Development of Load and Resistance Factors for Reinforced Concrete Structural Members in North Cyprus

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ABSTRACT

Load and Resistance Factor Design (LRFD), is a widely used procedure in the design of reinforced concrete, wood and steel structures. It is a reliability-based procedure for design, which gives a framework that is consistent with civil engineering design codes, in accordance with reliability theories. In this study, Advance First Order Second Moment (AFOSM) approach is used as the reliability approach in carrying out the analysis. Uncertainties related to material properties (i.e. compressive strength of concrete, yield and ultimate strength of reinforcing steel bars.), dimensions of reinforced concrete structural members (beams and columns) and the effect of load variables (i.e. Dead and Live load), are considered. Under the framework of AFOSM the failure mode in different reinforced concrete structural members were analyzed, which focused mainly on flexure failure, shear failure and the combined action of flexure and axial load failure. Reliability indexes are calculated according to the flexure and shear failure modes in beams and columns, in addition to failure due to the combine action of flexure and axial load on columns.

Target reliability indexes are selected for different load combinations from values reported by other researchers from different countries, which are used as the safety level to evaluate the computed reliability indexes. New load and resistance factors are selected for different failure modes in different structural members, considering the design practice in North Cyprus and specifications given in the Turkish codes (e.g. TS500).

Keywords: Model Uncertainty, Reliability, LRFD, Reliability Index, Safety Level

ÖZ

Yük ve Dayanım Katsayılarına göre Tasarım (YDKT), betonarme, ahşap ve çelik yapı elemanlarının tasarımında yaygın olarak kullanılan bir yöntemdir. Bu yöntem güvenilirlik esasına dayanan, tasarım yönetmelikleri çerçevesinde ve güvenilirlik teorilerine uygun bir yöntemdir. Bu çalışmada analizler "Geiştirilmiş Birinci Mertebe İkinci Moment" (GBMİM) yaklaşımı ile yapılmıştır. Çalışma kapsamında malzeme özellikleri (beton basınç mukavemeti, beton çelik çubukları çekme dayanımı v.b), betonarme elemanların boyutları (kirişier ve kolonlar) ve yük değişkenlerinin etkisine (sabit ve hareketli yükler) ilişkin belirsizlikler dikkate alınmıştır. GBMİM yaklaşımı çerçevesinde farklı betonarme yapı elemeanlarının göçme durumu incelenmiştir. Bu çalışmada esas olarak eğilme, kesme ve eksenel kuvvet-eğilme etkileşimindeki elemanlar dikkate alınmıştır. Bu bağlamda betonarme kirişlerde eğilme ve kesme göçme durumları ile eksenel kuvvet ve eğilme etkisindeki betonarme kolonların "güvenilirlik indeksleri" hesaplanmıştır.

Farklı yük birleşimlerine göre daha önceki çalışmalarda farklı ülkeler için rapor edilen hedef güvenilirlik indeksi değerleri seçilmiştir. Hesaplanan güvenilirlik indeksi seçilen hedef güvenilirlik indeksi ile kıyaslanarak yeterli güvenlik seviyesine ulaşılmıştır. Yapılan çalışma kapsamında Kuzey Kıbrıs'ta tasarım uygulamaları ve TS500'de belirtilen kurallar çerçevesinde yeni yük ve dayanım katsayıları belirlenmiştir.

Anahtar Kelimeler: Belirsizlik Modeli, Güvenilirlik, YDKT, Güvenilirlik İndeksi, Güvenlik Seviyesi

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A	Cross sectional area
A	Peak ground acceleration
A_S	Reinforcement area
A_{SW}	Cross sectional area of stirrups
b	Member width
c	Distance from the neutral axis to outer compressive fiber in a T cross-section
c_b	Depth of neutral axis at the balanced case in reinforced concrete cross section
D	Dead load effect
D_f	Failure domain
D_S	Safety domain
d	Depth of the member (d_b for beam, d_c for column)
d_e	Effective depth of the member
F_X, f_x	Cumulative distribution function (CDF) and probability density function of variable X, respectively.
f_c	Concrete compressive strength
f_{ct}	Tensile strength of concrete
f_S	Steel stress
f_y	Yield strength of steel bars
f_{yw}	Yield strength of shear reinforcement
k_1	A dimensionless coefficient which is a function of strength of concrete

L	Live load effect of maximum live load
M	Safety margin
M_r	Bending moment capacity
N	Correction factor
N	Axial load
P_f	Probability of failure
P(E)	Probability of event E
P_S	Survival probability (reliability)
R	Generalized resistance
R	Rate of loading
s	Spacing of stirrups
t	Depth of the flange thickness
U	Effect of factored load
V	Total design base shear
V_c	Shear resistance of concrete
V_d	Maximum design shear force
V_r	Shear strength
V_w	Resistance of shear reinforcement
W	Width (W_b for beam, W_c for column)
X	Basic random variable
$\overline{X}$	Mean of X
X'	Nominal of X
X^*	Design value of X
$\widehat{X}$	The model used to estimate X
X_{apt}	Arbitrary point-in-time value of X

x_i	Distance between neutral axis and i^{th} steel layer in reinforced concrete cross section
β	Reliability index
β_T	Target reliability index
γ	Generalized load factor
Δ	Prediction uncertainty
δ	Basic variability
ε_{cu}	Ultimate strain in concrete
μ	Mean value
ρ	Steel ratio
ρ_b	Balanced steel ratio
ρ'	Compression reinforcement ratio
σ	Standard deviation
σ_S	Steel stress
Ω	Total variability
ϕ	Generalized resistance factor
AFOSM	Advanced First Order Second Moment Method
APT	Arbitrary point-in-time
a.p.t.	Arbitrary point-in-time
C.D.F.	Cumulative distribution function
c.o.v.	Coefficient of variation
FOSM	First Order Second Moment Method
JCSS	Joint Committee on Structural Safety
LRFD	Load and Resistance Factor Design
RC	Reinforced Concrete

TEC	Turkish Earthquake Code (Specification for Structures to be Built in Earthquake Areas)
EMU	Eastern Mediterranean University

Chapter 1

INTRODUCTION

1.1 Overview

In the design of structures the main priority is to have a structure that is both safe, serviceable and at the same time economical, this is the basis for every structural engineer when it comes to the design of any structure. Uncertainty of any kind that might be resulting from inadequate information, prediction error and at times human error, should be considered when an engineer is trying to make a design decision. Safety requirements are introduced in engineering designs to account for the risks associated with these uncertainties.

The load and resistant factors design (LRFD) accounts for uncertainty related the parameters of load and resistance by the combination of limit state design (LSD) and probabilistic approach. The limit state design is divided into two categories, serviceability limit state (SLS) and ultimate limit state (ULS) (Salgado, 2008). The SLS is related to the malfunctioning of structures, for example the differential or uniform settlement in structures, while the ULS is associated with lack of safety in structures, which includes failure or collapse of structures. ULS occurs when the load is equal to the resistance of a structural system, when this happens the system fails. Therefore, for a good engineering design to be successful the ULS has to be identified and prevented.

LRFD tries to keep the level of probability of failure from exceeding the allowable safety level for safety purposes (i.e. at target reliability). The LSD frame work is used to explain LRFD, by analyzing the ULS using some partial factors on resistance and load. The partial factors related to the load and the resistance are computed based on their uncertainties.

