RELIABILITY-BASED DESIGN MODEL FOR RUBBLE-MOUND COASTAL DEFENSE STRUCTURES

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Approval of the thesis:

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ABSTRACT

RELIABILITY-BASED DESIGN MODEL FOR RUBBLE-MOUND COASTAL DEFENSE STRUCTURES

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In this thesis, a new computer model (tool) for the reliability-based design of rubble-mound coastal defense structures is developed in which design is carried out in a user frienly way giving outputs on time variant reliability for the predetermined lifetimes and damage levels. The model aims to perform the following steps:

1. Determine the sources of uncertainties in design parameters

2. Evaluate the damage risk of coastal structures which are at design stage and are recently constructed.

3. Study the sensitivity of limit state functions to the design parameters.

Different from other reliability studies on coastal projects, a new design computer program is developed that can be easily used by everyone working in coastal engineering field.

Key words: Reliability-based design, reliability, rubble mound breakwaters, computer model.

ÖZ

TAŞ DOLGU KIYI KORUMA YAPILARI İÇİN GÜVENİLİRLİĞE
DAYALI TASARIM MODELİ

Arıkan, Gökçe
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Bu çalışmada, taş dolgu kıyı koruma yapılarının güvenilirliğe dayalı modellemesini, önceden belirlenmiş olan yapı hasar seviyeleri ve ekonomik ömürlerini kullanarak zamana bağlı güvenilirlik sonuçlarını veren ve kullanımı kolay olan bir bilgisayar programı geliştirilmiştir.

Bu model aşağıdaki başlıkları yapmayı hedeflemektedir:

1. Tasarım parametrelerindeki belirsizlikleri belirlemek.

2. Tasarım aşamasındaki ve halihazırda kullanılmakta olan taş dolgu kıyı yapılarının hasar yüzdesini ve yıkım risklerini değerlendirmek.

3. Limit durum fonksiyonlarının ve buna bağlı olarak güvenilirliğe dayalı tasarımı amaçlanan taş dolgu kıyı koruma yapılarının herbir tasarım parametresine olan hassasiyetini araştırmak.

Bu çalışmada, kıyı mühendisliğinde halen kullanılmakta olan güvenilirliğe dayalı tasarım çalışmalarından farklı olarak yeni bir tasarım yapan ve kıyı mühendisliği konusunda çalışan herkesin kolaylıkla kullanabileceği bir bilgisayar programı yazılmıştır.

Anahtar Kelimeler: Güvenilirliğe dayalı tasarım, güvenilirlik, taş dolgu dalgakıranlar, bilgisayar modeli.

To my be loved son Arif Arın and husband Arif Erdem

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TABLE OF CONTENTS

ABSTRACT.....IV

ÖZ VI

DEDICATION.....VIII

ACKNOWLEDGMENTS IX

TABLE OF CONTENTS.....X

CHAPTERS .....

1.INTRODUCTION 1

1.1 GENERAL DESCRIPTIONS 1

1.2 STATEMENT OF THE PROBLEM 2

1.3 SCOPE OF THE THESIS 3

2.LITERATURE REVIEW 5

2.1 FORMER STUDIES ON RELIABILITY EVALUATIONS 5

2.2 BASIC DEFINITIONS USED IN RELIABILITY-BASED DESIGN7

2.3 PRINCIPLE FAILURE MODES FOR RUBBLE MOUND BREAKWATERS 8

2.4 UNCERTAINTY CLASSIFICATION 11

2.4.1 SOURCES OF UNCERTAINTY 12

3.THE RELIABILITY-BASED DESIGN MODEL 14

3.1CLASSIFICATION OF UNCERTAINTIES IN THE DESIGN OF RUBBLE MOUND BREAKWATERS 15

3.2 RELIABILITY METHODS 21

3.3 FIRST ORDER SECOND MOMENT METHOD 22

3.4 RELIABILITY ASSESSMENT 25

3.5 RACKWITZ-FIESSLER PROCEDURE (EQUIVALENT NORMAL DISTRIBUTIONS) 31

3.6 FAILURE MODE RESPONSE FUNCTIONS 33

3.6.1 HUDSON FAILURE FUNCTION 33

3.6.2 VAN DER MEER FAILURE FUNCTION 34

3.7 EXTREME WAVE HEIGHT DISTRIBUTIONS 36

3.7.1 FISHER-TIPPETT TYPE I (GUMBEL) DISTRIBUTION.....37

3.7.2 FISHER-TIPPETT TYPE II DISTRIBUTION 37
3.7.3 WEIBULL DISTRIBUTION 38
3.7.4 LOG-NORMAL DISTRIBUTION... 39
3.8 THE ARIN DESIGN MODEL.. 40
3.8.1 PROGRAM SCHEME 43
3.9 UNCERTAINTY EVALUATION OF BASIC DESIGN VARIAB 48
3.9.1 UNCERTAINTIES IN LOAD PARAMETERS 48
3.9.2 UNCERTAINTIES IN RESISTANCE PARAMETERS 50
4.APPLICATION OF THE RELIABILITY-BASED DESIGN MODEL. 58
CASE I: GİRESUN PORT 58
4.1 CASE STUDY I: GIRESUN PORT 58
4.1.1 DETERMINISTIC DESIGN 62
4.1.1.1 DESIGN BY HUDSON EQUATION............................ 63
4.1.1.2 DESIGN BY VAN DER MEER EQUATION 65
4.2 RELIABILITY-BASED DESIGN 67
5.APPLICATION OF RELIABILITY-BASED DESIGN MODEL 79
CASE II: FOÇA-LEVENTLER MILITARY PORT 79
5.1 DETERMINISTIC DESIGN BY VAN DER MEER EQUATION... 79
5.2 RELIABILITY-BASED DESIGN 81
5.3 HYDRAULIC MODEL STUDIES 91
6.APPLICATION OF RELIABILITY-BASED DESIGN MODEL 96
CASE III: SİNOP DEMİRCİKÖY FISHERY HARBOR 96
6.1 DETERMINISTIC DESIGN BY VAN DER MEER EQUATION... 96
6.2 RELIABILITY-BASED DESIGN 99
6.3 HYDRAULIC MODEL STUDIES 108
6.4 EVALUATION OF RESULTS... 111
6.5 HYDRAULIC MODEL STUDIES ON TOE BERM STABILITY 113
7.CONCLUSIONS AND FUTURE RECOMMENDATIONS............... 117
REFERENCES 120
APPENDIX 125
A.EXTREME WAVE ANALYSIS 125

A.1 DISTRIBUTION FUNCTIONS FOR EXTREME WAVES........... 126
A.1.1 FISHER-TIPPETT TYPE I (GUMBEL) DISTRIBUTION.... 126
A.1.2 FISHER-TIPPETT TYPE II DISTRIBUTION...................... 127
A.1.3 WEIBULL DISTRIBUTION 127
A.1.4 LOG-NORMAL DISTRIBUTION....................................... 128
A.2 RETURN PERIOD AND RETURN VALUE 130
A.3 PARAMETER ESTIMATION BY THE LEAST SQUARES METHOD 131
A.4 SELECTION OF MOST PROBABLE PARENT DISTRIBUTIO. 133
A.4.1 MINIMUM RATIO OF RESIDUAL CORRELATION COEFFICIENT CRITERION (MIR)... 134
A.4.2 OUTLIER DETECTION BY THE DEVIATION OF OUTLIER (DOL) CRITERION.. 135
A.5 EXTREME WAVE ANALYSIS RESULTS OF CASE I: GIRESUN PORT 139
A.6 EXTREME WAVE ANALYSIS RESULTS OF CASE II: FOÇA-LEVENTLER MILITARY PORT 145
A.7 EXTREME WAVE ANALYSIS RESULTS OF CASE III: SINOP DEMIRCIKÖY FISHERY HARBOR 151
CIRRICULUM VITAE……………………………………..………………157

LIST OF TABLES

3.1 Variation coefficients for sea state parameters (Burchart,1992)…..... 20

3.2 Limit State Functions…………………………………………..…... 35

3.3 Response functions for rubble-mound breakwaters for Van der Meer equation……………………………………………………………… 49

3.4 Uncertainty variables of response functions……………...………... 52

3.5 Basic Random Variables of Limit State Equations and their Probabilistic Values…………………………………..………….. 53

3.6 Damage level by S for two-layer armor……………………..……... 54

3.7 Minimum design life of coastal structures…………………….….. 55

3.8 The lifetime maximum admissible failure probabilities for flexible structures……………………………………………………………. 56

3.9 The lifetime maximum admissible failure probabilities for rigid structures…………………………………………………………… 57

4.1 Deterministic design by Hudson equation………………………….. 64

4.2 Deterministic design by Van der Meer equation…………………… 66

4.3 Statistical values of design variables used in Van der Meer limit state equation of Giresun Port…………………...………………..… 68

4.4 Probability of failure given S=10 and Lifetime 50 years……........... 71

4.5 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model……………………………………… 75

4.6 Design parameters evaluated from deterministic and reliability-based designs for Giresun Port…………………………………..… 77

5.1 Deterministic design by Van der Meer equation …………………… 81

5.2 Statistical values of design variables used in Van der Meer limit state equation of Foça Leventler Military Port……………………... 83

5.3 Design parameters evaluated from deterministic and reliability-based designs for Foça-Leventler Military Port………………........ 88

5.4 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model for the case……………………..…. 89

5.5 Selection of Model Scale…………………………………….…….. 92
5.6 Comparison of deterministic and reliability-based designs with physical modeling……………………….……………………….. 93
5.7 Comparison of lifetime alternatives LT=30 and LT=50 years for a constant damage level of S=2………………………………………. 95
6.1 Deterministic design by Van der Meer equation ………………..… 99
6.2 Statistical values of design variables used in Van der Meer limit state equation of Demirciköy Fishery Harbor……………………… 101
6.3 Statistical values of design variables used in Van der Meer limit state equation of Demirciköy Fishery Harbor……………………… 106
6.4 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model for the case ……………………….. 107
6.5 Wave characteristics of the model studies………………………….. 109
6.6 Comparison of Physical Modeling, Deterministic and Relibility Based Design Models for selected damage levels………..……… 110

LIST OF FIGURES

2.1 Cost Optimization Curve 10

2.2 Overview of rubble mound breakwater failure modes (CEM,2003) 11

3.1 Definition of safe and failure regions and reliability index……….. 26

3.2 Definition of the Hasofer and Lind reliability index, β_{HL}………….. 27

3.3 Starting Window of Model……………………………………….. 41

3.4 The iterative algorithm of the ARIN model………………………. 42

3.5 An example of a graphical output window from ARIN…………... 44

3.6 An example of a tabular output window from ARIN……………… 45

4.1 Location of Giresun Port……………………………………….. 59

4.2 Layout of Giresun Port………………………………………….. 60

4.3 Trunk section of the main breakwater in Giresun Port at a water depth of d=20.0 meters at the toe of the structure………………….. 63

4.4 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=10……………………………………………………………. 70

4.5 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2……………………………………………... 72

4.6 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2……………………………………………... 73

5.1 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2………………………………………………………… 85

5.2 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10, 16 and 18 damage levels in LT=50 years lifetime of structure………………………………….. 86

5.3 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2…………………………………….. 87

5.4 Location of Foça-Leventler Military Port………………………….. 91
5.5 Cross-section of Foca-Leventler Military Port……………………. 93
6.1 Location of Sinop Demirciköy Fishery Harbor…………………… 97
6.2 Cross-section of Sinop Demirciköy Fishery Harbor……………… 98
6.3 Exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} for S=2 damage level and 50 years lifetime of structure.. 103
6.4 Exceedance probabilities $P_{f(}\%)$ as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2,6,10,16,18.........................……………….............................. 104
6.5 Exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} for LT= 4,25,30,50 years lifetimes with damage level S=2…………………………………………………………. 105
6.6 Cross-section of Model 2 (Prototype Values)……………………… 115
6.7 Damage vs. Wave Height Curve of Set1 and Set2 Experiments Model 2…………………………………………………………... 116

CHAPTER 1

INTRODUCTION

1.1 General Descriptions

Coastal regions have always been popular places for civilization, recreation and habitation. In many countries, land that has connection to the sea is much more valuable and attractive than inland. However, low-lying coastal plains are subjected to flooding and cliffs are vulnerable to erosion. The prospect of accelerating sea level rise, unpredictable damaging storms and storminess associated with climate change has heightened public and academic awareness of the hazards faced by those living and working in and on coastal areas.

Turkey is surrounded by three seas and an inland adjacent sea having a big potential of transportation, tourism, food, trade etc. This makes Turkey to construct several number and types of coastal defense structures in order to be able to cope with this amount of economic fields rising the properties of our country. Therefore, numerous coastal structure projects have been carried out and will be planned in the forthcoming years. In addition, due to the effects of nature and damages of use, the existing structures' needs periodical maintenances. None of these structures that had been constructed or are planned to be constructed can be designed by inexperienced project or management teams since the dominating design parameters are forces of nature.

Economic and social pressures have led to the construction of coastal defenses to protect against storms, floods and erosions. There is a high degree of uncertainty in the conditions and parameters that may be experienced by a coastal structure, and a strong economic pressure to restrict the cost of defense structures. As a result, coastal defense structures are typically designed to withstand conditions of a specified severity (e.g. the storm conditions encountered once every 50 years on average), judged to balance between cost on the one hand and the level of protection or level of damage on the other hand (Reeve, Chadwick and Fleming, 2004).

In recent past, two major developments have had an influence on the way engineers approach design. First, the concept of certainty thus uncertainty was questioned. Second, methods to deal with uncertainties in design were developed. The application of probabilistic and reliability calculations to coastal structure design has become an integral part of design guidance for sea defenses in many countries.

1.2 Statement of the Problem

The safety evaluation of coastal regions thus coastal defense structures has been based on deterministic methods for several years. In these methods, randomness of load and resistance parameters is not taken into account. Deterministic design logic accompanies only the characteristic (mean) values of the design parameters. However, the design of coastal structures embodies a high degree of uncertainty and variability in the structural capacity itself and load intensities and characteristics. In addition, providing economical designs at specified levels of safety is gaining crucial importance in all civil and coastal engineering designs. This automatically introduced a probabilistic approach from which concepts of uncertainty, reliability and risk arose naturally. Thus, it has vital importance to evaluate safety of coastal defense structures by using a reliability-based design method in which the parameters are modeled as random variables using proper distributions.

Due to limited number of data collected, special attention should be given to the selection of parent distribution of design parameters, utilization of reliability-based design and decision making procedures for coastal defense structures. In addition, hydraulic physical testing is recommended for major structures in order to examine the parameters and failure modes that are not covered in the limit state functions which defines a boundary between the desired (safe domain) and the undesired (failure domain) performance of a structure. This is also proposed for the assurance of appropriateness of the reliability-based design model constructed.

In contrast with the importance of reliability-based design or risk assessment models that are developed in coastal engineering, these are mostly used only by the scientists working in that field since there exists insufficient corroborative results of the model executed in real-time projects.

A new user friendly reliability-based model that should be guidance for rubble-mound coastal defense structure design is therefore required according to the topics discussed above.

1.3 Scope of the Thesis

In this thesis a new approach to the reliability based design model of rubble-mound coastal defense structures will be studied in which the time dependent reliability will be treated within the predetermined lifetimes of the structures.

In the design of rubble-mound breakwaters there are different approaches used in coastal engineering field. The reliability evaluations of coastal defense structures are executed by three design levels considering the complexity of the modeling. A semi-deterministic method that uses characteristic values of basic variables and partial coefficients of safety is named as Level I design. In Level II design strength and load parameters are modeled with their first and second moments, i.e. their mean and variance. In addition, Level III design model deals with the complete joint probability density function of the response function in determining the reliability level of the problem.

In the thesis study, Level II reliability method is used for the reliability-based design of rubble-mound breakwaters for hydraulic stability of armor unit failure mode under the effect of wave action. Level II methods are more practically oriented and are quite suitable for design in spite of the fact that they are approximate compared to Level III methods. The results obtained from Level II analysis are compared with the physical modeling results.

In this study, reliability-based design computer model is developed for the design and safety evaluation of rubble-mound breakwaters taking into account the recommendations and proclamations on reliability design in coastal engineering field.

The Assessment of Reliability In Numerical Modeling of Rubble-Mound Breakwaters (ARIN) model aims to perform the following steps:

1) Determine the sources of uncertainties in design parameters.

2) Visualize the changes in reliability levels of rubble-mound coastal structures when design parameters changed one by one.

3) Evaluate the damage risk of coastal structures which are at design stage.

4) Evaluate the damage risk of coastal structures which are recently constructed.

5) Find out the probability of failure of the structure during or at any time of specified lifetime.

6) Study the sensitivity of limit state functions to the design parameters.

CHAPTER 2

LITERATURE REVIEW

2.1 Former Studies on Reliability Evaluations

Reliability is a word with different connotations. The original use of the term is quantitative and this implies the need for methods of measuring reliability. (Crowder, 1991). Reason for insisting on quantitative measure of reliability is that different standards of reliability is needed for different type of design works or projects. However, it is a newly applied methodology in coastal engineering field.

The trend in civil engineering, today, more than ever before, is toward providing economical designs at specified levels of safety thus reliability. In addition, there is an increasing awareness that the raw data, on which problem solutions are based, themselves exhibit significant variability (Harr, 1987).

The structural safety evaluations of coastal defense structures have been done with deterministic methods for several years. In these methods, semi-empirical design formulas are used where the design parameters are treated with their characteristic values. However, the design of coastal structures involves a large number of uncertainties in the load and resistance parameters. Therefore, the probabilistic approaches for the design of coastal structures have been gaining vital importance in recent times. Reliability-based method in which the parameters are modeled as random variables is one of the probabilistic methods used for the structural safety evaluations.

In most of the cases, the reliability of rubble mound structures is considered as the key element in evaluation of structural risks for coastal projects, because the reliability of such structures is known to depend on the highly variable and unpredictable nature of coastal storms (Balas and Ergin, 2002). The consequences of failure of breakwaters planned under widely different design conditions ranges from low to very significant economical losses (Sorensen et al., 1994).

Almost all coastal structure design formulae are semi empirical and based mainly on central fitting to model test results. The often considerable scatter in test results is not considered in general because the formulae normally express only the mean values. Consequently, the applied characteristic value of the resistance is then the mean value and not a lower fractile as is usually the case in other civil engineering fields. The only contribution to a safety margin in the design is inherent in the choice of the return period for the design load. (The exception is when the design curve is fitted to the conservative side of the data envelope to give a built-in safety margin.) It is now more common to choose the return period with due consideration of the encounter probability, i.e., the probability that the design load value is exceeded during the structure lifetime. This is an important step towards a consistent probabilistic approach (CEM, 2003)

In addition to design load probability, a safety factor (as given in some national standards) might be applied as well, in which case the method is classified as a Level I (semi-deterministic/quasi-probabilistic) method. However, this approach does not allow determination of the reliability (or the failure probability) of the design; and consequently, it is not possible to optimize structure design or avoid overdesign of a structure. This method only helps to check whether a defined level of safety is satisfied. In order to overcome this problem, more advanced probabilistic methods must be applied where the uncertainties (the stochastic properties) of the involved loading and strength variables are considered.

After obtaining the reliability functions for Hudson and Van der Meer equations (Van der Meer 1988b; Burcharth 1992) by utilization of First Order Second Moment Method, number of studies on reliability-based design and analysis of coastal structures have raised. In coastal engineering field, reliability-based design methodology has been so far used mostly for the vertical wall coastal defense structures.

The reliability-based risk assessment and structural design model (REBAD) (Balas, 1998) is a worthwhile tool in the preliminary design of maritime structures which are portrayed by vast failure consequences and significant resource expenditures (Ergin and Balas, 1999; Balas and Ergin, 2002).

2.2 Basic definitions used in reliability-based design

The term failure means different things to different people from different principles. It can be said that a structure fails if it cannot perform its intended function. It has vital importance to specify the definition of failure clearly before starting the reliability analysis in order to state the first point of the problem needed to be handled.

The concept of a ***limit state*** is used to help define failure in the context of structural reliability analyses. A limit sate is a boundary between the desired and the undesired performance of a structure. This boundary is often represented mathematically by a limit state function or performance function. Some structural members or structure itself may fail in a brittle manner, whereas others may fail in a ductile fashion. In the traditional approach, each mode of failure is considered separately, and each mode can be defined using the concept of a limit state.

In structural reliability analyses three types of limit states are considered.

1. **Ultimate limit sate:** The failure or collapse of the performance function arising from the loss of load-carrying capacity.
Ultimate limit state is implemented for the design of vertical wall structures in recent studies.

2. **Fatigue limit state:** The failure is related to loss of strength under repeated loads. Fatigue limit states are related to the accumulation of damage and eventual failure under repeated loads. In any fatigue analysis, the critical factors are both the magnitude and the frequency of load.

3. **Serviceability limit state:** This type is related to gradual deterioration of users' comfort or maintenance costs. They may or may not be related to structural integrity.

In this thesis serviceability limit state will be employed for the evaluation of safety of rubble mound breakwaters.

2.3 Principle Failure Modes for Rubble Mound Breakwaters

For many people, the word *failure* implies a total or partial collapse of a structure, but this definition is limited and not accurate when discussing design and performance of coastal structures. In the context of design reliability, it is preferable to define failure as the damage that results in structure performance and functionality below the minimum anticipated by design.

In the design process all possible failure modes (Figure 2.2) must be identified and evaluated in order to obtain a balanced design. An overview of the most important and common failure modes for the rubble mound breakwaters are given below.

-Overtopping

-Slip circles

-Core settlement

-Subsoil settlement and geotechnical instabilities

-Berm erosion

-Toe erosion

-Displacement, movement and breakage or armor units on the front and rear face of the armor layer due to the wave action.

Since there occurs interaction between components of the structural system, failure of one component may lead to the failure of the whole system. This is known as the series connection of the system components. In other case, components might compensate each other without a system failure, known as parallel connection. In this thesis interaction between failure modes are not considered due to the lack of information of interaction of different failure mechanisms and reliability-based design of rubble-mound breakwaters is limited to one failure mechanism limit state function till now. Hydraulic stability of the primary armor layer which is the most important failure mode is discussed with a particular emphasis on different sources of uncertainties.

In conventional design methodology for rubble-mound breakwaters no damage criteria has been accepted in practice (Hudson, 1974) However, in recent developments cost optimization is based on accepting some partial damage. Total cost of the structure has two components: Initial cost and maintanence cost. If no damage criteria is accepted for the design of rubble-mound breakwaters then the initial cost increases whereas if a partial damage is accepted in the design then the initial cost decreases yet partial damage cost is added to the total cost. In Figure 2.1 optimization of the total cost is shown with the curves of initial cost and maintanence cost giving the minimum point at the intersection called as the cost optimization point.

In recent developments where reliability-based design models are developed optimization of the total cost is taken as a base for the design of rubble-mound breakwaters. In view of this concept reliability based models give the probability of the assigned damage and the corresponding armor stone diameters.

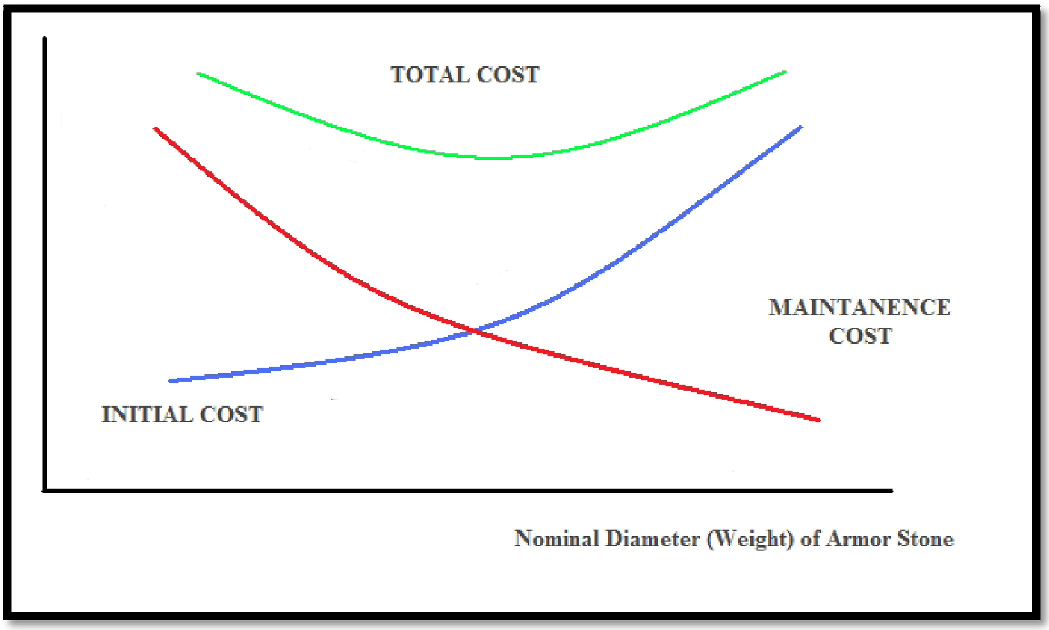

Figure 2.1 Cost Optimization Curve

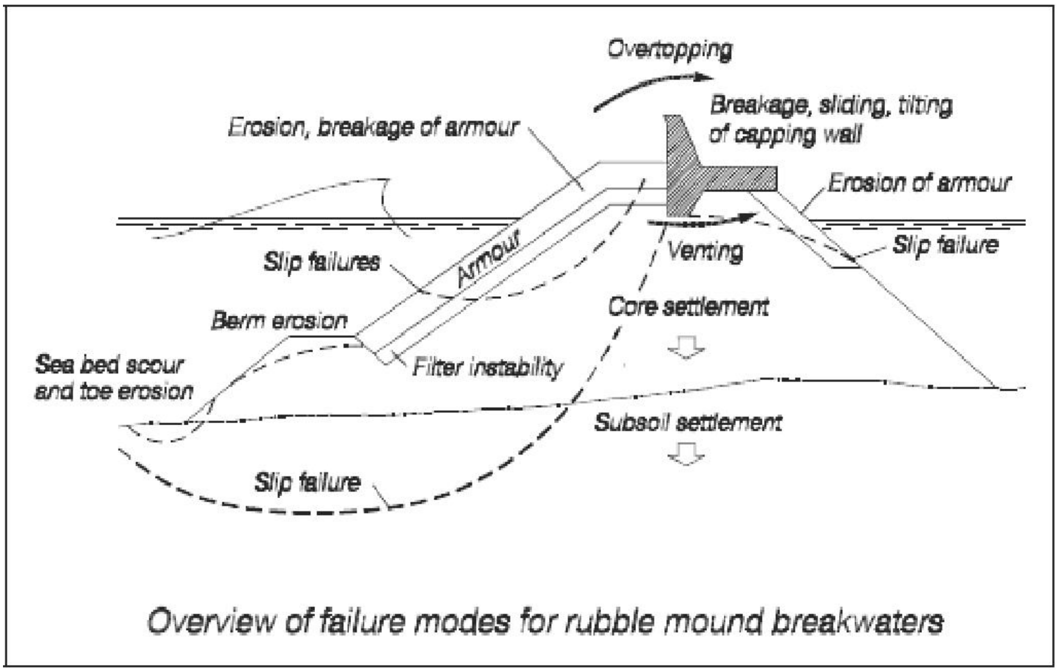

Figure 2.2 Overview of rubble mound breakwater failure modes (CEM, 2003)

2.4 Uncertainty Classification

Most of the engineering processes or phenomena contain randomness; that their outcomes are unpredictable. This inherent randomness thus variability is a state of nature which cannot be controlled since the exact realization of these processes are not possible. However, it is possible to exemplify the random behavior of the phenomena by statistical methods under the light of observational data.

Coastal projects mainly accompany the uncertainties at the design and planning stages. During the design stage the main source of uncertainties arises from the prediction of load and resistance parameters. The pivoting uncertainties at the design stage are the stochastic wind wave generation yielding in significant scatters of wave data and response of the structure against wave loading.
In this section types of uncertainties relevant to the assessment and design of coastal structures are summarized. It is important to acknowledge uncertainty wherever it lies in the design or assessment process. When applied correctly, a

probabilistic approach to design allows uncertainties to be quantified, even if not removed.

2.4.1 Sources of uncertainty

Incompleteness: If not all possible failure mechanisms have been identified then the risk assessment will be incomplete. For coastal structures detailed observations of failures are scarce, due to the relatively small number of failures and the difficulty of taking measurements during the physical conditions under which failure occur

Empiricism: The behavior of most coastal structures is predicted by design equations that are generally empirical, based on experiments performed at laboratory scale. Such experiments are rarely exactly repeatable, giving scatter in the results and errors in fitting an equation on the data

Extrapolation: In determining design conditions observations are used to specify the parameters of probability distribution. There are statistical errors associated with this procedure but in addition there is uncertainty when design values are estimated from extrapolating the distribution curve.

Measurement error: The observations used for the design will themselves have uncertainties due to the accuracy of measurement equipment. The accuracy of the measurements will affect the estimates of the design loads (e.g. wave heights and water levels) and the strength of the structures.

Compound failure mechanisms: Coastal structures in particular can be difficult to assess in terms of separate failure mechanisms. That is, failure may occur through a particular sequence of partial failures. Analysis of chain of events is difficult since design equations are formulated to represent a single mechanism. In practice, designs are governed by a small number of mechanisms that are treated independently.

Stationarity: The design loads and corresponding structure derived from these have, in the past, been taken as being applicable for the duration of the structured design life. That is, there is an assumption that the statistics of say, wave heights, remains constant over time. However, the effects of sea level rise and long term climate change caused a reappraisal of this assumption. If there is a long term underlying trend, or if the variance of a variable changes over time (such as changes in typical storm intensity, duration or frequency), then these can have significant effect on the design life of the structure.

Calculation of reliability or failure probability of a structure is based on formulae describing the structure's response to loads and on information about the uncertainties related to the formulae and relevant parameters.

CHAPTER 3

THE RELIABILITY-BASED DESIGN MODEL

Satisfaction of the optimum safety, economy and serviceability criteria is the logic behind the design of coastal structures. In the conventional design practice structural safety thus stability is treated in deterministic design basis, where the design parameters are used as deterministic not stochastic variables. In this methodology, the coastal defense structure is designed to survive under the extreme wave conditions with sustaining the predetermined damage and serviceability limits during its lifetime. The dominating parameter which is the design wave height is selected and calculated with the help of extreme wave analysis for the selected region and return period. Therefore, significant wave height for a predetermined return period is the only uncertainty inherent in deterministic design.

The reliability-based design model developed for rubble-mound breakwaters deals with uncertainties inherent in design parameters due to the large degree of uncertainty existent in both load and resistance parameters of design.

The purpose of the design of a revetment or breakwater is to obtain a structure which, during its construction and throughout its intended service life, has a sufficiently low probability of failure and of collapse. In order to achieve the best possible assessment of this, a risk analysis can be performed. The three main elements of the risk analysis are hazard, mechanisms, and consequence. A risk analysis begins with the preparation of an inventory of the hazards and mechanisms. A mechanism is defined as the manner in which the structure responds to hazards. A combination of hazards and mechanisms leads, with a particular probability, to failure or collapse of the structure as a whole or of its components. (Van der Meer, 1988b)

The uncertainties inherent in coastal projects are accumulated more significantly compared to other construction operations. Therefore it is much more difficult to evaluate and examine the safety level of a coastal defense structure. These uncertainties are quantified by use of standard deviation which is the measure of scatter around the mean value.

In this thesis, in order to reduce the complexity of parameter dimensions a dimensionless parameter called variation coefficient which is the ratio of the standard deviation to the expected (mean) value is used as the measure of uncertainty.

Basically, uncertainty is best given by a probability distribution; but because the true distribution is rarely known, it is common to assume a normal distribution and a related coefficient of variation,δ, defined as the measure of the uncertainty.

$$\delta = \sigma/\mu = \text{standard deviation / mean value}$$

3.1 Classification of Uncertainties in the Design of Rubble Mound Breakwaters

a) Uncertainty related to failure mode formulae

The uncertainty associated with a formula can be considerable. This is clearly seen from many diagrams presenting the formula as a smooth curve covered by a wide scattered cloud of data points (usually from experiments) that are the basis for the curve fitting. Coefficients of variation of 15 - 20 percent or even larger are quite normal. The range of validity and the related coefficient of variation should always be considered when using a design formula.

b) Uncertainty related to environmental parameters

The sources of uncertainty contributing to the total uncertainties in environmental design values are categorized as follows:

(1) Errors related to instrument response (e.g., from accelerometer buoy and visual observations).

(2) Variability and errors due to different and imperfect calculations methods (e.g., wave hindcast models, algorithms for time-series analysis).

(3) Statistical sampling uncertainties due to short-term randomness of the variables (variability within a stochastic process, e.g., two 20-min. records from a stationary storm will give two different values of the significant wave height)

(4) Choice of theoretical distribution as a representative of the unknown long-term distribution (e.g., a Weibull and a Gumbel distribution might fit a data set equally well but can provide quite different values for a 200-year event).

