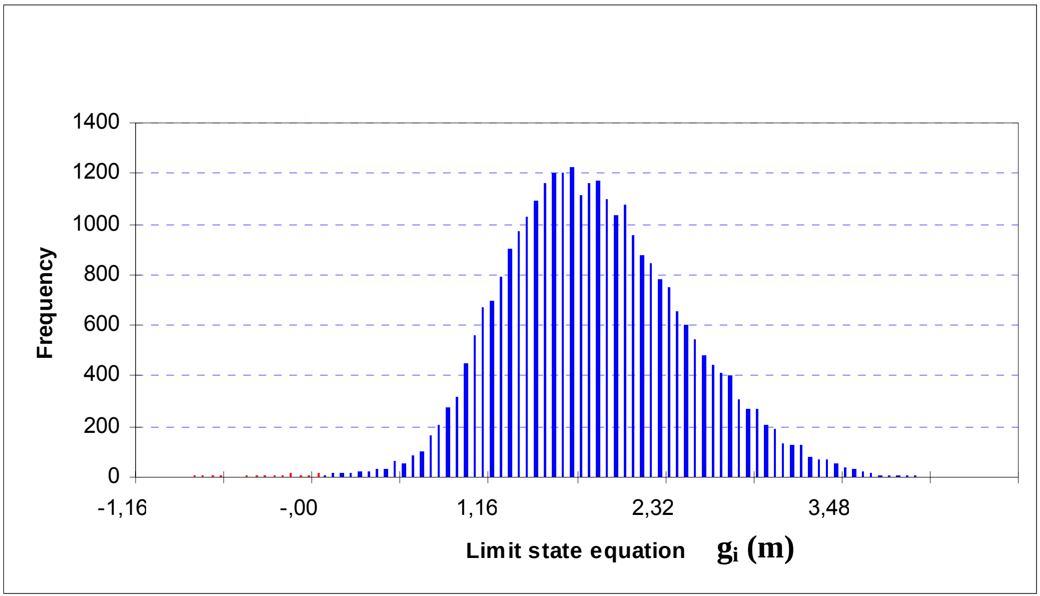

Figure (4.15) Frequency distribution for Case 2: Tsunami risk included, Damage Level 40-50%

The results obtained from Monte Carlo Simulation for Case 1 and 2, and corresponding damage level probabilities per year are tabulated in Table (4.5).

Table (4.5) Results of Monte Carlo Analysis.

	Tsunami risk included			No tsunami risk		
Damage Level (%)	P_f (%)	P_f10 (%)	P_f25 (%)	P_f (%)	P_f10 (%)	P_f25 (%)
0-5	1,68	15,59	34,53	0,35	3,45	8,39
5-10	1,28	12,09	27,53	0,15	1,49	3,68
10-15	0,93	8,92	20,83	0,05	0,50	1,24
15-20	0,74	7,16	16,95	0,01	0,10	0,25
20-30	0,49	4,79	11,56	0,00	0,00	0,00
30-40	0,43	4,22	10,21	0,00	0,00	0,00
40-50	0,33	3,25	7,93	0,00	0,00	0,00

Where,

P_f : failure probability of structure in 1 year,

P_f10 : failure probability of structure in 10 years,

P_f25 : failure probability of structure in 25 years.

The results of Monte Carlo simulation yielded the probabilities of failure in 1 year. The failure probabilities of structure in more than 1 year are found by the equation $P_f\, n = 1- (1- P_f)^n$, where n represents the number of years in which the maximum probability of failure is computed. It is assumed that for each year, structure performance is statistically independent and the simulation results are valid.

For each damage level, the probability of failure was found for Case 1 and 2 are shown on figures (4.16-4.19).