Design codes are created to provide a safe and economical guide in engineering designs. Design codes provides a probabilistic approach where design is concerned, this is due to the short comings in deterministic approach of solving structural safety problems. Since the design of structures has to be done in the presence of some uncertainties, probabilistic approach is used to quantify those uncertainties for safety purposes.

The reliability of a structure is determined by comparing the load effect to the resistance effect. In probabilistic approach load and resistance parameters are treated as random variables and the safety is determined by using a tolerable reliability index. The reliability of a whole structure is the sum of the reliabilities of individual structural members. In this approach, the ratio of mean to nominal value and the total uncertainties is computed for the purpose of calibration.

1.2 Literature Review

The concepts of probability assessment on structural safety was first introduced in the beginning of the 20th century. In the early 60s of that century, the American Concrete Institute (ACI) building code introduced a design that is based on the ultimate limit state, which used load factors to increase the load and strength reduction factors to reduce the strength, this design approach is known as the ultimate strengths design (USD). All studies related to this field started in the late 60s and an increase in interest on this structural reliability topics has been shown ever since then.

In the year 1979, Drs. Cornell, Galambos, Ellingwood and MacGregor all came together in the center for building technology at the National Bureau of Standards, which is currently knows as National Institute of Standards and technology with that purpose of recommending some set of universal load and resistance factors, these factors will be utilized during the design stage of structures. The meeting of the above Drs lead to several outcomes that was published in different papers some examples are;

- Ellingwood et al in 1980.
- Galambos et al in 1982.
- Ellingwood et al in 1982.

The results of that meeting lead to the development of the fundamental sets of load and resistance factors that were amalgamated in the 1982 ANSI A58.1.

After the fundamental factors for the load and resistance were created, further research continued in order to develop the load and resistance for different regions. Rackwitz (2000), in his study used the total cost minimization as a decision model to assess the target reliability in the process of developing a design code, the cost he considered involves both the maintenance and the reconstruction cost. A joint committee on structural safety (JCSS) was created by a Liaison committee that is responsible with the coordination of the activities of the six international association in civil engineering. The aim of creating the committee is to improve the knowledge related to structural safety, which lead to the development of a code in 2001, which was based on probabilistic design but there was no consideration of some information that includes the mechanical models such as the shear capacity, buckling and flexural failure.

The calibration of the Building code Requirement for Structural Concrete (ACI 318), was conducted based on the study of Nowak & Szerszen (2003a and b). The study was done in two parts 'a' and 'b', the first publication discussed the topic of statistical model for the resistance parameters, which creates the basics of the selection of the resistance factors (strength reduction factors). The second publication discussed reliability analysis and methods of selecting a resistance factor.

Design codes and specifications are not available for North Cyprus, hence the Turkish design code (TS500) is adopted. Thus research in this topic is not readily available here. The Turkish design codes and specifications are used as the guide for design here in North Cyprus, the design code follows a deterministic approach and reliability based design were not implemented during its calibration. Yücemen and Gulkan (1989), were the first to start any significant research on this topic in Turkey. Their study was based on suggesting some sets of reliability based load and resistance factors related to the design of reinforced concrete beams. Kömürcü (1995) followed up with a research on developing a reliability based design using the local conditions and the design practice in Turkey for reinforced concrete beams considering the flexural failure. Later, Firat (2007) in his study developed a reliability based design criterion for reinforced concrete beams, columns and shear walls, considering different failure modes in accordance to the conditions and design practices in Turkey, which is presently the most updated study in this field in Turkey.

In this study, to a certain extent is a follow up on the research conducted by Firat (2007), here the local data and the condition regarding the resistance variables of North Cyprus were used, also the load variable were selected and compared for different cities and with the results of other international researchers. The use of the local conditions in North Cyprus in determining appropriate load factors is the main objective of this study.

1.3 Aim of this Study

The aim of this study is to develop a reliability based load and resistance factors design criterion for reinforced concrete structural members considering the local conditions in North Cyprus using a probabilistic approach. For the purposes of this study, load and resistance parameters are treated as random variables. Local data used for the evaluation of these parameters were collected in North Cyprus and from values reported in international literature.

The Advance First Order Second Moment Method (AFOSM) is the approach adopted as the structural reliability model to propose the new sets of load and resistance factors. The effects of the loads on the structure coming from dead load and live load are estimated and also the resistance parameters that include reinforced concrete beams and columns in flexure and shear failure mode, together with the combine action of flexure and axial load on the column. The ratio of mean to nominal values is computed for both the load and resistance parameters solely for the purpose of calibration.

The Turkish design code and specifications is used as the guide in this study and the local conditions in North Cyprus are used to quantify the uncertainties related to the resistance and load parameters.

1.4 Outline of this Study

This study follows the development of LRFD based design procedure within the framework of LSD for reinforced concrete structural members. Chapter 1 gives an overview and a literature survey of work that has been done in this field. Chapter 2 give the background of the structural reliability models and probabilistic approach used in the analysis of uncertainties, it goes on to give definitions of basic headings such as reliability index and methods of computing it. Then in the conclusion part of the chapter it states the method selected for this study and the reason of this selection.

Chapter 3 is devoted on computing the ratio of mean to nominal values and the total uncertainty in the resistance variables, the assessment of local data on yield strength of reinforcing steel bars, dimension of structural members and the compressive strength of concrete were used in order to quantify the uncertainties in the resistance variables.

Chapter 4 uses the values determined in the analysis of the resistance variables from the previous chapter to compute the value of total uncertainties and the ratio of mean to nominal values in reinforced concrete structural members such as beams and columns for different failure modes which includes; flexure and shear failure mode and the combined action of flexure and axial load, all within the structural reliability framework.

In chapter 5, modeling loads used basically for design are analyzed with that help of local data and reports in international literature and those in Turkey. In chapter 6, the resistance criterion for different failure mode of the structural members are evaluated in compliance with reliability based design, target reliability index are selected. Then using the target reliability index a new set of resistance and load factors are selected for the different failure modes in the reinforced concrete structural members. Chapter 7 gives a summary of all content of the report, conclusions on results are also given in this chapter.

Chapter 2

RELIABILITY THEORY

2.1 Introduction

Serviceability and safety are important millstones required by an engineering design to ensure that the structural performance is in conformance with the life time design expectations.

A reliability analysis method gives a theoretical frame work used to quantify uncertainties in order to make a better design decision. It aims at evaluating the ability or capability of a system to operate under safety conditions throughout the structure's life cycle. Computing the probability of failure and reliability index helps in quantifying the risk involved and therefore providing the possible consequences if failure should occur. Problems related to reliability methods are modeled as random variables, a random vector is a group of random variables denoted by X, where $f_x(X)$ is called the joint probability density function (PDF).

A structure is said to be safe when the strength (R), is able to resist the maximum load effect (S), acting on the structure. In reliability theory failure in a member occurs when the strength of the member is less than the load applied on it (i.e. R< S), or the safety margin (M) is less than zero (i.e. M < 0, where M = R- S). The failure probability can be computed by the use of the Eq. (2.1).

$$\mathrm{P_f} = \mathrm{P}(\mathrm{R} \leq \mathrm{S}) = \mathrm{P}(\mathrm{R} - \mathrm{S} \leq 0) = \mathrm{P}(\mathrm{M} \leq 0) \quad (2.1)$$

Where; P_f is the probability of failure and P is the probability that the events in the brackets will occur.