(5) Statistical uncertainties related to extrapolation from short samples of data sets to events of low probability of occurrence.

(6) Statistical vagaries of the elements.

Distinction must be made between short-term sea state statistics and long-term (extreme) sea statistics. Short-term statistics are related to the stationary conditions during a sea state, e.g., wave height distribution within a storm of constant significant wave height, H_s. Long-term statistics deal with the extreme events, e.g., the distribution of H_s over many storms.

Related to the short-term sea state statistics the following aspects must be considered:

• The distribution for individual wave heights in a record in deepwater and shallow-water conditions, i.e., Rayleigh distribution and some truncated distributions, respectively.

• Variability due to short samples of single peak spectra waves in deep and shallow water based on theory and physical simulations.

• Variability due to different spectral analysis techniques, i.e., different algorithms, smoothing and filter limits.

• Errors in instrument response and influence of measurement location. For example, floating accelerometer buoys tend to underestimate the height of steep waves. Characteristics of shallow-water waves can vary considerably in areas with complex seabed topography. Wave recordings at positions with depth-limited breaking waves cannot produce reliable estimates of the deepwater waves.

• Imperfection of deep and shallow-water numerical hindcast models and quality of wind input data.

Evaluation of the uncertainties related to the long-term sea state statistics, and use of these estimates for design, involves the following considerations:

• The encounter probability.

• Estimation of the standard deviation of a return-period event for a given extreme distribution.

• Estimation of extreme distributions by fitting to data sets consisting of uncorrelated values of H_s from;

- Frequent measurements of H_s equally spaced in time.

- Identification of the largest H_s in each year (annual series).

- Maximum values of H_s for a number of storms exceeding a certain threshold value of H_s using peak over threshold (POT) analysis.

The methods of fitting are the maximum likelihood method, the method of moments, the least square method, and visual graphical fit.

• Uncertainty on extreme distribution parameters due to limited data sample size.

• Influence on the extreme value of H_s on the choice of threshold value in the POT analysis. (The threshold level should exclude all waves which do not belong to the statistical population of interest).

• Errors due to lack of knowledge about the true extreme distribution. Different theoretical distributions might fit a data set equally well, but might provide quite different return period values of H_s. (The error can be estimated only empirically by comparing results from fits to different theoretical distributions).

• Errors due to applied plotting formulae in the case of graphical fitting. Depending on the applied plotting formulae quite different extreme estimates can be obtained. The error can only be empirically estimated.

• Climatological changes.

• Physical limitations in extrapolation to events of low probability. The most important example might be limitations in wave heights due to limited water depths and fetch restrictions.

• The effect of measurement error on the uncertainty related to an extreme event.

It is beyond the scope of this thesis to discuss in more detail the mentioned uncertainty aspects related to the environmental parameters. Additional information is given in study carried out by Burchart (Burchart, 1992).

c) Uncertainty related to structural parameters.

The uncertainties related to material parameters (such as density) and geometrical parameters (such as slope angle and size of structural elements) are generally much smaller than the uncertainties related to the environmental parameters and to the design formulae.

Estimates of overall uncertainties for sea state parameters (first three items) are presented in Table 3.1 for use when more precise site specific information is not available.

Table 3.1 Variation coefficients for sea state parameters (Burchart, 1992)

Parameter	Method of determination	δ (%)	Comments
Significant Wave Height Offshore	Accelerometer Buoy, pressure cell, vertical radar	5-10	-
	Horizontal radar	15	
	Hind cast, numerical models	10-20	Very dependent on quality of weather maps
	Hind cast SMB method	15-20	Valid only for storm conditions in restricted sea basins
	Visual observation from ships	20	-
Significant Wave Height Near shore	Numerical models	10-20	δ can be much larger in some cases
	Manual calculations	15-35	
Mean wave period Offshore (significant wave height is fixed)	Accelerometer buoy records	2-5	-
	Estimates from amplitude spectra	15	
	Hind cast, numerical models	10-20	
Duration of sea state (significant wave height is exceeding a specific level)	Direct measurements	2	-
	Hind cast, numerical models	5-10	
Spectral peak frequency Offshore	Hind cast, numerical models	5-15	-
	measurements	10-20	

3.2 Reliability Methods

The reliability design method is classified into three categories depending on the level of probabilistic concepts being employed. These are Level I, Level II, and Level III methods (Goda and Takagi, 2000).

The Level I design is the semi-deterministic method, which employs the partial safety factors. The values of external loads are increased by multiplying them with the partial safety factors, while the values of resistance strength are reduced by dividing them with their partial safety factors. Both the factors are assigned values equal or greater than 1, in consideration of the probabilistic behaviors of loads and resistances. Evaluation of the partial safety factors is the crucial point in the success of the reliability method with Level I. Once the partial safety factors are determined the design calculation proceeds in a deterministic manner.

In the Level II reliability design method, the load and resistance are taken as random variables and assumed to distribute normally. In this method, basic variables are characterized by their first and second moments, i.e. their mean and variance. First Order Second Moment (FOSM) method is an example of level II design where only the first terms in Taylor series expansion and second moments i.e. mean and standard deviation values of parameter are used. Level II design scheme is suitable for the design of coastal structures (Balas and Ergin, 2002).

In the Level III reliability design method, 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. In fact, the use of this design method is more complicated and requires unrealistic capabilities of computation due to the excess number of random variables in the response function. This complexity is due to the integration of joint probability density functions over the unsafe region. Monte Carlo Simulation technique is employed to run Level III design concept.

The quantity that is traditionally used to maintain a proper degree of safety in structural design is factor of safety. Generally the factor of safety is understood to be the ratio of the expected strength to the expected load. The strength of the system and the acting load on the system are assumed to be unique in conventional design. However, in practice, both the strength and load are variables; the values of which are scattered about their respective mean values. This results in an overlap in the distribute values of strength and load that might lead to the failure of the system. Moreover, the safety factors used in Level I method may not lead to adequate and economical designs. Level II methods are more practically oriented although they are approximate with respect to Level III methods. Level III methods use full joint probabilistic descriptions of the random variables.

In this work the evaluation of reliability is based on a Level II method, i.e. the First Order Second Moment method and Level II method will be treated together in order to carry out reliability based design for rubble-mound coastal defense structures. Structural reliability design model and the developed computer program will be presented in the following sections of the chapter.

3.3 First Order Second Moment Method

One of the methods of Level II analysis is First Order Second Moment Method (FOSM). In this approach, uncertainty in performance is taken to be a function of uncertainty in model parameters or in the model itself. The expected values and standard deviations of the random variables are used to estimate the expected value and standard deviation of a performance function. The performance function, which is also called the **limit state function**, is the random function of resistance and loading, describing the system performance related to its possible failure, or limit state of interest as stated in Equation below:

$$F\ (\text{loading - resistance}) = 0$$

where F is defining the limit state function.

In this method the joint probabilities of loading and resistance variables are considered in obtaining the limit state. On the other hand, in conventional deterministic design, the limit state of interest is represented by a predefined factor of safety as stated in Equation below; without considering the uncertainties involved in loading and resistance parameters.

$$\frac{Resistance}{Loading} = Factor\ of\ safety$$

The usual output of FOSM methods is the reliability index, β, which is the shortest distance from the linearized limit state failure surface to the origin of the reduced variable space. The reliability index provides a measure of relative or comparative reliability without having to assume a probability distribution for the performance function. A complete distribution would be required to calculate the probability of failure, but its form is generally unknown.

To obtain the probability of failure (P_f) from β, a probability distribution on the performance function must be assumed. A normal distribution is generally used for the ease of calculation; however, the performance function is often taken as ln (FS) [or ln (resistance/load)], implying that the factor of safety is log normally distributed. Given this assumption and the value of β, the required probability values are easily calculated from the properties of the assumed distribution.

The analytical solutions of the failure integral are limited to a few of very special cases. However, for ***standard*** normally distributed basic variables and the linear failure function the analytical solution of the failure integral is given by:

$$P_f = \Phi(-\beta) \tag{3.1}$$

Φ ( ) being the cumulative standard normal distribution and ***β*** the ***so*** called ***reliability*** *index,* which represents the distance between the origin of the space of basic variables and the ***design point*** **x*** on the failure surface (Fig. 3.1). The design point **(x*)** is a point on the failure surface having the minimum distance to the origin of the space of basic variables, thus contributing most to the failure probability.

In the case of the non-linear failure function, a linearization at the design point provides an approximate value of the failure probability:

$$P_f \approx \Phi(-\beta)$$

For non-normal basic variables, a transformation from the physical **(x)** space to the standard normal ***(u)*** space is performed. If the basic variables are assumed to be stochastically independent, the transformation is defined by:

$$U_i = \Phi^{-1}\big(F_i(X_i)\big) \tag{3.2}$$

with standard normal variables $\mathbf{U_i}$, and its inverse by:

$$X_i = F_i^{-1}\big(\Phi(U_i)\big) \tag{3.3}$$

Application of the inverse transformation of the equation shown above allows the failure function in the physical **(x)** space to be evaluated.

3.4 Reliability Assessment

In 1974, Hasofer and Lind (Hasofer and Lind, 1974) proposed a modified reliability index that did not exhibit the invariance problem. In this new approach the limit state function is evaluated at a point known as the design point instead of mean values. Since this design point is generally not known a priori, an iteration technique must be used to solve for the reliability index. This forms the skeleton of main algorithm of the program which is used to evaluate the reliability analysis of rubble mound breakwaters that is the aim of this thesis.

A limit state function $g(X_1, X_2,..., X_n)$ where the random variables X_i are all uncorrelated is considered. The limit state function is rewritten in terms of the standard form of the variables (reduced variables) using

$$z_i = \frac{X_i - \mu_{X_i}}{\sigma_{X_i}} \tag{3.4}$$

where;

z_i: is the reduced variable of the design parameter

x_i: design parameter value

σ_{xi}: standard deviation of the design parameter

μ_{xi}: expected value of the design parameter.

The Hasofer-Lind reliability index is defined as the shortest distance from the origin of the reduced variable space to the limit state function g=0.

If the limit state function is linear, the reliability index is calculated as

$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i \mu_{X_i}}{\sqrt{\sum_{i=1}^{n} \left(a_i \sigma_{X_i}\right)^2}} \tag{3.5}$$

If the limit state function is non-linear, iteration is required to find the design point $\{z_1^*, z_2^*, \ldots., z_n^*\}$ in reduced variable space such that β still corresponds to the shortest distance. This is described in Figure 3.1 and Figure 3.2.

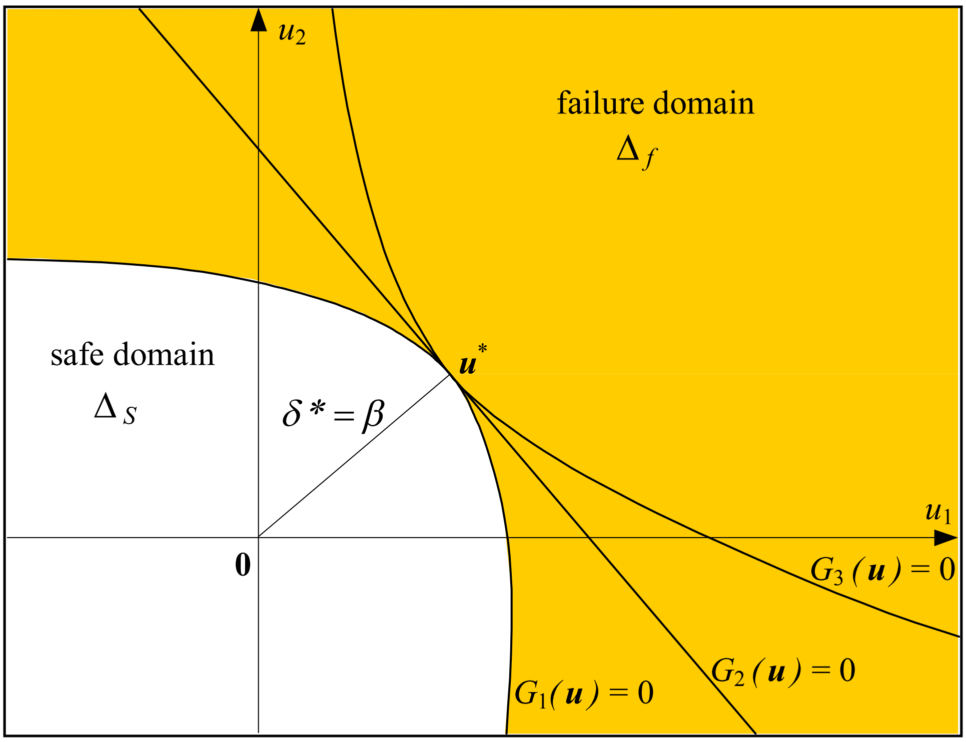

Figure 3.1 Definition of safe and failure regions and reliability index

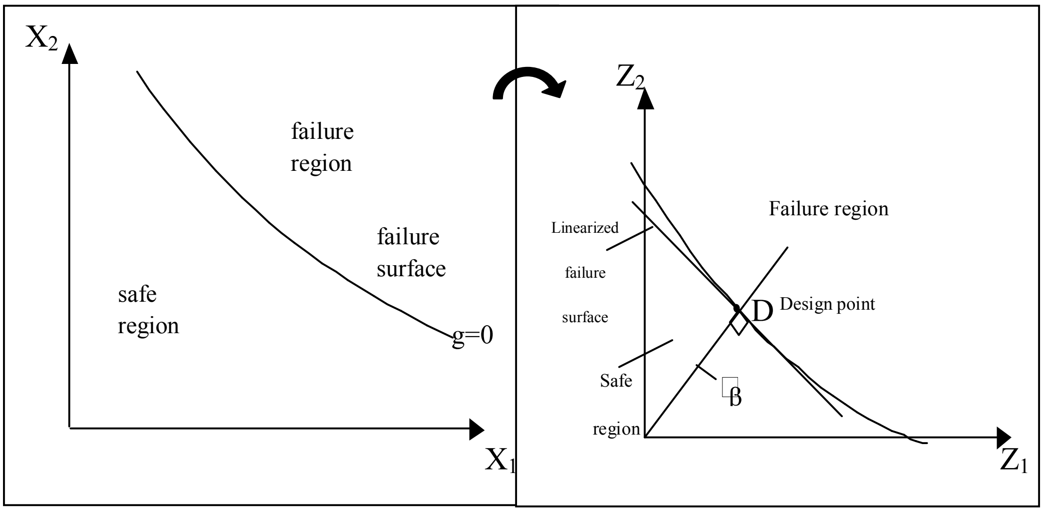

Figure 3.2 Definition of the Hasofer and Lind reliability index, β_{HL}

The iterative procedure requires to solve a set of (2n+1) simultaneous equations with (2n+1) unknowns: β, α_1, α_2,........, αn , $z_1^*, z_2^*, \dots, z_n^*$ where

$$\alpha_i = \frac{-\frac{\partial g}{\partial Z_i}}{\sqrt{\sum_{k=1}^{n}(\frac{\partial g}{\partial Z_k})^2}} \tag{3.6}$$

evaluated at design point

$$\sum_{i=1}^{n}(\alpha_i)^2 = 1 \tag{3.7}$$

$$z_i^* = \beta\alpha_i \tag{3.8}$$

g $(z_1^*, z_2^*, \dots, z_n^*)$ =0 is a mathematical statement of the requirement that the design point is on the failure boundary.

There are two alternative procedures for performing the iterative analysis: the simultaneous equation procedure and the matrix procedure. The steps in the simultaneous equation procedure are as follows:

1) Formulate the limit state function and appropriate parameters for all random variables involved.

2) Express the limit state function in terms of reduced variates Z_i

3) Express the limit state function in terms of β and α_i

4) Calculate the α_i values.

5) Conduct the initial cycle: Assume numerical values of β and all α_i, noting that the α_i values must satisfy $\sum_{i=1}^{n}(\alpha_i)^2 = 1$

6) Use the numerical values of β and α_i on the right-hand sides of the equations formed in steps 3 and 4 above.

7) Solve the n+1 simultaneous equations in Step 6 for β and α_i

8) Go back to step 6 and repeat. Iterate until the β and α_i values converge.

The matrix procedure consists of the following steps:

1) Formulate the limit state function and appropriate parameters for all random variables X_i (i=1, 2,....., n) involved.

2) Obtain an initial design point $\{x_i^*\}$ by assuming values for n-1 of the random variables X_i. (Mean values are often a reasonable choice).

3) Solve the limit state equation g=0 for the remaining random variable. This ensures that the design point is on the failure boundary.

4) Determine the reduced variates $\{z_i^*\}$ corresponding to the design point $\{x_i^*\}$ using

$$z_i^* = \frac{X_i^* - \mu_{X_i}}{\sigma_{X_i}} \tag{3.9}$$

5) Determine the partial derivatives of the limit state function with respect to the reduced variates. For convenience, a column vector {G} as the vector whose elements are these partial derivatives multiplied by -1 is defined.

$$\{G\} = \begin{Bmatrix} G_1 \\ G_2 \\ \cdot \\ \cdot \\ \cdot \\ G_n \end{Bmatrix} \text{ where } G_i = -\frac{\partial g}{\partial Z_i} \text{ evaluated at the design point} \tag{3.10}$$

6) Calculate an estimate β using the following formula:

$$\beta = \frac{\{G\}^T\{z^*\}}{\sqrt{\{G\}^T\{G\}}} \tag{3.11}$$

Where

$$\{z^*\} = \begin{Bmatrix} z_1^* \\ z_2^* \\ \cdot \\ \cdot \\ \cdot \\ z_n^* \end{Bmatrix} \tag{3.12}$$

The superscript T denotes the transpose.

7) Calculate a column vector containing the sensitivity factors using

$$\{\alpha\} = \frac{\{G\}}{\sqrt{\{G\}^T\{G\}}} \tag{3.13}$$

8) Determine a new design point in reduced variates for n-1 of the variables using

$$z_i^* = \beta\alpha_i \tag{3.14}$$

9) Determine the corresponding design point values in original coordinates for the n-1 values in step 7 using

$$x_i^* = \mu_{x_i} + z_i^*\sigma_{x_i} \tag{3.15}$$

10) Determine the value of the remaining random variables by solving the limit state function g=0.

Repeat steps 3 to 9 until β and the design point $\{x_i^*\}$ converge.

In the simultaneous equation procedure, the assumption on the reliability index parameter β is the starting point of the iteration. Since the procedure starts with an assumption, the procedure accompanies more uncertainty than the matrix procedure where the progress is set up to calculate β using given information on all the parameters of the limit state equation.

In this study, matrix procedure is preferred to perform the iterative analysis to find out the reliability thus the probability of failure of rubble-mound coastal defense structures, since the number of parameters used in the performance function is high and the uncertainty of the iteration procedure is lowered by minimizing the assumptions.

3.5 Rackwitz-Fiessler Procedure (Equivalent Normal Distributions)

If some or all of the random variables are non-normally distributed, β_{HL} can still be used but an extra transformation of the non-normal basic variables into normal basic variables must be performed before β_{HL} can be determined. A commonly used transformation is based on the substitution of the non-normal distribution of the basic variable X_i by a normal distribution in such a way that the density and distribution functions f_{Xi} and F_{Xi} are unchanged at the design point (Nowak and Collins, 2000). If the design point is given by x_1^d, x_2^d, …, x_n^d , then the transformation reads:

$$F_{Xi}(x_i^d) = \Phi\left(\frac{x_i^d - \mu'_{Xi}}{\sigma'_{Xi}}\right) \tag{3.16}$$

$$f_{Xi}(x_i^d) = \frac{1}{\sigma'_{Xi}}\varphi\left(\frac{x_i^d - \mu'_{Xi}}{\sigma'_{Xi}}\right) \tag{3.17}$$

where:

F_{Xi}	: cumulative distribution function of the non-normal variate
f_{Xi}	: probability density function of the non-normal variate
Φ	: cumulative distribution function of the standard normal variate
φ	: probability density function of the standard normal variate
μ'_{Xi}	: mean of the transformed normal distribution.
σ'_{Xi}	: standard deviation of the transformed normal distribution.

By using Equations given above the mean and the standard deviation of the fitted normal distribution can be obtained as:

$$\sigma^{N}{}_{Xi} = \frac{\varphi(\Phi^{-1}(F_{Xi}(x_i^d)))}{f_{Xi}(x_i^d)} \tag{3.18}$$

$$\mu^{N}{}_{Xi} = x_i^d - \Phi^{-1}(F_{Xi}(x_i^d))\sigma^{N}{}_{Xi} \tag{3.19}$$

$F_{Xi}(x_i^d) = \Phi\left(\dfrac{x_i^d - \mu'_{Xi}}{\sigma'_{Xi}}\right)$ can also be written as:

$$F_{Xi}(x_i^d) = \Phi\left(\frac{x_i^d - \mu'_{Xi}}{\sigma'_{Xi}}\right) = \Phi(z_i^d) = \Phi(\beta_{HL}\alpha_i) \tag{3.20}$$

Solving with respect to x_i^d gives

$$x_i^d = F_{x_i}^{-1}[\Phi(\beta_{HL}\alpha_i)] \tag{3.21}$$

where;

$$\alpha_i = \frac{-(\partial g / \partial zi)}{\left[\sum_{i=1}^{n} (\partial g / \partial z)^2\right]^{1/2}} \tag{3.22}$$

are the direction cosines along the axes z.

The basic idea behind this procedure is to calculate the equivalent normal values of the mean and standard deviation for each non-normal random variable.

After performing calculations above, the outcome of the will then is the probability of failure in a 1-year period, P_f *(1 year)*. If the failure events of each year are assumed **independent** (this assumption is valid throughout this study) for all variables then the failure probability in *T* years is

$$P_f^T = 1 - \left(1 - P_f\right)^T \tag{3.23}$$

This assumption simplifies the probability estimation for rubble mound breakwaters where it is reasonable to assume failure events as independent such as rubble-mound stone armor stability and will help us to make comments on the reliability, probability of failure, damage levels etc. during the construction period and lifetime of the project. These calculations will be helpful to visualize the reliability of constructed projects, since it will be able to evaluate the reliability within a given time domain.

3.6 Failure Mode Response Functions

Wave forces acting on a rubble-mound slope can cause armor unit movement. This is called hydraulic instability. Hydraulic stability failure mode will be employed throughout this study and the model that is developed, as the principle failure mode.

The response functions which are mostly employed for the design of front face slope or another saying the armor stability of rubble mound breakwaters are Hudson and Van der Meer equations.

3.6.1 Hudson Failure Function

The failure function can be obtained from Hudson equation (CEM, 2003), as a function of its basic variables: g_1=f (H_d, Y_1, D_{n50}, K_D, Δ, θ). In the presented reliability model, Hudson failure function is generated as follows:

$$g_{1=}Y_1 \Delta D_{n50}(K_D cot\theta)^{1/3} - H_d \tag{3.24}$$

where, in general:

g_i: Failure functions $g\leq0$: failure and $g>0$: no failure.

Y_i: Uncertainty variables signifying the uncertainty of the equation.

The uncertainty of the design wave height is implicitly considered in the failure function by using the total variation coefficient (δ_T) of the wave height distribution.

3.6.2 Van der Meer Failure Function

In recent studies carried out in reliability studies in coastal engineering field, Van der Meer failure (response) function is mostly preferred since it is based on the dynamic stability logic where wave period and duration of storms are taken into consideration. Therefore, in this study Van der Meer limit state function will be used in order to accomplish reliability based design of rubble-mound coastal defense structures.

In the presented reliability model, failure functions are obtained from Van der Meer equations as a function of basic variables (CEM, 2003):

$$g=f\,(H_s, D_{n50}, \Delta, \xi z, cot\alpha, S, N, P, Y)$$

The failure function of plunging waves (g_1) is:

$$g_{1=}Y_1 S^{0.2} P^{0.18} cot\alpha^{0.5} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{-0.25} N^{0.1} \tag{3.25}$$

The failure function for surging waves (g_2) is:

$$g_{2=}Y_2 S^{0.2} P^{-0.13} cot\alpha^{(0.5-P)} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{0.5P} N^{0.1} \quad (3.26)$$

Surging waves occur along extremely steep shores . The breaker zone much of the wave energy is back out at deeper water. The curling top is characteristics of the plunging type of waves. When it breaks much energy is dissipated in turbulence; little is reflected back to sea (Ergin, 2009)

Table 3.2 Limit State Functions

Equation	Limit State Functions
Hudson	$g = Y\Delta D_{n50}(K_D cot\theta)^{1/3} - H_d$
Van der Meer (plunging)	$g_{1=}Y_1 S^{0.2} P^{0.18} cot\alpha^{0.5} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{-0.25} N^{0.1}$
Van der Meer (surging)	$g_{2=}Y_2 S^{0.2} P^{-0.13} cot\alpha^{(0.5-P)} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{0.5P} N^{0.1}$
Toe berm	$g_3 = Y_3 \Delta D_{n50} \left(\frac{h_t}{h_s}\right)^{1.43} - H_s$

Toe berm stability is affected by wave height, water depth at the top of the toe berm, width of the toe berm, and block density. However, wave steepness does not appear to be a critical toe berm stability parameter.

Model tests with irregular waves indicate that the most unstable location is at the shoulder between the slope and the horizontal section of the berm. The instability of a toe berm will trigger or accelerate the instability of the main armor. Studies on toe berm stability showed that moderate toe berm damage has almost no influence on armor layer stability, whereas high damage of the toe berm severly reduces the armor layer stability (CEM, 2003). Therefore, in practice it is economical to design toe berms that allow for moderate damage.

In this study, Van der Meer failure functions for surging and plunging waves are used as the response functions. Toe berm failure function is not evaluated during the works. It will be implemented in the development stage of the study due to its effect on the economical considerations to the design methodology of rubble-mound coastal defense structures.

3.7 Extreme Wave Height Distributions

In reliability based risk assessment models identification of the basic variables of limit state function and development of appropriate probability distributions are very critical steps. The physical uncertainty in a basic random variable is represented by adopting a suitable probability distribution function. A proper decision on the selection of distribution type is therefore vital.

Wave height is the most important parameter in the design of coastal structures. In this part, frequently used long term extreme probability distributions of wave height are summarized.

In coastal engineering design works, to apply various distribution functions to a set of sample wave height data and select the best fitting one as the most probable distribution function is the widely used technique to apply extreme wave analysis. Four candidate functions employed in this study to make extreme wave analysis are briefly described including their probability density functions and cumulative density functions (Goda, 2000).

3.7.1 Fisher-Tippett type I (Gumbel) distribution

In probability theory and statistics, the Gumbel distribution is used to model the distribution of the maximum (or the minimum) of a number of samples of various distributions. It is useful in predicting the chance that an extreme wind speed, wave height, extreme earthquake, flood or other natural disaster will occur.

Cumulative distribution function;

$$F(x) = exp\left[-\exp\left(-\frac{x-B}{A}\right)\right] \tag{3.27}$$

Probability density function

$$f(x) = \frac{1}{A} exp\left[-exp\left(-\frac{x-B}{A}\right)\right] exp\left(-\frac{x-B}{A}\right) \tag{3.28}$$

where;

x is a random variable
A> 0 is the *scale parameter* and
B > 0 is the *location parameter* of the distribution.

3.7.2 Fisher-Tippett type II distribution

Cumulative distribution function

$$F(x) = exp\left[-\left(1 + \frac{x-B}{kA}\right)^{-k}\right] \tag{3.29}$$

Probability density function

$$f(x) = \frac{1}{A} exp\left[-\left(1 + \frac{x-B}{kA}\right)^{-k}\right]\left(-\frac{x-B}{kA}\right)^{-k-1} \tag{3.30}$$

where;

x is a random variable
k > 0 is the *shape parameter*
A > 0 is the *scale parameter* and
B > 0 is the *location parameter* of the distribution.

3.7.3 Weibull distribution

In probability theory and statistics, the Weibull distribution is a continuous probability distribution. The probability density function of a Weibull random variable x is:

Cumulative distribution function

$$F(x) = 1 - exp\left[-\left(\frac{x-B}{A}\right)^{k}\right] \tag{3.31}$$

Probability density function

$$f(x) = \frac{k}{A} exp\left[-\left(\frac{x-B}{A}\right)^{k}\right]\left(\frac{x-B}{A}\right)^{-k-1} \tag{3.32}$$

where;

x is a random variable
k > 0 is the *shape parameter*
A > 0 is the *scale parameter* and

$B > 0$ is the *location parameter* of the distribution.

The Weibull distribution is related to a number of other probability distributions; in particular, it interpolates between the exponential distribution ($k = 1$) and the Rayleigh distribution ($k = 2$).

The Weibull distribution is often used in the field of life data analysis due to its flexibility—it can mimic the behavior of other statistical distributions such as the normal and the exponential. If the failure rate decreases over time, then $k < 1$. If the failure rate is constant over time, then $k = 1$. If the failure rate increases over time, then $k > 1$.

3.7.4 Log-normal distribution

In probability theory, a log-normal distribution is a probability distribution of a random variable whose logarithm is normally distributed. If Y is a random variable with a normal distribution, then $X = \exp(Y)$ has a log-normal distribution; likewise, if Y is log-normally distributed, then $\log(Y)$ is normally distributed. (The base of the logarithmic function does not matter: if $\log_a(Y)$ is normally distributed, then so is $\log_b(Y)$, for any two positive numbers a, $b \neq 1$.)

Cumulative distribution function

$$F(x) = \frac{1}{2} erfc\left[-\left(\frac{lnx - B}{A\sqrt{2}}\right)\right] \tag{3.33}$$

Probability density function

$$f(x) = \frac{1}{Ax\sqrt{2\pi}} exp\left[-\frac{(lnx - B)^2}{2A^2}\right] \tag{3.34}$$

In this study, these four candidate functions discussed above are employed for the distribution of design wave height. The user is guided to select between these distributions as starting point of the model.

3.8 The ARIN Design Model

The model ARIN (Assessment of Reliability in Numeric Modeling of Rubble-Mound Breakwaters) is developed for the application of reliability-based design algorithm in sections 3.2-3.5 to the design of rubble-mound coastal structures. The tool is written in Delphi language. This tool can be executed in any type of computer since the user can operate the tool easily for the design from execution window. The model is computerized and built in a way that the user does not have to remember steps or formulas. The only thing the user does is to select the appropriate design parameters and their characteristic values. In addition, characteristic values of some design parameters are predefined during the development of the model and the user is given these values in the execution window (starting window). The starting window is presented in Figure 3.3.