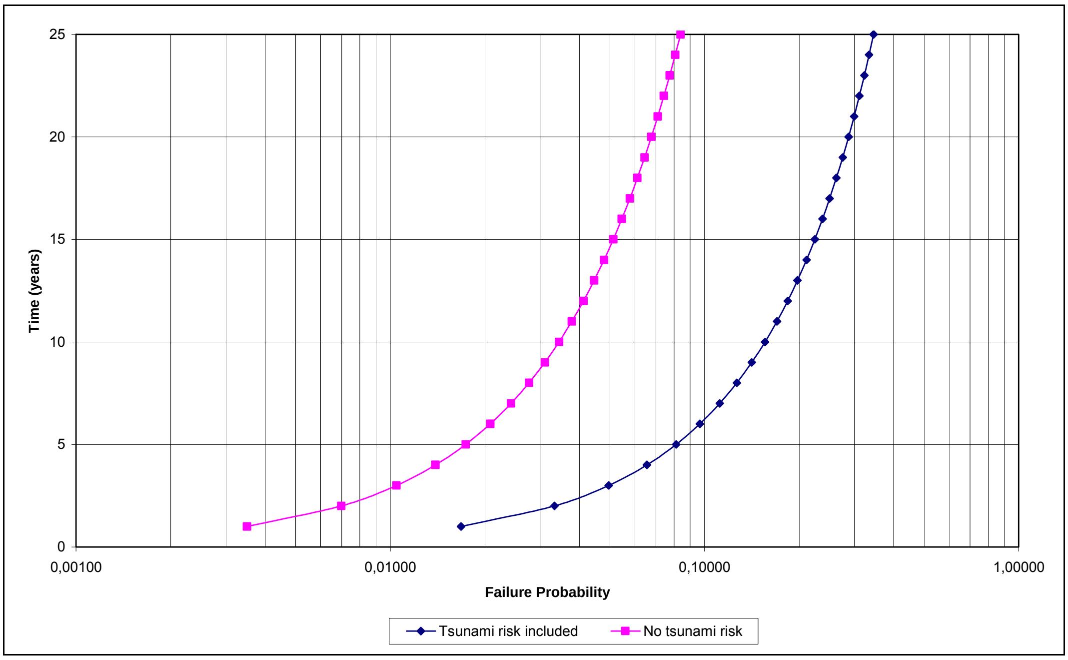

Figure (4.16) Probability of failure in 25 year period with damage level 0-5%.

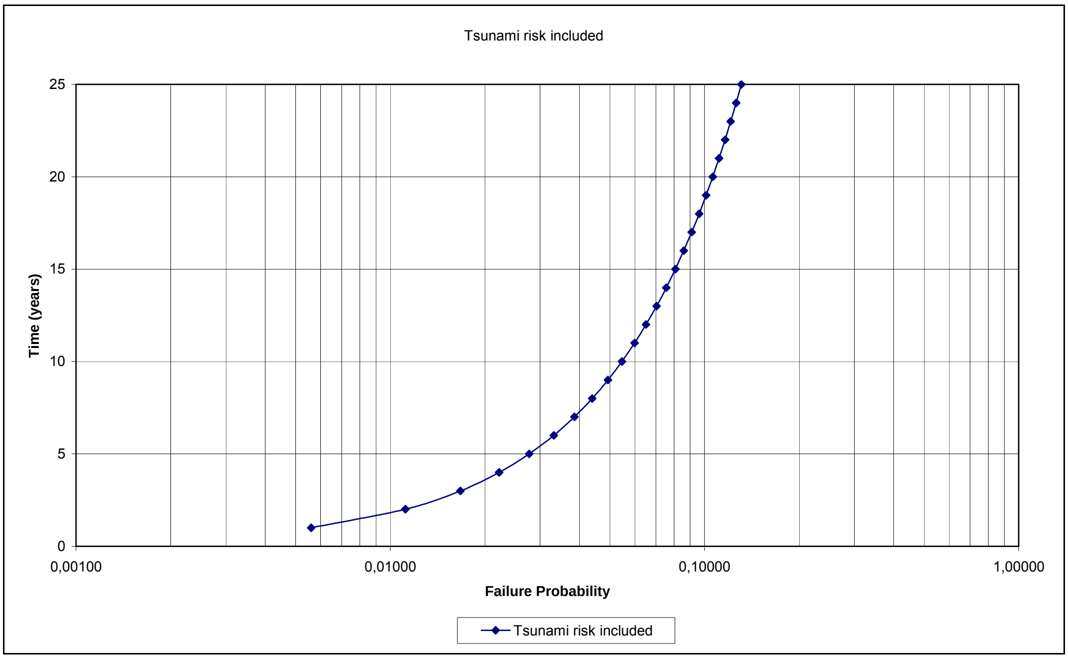

Figure (4.17) Probability of failure in 25 year period with damage level 20-30%.