The failure probability where R and S are independent and normally distributed given that $\bar{R}$ and $\bar{S}$ are the mean and σ_R and σ_S denotes standard deviation of the strength and load effect, respectively, is computed with the following relationship.

$$P_f = 1 - \phi\left(\frac{\bar{M}}{\sigma}\right) = 1 - \phi\left(\frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}}\right) \qquad (2.2)$$

Where ϕ denotes the probability distribution of the standard normal variate.

2.2 Uncertainty Modeling

Uncertainty is said to be involved in almost everything man does, including the structural design and other things that has to do with decision making. (Bulleit, 2008) categorized uncertainties into two;

- Aleatory Uncertainty,
- Epistemic Uncertainty.

Aleatory Uncertainty is referred to as the inherent or natural variability, it occurs due to randomness inherent in nature.

Epistemic Uncertainty occurs due to inadequate information or knowledge, an increase in information and data can be used to reduce this type of uncertainty.

Melchers (1999) further classified uncertainties into different types and how they affect the structural design and its performance. These classifications include;

- Phenomenological Uncertainty,
- Decision Uncertainty,

- Modeling Uncertainty,
- Prediction Uncertainty,
- Physical Uncertainty,
- Statistical Uncertainty,
- Human Error.

The uncertainties that are normally considered for the purpose of design are mainly physical uncertainty, statistical uncertainty and model uncertainty. Physical uncertainties mainly involve uncertainties that are associated with the type of loading environment, the geometry of a structure and material properties used. The statistical uncertainties occur due to incomplete information e.g. the number of sample needed for the test is not adequate. Finally, model uncertainties have to be considered for the purpose of accounting for those uncertainties related to the mathematical descriptions that are used to approximate the real physical behavior of the structure.

In order to model the various sources of uncertainties the prediction error and the inherent variability is combined to find the total uncertainty involved in the structural member. According to the formulation given by the First Order Second Moment (FOSM) method, the total uncertainty can be computed by the relationship in Eq. (2.3) below;

$$\Omega_{X_i} = \sqrt{\delta_{X_i}^2 + \Delta_{X_i}^2} \tag{2.3}$$

Here Δ_{X_i} denotes the coefficient of variation for the effect related to the epistemic uncertainty (N_i). δ_X denotes the coefficient of variation related to the inherent variability (X_i).

The inherent variability (X_i), is computed by the relationship suggested by (Ang & Tang, 1984):

$$X_i = N_i \hat{X}_i \tag{2.4}$$

Here N_i is a random factor used for the correction of the uncertainty value; $\hat{X}_i$ is to estimate the value of X_i.

The epistemic uncertainty (Ni) is computed by first computing all the possible sources that can cause this type uncertainty and labeling them accordingly to give a sum total of the correction value to be used (i.e. $N_i = N_{i1} N_{i2},..., N_{in}$). According to the formulation given in FOSM, the epistemic uncertainty can be computed with the following relationship given in Eq. (2.5) and Eq. (2.6). (Ellingwood et al., 1980)

$$\overline{N}_i = \overline{N}_{i1}\overline{N}_{i2}, \dots \dots, \overline{N}_{in} \tag{2.5}$$

$$\Delta_{X_i} = \sqrt{\Delta^2_{X_{i1}} + \Delta^2_{X_{i2}} +, \dots\dots, + \Delta^2_{X_{in}}} \tag{2.6}$$

2.3 The Reliability Index

The reliability index is denoted with the symbol β. It is often used as a substitute for the probability of failure. β may be used to compare different structures and can be used as the target in reliability-based design without mentioning a specific probability of failure.

Cornell (1969) defined the reliability index, or safety index, as the mean of the safety margin which is divided by the standard deviation of the safety margin. This formulation of the reliability index is referred to as the Cornell reliability index.

$$\beta = \frac{\overline{M}}{\sigma} \tag{2.7}$$

In this formulation β can be interpreted as the distance between the mean of the safety margin and the point of failure, measured in terms of the standard deviation of the safety margin. However, for all definitions, the basic concept of β remains the same; it is a measure of the distance between the most likely state of the structure (mean) and the most likely failure point, in terms of the variation.

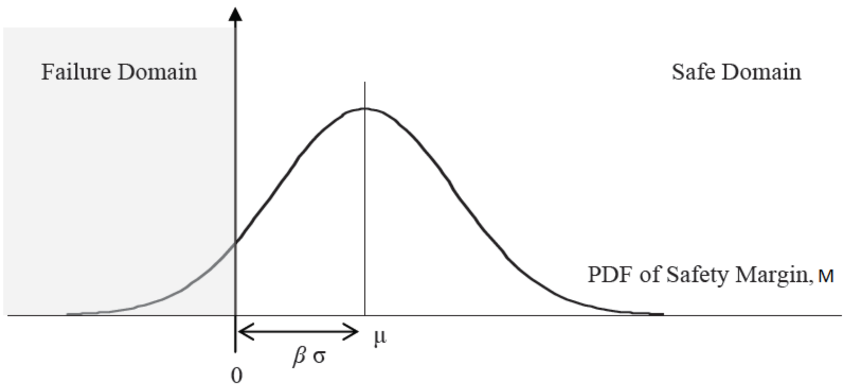

Figure 1. Definition of β using the Safety Margin

2.4 Computing the Reliability Index

There are numerous ways to compute the reliability indexes, but here the most popular technique are briefly discussed based on literature.

2.4.1 First Order Reliability Method (FORM)

First Order Reliability (FORM) deals with the first moment and second moment random variables. The procedure entails of two approaches which includes;

- First Order Second Moment (FOSM) approach,
- Advanced First Order Second Moment (AFOSM) approach.

The distributional information in AFOSM is appropriately used, while in the FOSM approach, distributional information on the random variables is ignored.

2.4.1.1 First Order Second Moment (FOSM)

First Order Second Moment (FOSM) approach uses the mean value and the standard deviation of the random variables. The linear function of the performance at the mean of the random variables is needed in this method. The linearization of the performance function at the mean of the random variables utilizes the first order Taylor series method of approximation.

Cornell's reliability index is extended into the formulation used to compute the reliability index. β is defined as the mean of the safety margin divided by the standard deviation of the safety margin. Therefore, to compute the reliability index the mean and standard deviation of the design variables are required. However, to give space for non-linear limit state functions, the mean and standard deviation of the safety margin are computed by using the Taylor series expansion to linearize the safety margin. The value computed for β depends on the point that is chosen to linearize the limit state function. A common choice is the point where each random variable takes on its mean value, resulting in the mean-value, first-order, second-moment reliability index, even though this method is very simple, it has several shortcomings. The most significant shortcoming is that the value of β is not invariant with respect to the limit state functions. For example two mechanically equivalent functions of the same limit state can produce different result for the reliability index.

2.4.1.2 Advanced First Order Second Moment (AFOSM)

Advance First Order Second Moment (AFOSM) approach, can be referred to as 'Hasofer Lind' approach of computing β. When all variables and limit state function are transferred to a standard Normal space, the design point is computed by selecting a minimization procedure that gives a mark on the limit state surface and gives the minimum distance to the origin. When the reliability index is computed in this procedure is referred to as β_{HL}, the Hasofer Lind reliability index (Hasofer & Lind, 1974). This measure of reliability is invariant with respect to the limit state function, but it utilizes only the information given by the second-moment about the variables, it can not be compared because the procedure does not depend on the curvature of the limit state function at the design point.