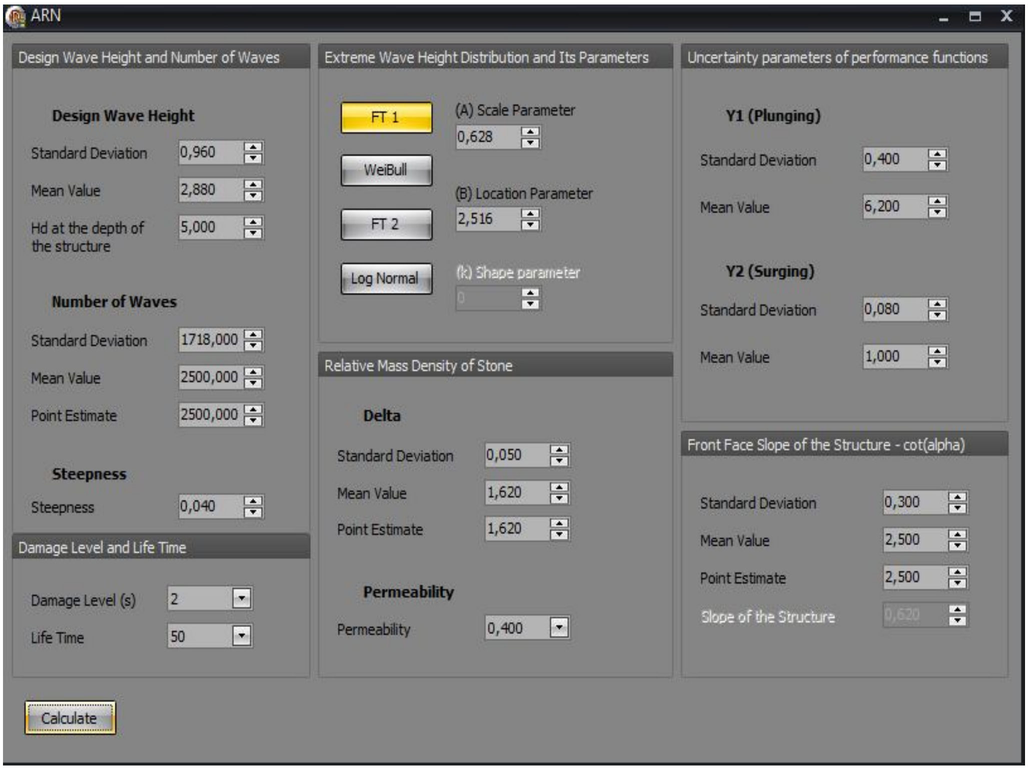

Figure 3.3 Starting Window of Model

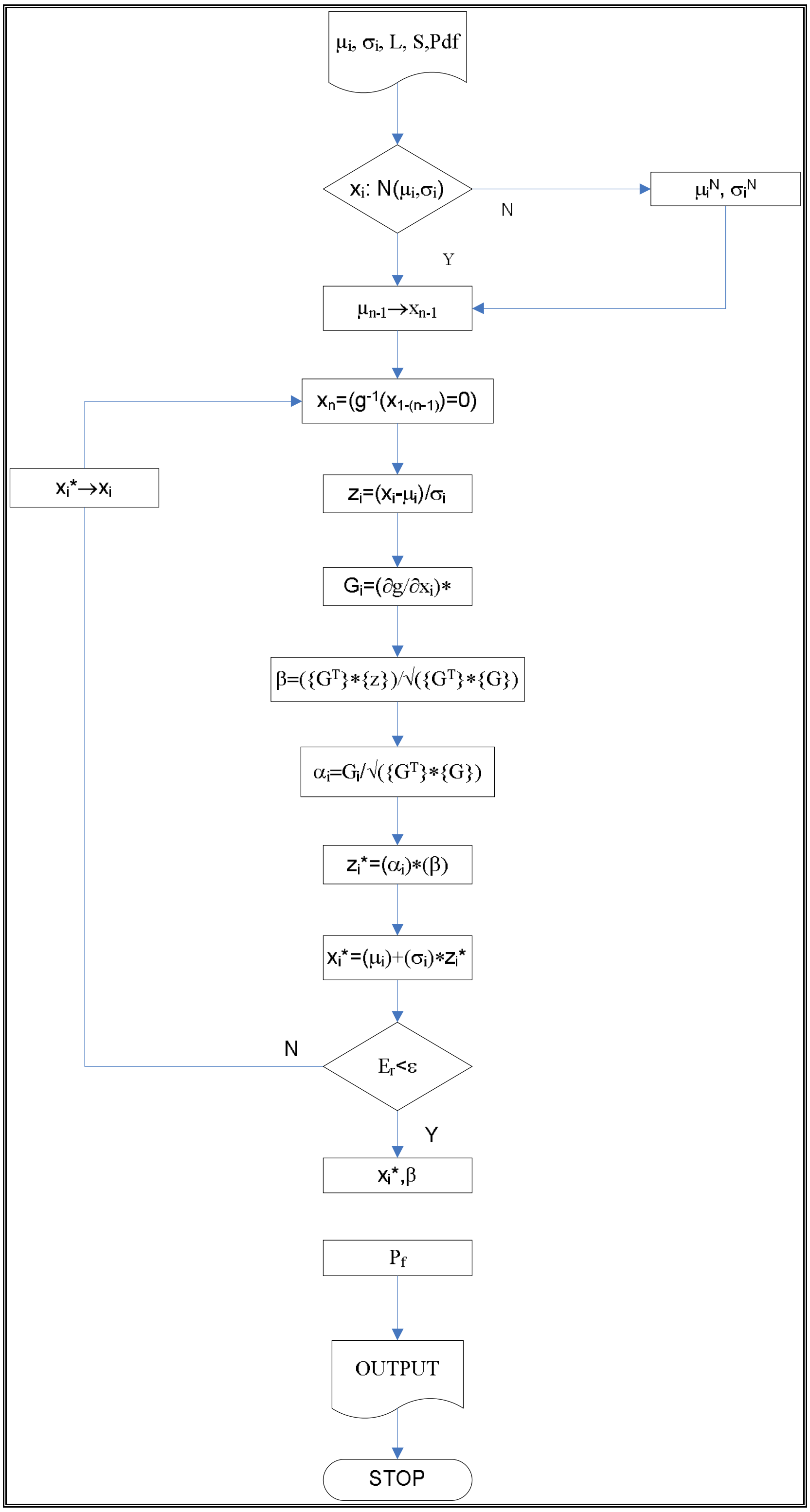

Figure 3.4 The iterative algorithm of the ARIN model

3.8.1 Program Scheme

The program is written in a form that the user gives the input values and gets the preferred output data and graphics for the design of rubble-mound coastal defense structures. Thus, model is developed in user friendly logic. In addition, the output values and graphics can be easily exported in any place of the user's computer in order to give the user permission to examine the output data.

The input parameters of the tool are the mean and standard deviation values of the design parameters in the response function of Van der Meer, lifetime of the structure (LT) and damage value (S). Damage values and lifetimes are selected from the starting window between the values previously entered during the composition of the code.

The user is directed to make selections for damage level and lifetime for S=2, 6, 10, 16 and 18 and LT=1, 4,25,30,50 and 100 years. These values are carefully selected according to the studies made on this field and tables presented in CEM, 2003. If the designer aims to analysis an existing structure with lifetime not included in the defined lifetimes at the model's starting window, it is recommended to select 1 year lifetime and then to calculate probability of failure of the structure for example for 40 years by using Equation 3.23. Thus, LT=1 year alternative in the design tool execution window is given for this purpose.

Starting window presented in Figure 3.3 shows the basic design parameters that the user will enter and the buttons will be directed to select.

The importance of selection of extreme wave height distributions has been discussed in section 3.7. This tool is developed in a way that the user must select one of the extreme wave height distributions and its scale, location and shape parameters. Some of the distributions do not have shape parameter in the probability and cumulative density functions. Therefore, when one of these distributions is selected the user is permitted to write on only the spaces for necessary parameters and the other unnecessary buttons are frozen. Therefore, personal mistakes on the entrance of the distribution parameters are reduced.

An example of output window is shown in Figure 3.5. There are different selection alternatives between outputs as seen from figure. The output of log button is also shown in Figure 3.6 to visualize the optional form of output window.

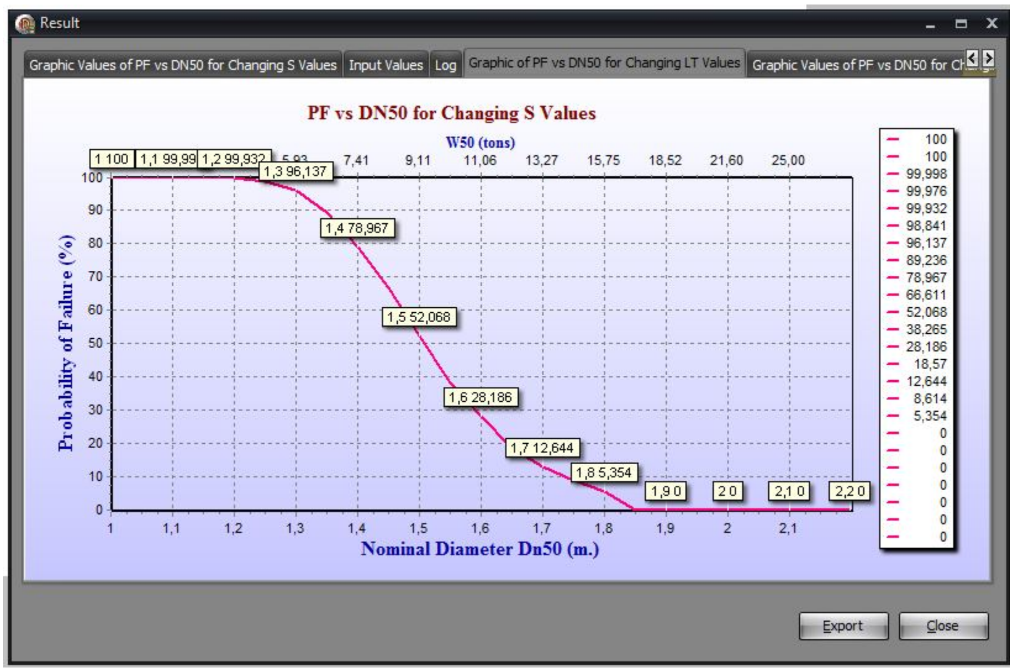

Figure 3.5 An example of a graphical output window from ARIN

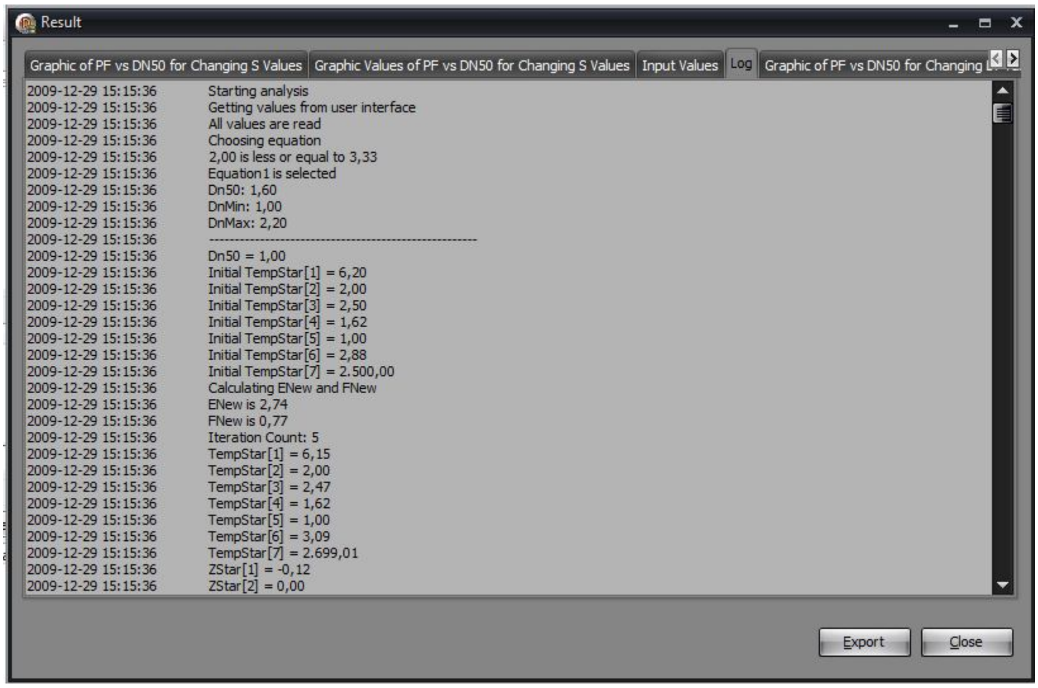

Figure 3.6 An example of a tabular output window from ARIN

In the design tool, mathematical formulation of the reliability analysis is executed by the iterative algorithm illustrated in Figure 3.4. The algorithm is given below as:

1) The input parameters of the design variables of the response function are selected. These input parameters are the mean and standard deviations thus characteristic values of the design parameters.

2) Parameters having distribution functions different from normal (Gaussian) distribution are selected in order to calculate the equivalent normal characteristic values $(\pi_{xi}^{N}, \sigma_{xi}^{N})$ of the non-normally distributed parameters by using Equations 3.18 and 3.19.

3) Initial design point $\{x_i^*\}$ is obtained by assuming mean values for n-1 of the random variables X_i. Then the $(x_i^*)_n$ value is calculated by equating the limit state function $g((x_i)_1, \ldots, (x_i)_{n-1}) = 0$.

4) The reduced variates $\{z_i^*\}$ corresponding to the design point $\{x_i^*\}$ are determined by using Equation 3.9.

5) The partial derivatives of the limit state function with respect to the reduced variates are determined by Equation 3.10. For convenience, a column vector {G} as the vector whose elements are these partial derivatives multiplied by -1 is defined.

6) Reliability index β is calculated by utilizing Equation 3.11.

7) Direction cosines are determined by Equation 3.13 satisfying the condition in Equation 3.7 where the summation of squares of direction cosines is equated to 1.0.

8) The new design points in the z-coordinate system for n-1 of the variables are obtained by using the calculated reliability index and direction cosine values in Equation 3.14.

9) Corresponding design point values in original coordinates for the n-1 values are determined by Equation 3.15 using design points in z-coordinate system and characteristic values (μ_{xi},σ_{xi}) of the design parameters.

10) Finally, the value of the remaining random variable is calculated by solving the limit state function g=0.

11) In this iterative procedure, iteration continues until the reliability index β and the design point $\{x_i^*\}$ converge. Absolute relative error is used to determine the convergence and to get the final output values of the algorithm. The absolute relative error (E_r) in the initial design points $\{x_{ia}^*\}$ assumed and $\{x_{ic}^*\}$ calculated values is evaluated as:

$$E_r = \left|\frac{x_{ic}^* - x_{ia}^*}{x_{ic}^*}\right|$$

where;

x_{ic}^* is the calculated design point by the iteration
x_{ia}^* is the previously calculated or assumed design point in the iteration

The absolute error limit is used as 0.001 as proposed in thesis work on reliability of coastal structure subject (Balas C.E, 1999).

12) The annual probability of failure is obtained from Equation 3.3 and exceedance probability of damage levels in the lifetime of the structure is evaluated as the final outcome of the reliability based design algorithm for rubble-mound coastal defense structures.

This reliability-based design model ARIN for rubble-mound coastal defense structures which is aimed to be a first user friendly design tool in reliability-based design that can be easily executed by every user working in coastal engineering field.

Giresun Port (Black Sea), Foça-Leventler Military Port (Aegean Sea) and Sinop Demirciköy Fishery Harbor (Black Sea) are selected as the case studies between several locations within the scope of the research for the application of the model. Case selection is done in a manner that projects in Turkey which have been constructed (Foça-Leventler Military Port), designed to be constructed (Sinop Demirciköy Fishery Harbor) and constructed and damaged (Giresun Port) are studied in the application of numerical reliability-based design model.

3.9 Uncertainty Evaluation of Basic Design Variables

The application of the reliability-based design model and the outcomes are described in this part.

Reliability-based design works with probability distributions of design parameters in order to evaluate the uncertainties. However, it is not always possible to select and also to obtain appropriate probability distributions of these parameters due to the lack of information at coastal engineering field.

Therefore, coefficient of variation is used as the measure of uncertainties of basic parameters of the response functions. Standard deviations of variables are calculated by using variation coefficient values in order to be able to handle the analysis with reduced variates.

Uncertainties in the limit state functions are subdivided into two groups considering the load and resistance variables separately. Load and resistance uncertainties are described in this subsection as follows:

3.9.1 Uncertainties in load parameters

Load parameters of the Van der Meer limit state functions for surging and plunging breakers tabulated in Table 3.3 are design wave height, average number of waves in storms and the deep water wave steepness. These are described in detail in this chapter. Therefore, the final outcomes of uncertainties will be presented.

Table 3.3 Response functions for rubble-mound breakwaters for Van der Meer equation

Equation	Limit State Functions
Van der Meer (plunging)	$g_{1=}Y_1 S^{0.2} P^{0.18} cot\alpha^{0.5} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{-0.25} N^{0.1}$
Van der Meer (surging)	$g_{2=}Y_2 S^{0.2} P^{-0.13} cot\alpha^{(0.5-P)} \Delta D_{n50} - H_s \left(\frac{H_s}{L_z}\right)^{0.5P} N^{0.1}$
Toe berm	$g_3 = Y_3 \Delta D_{n50} \left(\frac{h_t}{h_s}\right)^{1.43} - H_s$

The sources of uncertainties in design wave height originate from wind data, near shore calculations, prediction model and statistical model used for analysis of wind data.

The variation coefficient values for model predictions and near shore calculations are given in Table 3.1 as δ_{MP}= (10-20) % and δ_{NS}= (15-35) %, respectively.

Uncertainties in extreme value statistics δ_{ES} for all cases are calculated in the extreme wave analysis described in Appendix A.

As a result ranges of total coefficient of variations for the cases are obtained using:

$$\delta_{H_s} = \sqrt{\delta_{MP}^2 + \delta_{NS}^2 + \delta_{ES}^2} \quad (3.35)$$

This calculation assumes that the variations are distributed normally. In addition, the pivoting sources of uncertainty thus the coefficient of variation value of design wave height are stated as near shore calculation and prediction models according to the uncertainty calculations done for several case studies unless the wave data shows large variations from mean values i.e. there is lack of information of the data collected from field.

Mean and standard deviation calculations of number of waves parameter is done using Taylor series transformation. Mean value is obtained by the division of storm duration to the mean value of the significant wave period if detailed data cannot be found. Coefficient of variation is used as δ=30% as proposed by Ergin A. and Balas. C.E.

In this study, deep water wave steepness design parameter is not treated as an uncertainty parameter since it does not give large values of sensitivity factors on the reliability analysis according to the calculations carried for several cases.

3.9.2 Uncertainties in resistance parameters

The resistance parameters in Van der Meer response functions for surging and plunging breakers are nominal diameter of armor unit, the slope of front armor layer and the permeability coefficient. All of these parameters directly affect the stability of the structure.

Nominal diameter of armor units is described in a range of uniform distribution, since the variation between upper and lower limits of the range is not known. Coefficient of variation is calculated as:

$$\delta_{D_{n50}} = \frac{1}{\sqrt{3}}\left(\frac{D_u - D_l}{D_u + D_l}\right) \tag{3.36}$$

where, D_u and D_l stands for upper and lower limits of the armor unit diameter range, respectively. The coefficient of variation is therefore evaluated as δ_{Dn50}=4% by using Equation 3.36.

Angular deviation in structure slope cotθ is directly related to the talent and supervision of the worker that construct the breakwater and the occurrence of damage of the structure resulting in coefficient of variation ranging from $\delta_{cot\theta}$= (5-10) % to $\delta_{cot\theta}$= 20% (Özhan and Yalçıner, 1995). The variation range is described by upper triangular distribution in order to include minimum and maximum values in case of damage. Expected value of the variation coefficient range of face slope is:

$$\mu_{\delta_{cot\theta}} = \frac{(\delta_{cot\theta})_u + (\delta_{cot\theta})_l}{3} \tag{3.37}$$

where, $(\delta_{cot\theta})_u$ and $(\delta_{cot\theta})_l$ are upper and lower limits of the variation coefficient range, respectively. The angular deviation is modeled by uniform distribution giving variation coefficient of $\delta_{cot\theta}$= 12%.

The estimation of permeability P for a particular structure must, more or less is based on engineering judgment. Van der Meer suggests using P as 0.40 and 0.42 for rubble-mound structures (Van der Meer, 1988a). Therefore, permeability is given as a deterministic value P=0.40 and 0.42 for the numerical reliability-based model calculations.

To conclude, it is possible to observe the high number of uncertainties in the design parameters of limit state function stressing on the importance of reliability-based thus probabilistic design of coastal defense structures that inheres irreducible uncertainties since the facing loading parameters are forces of nature. It is summarized in Table 3.5.

In addition, the uncertainties in limit state functions for surging and plunging waves are introduced by giving coefficients 6.2 and 1.0 in the equations as normally distributed with a certain standard deviation. These uncertainty variables are given in Table 3.4.

Table 3.4 Uncertainty variables of response functions

Uncertainty Variable (Y_i)	Mean Value (μ_{Yi})	Standard deviation (σ_{Yi})	Variation coefficient (δ_{Yi}) %
Plunging of Van der Meer (Y_1)	6.2	0.4	6.5
Surging of Van der Meer (Y_2)	1.0	0.08	8

Table 3.5 Basic Random Variables of Limit State Equations and their Probabilistic Values

Basic Variable (X_i)	Distribution	Mean (μ_{xi})	Standard Deviation (σ_{xi})	Variation Coefficient (δ %)
Y_1	Normal	6.2	0.4	6.5
$(D_{n50})_{Hudson}$	Normal	Will be found according to the case		
Y_2	Normal	1.0	0.08	8
$(D_{n50})_{VanD.Meer}$	Normal	Will be found according to the case		
H_s	Changes according to cases	Will be found according to the case		
H_s/L_z	Normal	Will be found according to the case		
Δ	Normal	1.62	0.05	3
P	Normal	0.42 and 0.40		
N	Normal	Will be found according to the case		0.30 (in the case of lack of data)
$\cot\theta$	Normal	Will be found according to the case		0.12

where,

Y_1: stochastic variables signifying the uncertainty of the equation

H_s: design significant wave height

θ: structure front face slope angle

L_z: deep water wave length

D_{n50}: nominal diameter of armor unit

Δ: relative density of the armor unit

P: permeability coefficient of the structure

N: number of waves and

S: damage level

Damage level (S) values according to the rock and face slope characteristics of the structure are presented in Table 3.6 (CEM, 2003)

Table 3.6 Damage level by S for two-layer armor

Unit	Slope	Initial Damage	Intermediate Damage	Failure
Rock	1:1.5	2	3-5	8
Rock	1:2	2	4-6	8
Rock	1:3	2	6-9	12
Rock	1:4-1:6	3	8-12	17

The selection of appropriate lifetime L for the structure is a very important step in the design of coastal structures. Thus, selection of design lifetime has to be made carefully not to have over-design or under-design. It is recommended to make selection of the design lifetime according to table 3.7 which is prepared for the general utilization of Level I methods by the Maritime Works Recommendations (ROM, 1990) for the preliminary design.

Table 3.7 Minimum design life of coastal structures

Structure Type			
General Use		Specific Industrial Use	
Risk Level	Design Life (year)	Risk Level	Design Life (year)
I	25	I	15
II	50	II	25
III	100	III	50

The type of the coastal structure is categorized in Table 3.8 according to its use:

1) General use: Permanent structure that is not associated with a particular industrial installation.

2) Specific industrial use: Structures associated with transit transportation of natural resource deposits or particular industrial installation, such as loading platform of a mineral deposit, industrial service facilities.

The risk levels for these structures are defined as follows:

1) Level I: The risk of human life loss or environmental damage is small in case of failure, such as coastal regeneration works, local outfalls, and industrial service installations.

2) Level II: There is a moderate risk of human life loss or environmental damage in case of failure, such as harbors and city outfalls.

3) Level III: There is an elevated risk of human life loss or environmental damage in case of failure, such as flood protection structures for the defense of urban or industrial areas against sea flooding.

It is recommended to make selection of the damage levels according to Table 3.8 and Table 3.9 that are prepared for the general utilization of Level I and Level II methods by the Maritime Works Recommendations (ROM, 1990) for the preliminary design. The maximum admissible failure probabilities for rigid (sea walls, vertical wall breakwaters) and flexible (repairable) coastal structures (rubble-mound breakwaters) are presented in separate tables including benefit-cost ratios in the case of failure.

Table 3.8 The lifetime maximum admissible failure probabilities for flexible structures

Risk Levels			
Level I		Level II	
Benefit/Cost (BC)	Failure Probability (%)	Benefit/Cost (BC)	Failure Probability (%)
BC<5	50	Low	30
5<BC<20	30	Average	20
BC>20	25	High	15

Table 3.9 The lifetime maximum admissible failure probabilities for rigid structures

Risk Levels			
Level I		Level II	
Benefit/Cost (BC)	Failure Probability (%)	Benefit/Cost (BC)	Failure Probability (%)
BC<5	20	Low	15
5<BC<20	15	Average	10
BC>20	10	High	5

It is appropriate to give a short definition of benefit-cost analysis.

A cost benefit analysis finds, quantifies, and adds all the positive factors. These are the benefits. Then it identifies, quantifies, and subtracts all the negatives, the costs. The difference between the two indicates whether the planned action is advisable. The real trick to doing a cost benefit analysis well is making sure you include all the costs and all the benefits and properly quantify them.

Should we hire an additional sales person or assign overtime? Is it a good idea to purchase the new stamping machine? Will we be better off putting our free cash flow into securities rather than investing in additional capital equipment? Each of these questions can be answered by doing a proper cost benefit analysis.

These uncertainty calculations and outputs are used for all case studies and valid throughout the study.

CHAPTER 4

APPLICATION OF THE RELIABILITY-BASED DESIGN MODEL
CASE I: GİRESUN PORT

This study is primarily focused on the application of the developed reliability-based design model ARIN in the design of rubble-mound coastal defense structures which are widely used coastal structures in Turkey due to the availability of natural armor stones and the easiness of maintenance and construction of rubble-mound type of breakwaters.

In this chapter, application of developed model to cross-sections of rubble-mound breakwater in the selected Giresun Port case together with the results of the physical modeling works are given. In addition, implementation and results of reliability-based design model (ARIN) are reported with comparison and discussions between deterministic and probabilistic design approaches.

4.1 Case Study I: Giresun Port

Being a major port in Black Sea Region (Figure 4.1), Giresun Port is in service since 1959. It is protected with a rubble mound breakwater and has 5 quays; for general cargo, for passenger ships, tugboats, fishermen and bulk cargo. The layout of the port is given in Figure 4.2. With its approximate annual loading capacity of 1,800,000 tons, it has an important role in economy of the region. However, the existing main breakwater and the basin of the port have been damaged under the action of destructive storm in 1999. Damage maintenance works and capacity rising works has been carried out within the port.

There have been enormous amount of destruction and loss in the region beyond and at the inner parts of the port area which are protected by rubble mound coastal defense structure. A forecast for the economical consequences was done by Yüksek Ö. et. al. (2000) and it is found out that the total economical loss is about 3.5 million dollars in Giresun region. The reasons behind such an amount of destruction have been an important argument within the coastal researchers from the 1999 storm till now.

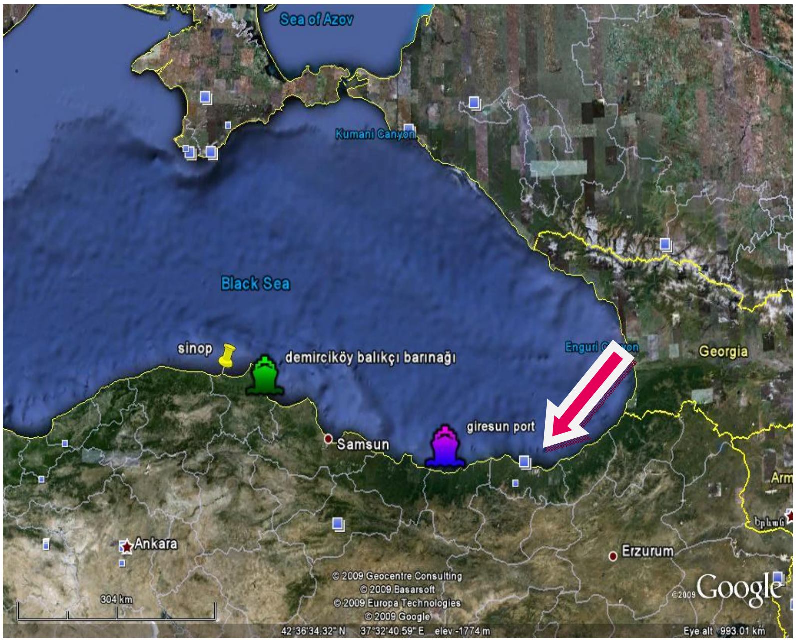

Figure 4.1 Location of Giresun Port

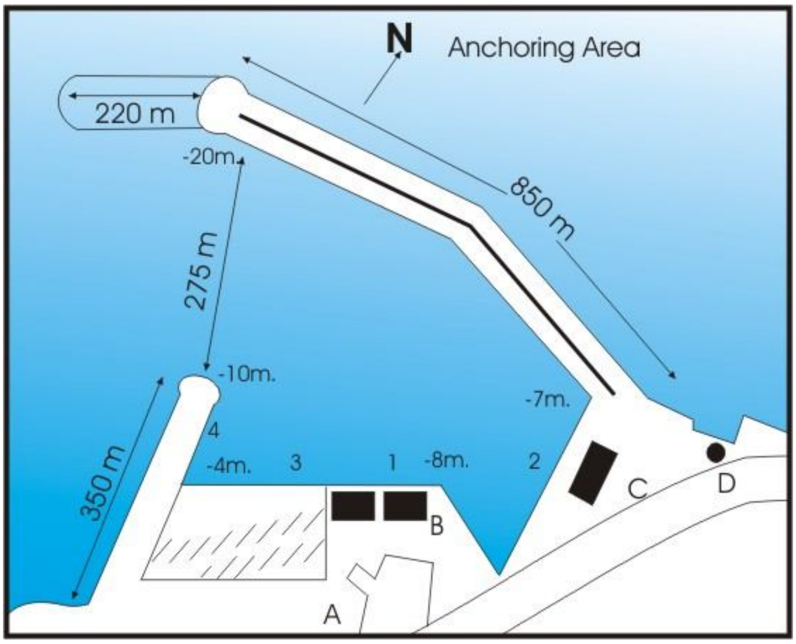

Figure 4.2 Layout of Giresun Port

The major reasons behind the destructions along the coastal regions during the 1999 storm occurred at Black Sea coasts especially at Giresun, Trabzon and Rize districts have been searched by so many people. It has been found that the pivotal reason is the selection of the design wave height for the coastal defense structures of the region and the lack of maintenances on the small damages occurred during the lifetimes. Most of the rubble mound coastal defenses structures along the costs of that region are designed with waves of 50 years return periods. From the wind and wave data collected form meteorological stations around Giresun it is clearly reported that the wave with a 50 years return period was occurred during the storm. The given cross-section of Giresun port presented in Figure 4.3 (drawn in 1959) shows that the weights of armor units used at the front armor layer has a mean of 13.5 tons. In these years, Hudson deterministic design scheme had been used for the preliminary designs. However, it is computed to be nearly 23 tons as the armor unit weight for the breaking condition as described in forthcoming subchapters. This calculation also summarizes the tremendous destructive effects on the structure.

The total damage occurred in primary rubble-mound breakwater of the port was 60% by volume according to the replaced armors and damaged cross-section details (Kılıçoğlu, Aral and Yalçıner, 2004).

Some researches based on physical modeling have been carried out at the Middle East Technical University Coastal and Harbor Engineering laboratory on rubble-mound coastal structures along the Black Sea coasts of Turkey (Fışkın G. (2004) and Özler B. (2004)). The model studies were carried out namely on the cross-section of coastal defense structures (rubble-mound breakwaters) for the Eastern Black Sea Highway Project. In these studies, Ordu-Giresun area had been selected as the pilot area to apply the model investigations. The major aim was to apply experimental studies to improve the previously proposed cross-sections and to obtain the most economical, safe and serviceable cross-section.

The outcomes of the experimental works proved that some fatal design mistakes such as design wave height selection, armor unit weight calculations etc. were done for the coastal structures along the region. Therefore, design studies for the Giresun region especially for the Giresun port and Eastern Black Sea Coastal highway project have become important issues in coastal engineering practices.

Since, the reliability-based design model in this study is primarily developed for the evaluation of safety levels of rubble-mound structures during the lifetime of the structures, this model can also be used for the determination of reliability thus safety levels of existing rubble-mound coastal defense structures. In this case, Giresun Port is treated from reliability point of view in order to examine new design methodology including variability thus uncertainties of design parameters. This model provides a valuable tool in the design and safety evaluations of structures for which the consequences of failure results are large and predicted as irreducible.

4.1.1 Deterministic design

In the model, hydraulic failure mode of the armor layer, which is the displacement of armor stones under the wave action, is studied to compare the effects of failure mode response function derived from Van der Meer equations for surging and plunging waves on the preliminary design of rubble-mound coastal defense structures.

The reliability-based risk assessment model results are compared with the conventional deterministic design procedure and hydraulic model tests. Therefore, the first part of this chapter is dealed with deterministic design.

For Giresun Port, design wave height is obtained from appropriate extreme wave height probability distribution by using highest values of characteristic wave heights selected from storm data records for every year. Extreme wave analysis is carried out as described in Appendix (A). According to the results of extreme wave analysis, deep water design wave is distributed log-normally with $(H_s)_D$= 6.25 m. significant wave height and $(T_s)_D$= 9.83 sec. significant wave period for 50 years return period (Table A.10).

In Figure 4.3, trunk section of the main breakwater in Giresun Port at a water depth of d=20.0 meters at the toe of the structure is presented.

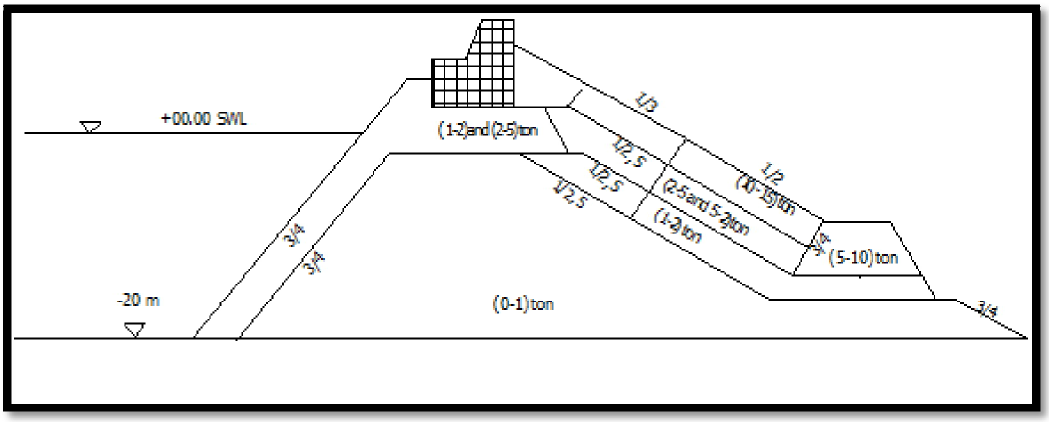

Figure 4.3 Trunk section of the main breakwater in Giresun Port at a water depth of d=20.0 meters at the toe of the structure

4.1.1.1 Design by Hudson equation

The reliability-based design model programmed in this study uses Van der Meer limit state functions to evaluate the safety of rubble mound coastal defense structures for surging and plunging breakers. Since the Hudson design scheme is the mostly used conventional deterministic method of design in Turkey and the design of Giresun Port was carried out in 1959 that is based on Hudson design scheme, this method is additionally evaluated for this case study in order to be able to compare the results with Van der Meer deterministic design methodology.