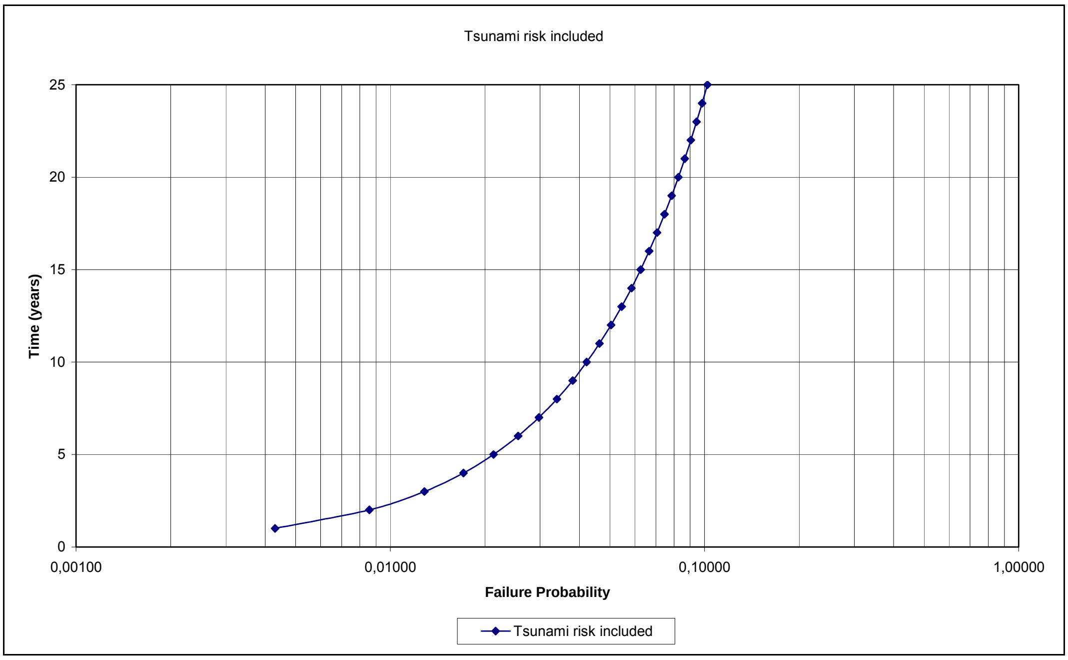

Figure (4.18) Probability of failure in 25 year period with damage level 30-40%.