In this approach, the reliability index is assessed mainly by transforming the problem to a standard coordinate system. Therefore, the random variable X_i is transformed to Z_i with the relationship given in Eq. (2.8) below:

$$Z_i = \frac{X_i - \overline{X}_i}{\sigma_{X_i}}, i = 1,2, \ldots, n \tag{2.8}$$

Where Z_i = random variable with mean equal to zero and a unit standard deviation.

Hence, the written equation above can be used to transform an initial limit state surface that is given by g(X) = 0 into a reduced form of the limit state surface given by g(Z) = 0. Therefore, X stands for the ‘original coordinate system while Z represents the ‘transformed or reduced form of the coordinate system’. Due to shortcomings in the FOSM method, (Hasofer and Lind, 1974) proposed a new reliability index denoted by β_{HL}. The approach used in calculating β_{HL} is referred to as the Advanced First Order Second Moment (AFOSM) approach. β_{HL}, can be defined as the minimum distance measured from the origin to the failure surface on the Z coordinate. A design point can be defined as any point yielding the shortest distance from the origin to any point on the failure surface.

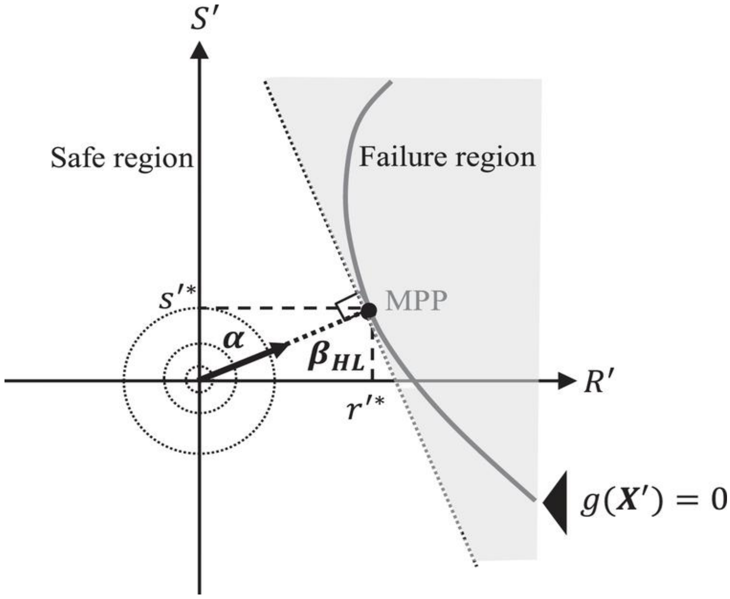

Figure 2. Hasofer-Lind Reliability Index β_{HL}

Figure 2 gives a graphical representation of the Hasofer-Lind reliability index β_{HL}, which is valid only in the case where two variables are involved.

When a nonlinear failure surface is involved, the seeking of a design point and the computation of a reliability index is done by the use of an iterative approach. This is done by first computing the values of Z_i's using Eq. (2.8) and substituting the value into Eq. (2.9).

$$Z_i^* = \alpha_i \beta_2 \tag{2.9}$$

The value of α_i's are the directional cosines that are used in the minimization of the value of β_2. The relationship given in Eq. (2.10) is used to compute the value of α_i;

$$\alpha_i = \frac{-\,\partial g/\partial z_i}{\left(\Sigma\left(\partial g/\partial z_i\right)^2\right)^{1/2}},\, i = 1,2, \dots, n \tag{2.10}$$

The iterations are made until the values start to converge in the value of β_2 then the probability of failure can be computed with the formulation given below.

$$P_f = 1 - \phi(\beta) \tag{2.11}$$

2.4.2 Monte Carlo Simulation (MCS)

Monte Carlo Simulation (MCS) is a technique with different applications to many different problems. In general the Monte Carlo approach involves the use of a random sampling technique to generate a set of data to be used in the analysis. In application to structural reliability, a random value is generated for each design variable based on the type of distribution that variable follows. These random variables are utilized in order to analyze the limit state equation. For a limit state function less than zero, can be interpreted that the structure has “failed”.

This procedure is iterated several times, the probability of failure is computed by dividing the number of samples that failed by the total amount of simulations. This procedure is very tough and it can be applied in almost any type of limit state function. Thus, the reliability of this procedure lean on the amount of simulations made, and for probabilities of failure that are small the required time for the computation can also be very ambitious. With additional knowledge about the failure region, variance reduction techniques, that include importance sampling, is used to concentrate the simulations in the region of interest and reduce the necessary amount of simulations required to compute the reliability.

2.4.3 Reliability Analysis Method Used Within the Scope of this Study

The Advance First order reliability approach (AFOSM) is the method selected for the course of this study. This procedure was chosen for several reasons, they are;

- It is easy to compute,
- It considers the short comings of FOSM,
- It gives room for many variables without overlapping,
- Most importantly for the assessment of the reliability of a structure.

Though the intent of this study is to calibrate based on component reliability, the limit state chosen for explicit consideration herein is the flexural and shear limit state function, which is used for the flexural and shear failure of a reinforced concrete structural members considering different failure modes. Hence, to make use of other reliability methods would require assessing the reliability against all the different failure modes separately and using principles of system reliability to calculate the reliability. AFOSM can consider all the failure modes simultaneously based on the particular random variables provide during the computation and provide the total reliability against the flexural and shear failure.

Chapter 3

UNCERTAINTY QUANTIFICATION OF THE RESISTANCE VARIABLES

3.1 Introduction

In this chapter uncertainties related to the resistance variables are quantified, they include;

- Concrete,
- Reinforcing Steel Bars,
- Dimensions,
- Area of Reinforcing Steel Bars.

Analyses were carried out using local data and design condition present in North Cyprus. The mean to nominal ratio and the total uncertainties in each resistance variable is computed for the purpose of calibration

3.2 Concrete

Concrete is defined as a mixture of aggregates and paste, which the paste consists of cement and water, the aggregates consists on of sand, gravel and crushed stones. For the purpose of safety in reinforced concrete structures the quality of concrete is important, the quality of concrete can be measured by considering it’s workability, compressive strength, durability, performance, and setting time.

The use of statistical analysis on the data obtained from concrete test is a more efficient way of evaluating the quality of concrete, a good quality concrete should result in a high mean value and a low coefficient of variation (c.o.v). The variation in concrete quality starts with variation in the properties of the materials used during the mixing stage, these variations in material properties of concrete can be due to the existence of some factors that are present at the mixing stage, such as;

- Temperatures.
- Methods of mixing.
- Mixing proportion.

In order to produce a high quality concrete careful observations should be made on factors that lead to the production of low quality concrete, these factors include:

- Lack of proper control.
- Lack of supervision.
- Not paying attention to details.
- Poor workmanship.

Most importantly compressive strength is the most affected property of concrete in terms of its mechanical properties, so to control the quality of concrete the compressive strength test is examined on cubic specimens (Ergün & Kürklü, 2012).

3.2.1 Data Evaluation and Analysis

Data were collected from the laboratory of Civil Engineering Department of Eastern Mediterranean University (EMU) North Cyprus, on the compressive strength of cubic concrete specimens with dimension of 150 x 150 x 150 mm. The tests were conducted accordance with the specifications given in TS 3114, samples to be used for this test were cured in the laboratory for the number of days required before testing, stored in accordance with the provisions given in TS 3068. A total of 5705 samples were test within the period of 8 years (i.e. 2004 to 2014). The samples of the specimens were obtained from firms around North Cyprus.