Deterministic design by Hudson equation starts with the determination of the nominal rock diameter (D_{n50}) thus the armor unit weight (W). In the preliminary design stage, the (0-5%) damage level is selected as no damage criteria since it is a common use in Turkey. The cross-section of the breakwater constitutes of two units of randomly placed rough angular quarry stones.

The most important point in deterministic preliminary design using Hudson equation is the determination of stability coefficient K_D. It should be selected according to the breaking condition of design wave at the toe of the structure, front face slope of the structure and also the material and cross-sectional properties of the structure. When the breaking condition is examined according to the design wave height and construction depth properties, it is found out that the non-breaking condition occurs. Therefore, K_D=4.0 is used as proposed in CEM, 2003.

Hudson equation parameters and their values for the case are tabulated in Table 4.1 as:

Table 4.1 Deterministic design by Hudson equation

Parameter	Unit	Values
Slope of armor layer (cotθ)	-	2.0
Unit weight of armor stone (γ_r)	tons/m^3	2.70
Unit weight of sea water (γ_w)	tons/m^3	1.03
Stability coefficient (K_D)	-	4.0
Damage level (DL)	%	0-5
Placement of rock	-	Random
Quarry stone texture	-	Rough, angular
Section	-	Trunk
Breaking condition	-	Non-Breaking
Design significant wave height $(H_s)_D$	m.	5.73
Nominal rock diameter (D_{n50})	m.	1.77
Median weight of the armor unit (W_{50})	tons	14.90

4.1.1.2 Design by Van der Meer equation

In the second part of the deterministic design nominal rock diameter of the main breakwater of Giresun Port is calculated by using Van der Meer design scheme. In the preliminary design two layered randomly placed quarry stone armor units are selected as the sectional details.

The permeability coefficient is selected as P=0.4 by utilizing the procedure given by Van der Meer (Van der Meer, 1992), in which the wave period, relative dissipation and permeability values are intercorporated.
The initial damage S=2 used in Van der Meer equation for $\cot\theta = 2.0$ conforms the no damage criteria in Hudson as described in Table 3.6. Therefore, in order to make calculations in the same domains damage coefficient is selected as S=2.

Outcomes of the extreme wave analysis resulted in an average 8 hours duration of storms for the selected area. Therefore, by using average period expected value of number of waves is calculated as μ_N=3000 waves. The standard deviation of number of waves is calculated as σ_N=900 assuming coefficient of variation δ=0.30 by expert opinion (Balas and Ergin, 2002).

The results of the calculations are summarized in Table 4.2 by giving the detailed information on design parameters.

Table 4.2 Deterministic design by Van der Meer equation

Parameter	Unit	Values
Slope of armor layer ($\cot\theta$)	-	2.0
Unit weight of armor stone (γ_r)	tons/m^3	2.70
Unit weight of sea water (γ_w)	tons/m^3	1.03
Design significant wave height $(H_s)_D$	m.	5.73
Deep water wave steepness $(H_s)_0/L_0$	-	0.038
Significant wave period (T_s)	sec.	9.83
Mean wave period (T_z)	sec.	7.79
Surf similarity parameter (ξ_z)	-	2.56
Critical surf similarity parameter (ξ_c)	-	3.77
Wave breaking classification	-	Plunging
Average number of waves (μ_N)	-	3000
Permeability coefficient (P)	-	0.4
Placement of rock	-	Random
Damage level (S)	-	2
Nominal rock diameter (D_{n50})	m.	1.86
Median weight of armor units (W_{50})	tons	17.31

In design with Van der Meer equation, larger armor unit weight is obtained for the same damage level used in Hudson design scheme. This difference emanates from the lack of storm duration thus period and number of waves parameter implementation in Hudson equation. Therefore, for locations having long storm durations like Giresun, use of Van der Meer design scheme has to be preferred to use for preliminary deterministic design of coastal structure.

4.2 Reliability-based Design

This part of the study is composed of the determination of nominal diameter of armor unit (D_{n50}) of the trunk section of main breakwater of Giresun Port by using design wave parameters obtained from extreme analysis described in Appendix A. The outcome of the extreme analysis shows that the annual extreme wave data collected from the site for 20 years best fits to Log-normal distribution with significant wave height H_s=6.25 m. and significant wave period T_s=9.83 sec. with a steepness of 0.038.

The analysis is carried out by reliability-based design model ARIN using mathematical modeling in DELPHI project creator based on the Van der Meer limit state functions for the hydraulic stability of the primary armor layer under wave action. This user friendly design tool aims to find out nominal diameter of armor unit thus weight of armor for the described damage level and lifetime and to make visualization for the assessment of the exceedance probabilities of predefined damage levels. All of the graphics presented are in form of original outcome format of the design tool ARIN.

Statistical values of basic variables used in limit state equation are given in Table 4.3 as:

Table 4.3 Statistical values of design variables used in Van der Meer limit state equation of Giresun Port

Basic Variable (X_i)	Distribution	Mean (μ_{xi})	Standard Deviation	Variation Coefficient (δ%)
Y_1	Normal	6.2	0.4	6.5
Y_2	Normal	1.0	0.08	8
$(D_{n50})_{VanD.Meer}$	Normal	2.10	0.084	4
H_s	Log-Normal	3.825	1.137	30
H_s/L_z	Deterministic	0.038	-	-
Δ	Normal	1.62	0.05	3
P	Deterministic	0.42 and 0.40	-	-
N	Normal	2930	879	30
cotθ	Normal	2.00	0.24	12

Log-normal distribution shows a difference from other distributions for mean and standard deviation values due to the difference in algorithm from other extreme type distributions. Therefore, special attention must be given when working with log-normal distribution selected by the method proposed by Goda (Goda, 2000) to use appropriate shape and location parameters resulting in correct mean and standard deviations of the design wave.

The exceedance probabilities for the described damage level in different life time alternatives and exceedance probabilities for different damage levels in predetermined lifetime values are obtained by using parameter uncertainties in the equation.

According to the calculations made simultaneously in ARIN, response function for plunging waves is selected according to the surf similarity parameter range calculations.

In Figure 4.4, exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2 is presented.

Probability of failure given S=10 and lifetime 50 years in original tabular outcome format of the ARIN model is given in Table 4.4.

Figure 4.5 gives exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2.

In Figure 4.6, exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10, 16 and 18 damage levels in LT=50 years lifetime of structure is presented.

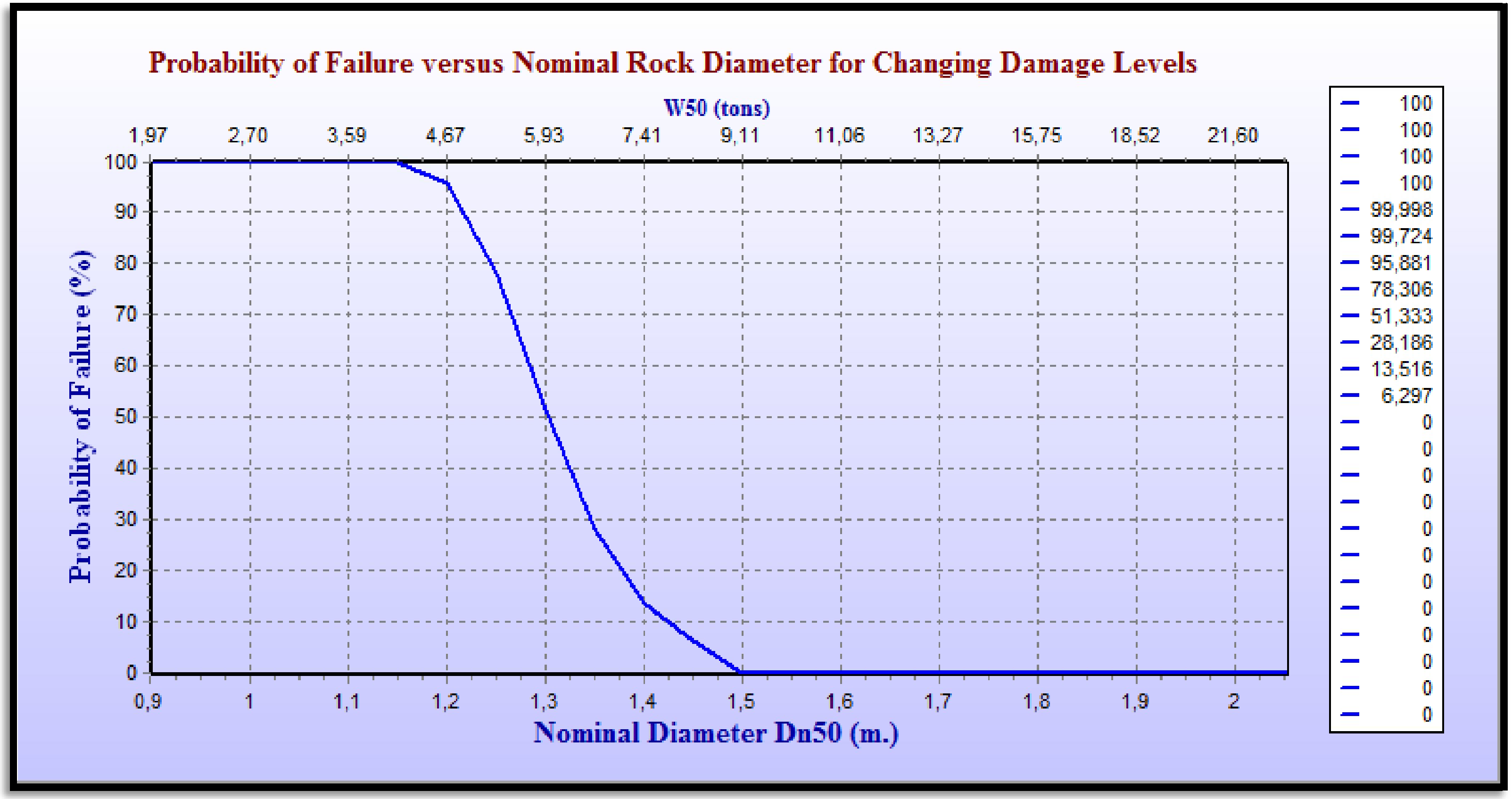

Figure 4.4 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=10

Table 4.4 Probability of failure given S=10 and Lifetime 50 years

D_{n50}	P_f	Beta	P_f Life Time
0.90	22.66	0.75	100.00
0.95	37.83	0.32	100.00
1.00	53.59	-0.10	100.00
1.05	68.79	-0.49	100.00
1.10	80.51	-0.86	100.00
1.15	88.88	-1.22	99.72
1.20	93.82	-1.55	95.88
1.25	96.99	-1.88	78.31
1.30	98.57	-2.19	51.33
1.35	99.34	-2.48	28.19
1.40	99.71	-2.76	13.52
1.45	99.87	-3.03	6.30
1.50	100.00	-3.29	0.00
1.55	100.00	-3.54	0.00
1.60	100.00	-3.78	0.00
1.65	100.00	-4.00	0.00
1.70	100.00	-4.22	0.00
1.75	100.00	-4.43	0.00
1.80	100.00	-4.63	0.00
1.85	100.00	-4.83	0.00
1.90	100.00	-5.02	0.00
1.95	100.00	-5.20	0.00
2.00	100.00	-5.37	0.00
2.05	100.00	-5.54	0.00

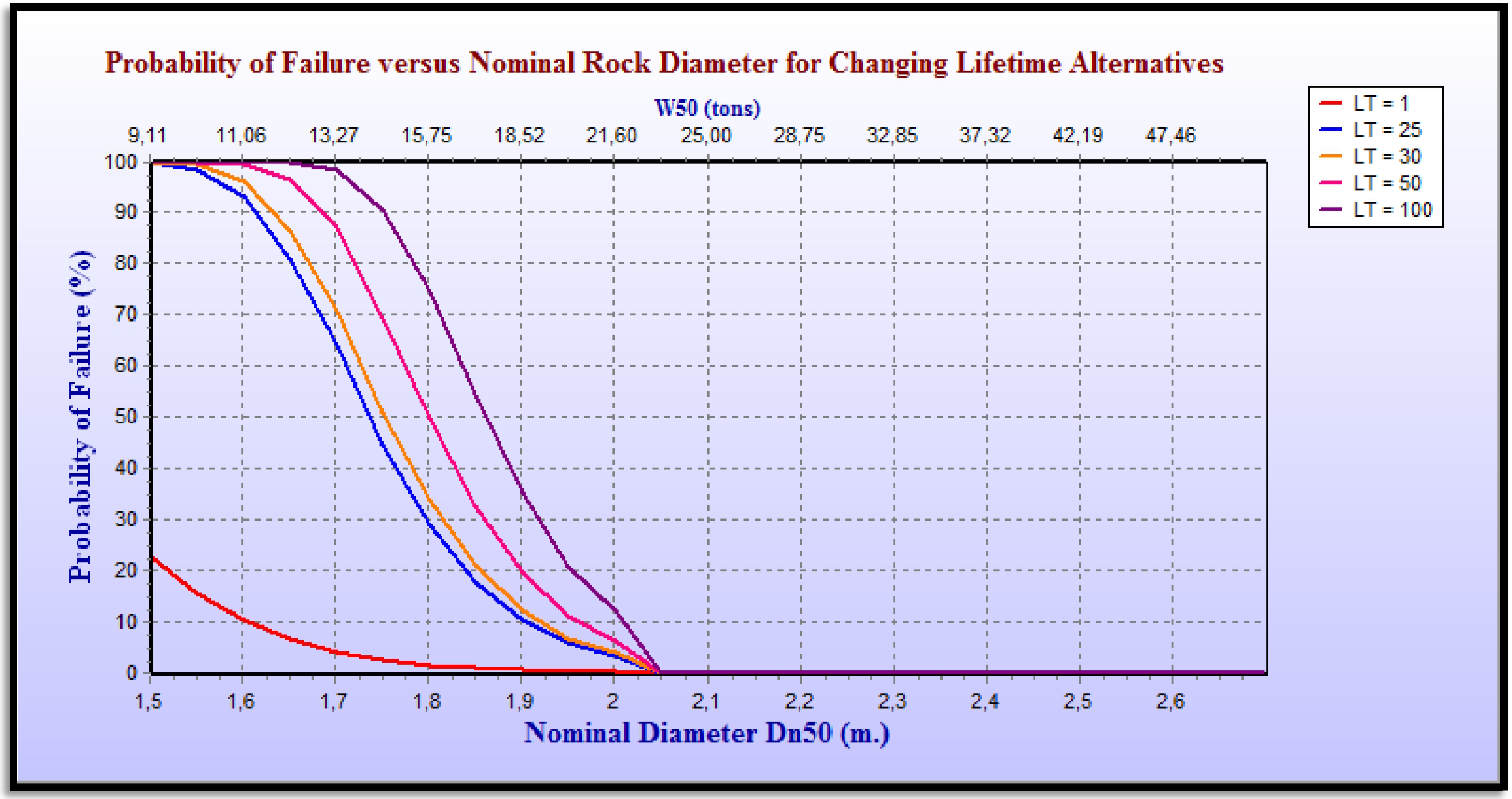

Figure 4.5 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2

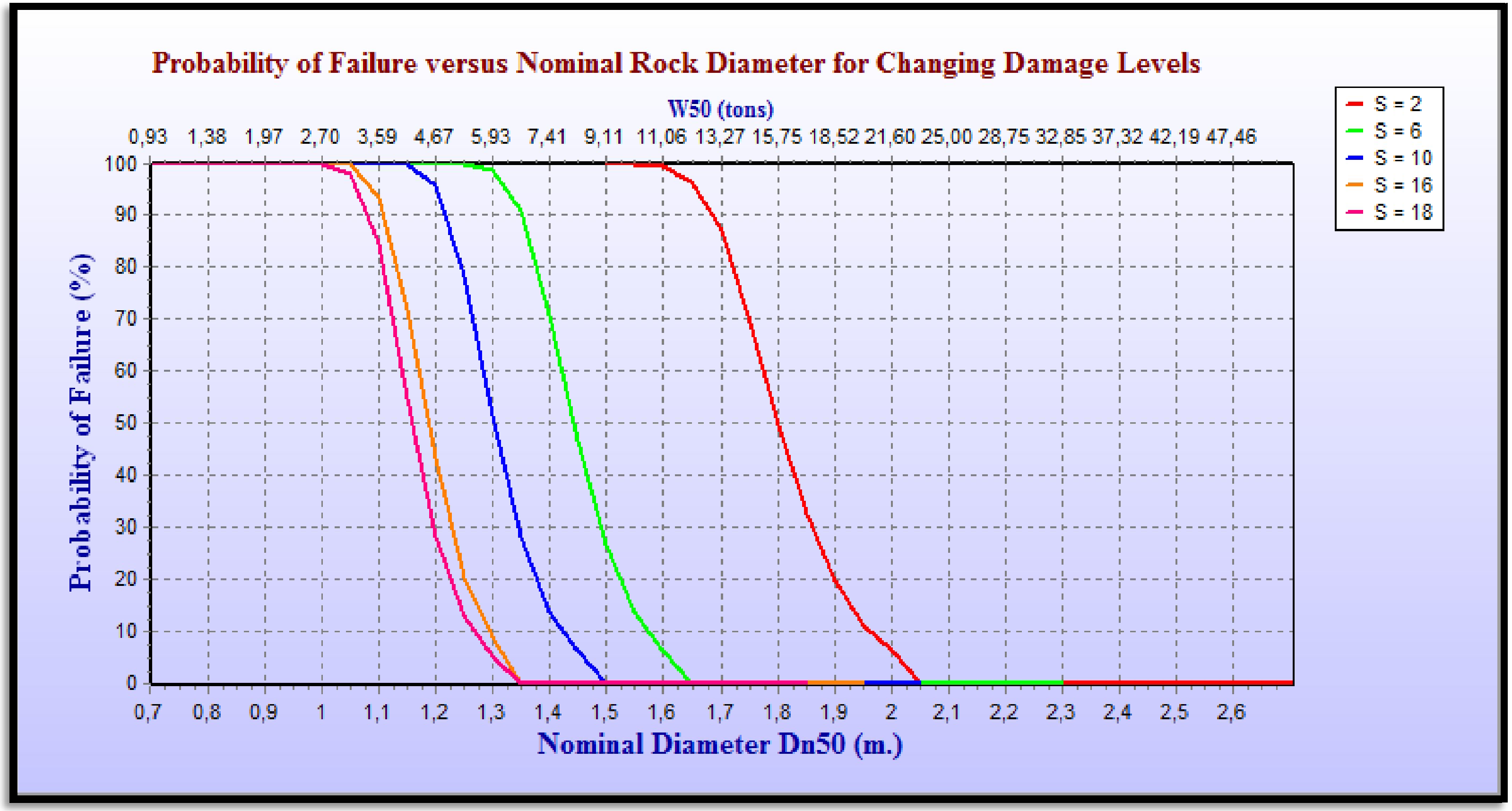

Figure 4.6 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10, 16 and 18 damage levels in LT=50 years lifetime of structure

The involvement of uncertainties in a design parameter can be examined by the relative change ratio R_c that is defined as the ratio of the reliability index β_{HL} to the reliability index that is obtained by taking the design parameter as constant value. It is accomplished by entering zero for the standard deviation value of the parameter in order to handle the variable as deterministic. The relative change ratio shows the measure of uncertainty level inherent in the design parameters. If the relative change ratio of a parameter is calculated approximately 1, then it is appropriate to take the design parameter as a deterministic quantity in the design.

The indicators for relative importance of a random variable on the reliability index is defined by direction cosines (α) and sensitivity factors (SF_i) which are squares of direction cosines. Influence of load and resistance parameters on the reliability level of a structure is well defined by these parameters. Thus, if the sensitivity factor of a parameter is small its influence on the reliability level can be neglected.

Sensitivity factors (SF_i), relative change ratios (R_c) and direction cosines (α_i) for all design parameters of the limit state function are given in Table 4.5.

Table 4.5 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model

Design parameter	SF_i	R_c	α_i
Y	0.30	1.197	0.55
Δ	0.07	1.036	0.26
cotθ	0.21	1.126	0.46
H	0.66	1.706	-0.81
N	0.05	1.027	-0.23
D_{n50}	0.01	1.007	0.12

It can be observed from Table 4.5 that, the reliability of the structure is mainly influenced by the design wave height, since the sensitivity factor of the parameter is SF_i=66%. Other variables, number of waves, relative density and nominal rock diameter of armor unit can be regarded as deterministic variables in the analysis due to the relative change ratios approximately equal to 1.

The design is also very sensitive to the reliability of the failure function since it has a 30% sensitivity factor according to Table 4.5.

Exceedance probabilities of selected constant damage level S=2 for changing lifetime of the structure LT=1, 25,30, 50 and 100 years is presented in Figure 4.5. The outcomes from Figure 4.5 are summarized as follows:

I) Exceedance probability P_f increases as the lifetime LT of the structure increases, for a given nominal diameter D_{n50} (armor unit weight)

II) As the design nominal rock diameter increases for a constant lifetime, exceedance probability decreases.

50 years lifetime is selected for Giresun Port case for making comparisons between deterministic and reliability-based design models from Table 3.7 according to the defined classification of risk levels and type of structures. (Structure: General use, Risk Level: 2).

When Figure 4.6 is examined in order to study the effects of damage level changes on reliability of the structure for a selected constant design lifetime, forthcoming results are obtained:

I) For a constant nominal diameter of armor unit, probability of failure (exceedance probability of predetermined damage level) increases as the damage level S decreases.

II) For a constant reliability level (probability of failure or damage level), the armor unit weight thus the nominal diameter has to be increased for decreasing damage levels.

III) This graph also helps the designer to examine the reliability level of an existing structure for a lifetime of 50 years if the preset damage level in design stage of structure is known. In addition, in ARIN reliability-based design tool, the user is given a flexibility to examine an existing structure at any time within its design lifetime.

This points out that the importance of damage level selection in preliminary design stage, since larger or smaller damage level selections give over-design or under-design conditions resulting in huge amounts of investment losses.

Using Tables 3.8 and 3.9 for Giresun Port failure probability of 20% is selected in our case since the rubble-mound structure is a flexible type with Level II risk identification resulting in average amount of financial loss.

Damage level S=2 which is used in deterministic design is exceeded with a probability of 35% in its 50 years lifetime as tabulated in Table 4.13.

Table 4.6 Design parameters evaluated from deterministic and reliability-based designs for Giresun Port

Design	Deterministic	Reliability-based
S	2	10
P_f(%)	35%	20%
D_{n50} (m)	1.85	1.39
W_{50} (tons)	17.31	7.25

In the reliability-based design by Van der Meer response function of plunging waves, the nominal diameter and weight of armor unit are evaluated from Figure 4.6 as approximately D_{n50}=1.39 meters and W_{50}= 7.25 tons, respectively for the maximum admissible failure probability of the breakwater for the damage level of S=10 that corresponds to (20-25)% damage in its lifetime.

This case study is on an existing port in Turkey, Giresun which had been damaged approximately totally under the action of a severe storm with a 50 years return period coinciding with the design lifetime of the structure. Therefore, it is a good implicator for the verification of reliability-based design tool. According to the given cross-section details of Giresun Port, (10-15) tons of armors are used at the front face of the structure resulting in a mean of 13.5 tons armors. It is clearly seen that under these conditions the probability of failure of the structure for damage level S=2 (DL=0-5 %) is 100%. Thus , it is not a surprise to see the destruction of Giresun Port under the attack of 1999 storm.

CHAPTER 5

APPLICATION OF RELIABILITY-BASED DESIGN MODEL
CASE II: FOÇA-LEVENTLER MILITARY PORT

This case study is on an existing port in Turkey, Foca which had been examined by physical modeling at the laboratories of General Directorate of Construction of Railways, Ports and Airports. Therefore, it is a good implicator for the verification of reliability-based design model.

Reliability-based design model ARIN is applied for Foça-Leventler Military Port as case II.

5.1 Deterministic design by Van der Meer equation

In the second part of the deterministic design nominal rock diameter of the main breakwater of Foça-Leventler Military Port is calculated by using Van der Meer design scheme. In the preliminary design two layered randomly placed quarry stone armor units are selected as the sectional details.

The permeability coefficient is selected as P=0.4 by utilizing the procedure given by Van der Meer (Van der Meer and Koster, 1988), in which the wave period, relative dissipation and permeability values are intercorporated.

The initial damage S=2 used in Van der Meer equation for cotθ= 2.0 conforms the no damage criteria in Hudson as described in Table 4.5. Therefore, in order to make calculations in the same domains damage coefficient is selected as S=2.

Outcomes of the extreme wave analysis resulted in an average 3 hours duration of storms for the selected area. Therefore, by using average period expected value of number of waves is calculated as μ_N=1516 waves. The standard deviation of number of waves is calculated as σ_N=454 assuming a coefficient of variation δ=0.30 by expert opinion (Balas and Ergin, 2002).

The results of the calculations are summarized in Table 5.1 by giving the detailed information on design parameters.

Table 5.1 Deterministic design by Van der Meer equation

Parameter	Unit	Values
Slope of armor layer (cotθ)	-	2.0
Unit weight of armor stone (γ_r)	tons/m^3	2.70
Unit weight of sea water (γ_w)	tons/m^3	1.03
Design significant wave height $(H_s)_D$	m.	3.69
Deep water wave steepness $(H_s)_0/L_0$	-	0.036
Significant wave period (T_s)	sec.	8.31
Mean wave period (T_z)	sec.	7.12
Surf similarity parameter (ξ_z)	-	2.64
Critical surf similarity parameter (ξ_c)	-	3.72
Wave breaking classification	-	Plunging
Average number of waves (μ_N)	-	1516
Permeability coefficient (P)	-	0.42
Placement of rock	-	Random
Damage level (S)	%	2
Nominal rock diameter (D_{n50})	m.	1.26
Median weight of armor units (W_{50})	tons	5.42

5.2 Reliability-based design

This part of the study is composed of the determination of armor unit diameter (D_{n50}, nominal diameter) of the trunk section of main breakwater of Foça Leventler Military Port (Figure 5.5) by using design wave parameters obtained from extreme analysis described in Appendix A and other design parameter properties and values proposed in the report prepared by General Directorate of Construction of Railways, Ports and Airports (Foça-Leventler Askeri Limanı, 2005). The design wave height is selected for a return period of 50 years. The outcome of the extreme analysis shows that the annual extreme wave data collected from the site for 17 years best fits to Log-normal distribution with significant wave height H_s=3.92 m.(H_d=3.69) and significant wave period T_s=8.31 sec. (Table A.14) with a steepness of 0.036.

The analysis is carried out by the implementation of Van der Meer limit state function for the hydraulic stability of the primary armor layer under wave action. Effects of the damage levels predefined in the preliminary design and failure probability distribution during different lifetimes are assessed according to the outcomes of ARIN. All of the graphics presented are in form of original outcome format of the design tool.

Statistical values of basic variables used in limit state equation are given in Table 5.2 as:

Table 5.2 Statistical values of design variables used in Van der Meer limit state equation of Foça Leventler Military Port

Basic Variable (X_i)	Distribution	Mean (μ_{xi})	Standard Deviation (σ_{xi})	Variation Coefficient (δ%)
Y_1	Normal	6.2	0.4	6.5
Y_2	Normal	1.0	0.08	8
$(D_{n50})_{VanD.Meer}$	Normal	1.30	0.052	4
H_s	Weibull (k=2)	2.911	0.48	0.16
H_s/L_z	Deterministic	0.036	-	-
Δ	Normal	1.62	0.05	3
P	Deterministic	0.42	-	-
N	Normal	1516	454	0.30
$\cot\theta$	Normal	2.00	0.24	0.12

Log-normal distribution shows a difference for mean and standard deviation values due to the difference in algorithm from other extreme type distributions. Therefore, special attention must be given when working with log-normal distribution selected by the method proposed by Goda (Goda, 2000) to use appropriate shape and location parameters resulting in correct mean and standard deviations of the design wave. The exceedance probabilities for the described damage level in different life time alternatives and exceedance probabilities for different damage levels in predetermined lifetime values are obtained by using parameter uncertainties in the equation.

According to the calculations made simultaneously in ARIN, response function for plunging waves is selected.

In Figure 5.1, exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2 is presented.

In Figure 5.2, exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10 and 18 damage levels in LT=50 years lifetime of structure is presented.

Figure 5.3 gives exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2.