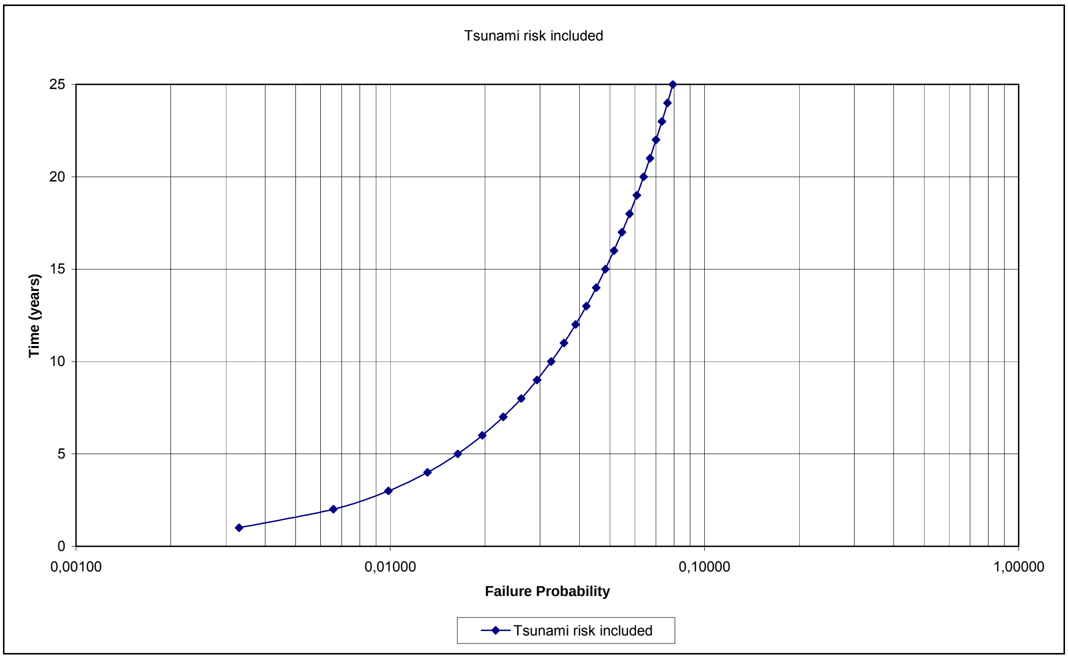

Figure (4.19) Probability of failure in 25 year period with damage level 40-50%.

Contribution of variables to the limit state is presented as pie chart in figures (4.20) and (4.21).

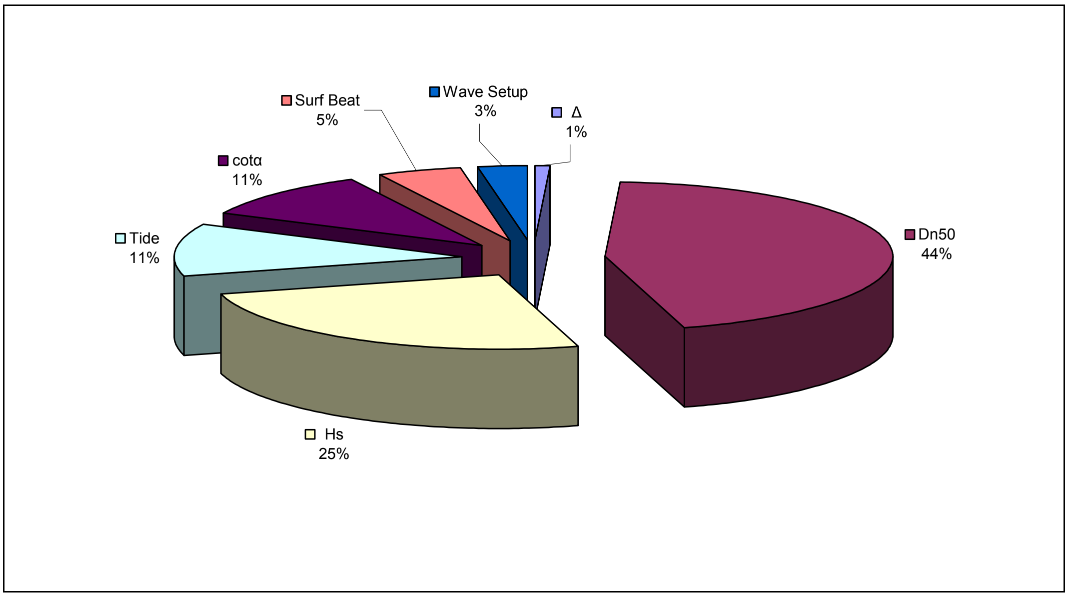

Figure (4.20) Contribution of parameters analysis for Case 1: No tsunami risk.