The concrete specimens test were conducted for different concrete age ranging from 3 to 55 days compressive strength, for the purpose of this study a 28-day compressive strength is considered since it’s the fundamental measure on the regulations for the strength of concrete. With regards to this regulation samples that were tested below and above the 28-day age, a 28-day compressive strength was approximately computed using a non-linear regressional analysis by utilizing the logarithmic best fit for the data which yielded a mathematical equations suggested in a study conducted by (Öztemel & Şensoy, 2004).

A graph of compressive strength (f_{ci}), versus the concrete age was plotted for concrete class, and a logarithmic function was derived from these graphs for each concrete class in order to get an approximate value of a 28-days compressive strength for data of each concrete class. Figures 3 - 7 shows the graph illustrated for each concrete class.

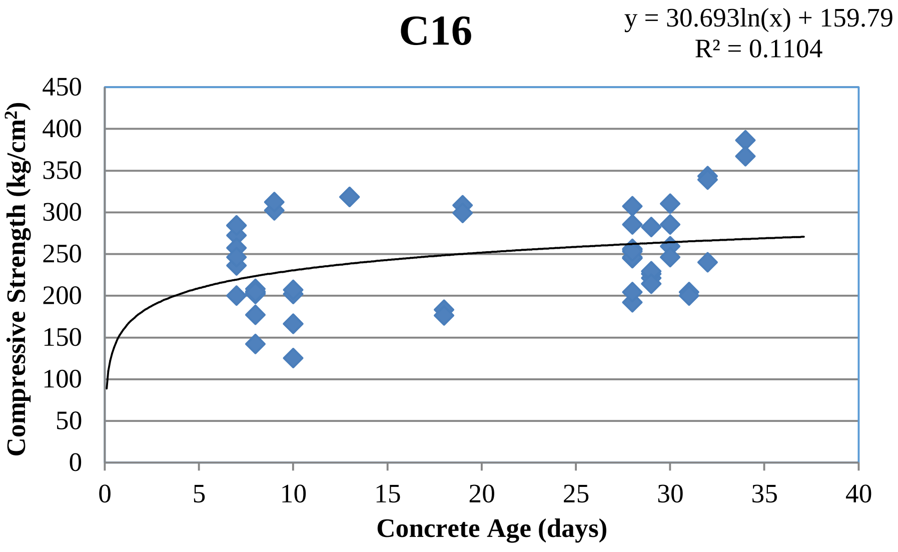

Figure 3. Compressive Strength Distribution for C16 Data

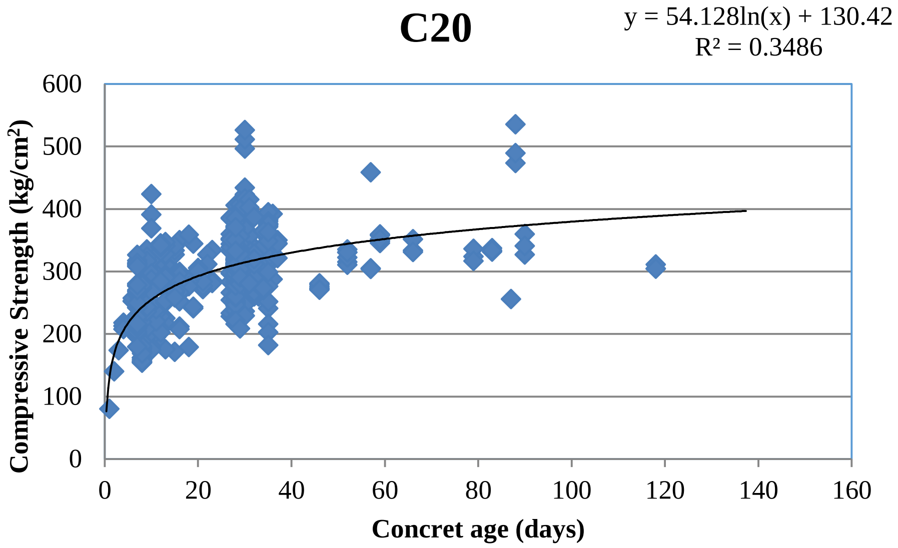

Figure 4. Compressive Strength Distribution for C20 Data.