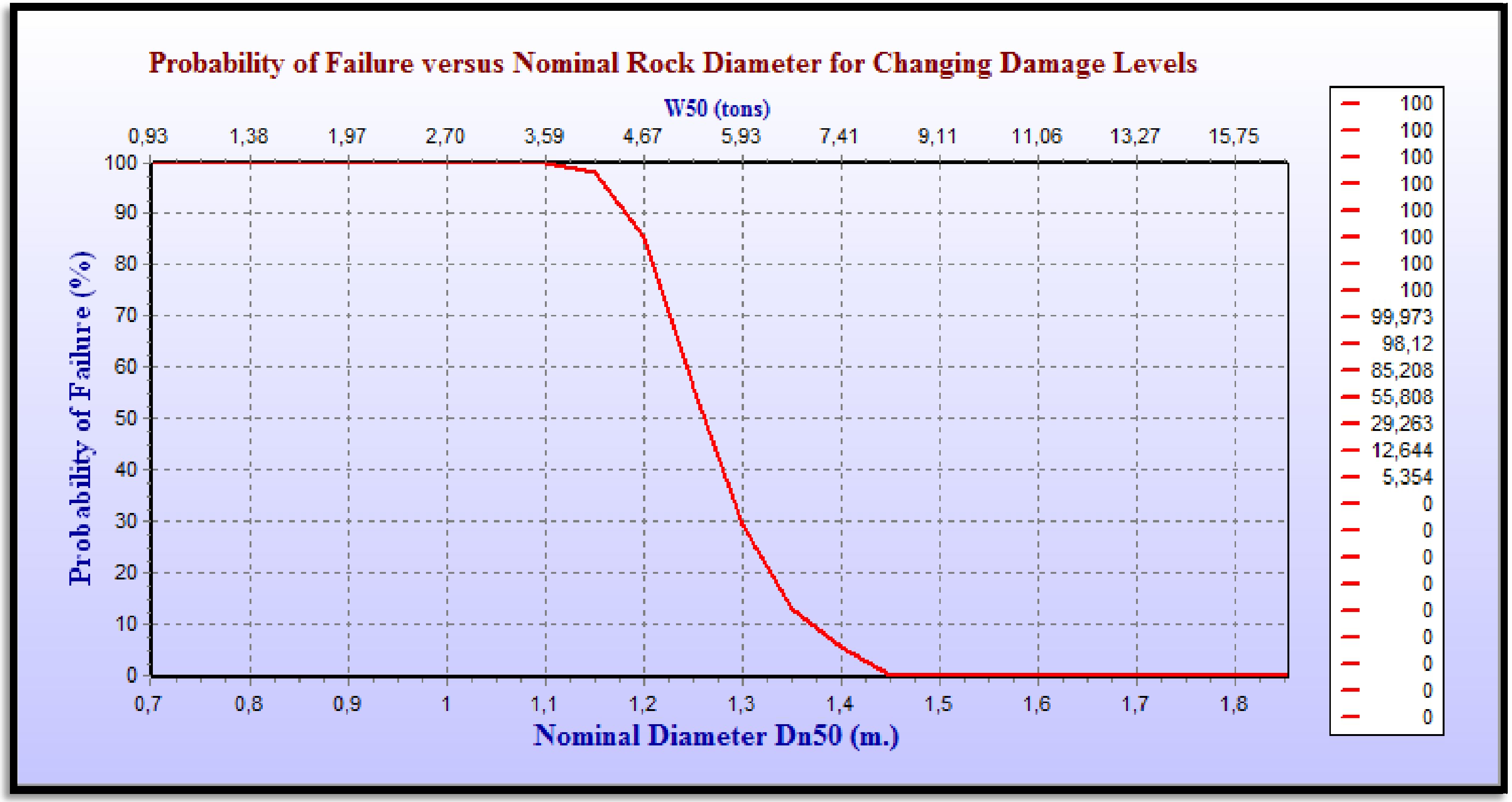

Figure 5.1 Exceedance probabilities $P_{f}(\%)$ as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2

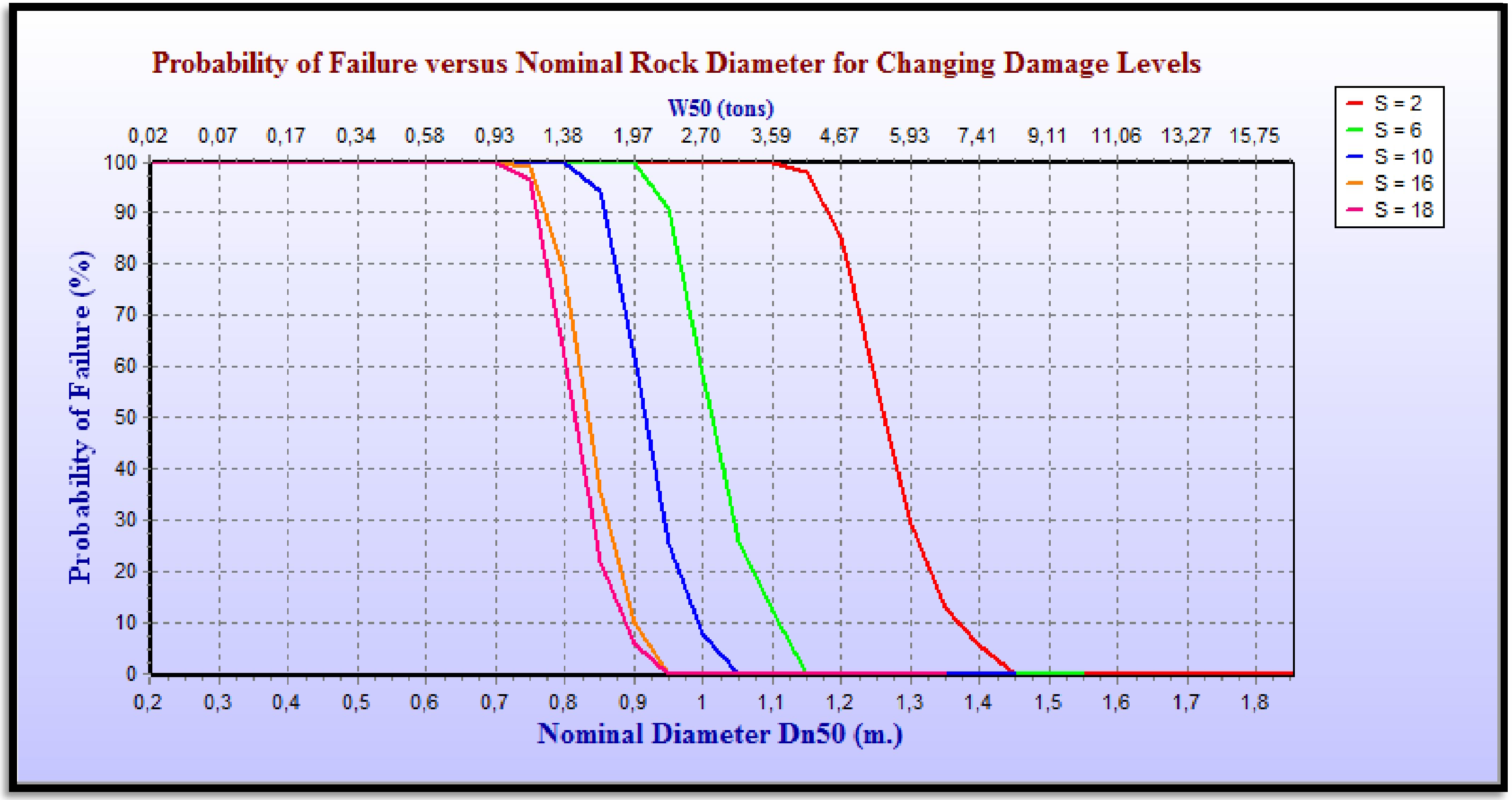

Figure 5.2 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10, 16 and 18 damage levels in LT=50 years lifetime of structure

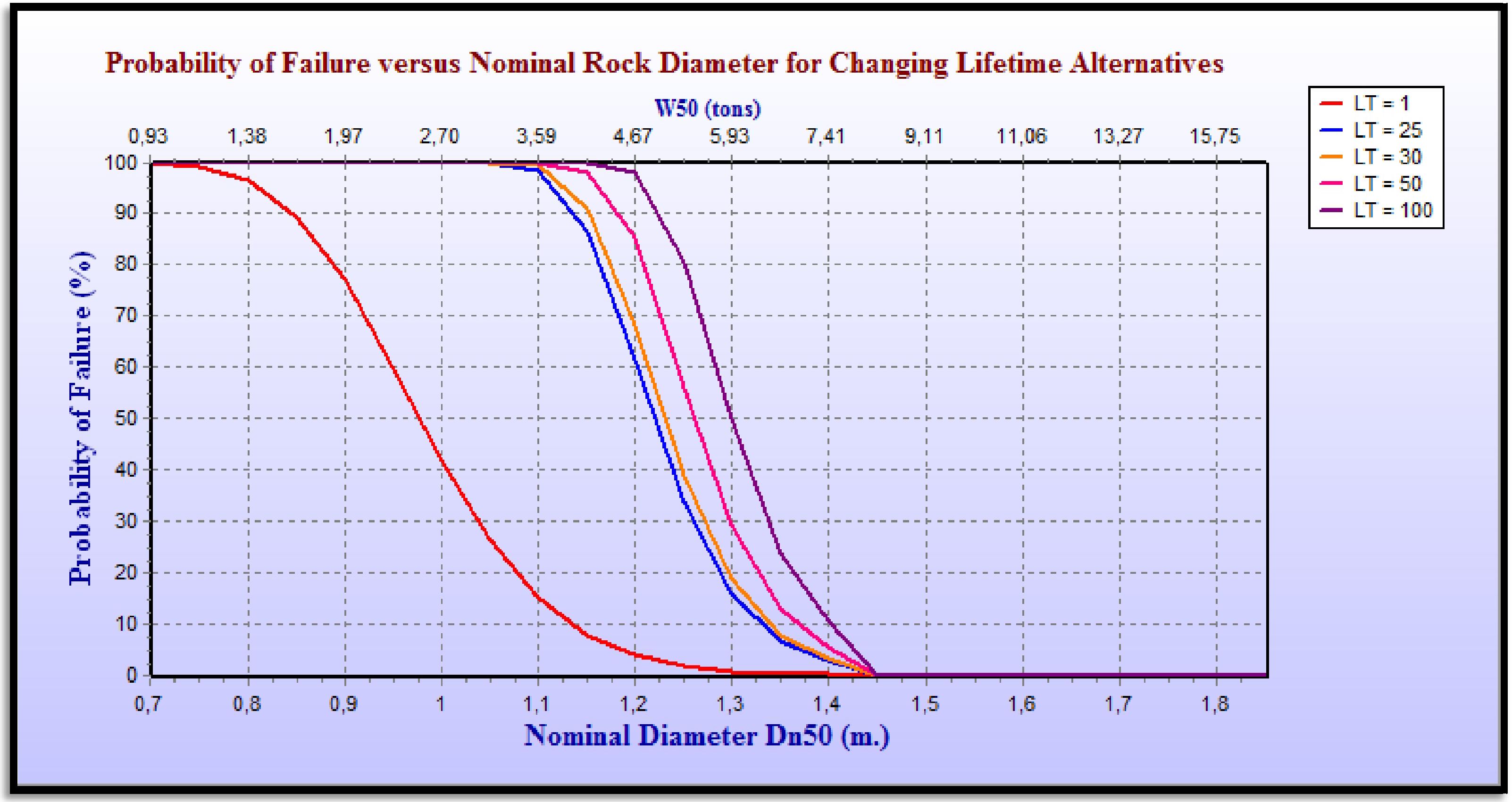

Figure 5.3 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=1,25,30,50 and 100 years lifetime of structure for damage level S=2

In the reliability-based design by Van der Meer response function of plunging waves, the nominal diameter and weight of armor unit are evaluated from Figure 5.2 as approximately D_{n50}=1.10 meters and W_{50}= 3.60 tons, respectively for the maximum admissible failure probability of the breakwater for the damage level of S=6 that corresponds to (20-25)% damage in its lifetime. Damage level S=2 which is used in deterministic design is exceeded with a probability of 70% in its 50 years lifetime as tabulated in Table 5.3.

Table 5.3 Design parameters evaluated from deterministic and reliability-based designs for Foça-Leventler Military Port

Design	Deterministic	Reliability-based
S	2	6
P_f(%)	70%	20%
D_{n50} (m)	1.26	1.10
W_{50} (tons)	5.40	3.60

Sensitivity parameters, relative change ratios and direction cosines of basic variables in the model for the case is tabulated in Table 5.4.

Table 5.4 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model for the case

Design parameter	SF_i	R_c	α_i
Y	0.16	1.091	0.40
Δ	0.06	1.031	0.25
$\cot\theta$	0.32	1.213	0.57
H	0.41	1.302	-0.64
N	0.04	1.021	-0.19
D_{n50}	0.008	1.000	0.09

It can be observed from Table 5.4 that, the reliability of the structure is mainly influenced by the design wave height, since the sensitivity factor of the parameter is SF_i=41%. Other variables (N,Δ,D_{n50},Y) number of waves, relative density and nominal rock diameter of armor unit can be regarded as deterministic variables in the analysis due to the relative change ratios approximately equal to 1.

The design is also very sensitive to the reliability of the failure function and slope of the front face of the structure since they have 16% and 32% sensitivity factors, respectively according to Table 5.4.

Exceedance probabilities of selected constant damage level S=2 for changing lifetime of the structure LT=1, 25,30, 50 and 100 years is presented in Figure 5.3. The outcomes for the probability of failure versus nominal diameter curves for Foça-Leventler Military Port (Figure 5.3) show the same trend with the outcomes of Giresun Port (Figure 4.5).

The outcomes from Figure 5.3 are summarized as follows:

III) Exceedance probability P_f increases as the lifetime L of the structure increases for a given nominal diameter D_{n50} (armor unit weight).

IV) As the design nominal rock diameter increases for a constant lifetime, exceedance probability decreases.

50 years lifetime is selected for Foça-Leventler Military Port case for making comparisons between deterministic and reliability-based design models from Table 3.7 according to the defined classification of risk levels and type of structures. (Structure: General use, Risk Level: 2).

When Figure 5.2 is examined in order to study the effects of damage level changes on reliability of the structure for a selected constant design lifetime, forthcoming results are obtained:

I) For a constant nominal diameter of armor unit, probability of failure (exceedance probability of predetermined damage level) increases as the damage level S decreases.

II) For a constant reliability level (probability of failure or damage level), the armor unit weight has to be increased for decreasing damage levels.

III) This graph also helps the designer to examine the reliability level of an existing structure for a lifetime of 50 years if the preset damage level in design stage of structure is known. In addition, in ARIN reliability-based design tool, the user is given a flexibility to examine an existing structure at any time within its design lifetime.

5.3 Hydraulic model studies

Major coastal defense structures should be tested with physical modeling phenomena thus hydraulic studies after the application of reliability based design works in order to see the effects of failure modes which are not included in the failure mode response functions.

Hydraulic model tests for Foça-Leventler Military Port (Figure 5.4) that were carried out at at the laboratories of hydraulic division of General Directorate of Construction of Railways, Ports and Airports are used to support the final design recommendations in a wave flume of 40 m. length, 0.60 m. width and 1.20 m. depth. Irregular wave generation is performed for the model studies.

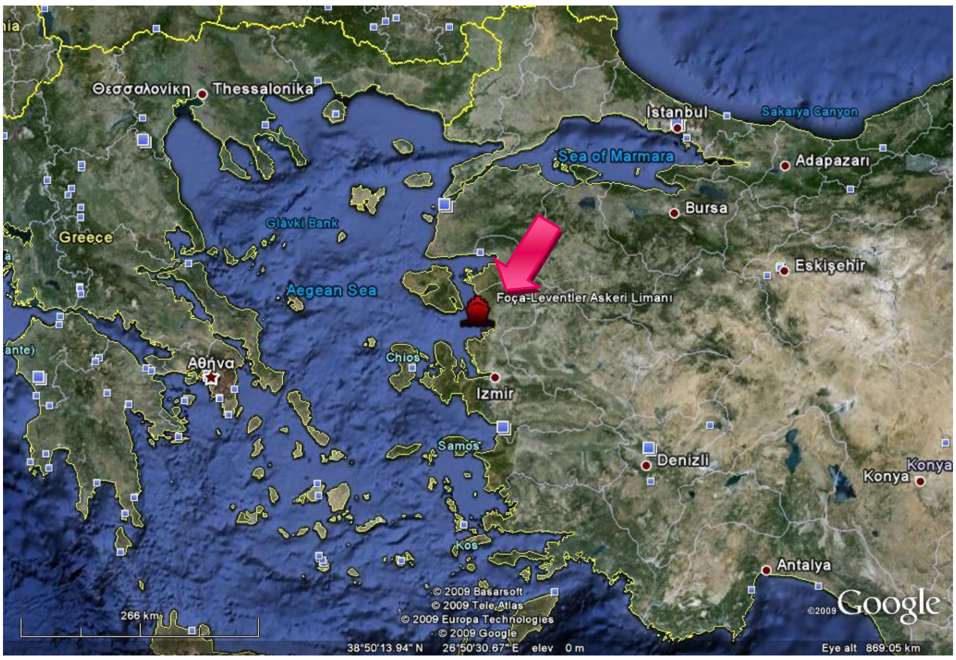

Figure 5.4 Location of Foça-Leventler Military Port

The model scale selection works are tabulated in Table 5.5 according to the limitations of the laboratory conditions and Froude law.

Table 5.5 Selection of Model Scale

	H_{min}	H_{max}	T_{min}	T_{max}
Scale	1.00	4.42	4.0	7.4
20	5.00	22.10	0.894	1.655
30	3.33	14.73	0.730	1.351
35	2.86	12.63	0.676	1.251
40	2.50	11.05	0.632	1.170

Most appropriate model scale is selected as 1/35 according to the wave properties, dimensions of the structure and the laboratory conditions (Foça-Leventler Askeri Limanı, 2005).

Unit weights of the armor unit used at the design of the structure and water are γ_{armor} = 2.69 ton/m^3 and γ_{water}= 1.00 ton/m^3, respectively.

Water depth at the toe of the structure is 31 m., front face slope is cotθ= 2.0 and the range of weight of the armor unit is 6-8 tons according to the given properties of the cross-section of the rubble-mound breakwater. Cross-section of the military harbor is shown at Figure 5.5.

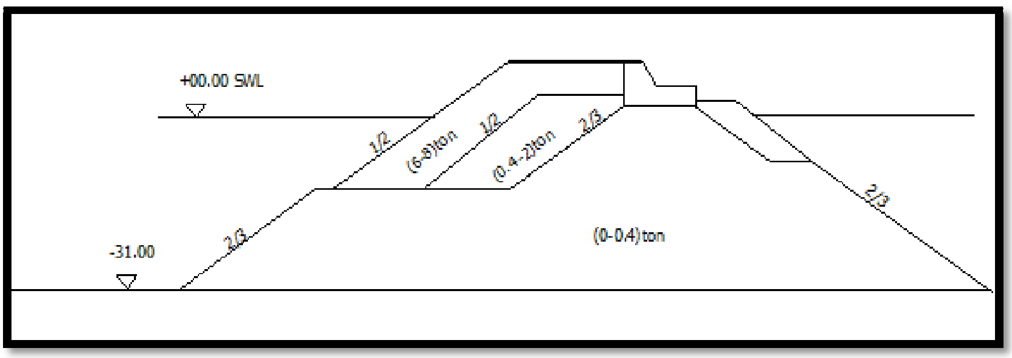

Figure 5.5 Cross-section of Foca-Leventler Military Port

Armor unit weights of the structure was obtained by Hudson design formula giving W_{50}=7.18 tons.

When the damage results of the physical modeling are examined, 1.5% of damage is observed under design wave of 50 years return period. This damage level satisfies predetermined (0-5)% no-damage criteria.

Nominal diameter of the armor unit is calculated as 1.37 according to the given armor unit weight. The results of physical modeling, deterministic design with Van der Meer formula and reliability-based design for lifetime of 50 years is summarized in Table 5.6

Table 5.6 Comparison of deterministic and reliability-based designs with physical modeling

Design	Physical Modeling	Deterministic	Reliability-based
S	2	2	6
P_f(%)	4%	70%	20%
D_{n50} (m)	(1.30-1.37)	1.26	1.10
W_{50} (tons)	(6-8)	5.40	3.60

Table 5.6 (LT_{50}) is examined in order to see the probability of failure P_f changes between reliability-based design and physical modeling. Probability of failure for the proposed range of nominal diameter of armor unit changes between (40-18)% for the predetermined 50 years lifetime for no-damage damage level according to the reliability-based design model.

Probability of failure percentage described for physical modeling is calculated for a duration of a storm. However, in reliability-based design the outcome of reliability levels are treated for design lifetimes of the structure. Thus, small damage percentage shown in Table 5.6 attributes from this logical difference between modeling types. In addition, when Figure 5.1 examined it is clearly seen that the structure gives approximately 40% probability of failure for no-damage criteria for 50 years lifetime. Therefore, it can be concluded that the design is an over-design situation resulting in economical loss from investment point of view according to the outcomes of reliability-based design tool developed in this study.

The computation of nominal rock diameter in reliability-based design is primarily affected by the selected damage level, lifetime and the exceedance probabilities. Nominal rock diameters obtained as a function of lifetime and exceedance probabilities under constant damage level S=2, are presented in Table 5.7.

Table 5.7 Comparison of lifetime alternatives LT=30 and LT=50 years for a constant damage level of S=2

		P_f(%)	60	70	80	90	100
Lifetime (years)	30	D_{n50} (m.)	1.24	1.22	1.20	1.18	1.10
	30	W_{50}	5.15	4.90	4.67	4.44	3.60
	50	D_{n50} (m.)	1.28	1.27	1.24	1.20	1.16
	50	W_{50}	5.70	5.53	5.15	4.67	4.21

It is clear that small changes in nominal diameter D_{n50} values results in large deviations in armor weights W_{50}. It is observed from the table that the combination of low exceedance probabilities and no damage level may result in uneconomical design conditions. When reliability-based design model is used at the preliminary design stage of a structure, special attention should be given to the economic consequences of failure if the consequences are serious. Therefore, this study highlights the importance of these parameters. As a result, selection of damage level, lifetime and exceedance probability alternatives have to be done very carefully not to under-design the structure.

CHAPTER 6

APPLICATION OF RELIABILITY-BASED DESIGN MODEL
CASE III: SİNOP DEMİRCİKÖY FISHERY HARBOR

This case study is on an fishery harbor designed to be constructed in Turkey, Sinop Demirciköy Fishery Harbor (Figure 6.1) which had been examined by physical modeling at the laboratories of General Directorate of Construction of Railways, Ports and Airports. Therefore, it is a good implicator for the verification of reliability-based design model.

Cross-section details of the main breakwater of Sinop Demirciköy Fishery Harbor is given in Figure 6.2.

Reliability-based design model ARIN is applied for Sinop Demirciköy Fishery Harbor as case III.

6.1 Deterministic design by Van der Meer equation

In this part the deterministic design nominal rock diameter of the main breakwater of Sinop Demirciköy Fishery Harbor is calculated by using Van der Meer design scheme. In the preliminary design two layered randomly placed quarry stone armor units are selected as the sectional details.

The permeability coefficient is selected as P=0.42 by utilizing the procedure given by Van der Meer (Van der Meer, 1988b), in which the wave period, relative dissipation and permeability values are intercorporated.

The initial damage S=2 used in Van der Meer equation for cotθ= 2.5 conforms the no damage criteria in Hudson as described in Table 4.5 . Therefore, in order to make calculations in the same domains damage coefficient is selected as S=2.

Outcomes of the extreme wave analysis resulted in an average 3 hours duration of storms for the selected area. Therefore, by using average period expected value of number of waves is calculated as μ_N=1158 waves. The standard deviation of number of waves is calculated as σ_N=348 assuming coefficient of variation δ=0.30 by expert opinion.

Figure 6.1 Location of Sinop Demirciköy Fishery Harbor

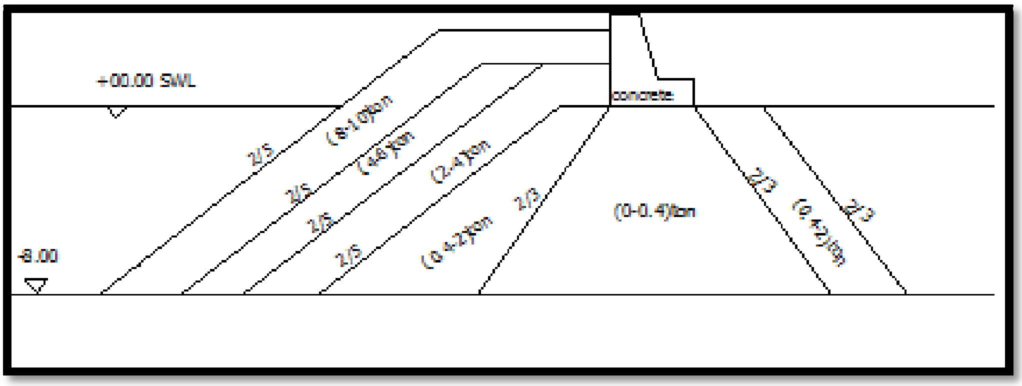

Figure 6.2 Cross-section of Sinop Demirciköy Fishery Harbor

The results of the calculations are summarized in Table 6.1 by giving the detailed information on design parameters.

Table 6.1 Deterministic design by Van der Meer equation

Parameter	Unit	Values
Slope of armor layer ($\cot\theta$)	-	2.5
Unit weight of armor stone (γ_r)	tons/m^3	2.70
Unit weight of sea water (γ_w)	tons/m^3	1.03
Design significant wave height $(H_s)_D$	m.	5.62
Deep water wave steepness $(H_s)_0/L_0$	-	0.039
Significant wave period (T_s)	sec.	11.58
Mean wave period (T_z)	sec.	9.32
Surf similarity parameter (ξ_z)	-	2.03
Critical surf similarity parameter (ξ_c)	-	3.30
Wave breaking classification	-	Plunging
Average number of waves (μ_N)	-	1158
Permeability coefficient (P)	-	0.42
Placement of rock	-	Random
Damage level (S)	%	2
Nominal rock diameter (D_{n50})	m.	1.65
Median weight of armor units (W_{50})	tons	12.22

6.2 Reliability-based design

This part of the study is composed of the determination of armor unit diameter (D_{n50}, nominal diameter) of the trunk section of main breakwater of Sinop Demirciköy Fishery Harbor (Figure 6.2) by using design wave parameters obtained from extreme analysis described in Appendix A and other design parameter properties and values proposed in the report prepared by General Directorates of Construction of Railways, Ports and Airports and (Sinop Demirciköy Barınağı Mendirek Kesitlerine ait Fiziksel Model Deneyleri Raporu, 2009). The design wave height is selected for a return period of 50 years. The outcome of the extreme analysis shows that the annual extreme wave data collected from the site for 48 years best fits to FT-I (Gumbel) distribution with significant wave height H_s=8.78 m. and significant wave period T_s=11.58 sec. with a steepness of 0.039

The analysis is carried out by the implementation of Van der Meer limit state function for the hydraulic stability of the primary armor layer under wave action. Effects of the damage levels predefined in the preliminary design and failure probability distribution during different lifetimes are assessed according to the outcomes of ARIN. All of the graphics presented are in form of original outcome format of the design tool.

Statistical values of basic variables used in limit state equation are given in Table 6.2.

Table 6.2 Statistical values of design variables used in Van der Meer limit state equation of Demirciköy Fishery Harbor

Basic Variable (X_i)	Distribution	Mean (μ_{xi})	Standard Deviation (σ_{xi})	Variation Coefficient (δ%)
Y_1	Normal	6.2	0.4	6.5
Y_2	Normal	1.0	0.08	8
$(D_{n50})_{VanD.Meer}$	Normal	1.90	0.076	4
H_s	FT-I (Gumbel)	5.822	1.119	0.20
H_s/L_z	Deterministic	0.039	-	-
Δ	Normal	1.62	0.05	3
P	Deterministic	0.42	-	-
N	Normal	1158	348	0.30
$\cot\theta$	Normal	2.5	0.24	0.12

The exceedance probabilities for the described damage level in different life time alternatives and exceedance probabilities for different damage levels in predetermined lifetime values are obtained by using parameter uncertainties in the equation.

In Figure 6.3, exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in LT=50 years lifetime of structure for damage level S=2 is presented.

In Figure 6.4, exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} for S=2, 6, 10, 16 and 18 damage levels in LT=50 years lifetime of structure is presented.

Figure 6.5 gives exceedance probabilities $P_f(\%)$ as a function of nominal armor unit diameter D_{n50} in LT=1, 25, 30, 50 and 100 years lifetime of structure for damage level S=2.

According to the calculations made simultaneously in ARIN, response function for plunging waves is selected.

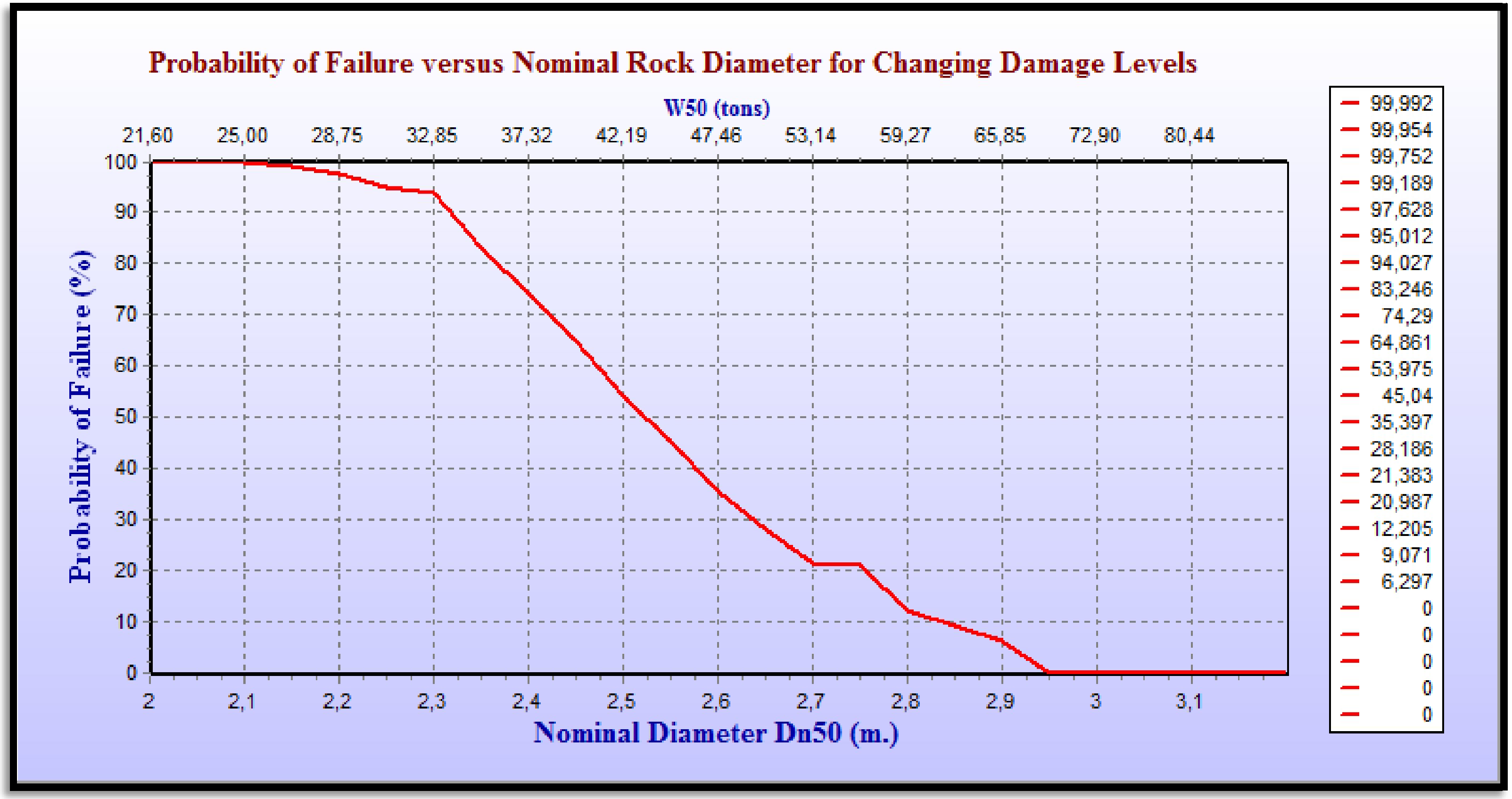

Figure 6.3 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for S=2 damage level and 50 years lifetime of structure

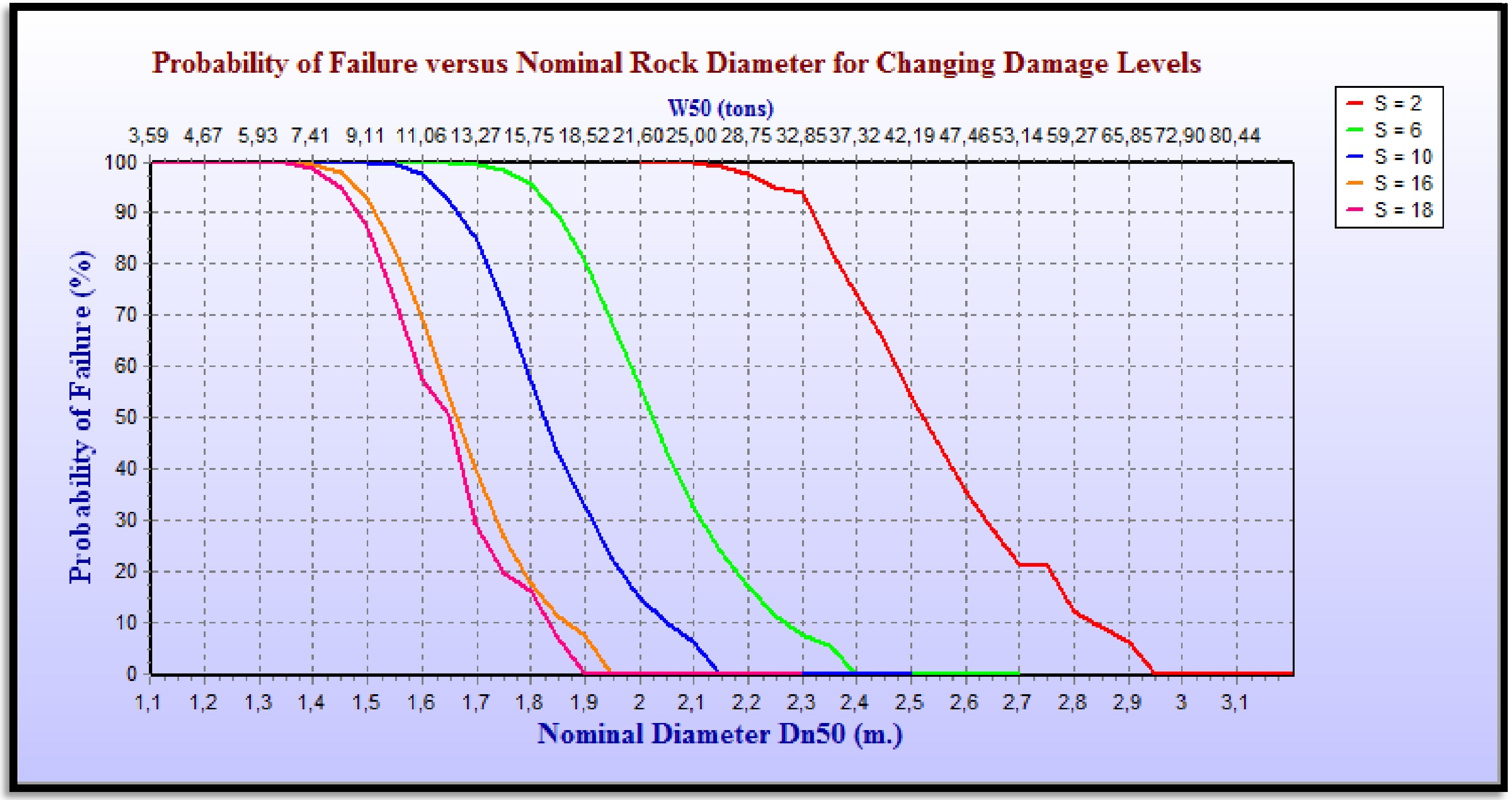

Figure 6.4 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} in L=50 years lifetime of structure for damage levels S=2,6,10,16,18.