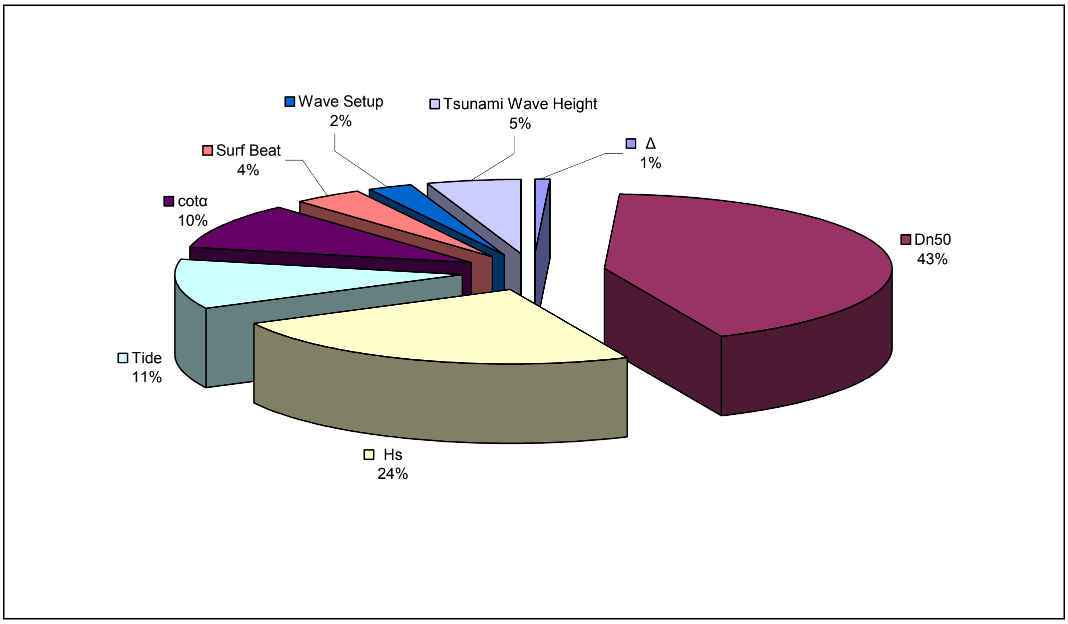

Figure (4.21) Contribution of parameters analysis for Case 2: Tsunami risk included.

4.3 Discussion of Results

The probable failure of a rubble mound breakwater in two cases (Case 1: No tsunami risk, Case 2: Tsunami risk included) is examined by the application of Monte Carlo Simulation.

1) Failure probability is higher when tsunami risk is included.

2) If maximum damage level (20-30%) for a rubble mound breakwater is considered, in 25 and 50 years, the probability that the breakwater will be unable to perform its functions is 11.56% and 21.78% respectively from Table (4.5). Cost-damage based feasibility work, can be applied for this particular case study.

3) The probability that of storm waves damaging the breakwater (exceeding 20-30%) was obtained almost zero.

4) From the contribution to the limit state function of design parameters, the influence of most important parameters in case of no tsunami risk, are as follows;

D_{N50} : 44%,

H_s : 25% from Figure (4.20)

This signifies the importance of the construction methodology and stone size used in accordance with the design.

5) The CPU time of the Monte Carlo Simulation for all 14 cases is 9 minutes, and 59 seconds with an Intel Celeron central processing unit. Since the simulation time is very short, it can be very effectively used during design process.

6) The risk level of the breakwater without tsunami depends mainly on the storm wave height. When tsunami exists, the risk level of the breakwater depends

on both parameters, because the order of magnitude of both parameters are the same, even, their time scales are different.

7) In this study, based on 1999 İzmit earthquake, the site of the Esenköy Fishery Harbor is least affected by the tsunami, generated in İzmit Bay. Because it was observed that the breakwater was not damaged and it stayed stable with tsunami.

This is reflected in our work by seeing that the probability of failure (damage level of 20-30%) is 11.56% as shown in Table (4.5). This probability decreased to 8% in case of total failure.

CHAPTER 5

CONCLUSIONS

This study was an application of a reliability based risk analysis of a rubble mound breakwater including tsunami risk. A statistical analysis is presented where probability distribution and magnitude of tsunami is included. This simulation application can be used for both determination of risk including tsunami effect for existing structures and design of new rubble mound breakwaters.

In order to include uncertainties in structural parameters, tsunami and storm wave characteristics, and time scale differences of tsunami and storm waves, a reliability based simulation was utilized; and the following results were obtained:

1) In this case study, tsunami risk increased the failure probability as a risk parameter, so it should be included in the reliability-based model. In regions with great seismic activity, where the magnitude of the tsunami and its occurrence frequency is high, it is expected that tsunami risk increases the risk noteworthy.