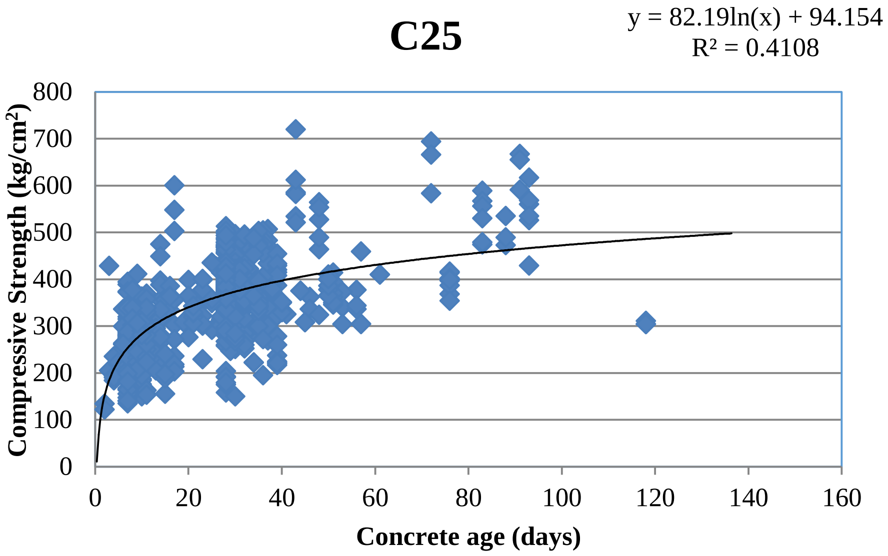

Figure 5. Compressive Strength Distribution for C25 Data

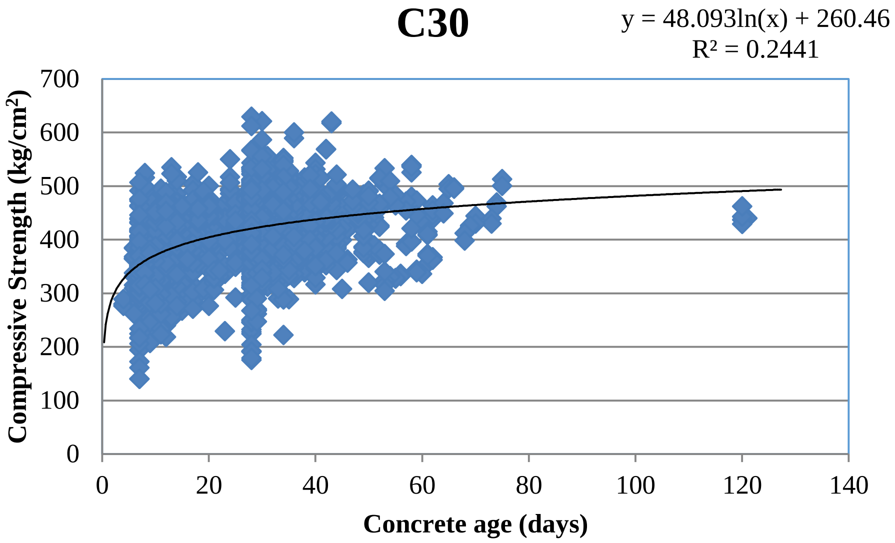

Figure 6. Compressive Strength Distribution for C30 Data

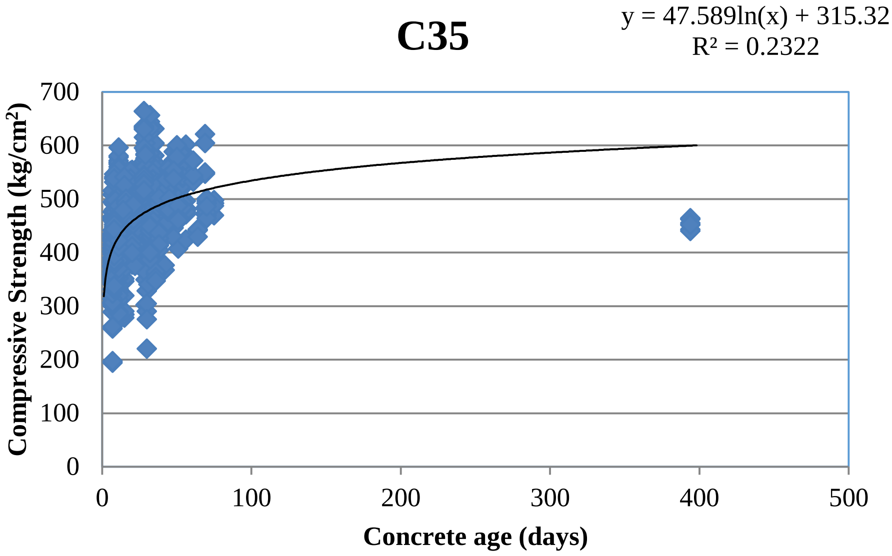

Figure 7. Compressive Strength Distribution for C30 Data

Utilizing the data from the graph, a logarithmic function is derived to calculate the approximate 28-days compressive strength. Below is the list of the logarithmic function for each concrete class

$$f_{ci}(x) = \begin{cases} y = 30.693\ln(x) + 159.79 \\ y = 54.128\ln(x) + 130.42 \\ y = 82.19\ln(x) + 94.154 \\ y = 48.093\ln(x) + 260.46 \\ y = 47.589\ln(x) + 315.32 \end{cases}$$

Where x represents the concrete age in days and f_{ci} (x) is the function that depends on the value of x, which represents the compressive strength corresponding to that concrete age in the particular concrete class.

The approximate 28-days compressive strength is computed using Eq. (3.1) as suggested by (Öztemel & Şensoy, 2004) in their study on compressive strength.

$$f_{ci_{28}}(x) = \frac{f_{ci} * f_{ci}(t=28)}{f_{ci}(t=t_{test})} \tag{3.1}$$

Here X_{test} is the relevant concrete age of testing, $f_{ci}(x)$ is the compressive strength tested for that age in the results from the laboratory, $f_{ci_{28}}$ is the approximate 28-days compressive strength.

After a complete analysis of the data and the approximate 28-days compressive strength was derived, a further analysis was carried out on the data to obtain the mean, standard deviation and c.o.v for each concrete class, as shown in Table 1.

Table 1. 28 day Statistical parameters on the compressive strength data

Concrete Class	**C16**	**C20**	**C25**	**C30**	**C35**	**Overall**
Concrete age	28	28	28	28	28	-
Number of Samples	53	506	1051	3127	968	5705
F_{ck},cyl(F_{ck},cub)	16(20)	20(25)	25(30)	30(35)	35(37)	-
Mean μ (Mpa)	26.2	31.1	368.15	420.52	473.78	408.69
Standard deviation	5.79	5.6	68.89	57.71	63.97	76.5
COV	0.22	0.18	0.19	0.14	0.14	0.19

3.2.2 Uncertainty Analysis of Concrete Compressive Strength

Analysis conducted on the collected data, gave a mean value of the cubic compressive strength of concrete to be 40.87 N/mm^2 as an Overall value with a c.o.v. of 19%. The equivalent cylindrical compressive strength can be computed using a conversion factor of 0.83, therefore the cylindrical compressive will be 33.92 N/mm^2 (40.87 x 0.83 = 33.92), with a c.o.v of 19%. In the content of this chapter, f_c denotes the mean compressive strength and c.o.v is denoted by δ_{fc}. The c.o.v is used to measure the inherent (basic) variability in the compressive strength of concrete. Considering the common construction conditions in Turkey and North Cyprus, the values taken for this study can be used to represent the whole of North Cyprus. In reality, variation occurs in concrete strength of a structure and the strength specified at the design stage. This variation may occur due to the following factors; Variations in the properties of materials.

- Proportions of concrete mix.
- Variations in mixing. Transporting.
- Placing and curing methods.
- Variations in testing procedures.

Additional uncertainties apart from the inherent variability may be caused by some other factors, will be considered according to international literature since there is no much local information to quantify these factors.

The consideration that the concrete strength in a structure is lower than the strength of cylinder specimen tested in the lab from the same sample of concrete. The deviation may arise due to the following effects;

- ✓ Curing and placing processes.
- ✓ Segregation of concrete in deep member.
- ✓ Size and shape.
- ✓ Stress conditions

Due to the effects of these factors, the in-situ strength of concrete is low compared with the strength measure in the laboratory. In order to consider the difference between the in-situ strength and the laboratory strength, a factor, $\overline{N}_1$, has been introduced, where $\overline{N}_1$ is the correction factor with a c. o. v denoted by Δ_1.

In a study conducted by (Ellingwood & Ang, 1972), they found the correction factor within a range from 0.83 to 0.92, to consider the difference between the in-situ strength and the standard cylindrical strength, with a value of Δ_1. to be 0.16, as the c.o.v associated with this correction factor. In another study of (Mirza 1979), he suggested the correction factor ($\overline{N}_1$), to be within the range of 0.74 to 0.94 and the value of $\Delta_1$as 0.1. In another study conducted by (Firat, 2007), $\overline{N}_1$ was computed as 0.86 and $\Delta_1$as 0.13. Here a mean value for the correction factor, $\overline{N}_1$, is taken as 0.85 as an average value from previous research, and the c.o.v (Δ_1), is taken as 0.13 as an average value of the reported values.

Strain effect in concrete due to micro cracking and due to creep is another factor causing decrease in the observed in-situ compressive strength of concrete; this can be corrected by using a factor suggested by (Mirza et al. 1979), in their study, denoted by $\overline{N}_2$, with its corresponding c.o.v denoted by Δ_2, they suggested a formula to be using in computing the value of $\overline{N}_2$ given by;

$$\overline{N}_2 = 0.89(1+0.08 \log (R)) \quad (3.2)$$

Here R is the rate of loading measure in psi/sec, R value used during the test as 0.5 psi/sec which is equivalent to 3.447 KN/m^2 /sec, therefor the value of $\overline{N}_2$ is computed as 0.89. In a study conducted by (Kömürcü, 1995) he found the value of $\overline{N}_2$ as 0.88 with a c.o.v as 0 (i.e. no prediction uncertainty was discovered so Δ_2=0). (Mirza et al. 1979) in their study also suggested that the uncertainty associated with the prediction of $\overline{N}_2$ can be neglected. In this study, $\overline{N}_2$ will be taken as 0.89 without considering its prediction uncertainty (i.e. Δ_2 =0).

Another factor will be consider which deals with the error that rises due to lack of standard testing method, proper timing , poor calibration of machine and human errors in general. This factor has been suggested by (Kömürcü, 1995), in his study as $\overline{N}_3$ with a value of 0.95 as the mean and the prediction uncertainty associated with this factor as Δ_3 with a value of 0.05. All these factors will be considered in this research.