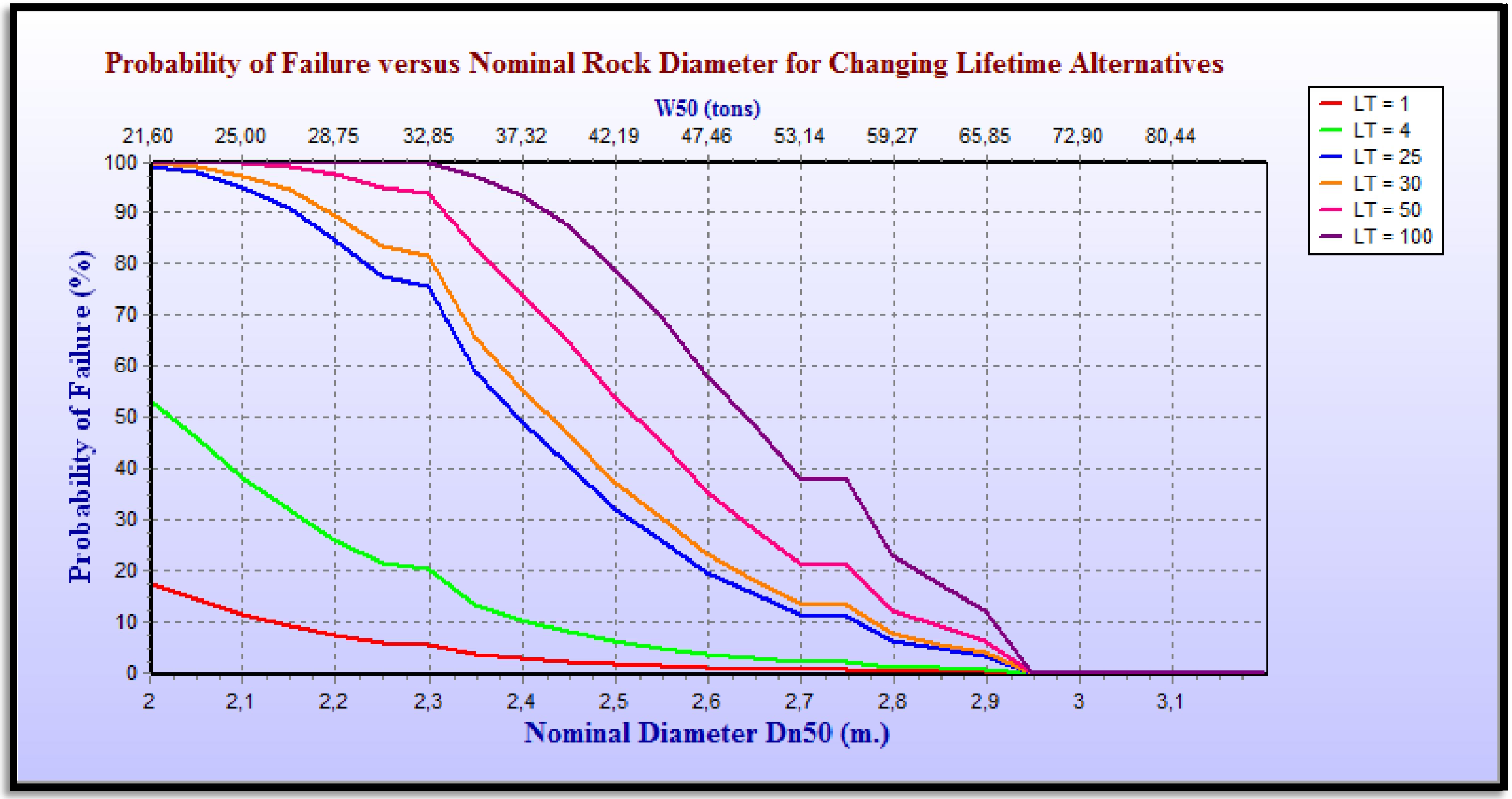

Figure 6.5 Exceedance probabilities P_f(%) as a function of nominal armor unit diameter D_{n50} for L= 4,25,30,50 years lifetimes with damage level S=2

In Table 6.3 design parameters evaluated from deterministic and reliability-based designs for Sinop Demirciköy Fishery Harbor giving probability of failure percentages for changing armor weights are presented.

Table 6.3 Design parameters evaluated from deterministic and reliability-based designs for Sinop Demirciköy Fishery Harbor

Design	Deterministic	Reliability-based
S	2	18
P_f(%)	92%	20%
D_{n50} (m)	1.65	1.75
W_{50} (tons)	12.22	14.47

Sensitivity parameters, relative change ratios and direction cosines of basic variables in the model for the case are given in Table 6.4.

It can be observed from Table 6.4 that, the reliability of the structure is mainly influenced by the design wave height, since the sensitivity factor of the parameter is SF_i=78%. Other variables, number of waves, relative density and nominal rock diameter of armor unit can be regarded as deterministic variables in the analysis due to the relative change ratios approximately equal to 1.

Different from other cases, the design is not sensitive to the reliability of the failure function and slope of the front face of the structure since they have 10% and 8% sensitivity factors, respectively according to Table 6.4 .

Table 6.4 Sensitivity parameters, Relative change ratios and direction cosines of basic variables in the model for the case

Design parameter	SF_i	R_c	α_i
Y	0.10	1.054	0.32
Δ	0.02	1.010	0.15
cotθ	0.08	1.087	0.28
H	0.78	2.132	-0.88
N	0.06	1.032	-0.24

Exceedance probabilities of selected constant damage level S=2 for changing lifetime of the structure LT=1, 25, 30, 50 and 100 years is presented in Figure 6.5. The lifetime curves in Figure 6.5 show the same trend as in Giresun and Foça cases since the probability of failure versus nominal rock diameter curves show same behavior for all cases.

The outcomes from Figure 6.5 are summarized as follows:

I) Exceedance probability P_f increases as the lifetime L of the structure increases for a given nominal diameter D_{n50} (armor unit weight).

II) As the design nominal rock diameter increases for a constant lifetime, exceedance probability decreases.

50 years lifetime is selected for Sinop Demirciköy Fishery Harbor case for making comparisons between deterministic and reliability-based design models

from Table 3.7 according to the defined classification of risk levels and type of structures. (Structure: General use, Risk Level: II).

When Figure 6.4 is examined in order to study the effects of damage level changes on reliability of the structure for a selected constant design lifetime, forthcoming results are obtained:

I) For a constant nominal diameter of armor unit, probability of failure (exceedance probability of predetermined damage level) increases as the damage level S decreases.

II) For a constant reliability level (probability of failure or damage level), the armor unit weight thus nominal diameter has to be increased for decreasing damage levels.

III) This graph also helps the designer to examine the reliability level of an existing structure for a lifetime of 50 years if the preset damage level in design stage of structure is known. In addition, in ARIN reliability-based design tool, the user is given a flexibility to examine an existing structure at any time within its design lifetime.

6.3 Hydraulic model studies

Hydraulic model tests for Sinop Demirciköy Fishery Harbor that were carried out at hydraulic division of General Directorate of Construction of Railways, Ports and Airports are used to support the final design recommendations.

The wave flume is 40 m. long, 0.60 m. wide and 1.20 m. deep and irregular wave generation is performed for the model studies.

According to the topographic studies on the region sea bed slope is selected as 1/100. Model scale 1/30 is used for the construction of the cross-section (Figure 6.2) of rubble-mound defense structure.

Unit weights of the armor unit used at the design of the structure and water are γ_{armor} = 2.706 ton/m^3 and γ_{water}= 1.018 ton/m^3, respectively.

Water depth at the toe of the structure is 8 m., front face slope is cotθ= 2.5 and the range of weight of the armor unit is 8-10 tons according to the given properties of the cross-section of the rubble-mound breakwater.

Armor unit weights of the structure was obtained by Hudson design formula giving W_{50}=37.3 tons.

Each test was carried out for a duration that corresponds to 3000 number of waves. These model studies and wave characteristics created in the flume by the wave generator is given in Table 6.5.

Table 6.5 Wave characteristics of the model studies

Test number	H_s (model)	T_s (model)	H_s (prototype)	T_s (prototype)
2	6.6	1.2	1.98	6.57
3	10.2	1.41	3.06	7.72
4	13.5	1.77	4.05	9.69
5	17.3	1.95	5.19	10.68
6	19	2	5.70	10.95

The damage during the experiments was recorded by counting the number of displaced armor units at the end of each set. The damage percentage was determined as the ratio of the number of displaced units to the total number of armor units.

When the damage results of the physical modeling are examined, 4.4% of damage is observed under design wave of 50 years return period. This damage level satisfies predetermined (0-5)% no-damage criteria.
Nominal diameter of the armor unit is calculated as 1.50 according to the given armor unit weight. The results of physical modeling, deterministic design with Van der Meer formula and reliability-based design for lifetime of 50 years are summarized in Table 6.6.

Table 6.6 (LT_{50}) is examined in order to see the P_f changes between reliability-based design and physical modeling. Probability of failure for the proposed range of nominal diameter of armor unit is 100% for the predetermined 50 years lifetime for no-damage damage level according to the reliability-based design model results obtained by the execution of ARIN.

Table 6.6 Comparison of Physical Modeling, Deterministic and Relibility-Based Design Models for selected damage levels

Design	Physical Modeling	Deterministic	Reliability-based
S	2	2	18
P_f(%)	100	91	20
D_{n50} (m)	1.50	1.65	1.70
W_{50} (tons)	9.00	12.22	13.25

In Table 6.6 the exceedance of damage level S=2 (no damage criteria) will be faced with a probability of 100% for the given final design outcomes from physical modeling studies for Sinop. However, the exceedance probability of S=18 which corresponds to (20-25%) damage in its lifetime is only 20%.

This result shows that the design of main breakwater is an under-design resulting in a high damage level when no damage criteria is preset for the structure. In order not to face with tremendous effects of damage in case of failure routine maintanences should be done during the lifetime of the structure. By this method, changes in depth at the toe of the structure due to the movement of armor units to the sea bottom, decreases of armor unit weights due to breakage e.tc. under wave attack can be reduced.

6.4 Evaluation of Results

In order to make appropriate decisions on the reliability-based and deterministic design models applied on cases of this study, the differences between two methodologies should be summarized briefly. Therefore, steps are discussed below for both models.

Steps of deterministic design model:

I) By using extreme probability analysis for the assigned return period, design wave height is determined.

II) Encounter probability of damage is examined for assigned lifetime of the structure and return period of the design wave height. If the encounter probability is high return period is re-selected and design wave height is re-calculated.

III) Nominal diameter thus weight of armor unit is directly calculated from design equation for a selected damage level without considering uncertainties of design parameters.

IV) Hydraulic model studies are carried out for the final design, as none of the design parameters are treated with associated uncertainties which may affect the preliminary design significantly.

Steps of reliability-based design model:

I) Design wave height characteristic values (μ,σ) are determined by utilizing extreme wave analysis for the selected region of construction.

II) Reliability level is decided for the extreme environmental condition encountered in the lifetime of the structure.

III) Characteristic values and probability density functions of remaining design parameters of the response function are determined.

IV) Limit state condition is evaluated for design parameters with their reliability levels.

V) Nominal diameter of armor unit is obtained as a function of various design parameter alternatives. Considering the optimum solution, design parameters can be re-selected.

VI) Since, all of the possible failure modes are not examined in the reliability-based model, hydraulic model tests are carried out for the final design.

6.5 Hydraulic model studies on toe berm Stability

In recent years, designers has focused on the importance of toe berm construction for rubble-mound breakwaters. The function of a toe berm is to support the main armor layer and to prevent damage resulting from scour. Armor units displaced from the armor layer may come to rest on the toe berm, thus increasing toe berm stability. Toe berms are normally constructed of quarry-run, but concrete blocks can be used if quarry-run material is too small or unavailable (CEM, 2003).

Toe berm stability is affected by wave height, water depth at the top of the toe berm, width of the toe berm, and armor unit density. However, wave steepness does not appear to be a critical toe berm stability parameter.

It is stated that, toe berm stability formulas are based exclusively on small scale physical model tests.

According to the given information on toe berm, it is decided to present the results of physical modeling on berm stability studies (Fışkın, 2004) carried out in Middle East Technical University Coastal and Harbor Engineering Laboratory on Eastern Black Sea Highway Project where the effects of wave are severe.

In these model investigations, the coastal defense structure is designed to survive under the extreme wave conditions with sustaining the predetermined damage and serviceability limits during its lifetime. The dominating parameter which is the design wave height is selected and calculated with the help of extreme wave analysis for the selected region and return period. Return period is defined as the interval of time in which the design wave height is exceeded once. Therefore, return period determination also dominates the dimensions of the coastal defense structure and outcomes of the design. For this study 50 years return period is selected. Design life 50 years is used according to table

4.5. which is prepared for the general utilization of Level I methods by the Maritime Works Recommendations (ROM, 1990) for the preliminary design.

Another important parameter in the determination of design wave characteristics is to determine the deep water wave steepness. It was determined to be H_0/L_0 = 0.038 (Özhan and Abdalla, 1999) representing the general wave characteristics of the region.
When the averages of total storm durations were taken, the storm duration for the region was found out as 8 hours. This value can be considered as the mean value of the total storm durations.

According to the extreme wave height studies carried out for the selected region for return periods of 25 and 50 years, maximum deep water significant wave height is ranging between H_s=5.75 meters with T_s=9.85 seconds (Sürmene) and H_s=6.40 meters with T_s=10.39 seconds (Poti offshore) and the maximum deep water significant wave ranges between H_s=6.30 meters, T_s=10.31 seconds (Sürmene) and H_s=7.00 meters, T_s=10.87 seconds (Hopa), respectively.

After finding out the significant wave heights and periods for the selected regions, the depth at the toe of the structure was determined to be as 7.50 m. That is the maximum reached depth of construction which is determined by examining the topographic maps of Giresun region.

Waves breaking on the structure have the most hazardous effect on the stability of the structure, since the waves have the maximum energy when they are breaking. This means that the most important point in designing the coastal defense structures is to determine the breaker wave height at the construction depth of the structure. In order to determine the breaking wave properties at the toe of the structure and their deep water properties, the charts given in Coastal Engineering Manual (CEM, 2003) were used. Breaker wave height at d=7.50 meters was found to be H_b=6.50 meters and deep water wave height correspondent was determined as H_s= 5.80 meters after the application of the computations. Significant wave period was found as T_s = 9.90 seconds with the wave steepness of 0.038 which was determined by Özhan and Abdalla 1999.

Several cross-sections were tested under regular wave generation in the flume and damages in the layers and total damages were reported. In this part, only the model giving minimum damage will be presented. In Figure 6.6 the cross-section of Model 2 (Fışkın, 2004) is given. The damage curve for the model is presented in Figure 6.7.

Minimum cumulative total damage was observed as 1% on the Model 2 (Figure 6.7) which was constructed with 15 meters berm width and (4-6) tons front and back armor layer stone sizes.

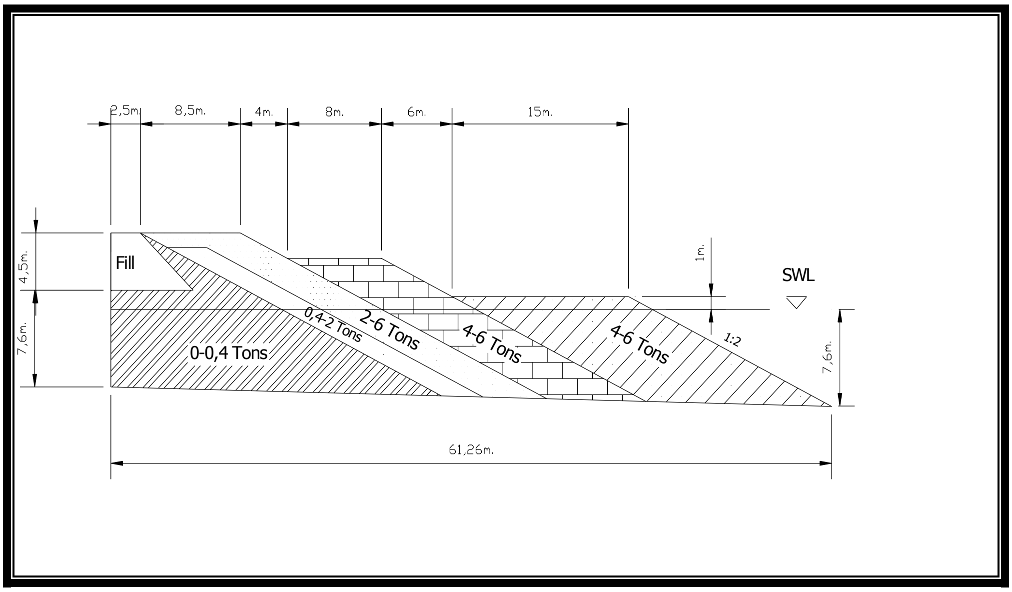

Figure 6.6 Cross-section of Model 2 (Prototype Values)

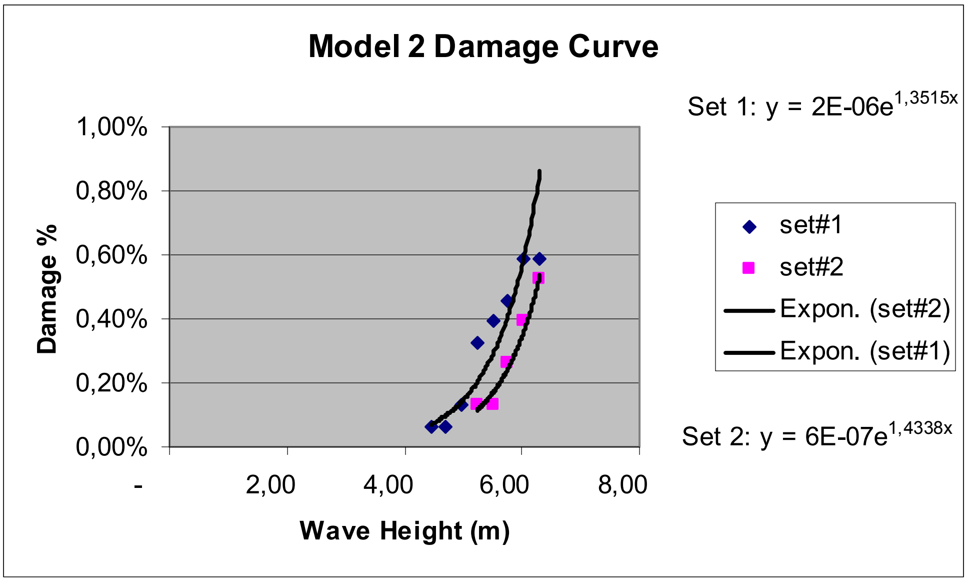

Figure 6.7 Damage vs. Wave Height Curve of Set1 and Set2 Experiments Model 2

To conclude, toe berm protection has a vital importance from both stability and economy point of view since toe berm construction results in smaller armor weights and smaller damage levels when compared with structures without berms.

CHAPTER 7

CONCLUSIONS AND FUTURE RECOMMENDATIONS

In this thesis, a new reliability-based design computer model (ARIN) is developed which practically evaluates the reliability levels of rubble-mound coastal defense structures, both existing and in design stage.

This model enables quick evaluations of reliability of rubble-mound structures for even for designer who do not have a deep knowledge about reliability-based design methodology.

In recent years, due to the existing trend of optimum cost and optimum safety design criteria in construction works, interest in probabilistic designs have increased. However, it is not the case in coastal works as practical models have not been introduced in coastal engineering for reliability-based design methodologies This computer program is developed in order emphasize the use of reliability-based design algorithm in coastal structure works carried out and designed in Turkey.

Computer models have to be verified by case studies because of the complexity of coded algorithm of problems to be solved. In this work, ARIN is verified for several cases and differing combinations of parameters in order to decrease errors and to increase reliability of the program.

Comparison of reliability-based and deterministic design models for all cases in this study shows that, the difference between the models is basically highlighted in the exceedance of predetermined damage levels, which reflects the difference in methodologies. In reliability-based design model, starting point is the determination of exceedance probability of damage level ending in the calculation of the corresponding nominal armor unit diameter D_{n50}. In contrast, deterministic design focuses on the estimation of nominal armor unit diameter using design equations written for a given damage level, and deterministic design ends with checking whether the exceedance probability is satisfied or not.

This model is a valuable design tool as it enables the user to determine outcomes of a preliminary design and also enables to evaluate an existing structure.

In the reliability-based design model, further studies are required for the development of the program. Basic design parameters are treated with selected probability distributions with their mean and standard deviation values. These distribution types are selected and implemented by expert opinion and collected data for some parameters. Therefore, in order to obtain the exact load and resistance responses of the rubble-mound structures, it is recommended to obtain better estimates of distributions by the accumulation of long term data.

In this study, level II reliability methods are implemented for the reliability-based design of rubble-mound breakwaters where first and second moments of the design parameters are used. However, evaluation of joint probability density function of limit state equation named as Level III method in reliability analysis is not performed. It is recommended to examine the behavior of rubble-mound breakwaters with Level III method if appropriate distributions of basic design variables and also correlation between these variables can be found under the light of accumulating long term data leading to better estimates of probability distributions and correlation coefficients.

Toe berm analysis has to be carried out for the structures designed by reliability-based design model as stated in part 6.5.

Further developments on this program will be on the design of rubble-mound structures by artificial units.

As stated before, storms are assumed to be independent throughout the study. However, for some resistance variables, such as concrete strength, it is unrealistic to assume the events of each year are independent. The calculated values of the failure probability in T-years using H_{s1} *year* and H_s^T will be different. The difference will be very small if the variability of H_s is much larger than the variability of other variables. Therefore, this recommendation has to be inserted in the development stage for artificial units.

Van der Meer limit state function does not include the depth effect which can be directly used when effects of swell, tsunami and sea level rise are demanded to be added as parameters to the reliability-analysis. Therefore, it is recommended to study the depth effect in Van der Meer equations.

The model can be upgraded by studying the changes in armor weight in lifetime of the project under wave attack since changes in shapes and weights of armor units are observed for existing rubble-mound breakwaters.
A complementary study is required for the determination of cost analysis of the coastal projects designed with reliability-based design model determining construction and maintanence costs for coastal projects.

Finally, reliability-based design model ARIN for rubble-mound breakwaters gives the chance to a designer to select the exceedance probability of the selected damage level for the design hence total cost optimization can be easily carried out considering the initial and the maintanence costs using the probability of failure versus nominal armor stone diameter

REFERENCES

Ang, A.H-S., Tang W.H., 1984. *Probability Concepts in Engineering Planning and Design*, Vol. 1 and 2, John Wiley and Sons, Inc., U.S.A.

Balas C. E., 1999. *Reliability-Based Risk Assessment Model for Coastal Projects*, Ph.D. Thesis, Graduate School of Natural and Applied Sciences, Middle East Technical University, Ankara, Turkey.

Balas, C.E. and Ergin, A., 2000, "A Sensitivity study for the Second Order Reliability-Based Design Model of Rubble Mound Breakwaters", *Coastal Engineering Journal*, pp. 57-86.

Balas, C.E. and Ergin, A., 2002, "Reliability-Based Risk Assessment in Coastal Projects: A Case Study in Turkey", *ASCE Journal of Waterway, Port, Coastal, & Ocean Engineering*, Vol.111., pp.678-898.

Bea, R.G., 1999, "Reliability Based Earthquake Design Guidelines for Marine Structures", *ASCE Journal of Waterway, Port, Coastal, & Ocean Engineering*, Vol. 125, No.5.

Burchart, H.F., 1992, "Reliability Evaluation Of a Structure At Sea", *Proceedings of the Short Course on Design and Reliability of Coastal Structures*, Venice, Italy.

Burchart, H.F., Sorensen, J.D. and Christiani, E., 1995, "Application of Reliability Analysis for Optimal Design of monolithic Vertical Wall Breakwaters", *International Conference on Coastal and Port Engineering in Developing Countries*, pp. 1321-1336.

CEM., 2003. *Coastal Engineering Manual*, Coastal Engineering Research Center, Dept. of Army Corps of Engineers, US, Chapter VI, Part 6.

Crowder, M.J., 1991. *Statistical Analysis of Reliability Data*, Chapman & Hall, London.

Dovich, R.A., 1990. *Reliability Statistics*, ASQC Quality Press, Milwaukee, Wis.

Ergin, A., 2009. *Coastal Engineering*, METU Press, Ankara, Turkey.

Ergin, A. and Abdalla, S., 1993. "Comparative Study on Breaking Wave Forces on Vertical Walls", *ASCE Journal of Waterways, Ports, Coastal and ocean Engineering*, Vol. 119, No. 5, pp. 560-567.

Ergin, A. and Abdalla, S., 1995. "A Comparative Study on Breaking Wave Forces on Vertical Walls Closure", *ASCE Journal of Waterways, Ports, Coastal and ocean Engineering*, Vol. 121, No. 5, pp.220-272.

Fışkın, G., 2004. *A Case Study on the Stability of Berm Type Coastal Defense Structures*, MS Thesis, METU,Ankara, Turkey.

General Directorate of Cocstruction of Railways, Ports and Airports, 2005. *Foça-Leventler Askeri Limanı Dalgakıran Stabilitesine ait Fiziksel Model Çalışmaları Raporu*, Ankara, Turkey.

General Directorate of Cocstruction of Railways, Ports and Airports, 2009. *Sinop Demirciköy Balıkçı Barınağı Fiziksel Model Deneyleri Raporu*, Ankara, Turkey.

Goda, Y., 2001. “Performance-Based Design of Caisson Breakwaters with New Approach to Extreme Wave Statistics”, *Coastal Engineering Journal*, Vol. 43, No.4, pp. 289-316.

Goda, Y. and Takagi, H., 2000. “A Reliability Design Method of Caisson Breakwaters With Optimal Wave Heights”, *Coastal Engineering Journal.*

Goda, Y., 2000. *Random Seas and Design of Maritime Structures*, University of Tokyo Press, Japan.

Hasofer, A.M. and Lind, N.C., 1974. “ An Exact and Invariant First Order Reliability Format”, *Journal of Engineering Mechanics*, A.S.C.E., Vol. 100, No.1., pp. 111-121, U.S.A.

Harr, M.E., 1987. *Reliability-Based Design in Civil Engineering*, McGraw-Hill Book Company, New York, U.S.A.

Kılıçoğlu, A., Aral, F. and Yalçıner, A.C., 2004. *Expert Report On Giresun Port,* Ankara, Turkey.

Kottegoda, N.T. and Rosso, R., 1998. *Statistics, Probability and Reliability for Civil and Environmental Engineers*, McGraw-Hill International Editions, Civil Engineering Series, Singapore.

Meeker, W.Q., 1998. *Statistical Methods for Reliability Data,* Wiley, New York.

Nowak, A.S., and Collins, K.R., 2000. *Reliability of Structure*, McGraw-Hill Higher Education, U.S.A.

Özhan, E., and Abdalla, S., 1999. *Türkiye Kıyıları Rüzgar ve Dalga Atlası* , ODTÜ İnş. Müh. Böl. Deniz Müh. Araştırma Merkezi, Ankara, Turkey.

Özhan, E., and Yalçıner, A.C., 1995. *Determination of Existing Situation in TEAŞ Akkuyu Harbor Breakwater Construction*, Technical Report No:52, Coastal and Harbor Engineering Research Center, Civil Engineering Department, METU, Ankara, Turkey.

Özler, B., 2004. *A Case Study on the Submerged Berm Type Coastal Defense Structures*, MS Thesis, METU,Ankara, Turkey.

Özyurt, G., and Özbahçeci, B., 2008. "Tasarım Dalgasının Bulunmasında Dağılım Modelinin Etkisi", *Türkiyenin Kıyı ve Deniz Alanaları 7. Ulusal kongresi,* 2008, Ankara.

Ranganathan, R., 1990, *Reliability Analysis and Design of Structures*, Tata McGraw-Hill Publishing Company Limited, New Delhi.

Rao, S.S., 1992. *Reliability-Based Design*, McGaw-Hill Inc., New York.

Reeve D., Chadwick, A. and Fleming, C., 2004. *Coastal Engineering*, Spon Press, New York, U.S.A.

ROM, 1990. *Maritime Works Recommendations (ROM 0.2-90)*, Actions in the Design of Maritime and Harbor Works, Madrid, Spain.

Sorensen, J.D., Kroon, I.B., and Faber, M.H., 1994. "Optimal reliability-based code calibration", *Structural Safety*, Vol.15., pp.197-208.

SPM. 1984. *Shore Protection Manual*, Coastal Engineering Research Center, U.S. Army, Corps of Engineers, Virginia, USA.

Williams, A.T.; Davies P.; Ergin A. and Balas C.E., 1998. "Coastal Recession and the Reliability of Planned Responses: Colhuw Beach, the Glamorgan

Heritage Coast, Wales, UK", *Journal of Coastal Research*, SI No.26, pp. 72-79.

Yüksek, Ö.; Karasu, S. and Kömürcü, M., 2000. "Bir Fırtınanın ve Neden Oldugu Hasarların Analizi", *III. Ulusal Kıyı Mühendisliği Kongresi Bildiriler Kitabı* , Çanakkale.

Van der Meer, J. W., 1988a. *Rock Slopes and Gravel Beaches Under Wave Attack,* Ph.D. Thesis, Delft University of Technology, Netherlands.

Van der Meer, J. W., 1988b. "Deterministic and Probabilistic Design of Breakwater Armor Layers", *Journal of Waterway, Port, Coastal Ocean Eng*., ASCE 114, Vol-1, pp. 66-80.

Van der Meer, J. W. and Koster, M. J., 1988. Application of computational model on dynamic stability. *Proc. Breakwaters 88*, Eastbourne, Thomas Telford.

Van der Meer, J. W., 1992. "Conceptual Design of Rubble Mound Breakwaters", *Design and Reliability of Coastal Structure*s, Proceedings of the short course on design and reliability of coastal structures, Venice, Italy.

APPENDIX A

EXTREME WAVE ANALYSIS

The first step in the design of all type of coastal structures is the selection of the design waves. In most cases, wave heights are chosen on the basis of statistical analysis of extreme events which is called extreme wave analysis. In this chapter, statistical technique used in selection of the design wave heights in case studies of thesis work is defined.

Depending on the method of selecting a set of wave data, there are three different approaches. One method tries to utilize the whole data of wave heights observed visually or instrumentally during a number of years. The data are analyzed in a form of cumulative distribution function. Once a best-fitting distribution function is found, the design wave height is estimated by extrapolating the distribution function to the level of probability which corresponds to a given period of years being considered in design process. This method is called the *total sample method* or *cumulative distribution function method* (Goda, 2000).

The other two methods are based on the use of maximum values of wave heights in time series data. The *annual maxima method* uses the largest significant wave height data in each year. On the other hand, *peaks-over-threshold method* picks up the peak wave heights during storms over a pre-defined threshold value.

Selection between these methods is a subjective approach. However, independency and homogeneity criteria of the sample should be treated as requisites during the selection of the methods. According to these two vital criteria total sample method is not recommended. Peaks-over-threshold (POT) method is the widely used one in coastal engineering works since it covers a large number of data in a sample resulting in a smaller range of confidence interval. The wave height values are selected by POT in this work field.

In this study extreme wave analysis is done by the method proposed by Goda (Goda, 2000) and applied with a tool written by Özyurt and Özbahçeci (Özyurt and Özbahçeci, 2008). The steps of the extreme analysis are described in a logical order in sub-sections with appropriate headings.

A.1 Distribution functions for extreme waves

In the extreme data analysis, many theoretical distribution functions are employed for fitting to samples. Since the data of extreme wave heights collected by POT carry no meaning of maxima of samples, there is no theoretical recommendation to use any distribution function.

Current consensus among people working on extreme wave analysis is to apply various distribution functions to a set of data and to select the best fitting one as the representative distribution of the whole wave data of the selected region.

Distribution functions that are mostly employed in extreme wave analysis are listed below.

A.1.1 Fisher-Tippett type I (Gumbel) distribution

Cumulative distribution function,

$$F(x) = exp\left[-\exp\left(-\frac{H-B}{A}\right)\right] \tag{A.1}$$

Probability density function

$$f(x) = \frac{1}{A} exp\left[-exp\left(-\frac{H-B}{A}\right)\right] exp\left(-\frac{H-B}{A}\right) \tag{A.2}$$

where;

H: extreme wave height

A: scale parameter of the distribution (has same unit with the wave height)

B: location parameter of the distribution (has same unit with the wave height)

A.1.2 Fisher-Tippett type II distribution

Cumulative distribution function,

$$F(x) = exp\left[-\left(1 + \frac{H-B}{kA}\right)^{-k}\right] \tag{A.3}$$

Probability density function,

$$f(x) = \frac{1}{A} exp\left[-\left(1 + \frac{x-B}{kA}\right)^{-k}\right]\left(-\frac{x-B}{kA}\right)^{-k-1} \tag{A.4}$$

where;

H: extreme wave height

A: scale parameter of the distribution (has same unit with the wave height)

B: location parameter of the distribution (has same unit with the wave height)

k: shape parameter (has no dimension)

A.1.3 Weibull distribution

Cumulative distribution function,

$$F(x) = 1 - exp\left[-\left(\frac{H-B}{A}\right)^{k}\right] \tag{A.5}$$

Probability density function,

$$f(x) = \frac{k}{A} exp\left[-\left(\frac{H-B}{A}\right)^{k}\right]\left(\frac{H-B}{A}\right)^{-k-1} \quad (A.6)$$

Where;

$k > 0$ is the *shape parameter* and

A > 0 is the *scale parameter* of the distribution

The Weibull distribution is related to a number of other probability distributions; in particular, it interpolates between the exponential distribution (k = 1) and the Rayleigh distribution (k = 2).