2) Using a deterministic design process to handle both storm and tsunami waves effect, will possibly yield an over-design, due to the time scale difference in the occurrence of the maximum probable values of tsunami and storm waves.

3) The application of Monte Carlo simulation is performed within few minutes of CPU times in computers having a Celeron type processor. This creates the opportunity to optimize risk and investment during the lifetime of structure and helps us to take precautions for emergency cases.

4) Recommendations for further studies on this subject can be the application of the reliability analysis on a limit state equation including period effect, and the analysis of harbor as a full system and investigating failure modes of all the structural elements.

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APPENDIX A

EXAMPLE INPUT AND OUTPUT FILE

A.1 Input File

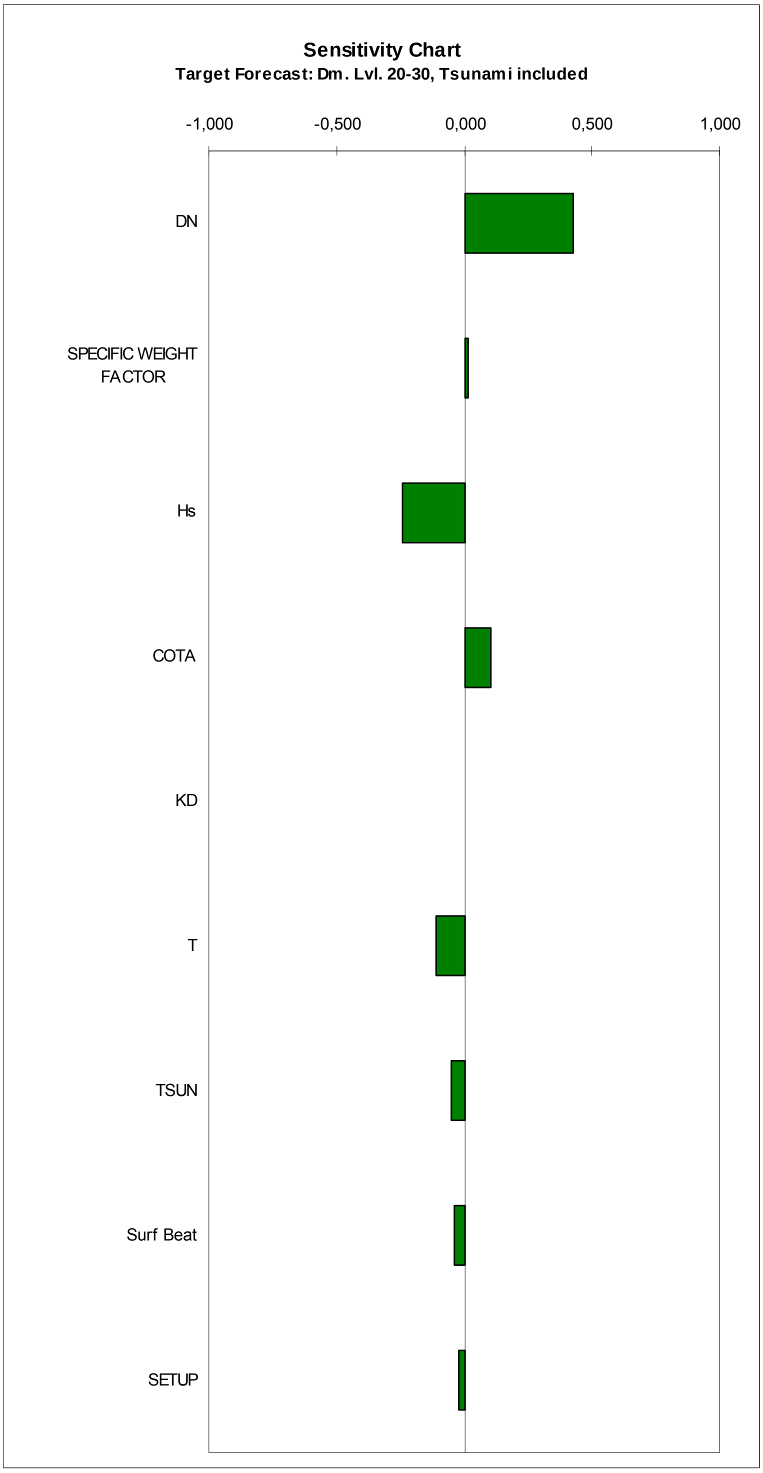
The program used for Monte Carlo Simulation was Crystal Ball 2000**.** It is a spreadsheet risk analysis program which is an Add-In of Microsoft Excel. Each parameter of simulation is defined in a cell with a distribution type and parameters of that distribution, named "Assumption." Parameters of distribution depends on the type of distribution. Then, the desired value of (in our case Limit State Equation) parameter is defined in terms of defined parameters. The program then uses the defined range in the simulation. In addition, the program keeps track of the results of each scenario. Finally, the report of simulation is generated. At the first glance, the distribution parameters cannot be seen on the input file, but are presented on report.