Therefor the overall mean bias in the compressive strength can be computed as is computed with the formula given in Eq. (3.3)

$$\overline{N}_{fc} = \overline{N}_1 + \overline{N}_2 + \overline{N}_3 \qquad (3.3)$$

$\overline{N}_{fc}$ = 0.70 (0.85x0.87x0.95 = 0.70), the in-situ mean of the compressive strength can be computed with the relationship given in Eq. (3.4) below;

$$\overline{f}_c = \overline{N}_{fc} \text{ x } \dot{f}_c \qquad (3.4)$$

Here $\overline{f}_c$ denotes the in-situ value of the compressive strength of concrete, $\dot{f}_c$ denotes the value of the compressive strength from the cylindrical specimens tested in the laboratory, and $\overline{N}_{fc}$ is the value of the overall mean bias in f_c. Therefor the mean in-situ compressive strength is computed as $\overline{f}_c$ = 23.74N/mm^2 (0.70 x 33.92 = 23.74), as the mean value of the in-situ compressive strength of concrete. The average of the concrete classes used in this study is found to be 25.2 N/mm^2 (C16, C20, C25, C30, C35).The nominal compressive strength of concrete can be computed by dividing the compressive strength with a factor of 1.5 which is a value taken from TS 500 (2000), corresponding to the average value of the concrete class. Therefore the nominal compressive strength is computed as $\dot{f}_c$ = 16.8 N/mm^2 (25.2/1.5 = 16.8), Thus the ratio of mean to nominal ratio is computed as $\frac{\overline{f}_c}{\dot{f}_c} = \frac{23.74}{16.8}$ = 1.41.

The total variability due to the prediction error related to those three uncertainty sources is computed as$\Delta_{fc} = \sqrt{0.132^2 + 0^2 + 0.05^2} = 0.14$.

Therefore the total uncertainty will be computed by combining the prediction uncertainty and the inherent uncertainty, as $\Omega_{fc} = \sqrt{0.19^2 + 0.14^2} = 0.24$. In a study conducted by (Firat, 2007), he took total uncertainty as 0.18, also in the study of (Kömürcü and Yücemen, 1996), the value of total uncertainty was computed as 0.21. Here, the value computed is related to the data collected from firms around North Cyprus.

The distribution of the compressive strength of concrete has been taken as normally distributed by researchers that worked on it in the past, for the purpose of this study a program Easy Fit 5.6 was used in the determination of the distribution of data on concrete compressive strength and it was found to follow a normal distribution.

3.3 Reinforcing Steel Bars

North Cyprus does not produce steel. Reinforcing steel bars used in North Cyprus are imported from other countries especially from Turkey.

The trades of these iron and steel products are carried out by individual firms. These firms order the steel products with specifications, standards and size required in the construction industries in North Cyprus. Table 2 shows the standards of some countries related to the reinforcing steel bars.

Table 2. Reinforcing steel bars required standards according to different countries (from Firat, 2007)

Country	Standard	Class	Minimum Yield Strength	Minimum Ultimate Limit Strength	Yield/ Ultimate Strength	Minimum Elongation
			(N/mm^2)	(N/mm^2)		(%)
Turkey	TS 708 (1996)					12
		III a	420	500	1.1	(Ø8-Ø28)
		IV a	500	550	1.08	10
						(Ø32-Ø50)
England	BS 4449 (1997)	Gr 460B	460	-	1.08	14
USA	ASTM A615 / A616M (1996)	Gr 40	300	500	-	11 ~ 12
		Gr 60	420	620	-	7 ~ 9
		Gr 75	520	690	-	6 ~ 7

Data used in this study on reinforcing steel bars are collected from firms around North Cyprus.

3.3.1 Analysis of Data

Data collected from individual firms on the yield strength, ultimate strength and elongation of reinforce steel bars, were further tested in the laboratory of Mechanical Engineering Department of Eastern Mediterranean University, the further testing was done in order to check the conformity with the data obtained from the firms.

A total of 3851 specimens of reinforcing steel bars were tested for yield strength, ultimate strength and elongation. The test was carried out in accordance with the specifications of TS 708.

Table 3. Analysis on yield strength for 420(a) reinforce steel bars data

Diameter (mm)	**Number of Samples**	**Mean Yield strength μ (Mpa)**	**Standard deviation**	**COV**
8	366	458.28	52.58	0.11
10	837	469.79	56.87	0.12
12	539	464.05	54.62	0.12
14	605	474.29	60.8	0.13
16	489	493.06	64.26	0.13
18	319	481.36	58.3	0.12
20	283	511.46	50.2	0.1
22	337	494.16	43.4	0.09
24	76	523.97	45.69	0.09
Overall	3851	478.78	58.4	0.12

Table 4. Analysis on Ultimate strength of 420(a) reinforcing steel bars data

Diameter (mm)	**Number of Samples**	**Mean Yield strength μ (Mpa)**	**Standard deviation**	**COV**
8	366	629.26	75.57	0.12
10	837	629.8	61.29	0.097
12	539	629.83	64.11	0.1
14	605	633.64	63.47	0.1
16	489	636.77	64.52	0.1
18	319	639.18	64.5	0.1
20	283	640.58	63.5	0.099
22	337	640.04	62.09	0.097
24	76	640.51	61.9	0.097
Overall	3851	640.52	61.87	0.096

3.3.2 Uncertainty Analysis of Reinforcing Steel Bars

The mechanical properties of reinforcing steel bars such as its strength parameter are the main characteristics used to classify it. Therefore, variation in these properties may be analyzed as an uncertainty associated with this strength parameters (i.e. Yield strength and Ultimate strength). In order to properly quantify these uncertainties, the sources of these variations should be considered, the following are some of the factors that cause these variations;

- Effect of bar diameter.
- Change in the strength of the material.
- Rate of loading during the test.
- Changes in the cross-sectional area.
- Effect of using a combination of bars belonging to another batch.

In a study conducted by (Mirza and MacGregor, 1976), on variations that occurs in reinforcing steel bars, they suggested that a value of 5% to 8% should be taken as coefficient of variation (c.o.v) for data collected from different producers on individual bar size. (Kömürcü, 1995), found variability range from 1% to 4% for individual bar size, provided that the reinforcing steel bars are from the same manufacturer.

In Turkey the c.o.v ranges from 2% to 7% for individual bar size from the same manufacturer. In this study the inherent variability in reinforcing steel bars yield strength is computed as 12% as the overall of the entire bar sizes collected from the firms (i.e. c.o.v denoted by δ_{fy} = 0.12).

Mirza and MacGregor (1979), pointed out that the yield strength of reinforcing steel bars can be overestimated since the test procedure is performed using a huge amount of strain in structures under static loads. Therefor the effect caused by strain and rate of loading can be corrected by using a factor with an overall mean bias, $\overline{N}_1$ = 0.9, (Ellingwood & Ang, 1972). Firat (2014), also took this value as 0.9 in his study on reinforcing steel bars. In another study carried out by (Kömürcü, 1995) he suggested that the prediction error Δ_1 associated with this bias factor can be neglected. Here, the mean bias factor $\overline{N}_1$ is taken as 0.9 and the prediction error Δ_1 is neglected.