A.1.4 Log-normal distribution

Cumulative distribution function,

$$F(x) = \frac{1}{2} erfc\left[-\left(\frac{lnH - B}{A\sqrt{2}}\right)\right] \quad (A.7)$$

Probability density function,

$$f(x) = \frac{1}{Ax\sqrt{2\pi}} exp\left[-\frac{(lnH - B)^2}{2A^2}\right] \quad (A.8)$$

where;

H: extreme wave height

A: scale parameter of the distribution (has same unit with the wave height)

B: location parameter of the distribution (has same unit with the wave height)

erfc: error function (defined in Figure 4.1)

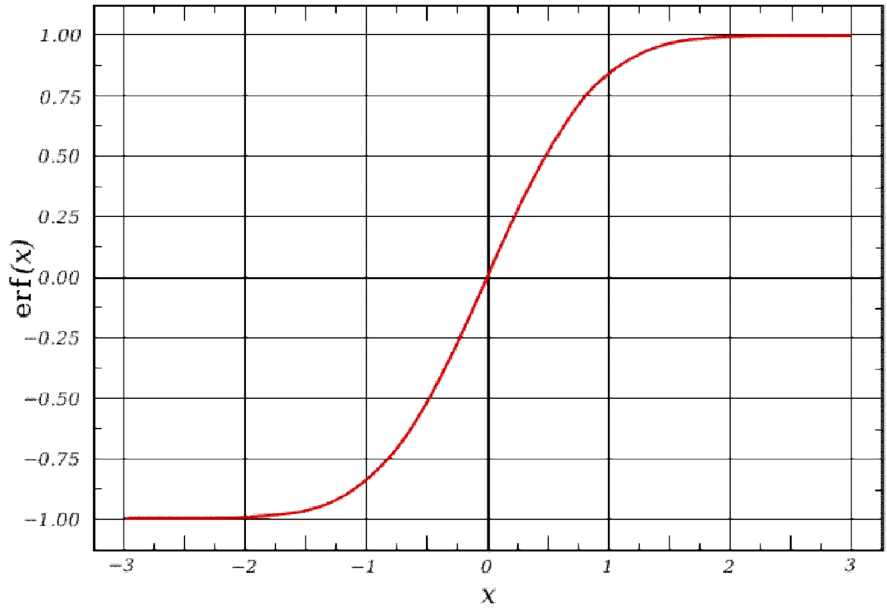

Figure A.1 Error function graph

The characteristics of the above four distributions are listed in Table A.1 for their modes, means and standard deviations.

Table A.1 Characteristics of distribution functions for extreme analysis

Distribution	**Mode**	**Mean**	**Standard deviation**
FT-II	$B + kA\left[\left(\frac{k}{1+k}\right)^{1/k} - 1\right]$	$B + kA\left[\Gamma\left(1-\frac{1}{k}\right) - 1\right]$	$kA\left[\Gamma\left(1-\frac{2}{k}\right) - \Gamma^2\left(1-\frac{1}{k}\right)\right]^{1/2}$
FT-I	B	$B + A\gamma$	$\frac{\pi}{\sqrt{6}}A$
Weibull	$B + A\left(1-\frac{1}{k}\right)^{1/k}$	$B + A\Gamma\left(1+\frac{1}{k}\right)$	$A\left[\Gamma\left(1+\frac{2}{k}\right) - \Gamma^2\left(1+\frac{1}{k}\right)\right]^{1/k}$
Lognormal	$\boldsymbol{exp\,(B - A^2)}$	$\mathbf{exp}\left(\boldsymbol{B + \frac{A^2}{2}}\right)$	$\boldsymbol{exp}\left(\boldsymbol{B + \frac{A^2}{2}}\right)\boldsymbol{(expA^2 - 1)^{1/2}}$

Note: Γ( ) is the gamma function and g is Euler's constant (0.5772)

In this study, this table is used to calculate the mean and standard deviation of best fitting extreme distribution for the selected region or site.

A.2 Return period and return value

The return period is defined as the average duration of time during which extreme events exceeding a threshold value would occur once. In addition, the return value is the threshold value which defines a return period.

The return period R is calculated derived from the distribution function as follows:

$$P_{n=}F^{n-1}(x_u)[1-F(x_u)] \tag{A.9}$$

F(x): distribution function
x_u : threshold value (return value)
n: number of years
$F^{n-1}(x_u)$: non-exceedance probability in (n-1) years
P_n: Probability of occurrence in the first year

The above event may occur in the first year of n=1. The expected value of n is the return period and is calculated as:

$$R = E[n] = \sum_{n=1}^{\infty} nP_n = [1-F(x_u)]\sum_{n=1}^{\infty} nF^{n-1}(x_u) = \frac{1}{1-F(x_u)}$$

The return value x_R is obtained with the inverse function of the distribution as:

$$x_R = F^{-1}\left(1-\frac{1}{R}\right) \tag{A.10}$$

In the case of POT with the mean rate l, each year is divided into equal segments. Then, the return period and return value is derived as:

$$R = \frac{1}{\lambda[1-F(x_u]} \tag{A.11}$$

$$x_R = F^{-1}\left(1 - \frac{1}{\lambda R}\right) \tag{A.12}$$

A.3 Parameter Estimation by the Least Squares Method

The first step is the selection of candidate distribution functions for the the parameter estimations of extreme wave data. The scale, location and shape parameters are estimated . There are several methods of fitting a distribution function to a sample of extreme wave data and estimating the distribution parameter values A,B and k. In this study, least squares method is preferred as it has a simple algorithm and applications.

The least squares method is the best estimate of two parameters in a single operation. Since FT-I distribution has two parameters of scale and location, least square method is applied directly for the parameter estimations. On the other hand, FT II and Weibull distributions have three parameters including shape (k), scale (A) and location (B). Thus, a small modification must be done to form a two parameter function.

In this study fixed values of shape parameter are used as k= 2.5, 3.33, 5.0 and 10.0 for the FTII distribution and k= 0.75, 1.0, 1.4 and 2.0 for the Weibull distribution in order to treat all the candidate distributions as independent to compete with other functions for best fitting.

The second step in the parameter estimation is the order statistics and defined as:

Arrangement f data in ascending order having the order number *m*
Assignment of the non-exceedance probabilities of order data calculated using the Weibull formula as:

$$P = \frac{m}{N+1}$$

where;

P: non-exceedance probability

m:order number

N: number of data

However, since Weibull plotting position formula produces a positive bias in the return value when the sample size is small, a new unbiased plotting position formula proposed by Goda is used in order to calculate the non-exceedance probabilities.

$$\hat{F}_{(m)} = \frac{m - \alpha}{N + \beta}$$

The values of α and β are given in Table A.2.

Table A.2 Constants of unbiased plotting position formula

Distribution	α	β	Authors
FT-II	$0.44 + 0.52/k$	$0.12 - 0.11/k$	Goda and Onozawa
FT-I	0.44	0.12	Gringorten
Weibull	$0.20 + 0.27/\sqrt{k}$	$0.20 + 0.23/\sqrt{k}$	Goda
Lognormal	$\mathbf{0.375}$	$\mathbf{0.25}$	Blom

Calculation of the reduced variate $y_{(m)}$ for the m^{th} ordered data by:

FT- I distribution: $y(m) = -\ln[-\ln\hat{F}(m)]$ (A.13)

FT- II distribution: $y_{(m)} = k\left[\left(-ln\hat{F}_{(m)}\right)^{-1/k} - 1\right]$ (A.14)

Weibull distribution: $y_{(m)} = \left[-ln\left(1 - \hat{F}_{(m)}\right)\right]^{1/k}$ (A.15)

The third step is the application of least squares method for the parameters of $\hat{A}$ and $\hat{B}$in the following equation:

$$x_{(m)} = \hat{B} + \hat{A}y_{(m)} \quad \text{(A.16)}$$

Then correlation coefficient between $x_{(m)}$ and $y_{(m)}$ is calculated.

A.4 Selection of Most Probable Parent Distribution

Selection of the most probable distribution for the extreme wave data is vital in this study as it directly affects the most important loading parameter in the reliability-based risk assessment model proposed in this study. Therefore special attention is given to this step.

Three test criteria are applied to find out the best fitting in this study. These are described in the subsections as follows:

A.4.1 Minimum ratio of residual correlation coefficient criterion (MIR)

Goda and Kobune proposed to use MIR criterion for judgment of best fitting; a distribution with the smallest ratio is a best fitting one. They derived an empirical formula for estimating the mean residue Δr_{mean} for a given distribution, sample size and censoring parameter from the data of simulation study. The formula is given as

$$\Delta r_{mean} = exp[a + blnN + c(lnN)^2] \qquad \text{(A.17)}$$

The coefficients a, b and c for different distributions are tabulated in Table A.3.

Table A.3 Empirical coefficients for Δr_{mean} in the MIR criterion

Distribution	a	b	c
FT-II (k=2.5)	-2.470+0.015$v^{3/2}$	-0.1530-0.00525$v^{5/2}$	0
FT-II(k=3.33)	-2.462-0.009v^2	-0.1933-0.0037$v^{5/2}$	-0.007
FT-II(k=5.0)	-2.463	-0.2110-0.0131$v^{5/2}$	-0.019
FT-II(k=10.0)	2.437+0.0285$v^{5/2}$	-0.2280-0.0300$v^{5/2}$	-0.033
FT-I	-2.364+0.54$v^{5/2}$	-0.2665-0.0457$v^{5/2}$	-0.044
Weibull (k=0.75)	-2.435-0.168$v^{1/2}$	-0.2083+0.1074$v^{1/2}$	-0.047
Weibull (k=1.0)	-2.355	-0.2612	-0.043
Weibull (k=1.4)	-2.277+0.056$v^{1/2}$	-0.3169-0.0499v	-0.044
Weibull (k=2.0)	-2.160+0.113v	-0.3788-0.0979v	-0.041
Lognormal	-2.153+0.059v^2	-0.2627-0.17165$v^{1/4}$	-0.045

A.4.2 Outlier detection by the Deviation of Outlier (DOL) criterion

This criterion is treated in order to eliminate the outliers which exhibit the value much larger than the rest of data in an extreme series. This criterion is also proposed by Goda and Kobune and/or a statistical test by Barnett and Lewis. The DOL criterion uses the following dimensionless deviation ξ in order to calculate the deviation of the largest value:

$$\xi = \frac{H_{(1)} - \bar{H}}{s} \tag{A.18}$$

where;

$\bar{H}$: is the mean of the sample
s: standard deviation of the sample
$H_{(1)}$: largest data in the set

After finding out the dimensionless deviation of the largest value in the data set, the cumulative distribution function curve of ξ is obtained by simulating 10000 artificial samples in order to calculate the upper and lower DOL's.

The threshold value $\xi_{5\%}$ of the population is called the lower DOL and the threshold value $\xi_{95\%}$ is called the upper DOL.

If the ξ of the data lies between upper and lower DOL values, the model is selected as representative for the data.

$\xi_{5\%}$ and $\xi_{95\%}$ values are calculated by the following empirical formula:

$$\xi_{5\%} \text{ and } \xi_{95\%} = a + b\ln N + c(\ln N)^2 \tag{A.19}$$

The empirical coefficients a, b and c are tabulated in Tables A.4 and A.5.

Table A.4 Empirical coefficients for the upper DOL criterion $\xi_{95\%}$

Distribution	a	b	c
FT-II (k=2.5)	4.653-1.076$v^{1/2}$	-2.047+0.307$v^{1/2}$	0.635
FT-II(k=3.33)	3.217-1.216$v^{\frac{1}{4}}$	-0.903+0.294$v^{1/4}$	0.427
FT-II(k=5.0)	0.599-0.038v^2	0.518-0.045v^2	0.210
FT-II(k=10.0)	-0.371+0.171v^2	1.283-0.133v^2	0.045
FT-I	-0.579+0.468v	1.496-0.227v^2	-0.038
Weibull (k=0.75)	-0.256-0.632v^2	1.269+0.254v^2	0.037
Weibull (k=1.0)	-0.682	1.600	-0.045
Weibull (k=1.4)	-0.548+0.452$v^{1/2}$	1.521-0.184v	-0.065
Weibull (k=2.0)	-0.322+0.641$v^{\frac{1}{2}}$	1.414-0.326v	-0.069
Lognormal	0.178+0.740v	1.148-0.480$v^{3/2}$	-0.035

Table A.5 Empirical coefficients for the lower DOL criterion $\xi_{5\%}$

Distribution	a	b	c
FT-II (k=2.5)	$1.481-0.126v^{1/4}$	$-0.331-0.031v^2$	0.192
FT-II(k=3.33)	1.025	$-0.077-0.050v^2$	0.143
FT-II(k=5.0)	$0.700+0.060v^2$	$0.139-0.076v^2$	0.100
FT-II(k=10.0)	$0.424+0.088v^2$	$0.329-0.094v^2$	0.061
FT-I	$0.257+0.133v^2$	$0.452-0.118v^2$	0.032
Weibull (k=0.75)	$0.534-0.162v$	$0.277+0.095v$	0.065
Weibull (k=1.0)	0.308	0.423	0.037
Weibull (k=1.4)	$0.192+0.126v^{3/2}$	$0.501-0.081v^{3/2}$	0.018
Weibull (k=2.0)	$0.050+0.182v^{3/2}$	$0.592-0.139v^{3/2}$	0
Lognormal	$0.042+0.270v$	$0.581-0.217v^{3/2}$	0

A.4.3 Rejection of candidate distribution by the REC criterion

Presence of an outlier suggests that a particular distribution is better eliminated from the candidates of parent distributions. When the distribution fitting is made with least squares method, the value of the correlation coefficient r between the ordered variate $x_{(m)}$ and the reduced variate $y_{(m)}$ can provide another test for the rejection of candidate distributions. The residue of correlation coefficient $\Delta r=1-r$ is employed for this purpose.

The exceedance probability of 0.95 is set for establishing the threshold value of Δr at the significance level of 0.05. $\Delta r_{95\%}$ is obtained by the empirical expression of :

$$\Delta r_{95\%} = exp[a + blnN + c(lnN)^2] \quad 20)$$

The coefficients of the equation (number) is listed in table A.6

Table A.6 Empirical coefficients for Δ $r_{5\%}$ in the REC criterion

Distribution	a	b	c
FT-II (k=2.5)	-1.122-0.037v	-0.3298+0.0105$v^{1/4}$	0.016
FT-II(k=3.33)	-1.306-0.105$v^{3/2}$	-0.3001+0.0404$v^{\frac{1}{2}}$	0
FT-II(k=5.0)	-1.463-0.107$v^{\frac{3}{2}}$	-0.2716+0.0517$v^{\frac{1}{2}}$	-0.018
FT-II(k=10.0)	-1.490-0.073v	-0.2299-0.0099$v^{\frac{5}{2}}$	-0.034
FT-I	-1.444	-0.2733-0.0414$v^{\frac{5}{2}}$	-0.045
Weibull (k=0.75)	-1.473-0.049v^2	-0.2181+0.0505v	-0.041
Weibull (k=1.0)	-1.433	-0.2679	-0.044
Weibull (k=1.4)	-1.312	-0.3356-0.0449v	-0.045
Weibull (k=2.0)	-1.188+0.073$v^{1/2}$	-0.4401-0.0846$v^{3/2}$	-0.039
Lognormal	-1.362+0.360$v^{1/2}$	-0.3439-0.2158$v^{1/2}$	-0.035

Extreme wave analysis is carried out as discussed above in a detailed manner. The outcomes of the extreme analysis for cases in the thesis are summarized in graphs and tables pointing out the selected best fitting distribution, distribution parameters and design significant wave height-period for appropriate return periods.

A.5 Extreme Wave Analysis Results of Case I: Giresun Port

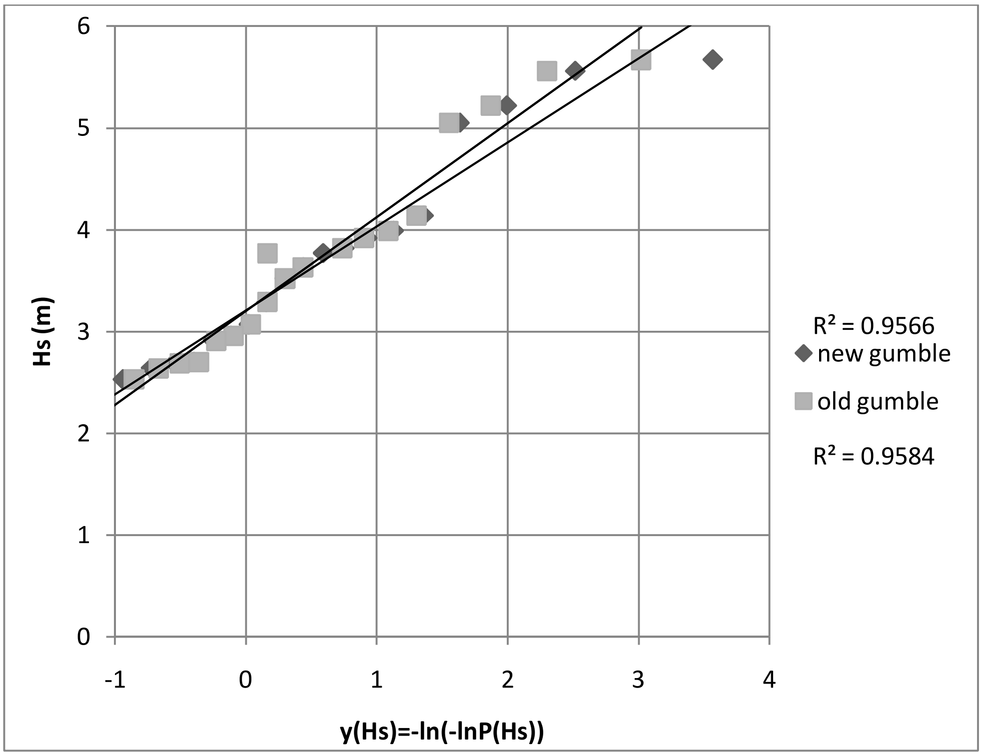

Figure A.2 Fisher-Tippet Type I (Gumble) probability distribution for extreme annual significant wave height for Giresun port with scale and location parameters of;

B (location parameter)	3.207 (new)	3.203 (old)
A (scale parameter)	0.826	0.922

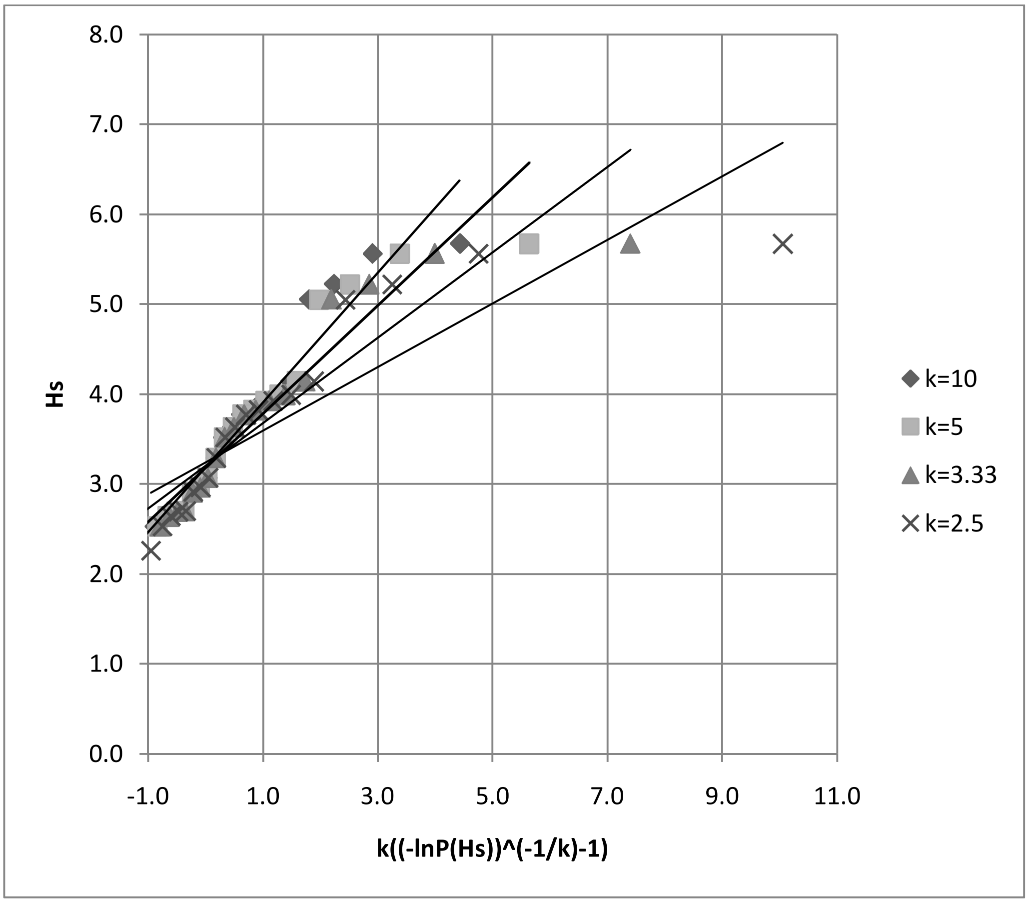

Figure A.3 Fisher-Tippet Type II probability distributions for shape parameters of k=10, k=5, k=3.33 and k=2.5 with scale and location parameters;

B (location parameter)	3,19	3,18	3,20	3,24
A (scale parameter)	0,72	0,60	0,48	0,35

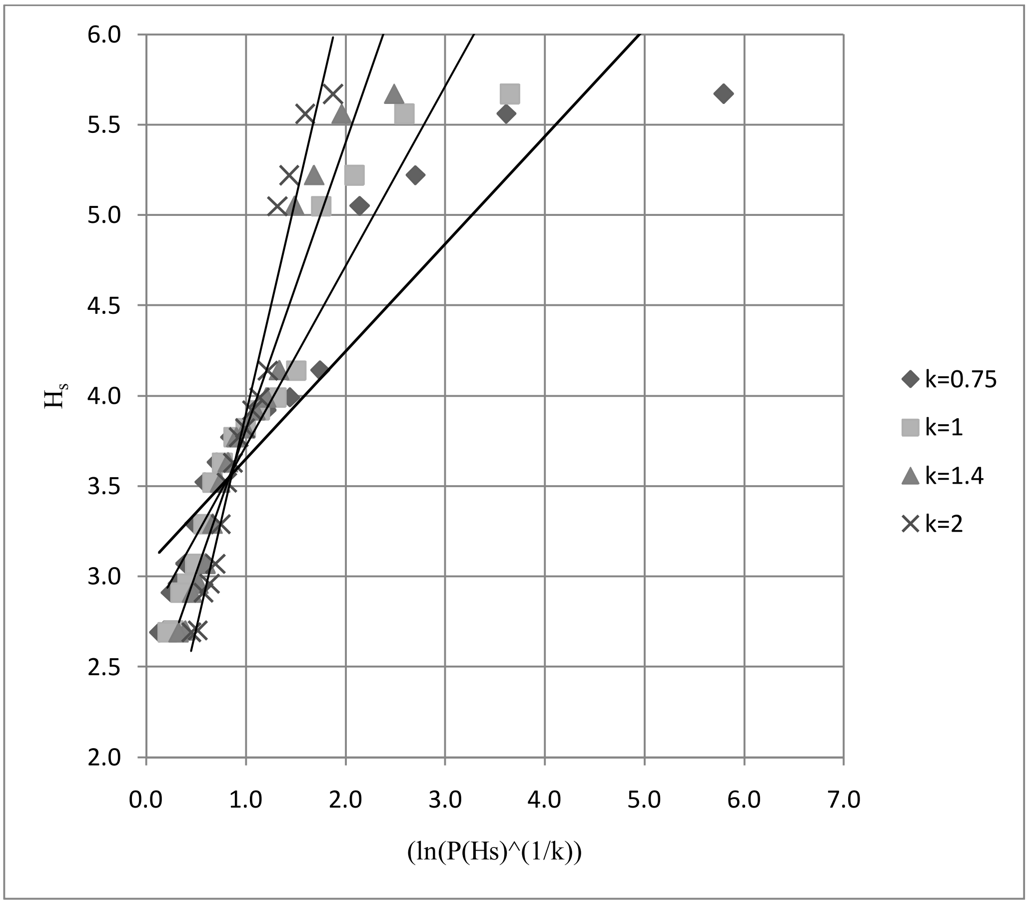

Figure A.4 Weibull probability distribution for annual extreme wave height for shape parameters of k=0.75, k=1.0, k=1.4 and k=2.0 with scale and location parameters of;

B (location parameter)	2,90	2,62	2,21	1,66
A (scale parameter)	0,65	1,05	1,60	2,26

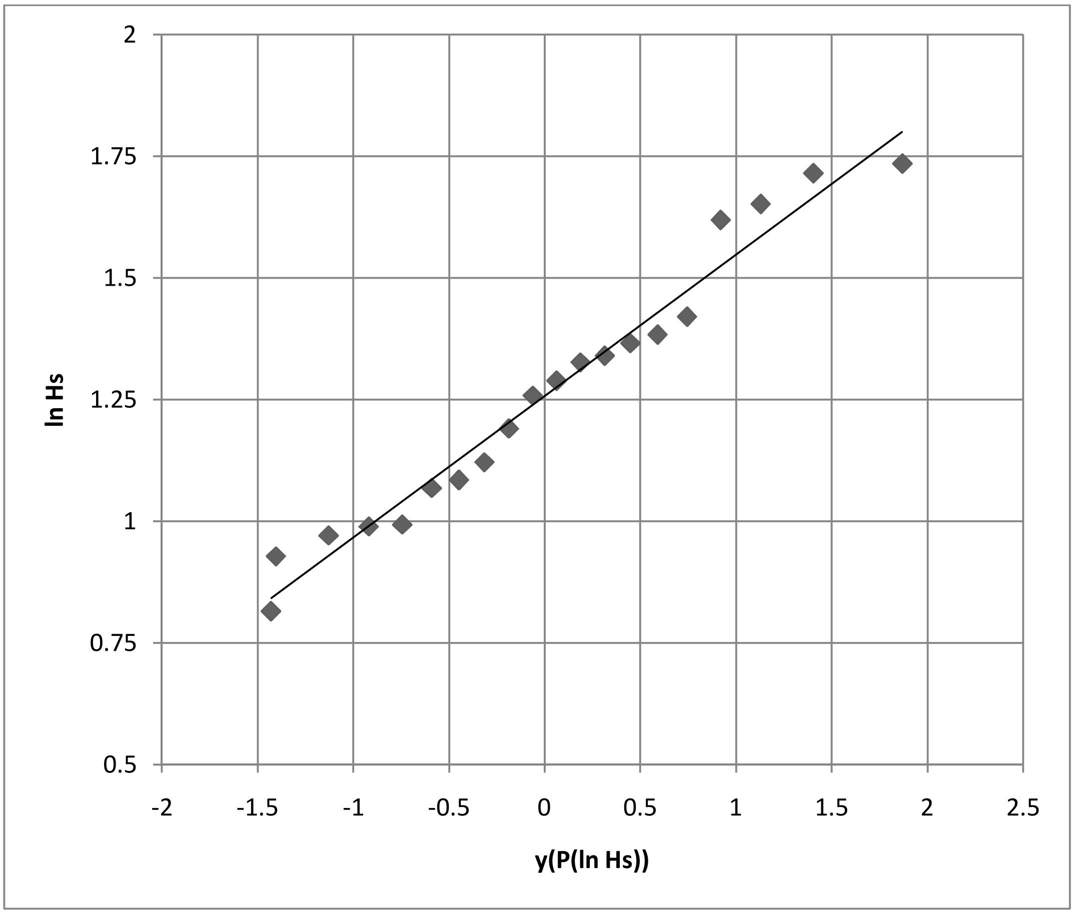

Figure A.5 Log-Normal probability distribution for extreme annual significant wave height with scale and location parameters of;

B (location parameter)	1,26
A (scale parameter)	0,29

Table A.7 Results of MIR criterion

Minimum ratio of residual correlation coefficient criterion (MIR)								
Distribution Type	R^2	R	Δr=1-R	r r_{mean}	a	b	c	Δr/r_{mean}
FT 2 (k1=2.5)	0,757	0,870	0,130	0,053	-2,455	-0,158	0,000	2,428
FT 2 (k2=3.33)	0,837	0,915	0,085	0,044	-2,471	-0,197	-0,007	1,934
FT 2 (k3=5.0)	0,898	0,948	0,052	0,037	-2,463	-0,224	-0,019	1,430
FT 2 (k4=10.0)	0,937	0,968	0,032	0,031	-2,409	-0,258	-0,033	1,033
Gumbel1 (old)	0,958	0,979	0,021	0,043	-1,824	-0,312	-0,044	0,492
Gumbel2 (new)	0,957	0,978	0,022	0,043	-1,824	-0,312	-0,044	0,515
LogNormal	0,970	0,985	0,015	0,022	-2,094	-0,434	-0,045	0,664
Weibull (k1=0.75)	0,836	0,914	0,086	0,036	-2,603	-0,101	-0,047	2,390
Weibull (k2=1.0)	0,918	0,958	0,042	0,029	-2,355	-0,261	-0,043	1,427
Weibull (k3=1.4)	0,961	0,980	0,020	0,024	-2,221	-0,367	-0,044	0,806
Weibull (k4=2.0)	0,964	0,982	0,018	0,021	-2,047	-0,477	-0,041	0,836

Table A.8 Results of REC criterion

REC (residue of correlation coefficient)								
Distribution Type	r^2	r	1-r	$r_{\%95}$	a	b	c	
FT 2 (k1=2.5)	0,757	0,870	0,130	0,139	-1,159	-0,319	0,016	ACCEPTED
FT 2 (k2=3.33)	0,837	0,915	0,085	0,112	-1,411	-0,260	0,000	ACCEPTED
FT 2 (k3=5.0)	0,898	0,948	0,052	0,092	-1,570	-0,220	-0,018	ACCEPTED
FT 2 (k4=10.0)	0,937	0,968	0,032	0,075	-1,563	-0,240	-0,034	ACCEPTED
Gumbel1 (old)	0,958	0,979	0,021	0,061	-1,444	-0,315	-0,045	ACCEPTED
Gumbel2 (new)	0,957	0,978	0,022	0,061	-1,444	-0,315	-0,045	ACCEPTED
LogNormal	0,970	0,985	0,015	0,050	-1,002	-0,562	-0,035	ACCEPTED
Weibull (k1=0.75)	0,836	0,914	0,086	0,091	-1,522	-0,168	-0,041	ACCEPTED
Weibull (k2=1.0)	0,918	0,958	0,042	0,072	-1,433	-0,268	-0,044	ACCEPTED
Weibull (k3=1.4)	0,961	0,980	0,020	0,058	-1,312	-0,381	-0,045	ACCEPTED
Weibull (k4=2.0)	0,964	0,982	0,018	0,048	-1,115	-0,525	-0,039	ACCEPTED

Table A.9 Results of R^2 test

Distribution Type	R^2	R	Comment
FT 2 (k1=2.5)	0,7573	0,8702	OUTLIER
FT 2 (k2=3.33)	0,8371	0,9149	NOT OUTLIER
FT 2 (k3=5.0)	0,8978	0,9475	NOT OUTLIER
FT 2 (k4=10.0)	0,9372	0,9681	NOT OUTLIER
Gumbel1 (old)	0,9584	0,9790	NOT OUTLIER
Gumbel2 (new)	0,9566	0,9780	NOT OUTLIER
LogNormal	0,9705	0,9851	NOT OUTLIER
Weibull (k1=0.75)	0,8358	0,9142	OUTLIER
Weibull (k2=1.0)	0,9176	0,9579	NOT OUTLIER
Weibull (k3=1.4)	0,9611	0,9804	NOT OUTLIER
Weibull (k4=2.0)	0,9645	0,9821	NOT OUTLIER

Table A.10 Parent distribution and design wave parameters for changing return periods

	Return Period	5	10	20	50	100	1000
Gumbel	H_s (m)	4,54	5,22	5,87	6,72	7,35	9,44
	T_s (sn)	8,27	8,86	9,40	10,05	10,52	11,92
LogNormal	H_s (m)	5,08	5,46	5,81	6,25	6,56	7,55
	T_s (sn)	8,87	9,20	9,49	9,83	10,08	10,81

Log-normally distributed design wave height with Hs=6.25 and Ts=9.83 with corresponding A and B parameters is selected to be used at the reliability based design model, ARIN.