A-1	B	C	D	E	F
2	Esenköy Fishery Harbor				
3			Surf Beat (m)	0,1	
4			Wave Setup (m)	0,05	
5			KD	2,8	
6			DELTA	1,63	
7			DN (m)	1,2	
8			Hs (m)	1,54	
9			COTA	2	
10			Y1	1	
11			Tide (m)	0	
12			Damage level	1,37	
13			TSUN (m)	1	
14	%20-30	damage level	G (m)	1,51	=D9*D5*D6*(D4*D8)^(1/3)-(D8+D11+D3+D4+D13)/D12
15	Tsunami included				

Figure (A.1) Input of Monte Carlo simulation program Case 2: Tsunami risk included, Damage Level: 20-30%.

A.2 Output File

Crystal Ball Report



Forecast: Dm. Lvl. 20-30, Tsunami included

Summary:

Certainty Level is 99,51%
Certainty Range is from 0,00 to +Infinity (m)
Display Range is from -1,69 to 4,31 (m)
Entire Range is from -1,69 to 4,31 (m)
After 30.000 Trials, the Std. Error of the Mean is 0,00

Statistics:	Value
Trials	30000
Mean	1,72
Median	1,68
Mode	---
Standard Deviation	0,61
Variance	0,37
Skewness	0,10
Kurtosis	3,68
Coeff. of Variability	0,35
Range Minimum	-1,69
Range Maximum	4,31
Range Width	5,99
Mean Std. Error	0,00

Forecast: Dm. Lvl. 20-30, Tsunami included (cont'd)

Percentiles:

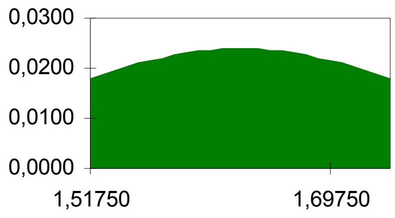
Percentile	(m)
0%	-1,69
10%	1,00
20%	1,22
30%	1,39
40%	1,54
50%	1,68
60%	1,84
70%	2,01
80%	2,22
90%	2,53
100%	4,31

Assumption: SPECIFIC WEIGHT FACTOR

Normal distribution with parameters:

Mean	1,63
Standard Dev.	0,15

Selected range is from -Infinity to +Infinity

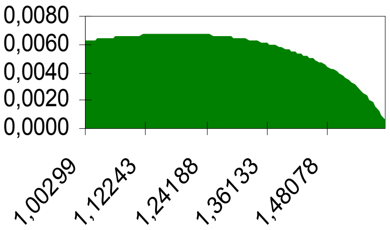


Assumption: DN

Beta distribution with parameters:

Alpha	3,00
Beta	1,70
Scale	1,60

Selected range is from 1,00 to 4,60

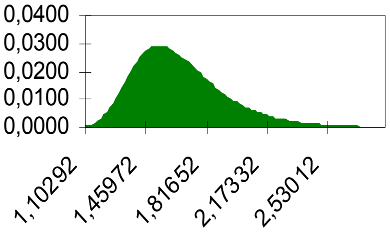


Assumption: Hs

Extreme Value distribution with parameters:

Mode	1,54
Scale	0,22

Selected range is from -Infinity to +Infinity

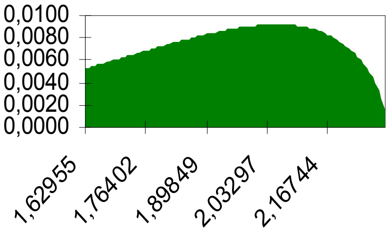


Assumption: COTA

Beta distribution with parameters:

Alpha	6,00
Beta	1,60
Scale	2,30

Selected range is from 1,63 to 2,83

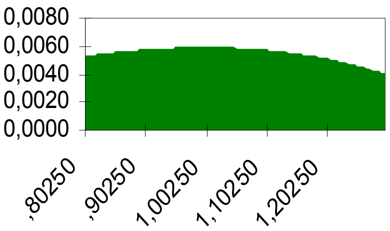


Assumption: Y1

Beta distribution with parameters:

Alpha	3,00
Beta	2,00
Scale	1,50

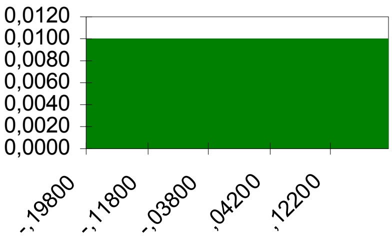
Selected range is from 0,80 to 1,30



Assumption: Tide

Uniform distribution with parameters:

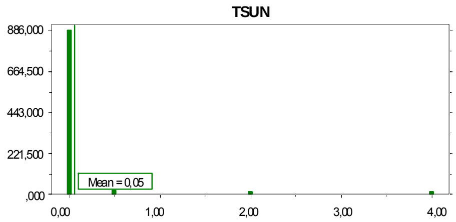
Minimum	-0,20
Maximum	0,20



Assumption: TSUN

Custom distribution with parameters:

		Relative Prob.
Single point	0,00	886,00
Single point	0,50	20,00
Single point	2,00	10,00
Single point	4,00	5,00
Total Relative Probability		921,00

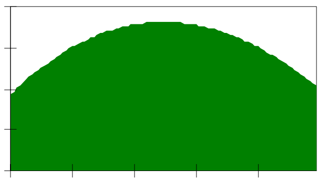


Assumption: SETUP

Extreme Value distribution with parameters:

Mode	0,05
Scale	0,30

Selected range is from 0,00 to 0,10

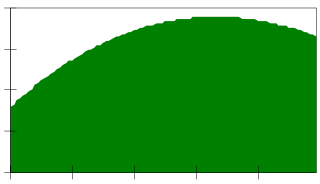


Assumption: Surf Beat

Extreme Value distribution with parameters:

Mode	0,10
Scale	0,30

Selected range is from 0,00 to 0,15



End of Assumptions

APPENDIX B

DETERMINISTIC DESIGN OF THE BREAKWATER

Determination of values of parameters:

Load Parameters:

1) Storm Wave Height:

For return period of 100 years, the storm wave height is found from Equation (4.1) as:

$H_s = 2,6$ m

2) Surf Beat (10% of storm wave height, (Goda, 2000)):

$H_{(surf\ beat)} = 0,26$ m

3) Wave Setup (5% of storm wave height, (Goda, 2000)):

$H_{(wave\ setup)} = 0,14$ m

4) Tide:

$H_{(tide)} = 0,2$ m

5) Tsunami Elevation:

Again for return period of 100 years,

$H_T = 2$ m

Finally, the design wave height is determined as:

$H_D = 5{,}2$ m

Resistance Parameters:

1) Structure Slope:

To be able to make comparison, angle of structural slope is taken as:

$\cot\alpha = 2$

2) For structural slope of 2, head section and non-breaking wave case, the stability coefficient is:

$K_D = 2{,}8$

3) As a rock characteristic:

$\Delta = 1{,}6$

From Equation (3.1):

$D_{N50} = 1{,}83$ m

Stone weight is found as:

W = 16 tons

It is obvious that 16 tons of stone weight on average is unfeasible compared to 4,5 tons on average, for a fishery harbor in 6 m of depth. Obtaining almost zero risk by this design, may be much costly because as the stone size increases, the price increases and availability decreases.