A.6 Extreme Wave Analysis Results of Case II: Foça-Leventler Military Port

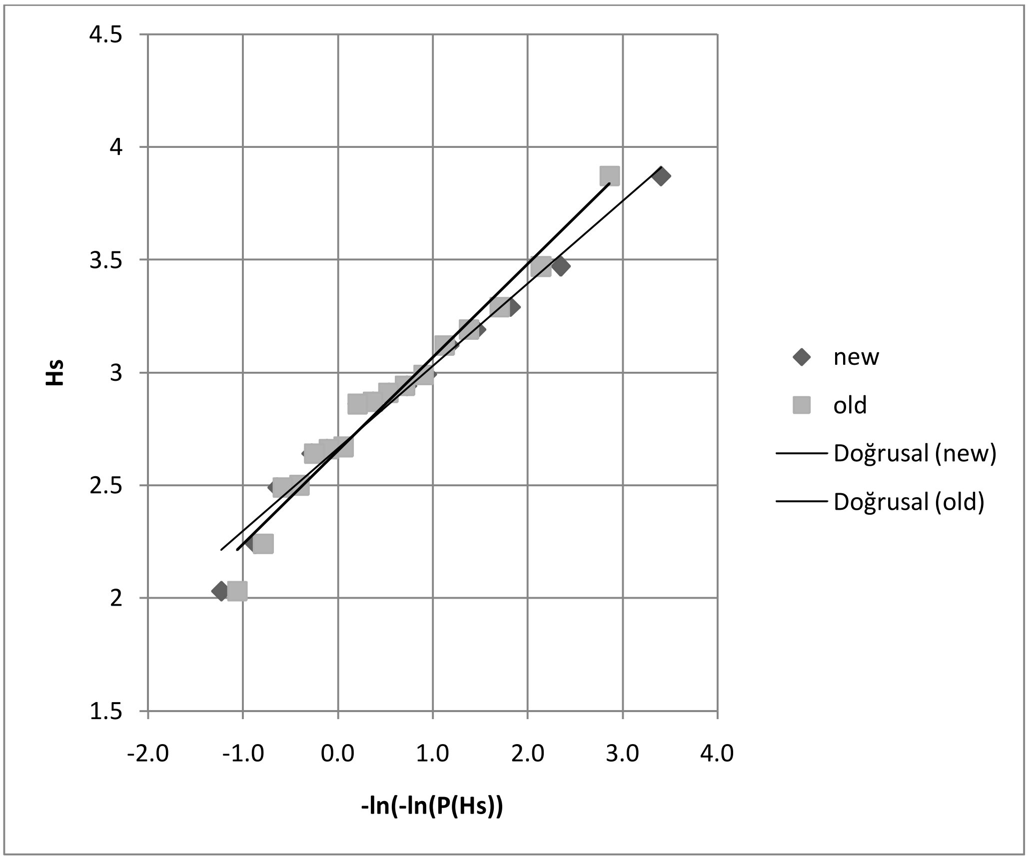

Figure A.6 Fisher-Tippet Type I (Gumble) probability distribution for extreme annual significant wave height for Foça-Leventler Military Port with scale and location parameters of;

B (location parameter)	2,664 (new)	2,653 (old)
A (scale parameter)	0,366	0,414

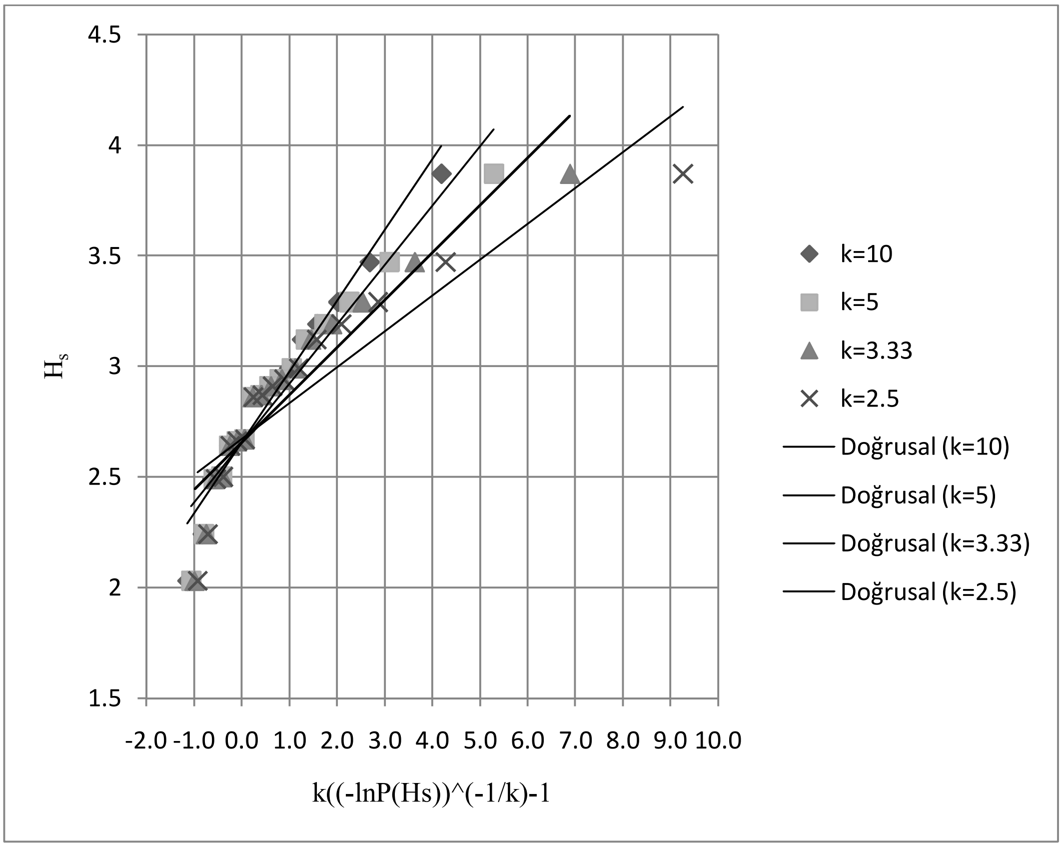

Figure A.7 Fisher-Tippet Type II probability distributions for shape parameters of k=10, k=5, k=3.33 and k=2.5 with scale and location parameters of;

B (location parameter)	2,65	2,65	2,66	2,67
A (scale parameter)	0,32	0,27	0,21	0,16

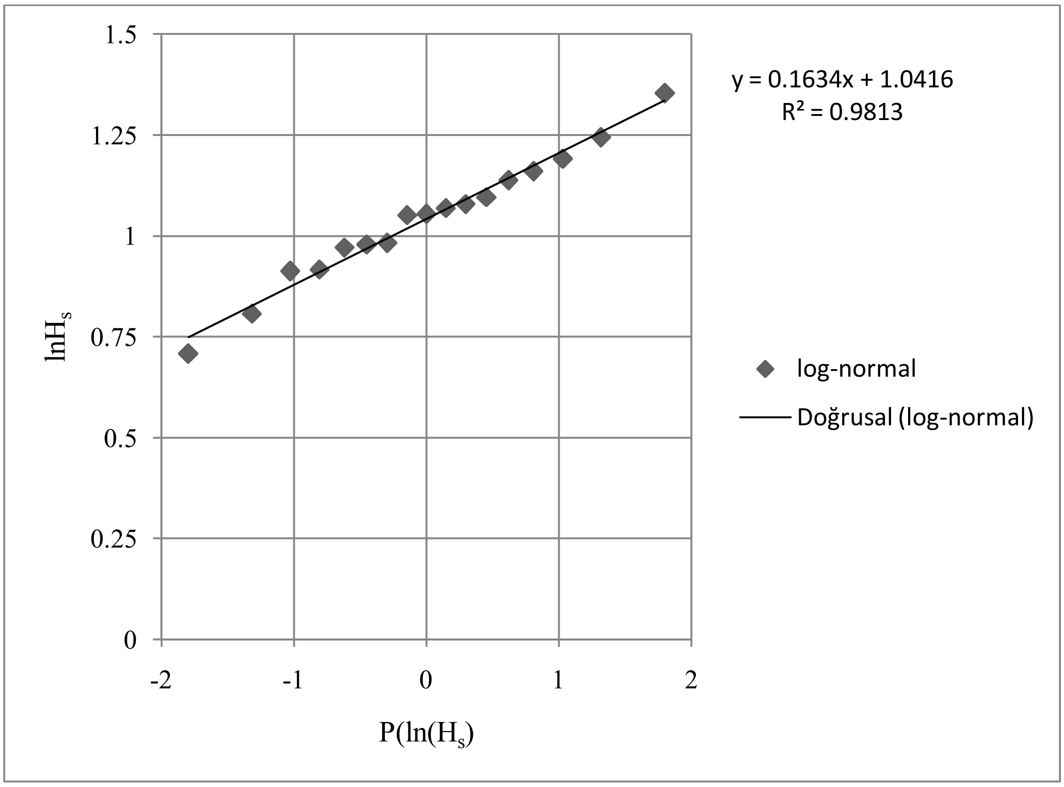

Figure A.8 Log-Normal probability distribution for extreme annual significant wave height with scale and location parameters of;

B (location parameter)	1,042
A (scale parameter)	0,163

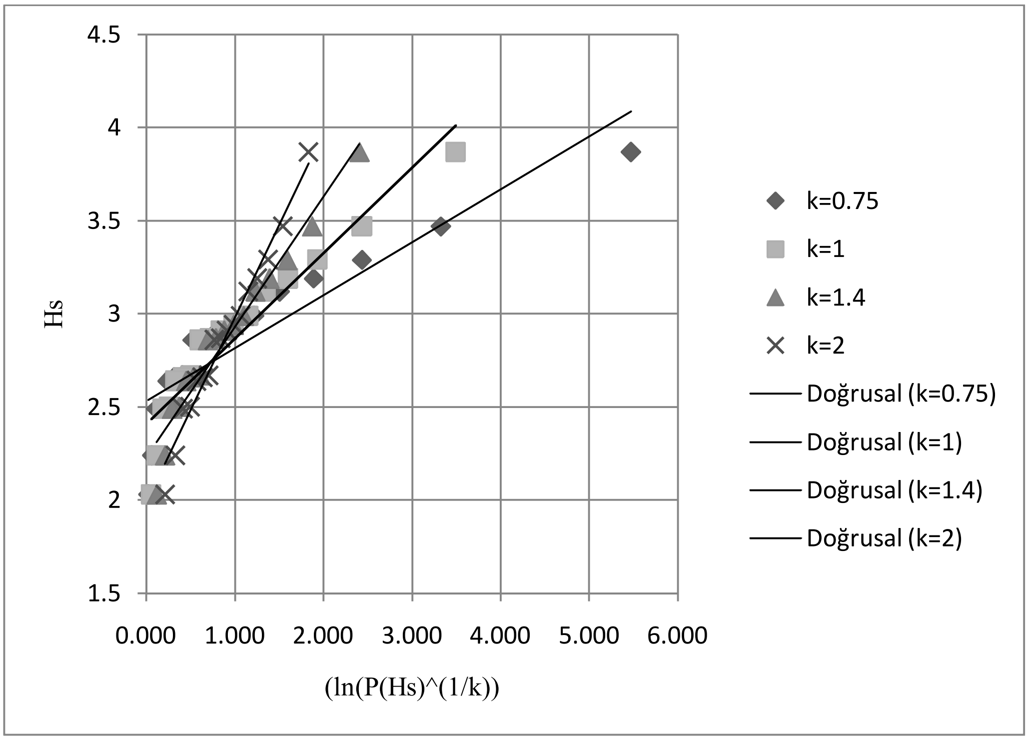

Figure A.9 Weibull probability distribution for annual extreme wave height for shape parameters of k=0.75, k=1.0, k=1.4 and k=2.0 with scale and location parameters of;

B (location parameter)	2,53	2,41	2,23	1,982
A (scale parameter)	0,284	0,459	0,699	0,998

Table A.11 Results of MIR criterion

MIR (Minimum ratio of residual correlation)								
Distribution Type	R^2	R	Δr=1-R	r_{mean}	a	b	c	$\Delta r/r_{mean}$
FT 2 (k1=2.5)	0,801	0,895	0,105	0,055	-2,455	-0,158	0,000	1,918
FT 2 (k2=3.33)	0,865	0,930	0,070	0,046	-2,471	-0,197	-0,007	1,528
FT 2 (k3=5.0)	0,916	0,957	0,043	0,039	-2,463	-0,224	-0,019	1,109
FT 2 (k4=10.0)	0,952	0,976	0,024	0,033	-2,409	-0,258	-0,033	0,736
Gumbel1 (old)	0,972	0,986	0,014	0,047	-1,824	-0,312	-0,044	0,301
Gumbel2 (new)	0,974	0,987	0,013	0,047	-1,824	-0,312	-0,044	0,284
LogNormal	0,981	0,991	0,009	0,025	-2,094	-0,434	-0,045	0,375
Weibull (k1=0.75)	0,825	0,908	0,092	0,038	-2,603	-0,101	-0,047	2,409
Weibull (k2=1.0)	0,899	0,948	0,052	0,032	-2,355	-0,261	-0,043	1,622
Weibull (k3=1.4)	0,951	0,975	0,025	0,027	-2,221	-0,367	-0,044	0,920
Weibull (k4=2.0)	0,975	0,987	0,013	0,024	-2,047	-0,477	-0,041	0,532

Table A.12 Results of REC criterion

REC (residue of correlation coefficient)								
Distribution Type	r^2	r	Δr=1-r	$r_{\%95}$	a	b	c	
FT 2 (k1=2.5)	0,801	0,895	0,105	0,144	-1,159	-0,319	0,016	ACCEPTED
FT 2 (k2=3.33)	0,865	0,930	0,070	0,117	-1,411	-0,260	0,000	ACCEPTED
FT 2 (k3=5.0)	0,916	0,957	0,043	0,097	-1,570	-0,220	-0,018	ACCEPTED
FT 2 (k4=10.0)	0,952	0,976	0,024	0,081	-1,563	-0,240	-0,034	ACCEPTED
Gumbel1 (old)	0,972	0,986	0,014	0,067	-1,444	-0,315	-0,045	ACCEPTED
Gumbel2 (new)	0,974	0,987	0,013	0,067	-1,444	-0,315	-0,045	ACCEPTED
LogNormal	0,981	0,991	0,009	0,056	-1,002	-0,562	-0,035	ACCEPTED
Weibull (k1=0.75)	0,825	0,908	0,092	0,098	-1,522	-0,168	-0,041	ACCEPTED
Weibull (k2=1.0)	0,899	0,948	0,052	0,078	-1,433	-0,268	-0,044	ACCEPTED
Weibull (k3=1.4)	0,951	0,975	0,025	0,064	-1,312	-0,381	-0,045	ACCEPTED
Weibull (k4=2.0)	0,975	0,987	0,013	0,054	-1,115	-0,525	-0,039	ACCEPTED

Table A.13 Results of R^2 test

Distribution Type	R^2	R	Comment
FT 2 (k1=2.5)	0.801	0.895	NOT OUTLIER
FT 2 (k2=3.33)	0.865	0.930	NOT OUTLIER
FT 2 (k3=5.0)	0.916	0.957	NOT OUTLIER
FT 2 (k4=10.0)	0.952	0.976	NOT OUTLIER
Gumbel1 (old)	0.972	0.986	NOT OUTLIER
Gumbel2 (new)	0.974	0.987	NOT OUTLIER
LogNormal	0.981	0.991	NOT OUTLIER
Weibull (k1=0.75)	0.825	0.908	NOT OUTLIER
Weibull (k2=1.0)	0.899	0.948	NOT OUTLIER
Weibull (k3=1.4)	0.951	0.975	NOT OUTLIER
Weibull (k4=2.0)	0.975	0.987	NOT OUTLIER

Table A.14 Parent distribution and design wave parameters for changing return periods

Distribution	Return Period	5 (yrs)	10(yrs)	20(yrs)	50(yrs)	100(yrs)	1000(yrs)
Log-normal	H_s (m)	3,49	3,63	3,76	3,92	4,02	4,35
	T_s (sn)	7,84	8,00	8,14	8,31	8,43	8,76
Gumbel Old	H_s (m)	3,21	3,49	3,75	4,09	4,35	5,19
	T_s (sn)	7,53	7,84	8,14	8,50	8,76	9,57

Log-normally distributed design wave height with H_s=3.92 and T_s=8.31 with corresponding A and B parameters is selected to be used at the reliability based design model, ARIN.

A.7 Extreme Wave Analysis Results of Case III: Sinop Demirciköy Fishery Harbor

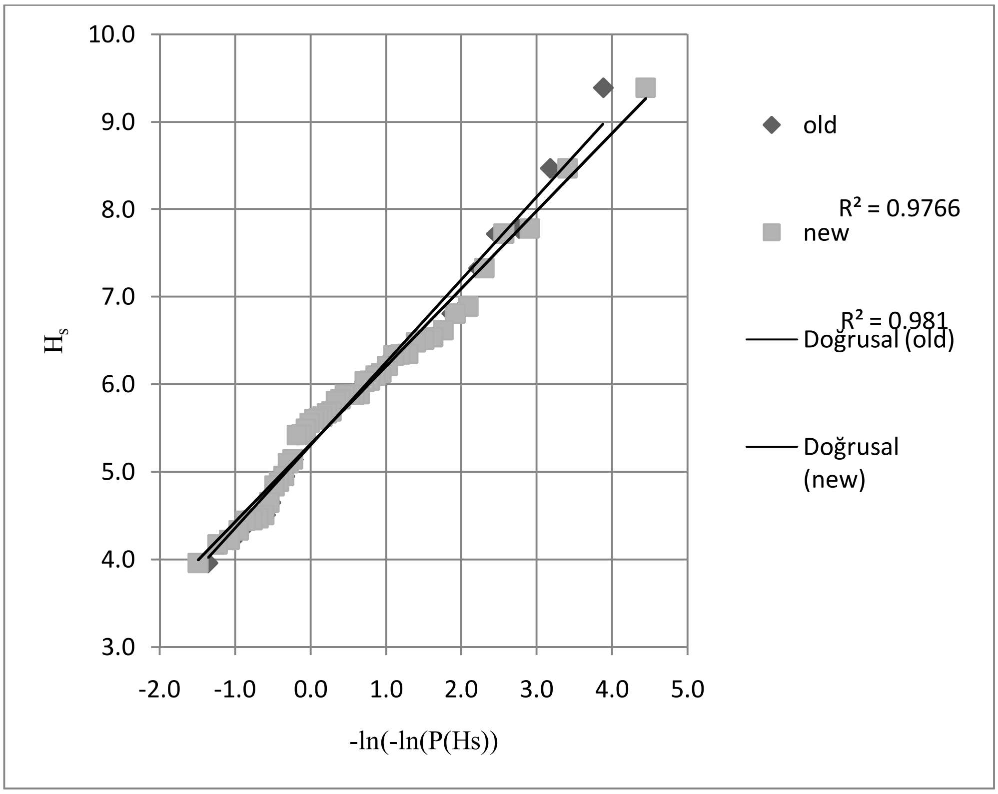

Figure A.10 Fisher-Tippet Type I (Gumble) probability distribution for extreme annual significant wave height for Sinop Demirciköy Fishery Harbor with scale and location parameters of;

B (location parameter)	3,189 (new)	3,182 (old)
A (scale parameter)	0,807	0,906

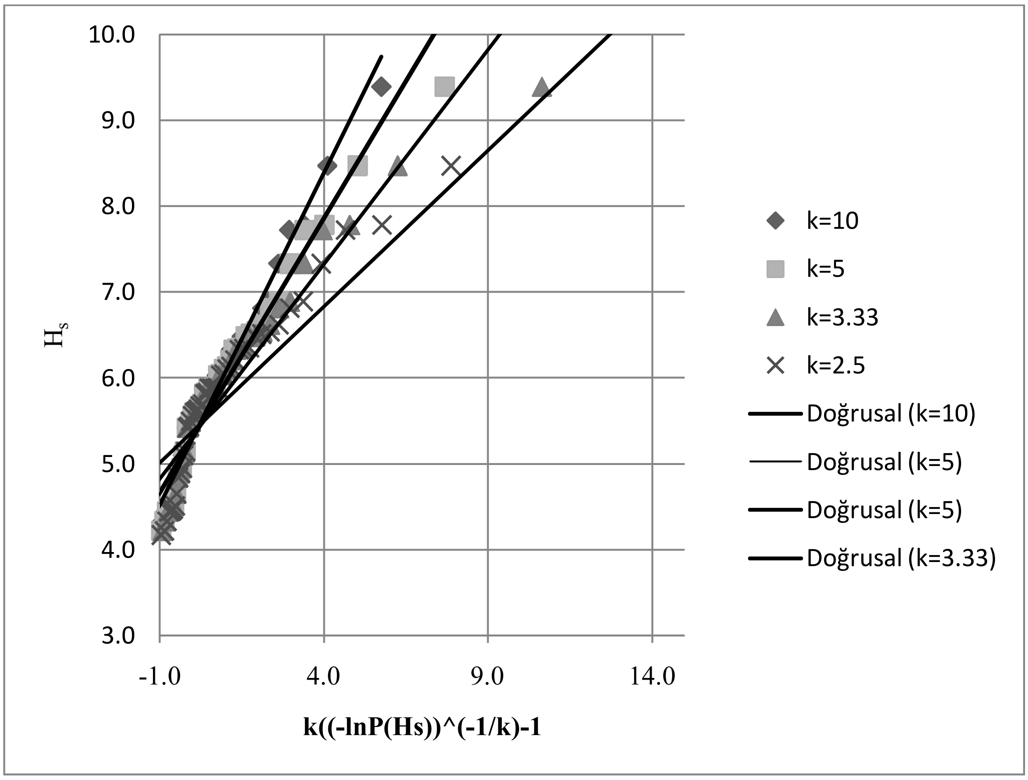

Figure A.11 Fisher-Tippet Type II probability distributions for shape parameters of k=10, k=5, k=3.33 and k=2.5 with scale and location parameters of;

B (location parameter)	5,29	5,30	5,33	5,38
A (scale parameter)	0,77	0,49	0,50	0,36

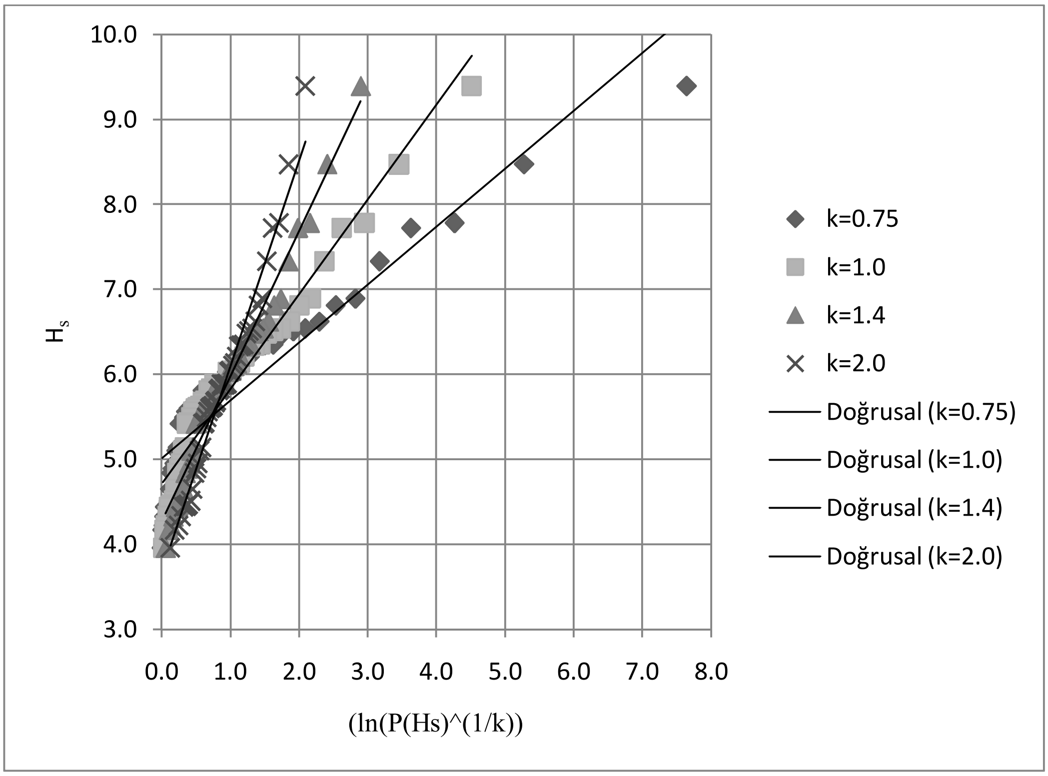

Figure A.12 Weibull probability distribution for annual extreme wave height for shape parameters of k=0.75, k=1.0, k=1.4 and k=2.0 with scale and location parameters of;

B (location parameter)	5,012	4,707	4,268	3,676
A (scale parameter)	0,682	1,117	1,706	2,422

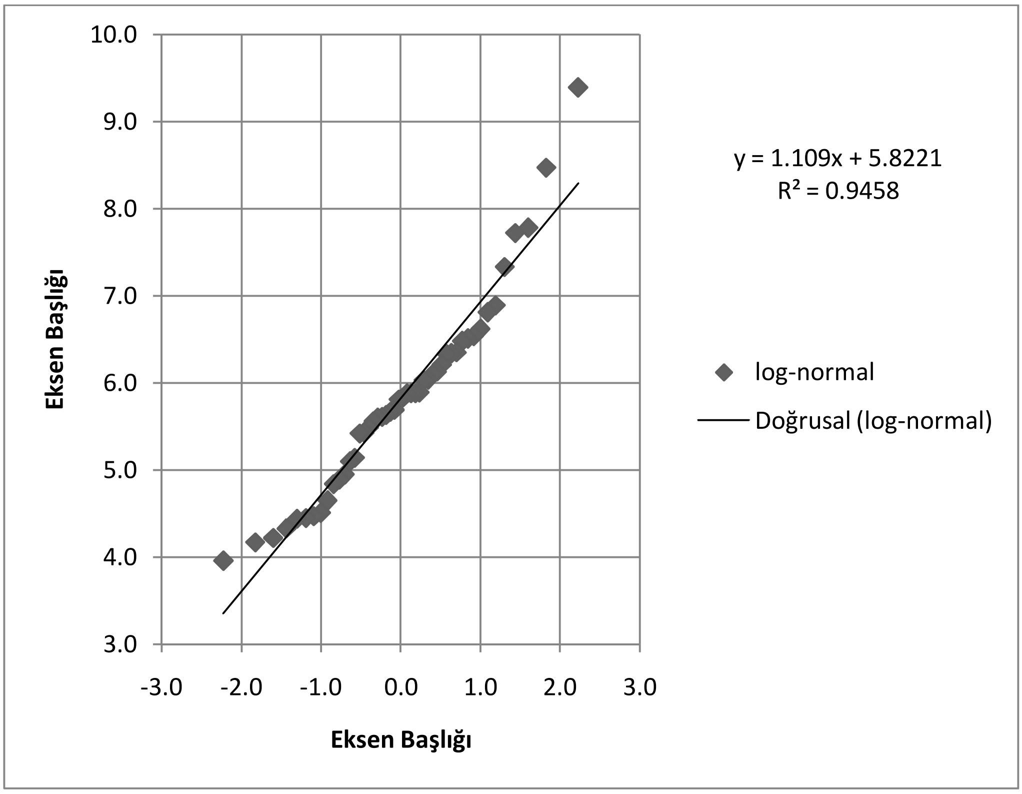

Figure A.13 Log-Normal probability distribution for extreme annual significant wave height with scale and location parameters of;

B (location parameter)	1,744
A (scale parameter)	0,188

Table A.15 Results of MIR criterion

MIR (Minimum ratio of residual correlation)								
Distribution Type	r^2	r	1-r	r_{mean}	a	b	c	$\Delta r/r_{mean}$
FT 2 (k1=2.5)	0,805	0,897	0,103	0,047	-2,455	-0,158	0,000	2,203
FT 2 (k2=3.33)	0,884	0,940	0,060	0,035	-2,471	-0,197	-0,007	1,678
FT 2 (k3=5.0)	0,939	0,969	0,031	0,027	-2,463	-0,224	-0,019	1,145
FT 2 (k4=10.0)	0,970	0,985	0,015	0,020	-2,409	-0,258	-0,033	0,741
Gumbel1 (old)	0,977	0,988	0,012	0,025	-1,824	-0,312	-0,044	0,471
Gumbel2 (new)	0,981	0,990	0,010	0,025	-1,824	-0,312	-0,044	0,383
LogNormal	0,978	0,989	0,011	0,012	-2,094	-0,434	-0,045	0,968
Weibull (k1=0.75)	0,864	0,929	0,071	0,025	-2,603	-0,101	-0,047	2,853
Weibull (k2=1.0)	0,931	0,965	0,035	0,018	-2,355	-0,261	-0,043	1,941
Weibull (k3=1.4)	0,967	0,984	0,016	0,014	-2,221	-0,367	-0,044	1,214
Weibull (k4=2.0)	0,971	0,986	0,014	0,011	-2,047	-0,477	-0,041	1,311

Table A.16 Results of REC criterion

REC (residue of correlation coefficient)								
Distribution Type	r^2	r	1-r	$r_{\%95}$	a	b	c	
FT 2 (k1=2.5)	0,805	0,897	0,103	0,116	-1,159	-0,319	0,016	ACCEPTED
FT 2 (k2=3.33)	0,884	0,940	0,060	0,089	-1,411	-0,260	0,000	ACCEPTED
FT 2 (k3=5.0)	0,939	0,969	0,031	0,068	-1,570	-0,220	-0,018	ACCEPTED
FT 2 (k4=10.0)	0,970	0,985	0,015	0,050	-1,563	-0,240	-0,034	ACCEPTED
Gumbel1 (old)	0,977	0,988	0,012	0,036	-1,444	-0,315	-0,045	ACCEPTED
Gumbel2 (new)	0,981	0,990	0,010	0,036	-1,444	-0,315	-0,045	ACCEPTED
LogNormal	0,978	0,989	0,011	0,025	-1,002	-0,562	-0,035	ACCEPTED
Weibull (k1=0.75)	0,864	0,929	0,071	0,062	-1,522	-0,168	-0,041	REJECTED
Weibull (k2=1.0)	0,931	0,965	0,035	0,044	-1,433	-0,268	-0,044	ACCEPTED
Weibull (k3=1.4)	0,967	0,984	0,016	0,031	-1,312	-0,381	-0,045	ACCEPTED
Weibull (k4=2.0)	0,971	0,986	0,014	0,024	-1,115	-0,525	-0,039	ACCEPTED

Table A.17 Results of R^2 test

Distribution Type	r^2	r	Comment
FT 2 (k1=2.5)	0,805	0,897	NOT OUTLIER
FT 2 (k2=3.33)	0,884	0,940	NOT OUTLIER
FT 2 (k3=5.0)	0,939	0,969	NOT OUTLIER
FT 2 (k4=10.0)	0,970	0,985	NOT OUTLIER
Gumbel1 (old)	0,977	0,988	NOT OUTLIER
Gumbel2 (new)	0,981	0,990	NOT OUTLIER
LogNormal	0,978	0,989	OUTLIER
Weibull (k1=0.75)	0,864	0,929	NOT OUTLIER
Weibull (k2=1.0)	0,931	0,965	NOT OUTLIER
Weibull (k3=1.4)	0,967	0,984	NOT OUTLIER
Weibull (k4=2.0)	0,971	0,986	NOT OUTLIER

Table A.18 Parent distribution and design wave parameters for changing return periods

	Return Period	5 (yrs)	10,00	20,00	50,00	100,00	1000,00
Gumbel	Hs (m)	6,65	7,32	7,95	8,78	9,40	11,45
	Ts (sn)	10,08	10,58	11,03	11,59	11,99	13,23
LogNormal	Hs (m)	4,99	5,34	5,66	6,06	6,35	7,26
	Ts (sn)	8,79	9,09	9,37	9,69	9,92	10,60

Log-normally distributed design wave height with H_s=8.78 and T_s=11.59 with corresponding A and B parameters is selected to be used at the reliability based design model, ARIN.

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Ergin A., Yalçıner A.C., Fışkın Arıkan G., Özler B., Basamak Tipi Kıyı Koruma Yapılarının Denge Durumlarının Uygulamalı Araştırması. *"5.Ulusal Kıyı Mühendisliği Sempozyumu Bildiriler Kitabı"*, 2, (2005), s.425-440.