The upper and lower yielding point of reinforcing steel bars are used to determine its yield strength, the yield strength of reinforcing steel bars is also affected by some factors. A correction factor$\overline{N}_2$, is introduced in order to deal with the effect of these factors. In the study of (Kömürcü & Yücemen, 1995) the correction factor$\overline{N}_2$, was taken as 1 with a prediction error $\Delta_2 as$ 9%. Ellingwood and Ang, (1972), used a value of 5% as the prediction error associated with this correction factor. In this study$\overline{N}_2$, is taken as 1 and the prediction error Δ_2 as 5%, as an average value.

The overall inherent variability in yield strength computed from the data analyzed was found to be 12%. If a structure is built using reinforcing steel bar from a single manufacturer the prediction error is assumed to be 0, data collected for the purpose of this study are not entirely from a single manufacturer, thus a prediction error Δ_3 is considered with a value of 6% as an average value between 0 and 12%, with a correction factor$\overline{N}_3$, as 1. The overall mean bias factor related to the yield strength can be computed as $\overline{N}_{fy} = \overline{N}_1 + \overline{N}_2 + \overline{N}_3 =$ 0.9 (0.9 x 1 x 1= 0.9). The overall prediction error $\Delta_{fy} = \sqrt{\Delta_1^2 + \Delta_2^2 + \Delta_1^2} = \sqrt{0.05^2 + 0.06^2} = 0.078$.

Results from the analysis conducted on data from the test on yield strength, a value of 478.78 N/mm^2 is computed as the mean. Considering the factors affecting the yield strength of reinforcing steel bar, the corrected mean yield strength is computed as $\overline{f}_y = \overline{N}_{fy} * \dot{f}_y$ = 0.9 x 478.78 = 430.90 N/mm^2 . The nominal yield strength corresponding to 420(a) reinforcing steel bars is 365 N/mm^2 (420/1.15= 365), according to TS 500 (2000). Therefore, the mean to nominal ratio is computed as 1.18 (i.e. $\frac{\bar{f}_y}{\acute{f}_y} = \frac{430.90}{365}$ = 1.18). Kömürcü and Yücemen (1996), computed 0.14 as the result of total prediction error on yield strength. A similar research by (Real et al., 2003) suggested a value between the range of 0.05 to 0.10 as a value of total prediction error in yield strength, Firat, 2007, used a value of 0.09 as the total prediction error. Here the total prediction error $\Delta_{fy}$is computed as 0.078. By combining the total prediction error and the inherent variability, the total uncertainty can be computed as $\Omega_{fy} = \sqrt{0.12^2 + 0.078^2} = 0.14$

Ultimate strength of reinforcing steel bars is almost similar in terms of source variation (Mirza and MacGregor, 1979). The mean ultimate strength in this study is computed as 640.52 N/mm^2, with an overall mean bias, $\overline{N}_{fu}$ = 0.9. The corrected value of the mean ultimate strength, $\bar{f}_u$= 0.9 x 640.52 = 576.47 N/mm^2, a nominal value of 435 N/mm^2 (500/1.15=435 N/mm^2) as stated in (TS 500, 2000). The ration of mean to nominal of the ultimate strength is computed as 1.33 ($\frac{\bar{f}\mathrm{u}}{fu} = \frac{576.47}{435}$ = 1.33). The overall prediction error related to ultimate strength is computed as $\Delta_{fu} = \sqrt{0.05^2 + 0.06^2 + 0.2^2} = 0.078$. A value of 0.096 was computed from the statistical analysis on ultimate strength as the overall inherent variability. Therefore the total uncertainty related to the ultimate strength is computed as $\Omega_{fu} = \sqrt{0.096^2 + 0.078^2} = 0.12$

Data used for the purpose of this study were found to follow a normal distribution using a computer program Easy Fit 5.6. Firat, 2007, used normal distribution for the yield strength of reinforcing steel bars, while Kömürcü, 1995 used lognormal distribution.

3.4 Dimensions

Variation in the dimension of reinforced concrete structural members computed during the design stage and that of the as-built, affects the resistance of the reinforced structure members. These variations are considered in terms of geometrical discrepancies, which occur mostly during the construction stage in the life-cycle of the structure. Geometrical discrepancies depend on the size, shape and quality of form work used during the construction. The method of concreting and vibrating these structural members at the construction stage is regarded as the primary source of the discrepancy (Atadero & Karbhari, 2006).

Unfortunately, local data to be used in quantifying the uncertainty related to dimensions was not found, therefore values used in this section are based on the research carried out in turkey on dimensions of reinforced concrete members. Based on engineering judgment, there are many similarities between the workmanship in Turkey and North Cyprus, given that the most construction workers are from Turkey. Prediction errors related to dimension were quantified based on three likely sources of variability, these sources include;

- ❖ Dimensional changes with change in different design values.
- ❖ Unfixity of forms.
- ❖ Difficulties in the direct measurement of effective depth.

Variability caused by changes in different design values can be accounted for with a prediction error of 0.02. A value of 0.02 is also taken as the prediction error associated with unfixity of forms, (Firat, 2007). A total prediction error of $\Delta_{bw}= \sqrt{0.02^2 + 0.02^2} = 0.03$. The variability due to difficulties in direct measurement of the effective depth is quantified with a prediction error of 0.06, this value is suggested by Ellingwood et al. (1980), Kömürcü (1995) and Firat (2014). Therefore, the total prediction error related to effective depth of both beams and columns is computed as$\Delta_{bd} = \sqrt{0.02^2 + 0.02^2 + 0.06^2} = 0.07$

Kömürcü (1995), in his study on dimensions of reinforced concrete structures in Turkey computed a value of 0.03, 0.03 and 0.07 as the prediction error associated with the depth, width and effective depth of beams and columns, respectively. In a similar study, Firat (2007) computed the same values for the prediction error of the depth, width and effective depth of beams and columns. In the study, considering the similarities in workmanship between Turkey and North Cyprus, the prediction error related to the depth, width and effective depth of beams and column is taken as 0.03, 0.03 and 0.07, respectively. The inherent variability associated with the width, depth and effective depth of beams and columns will be computed separately, in accordance with previous researches done in Turkey.

3.4.1 Beam Dimensions

In a recent study conducted by (Firat, 2007) on the dimensions of beam, he computed the inherent variability related to the width, depth and effective depth of beam 0.025, 0.045 and 0.024, the mean to nominal ratio was computed as 0.998, 0.996 and 1 for the depth, width and effective depth of beam. By combining the prediction error and the inherent variability a total variability is computed for the width, depth and effective depth of beam as 0.054, 0.04 and 0.074, respectively.

Yücemen and Gulkan, (1989), and Kömürcü (1995), in their research on beam dimension in Turkey, the computed a value of 1 as the ratio of mean to nominal for the depth, width and effective depth of beam. Here, the total variability is taken as

0.054, 0.04 and 0.074 and the mean to nominal ratio related to the width, depth and effective depth is taken as 0.998, 0.996 and 1, respectively.

Table 5. Statistical analysis results on column dimensions (Firat, 2007)

Beam Dimension	Inherent Variability	Prediction Error	Mean to nominal ratio	Total Variability
Width (W_b)	0.045	0.03	0.998	0.054
Depth (d_b)	0.025	0.03	0.996	0.04
Effective depth (d_{eb})	0.024	0.07	1	0.074

3.4.2 Column Dimension

Unfortunately, data related to the width and depth of column is not available for this study, therefore with regards to the similarities of quality control and workmanship between North Cyprus and Turkey, results of researches carried out on the column dimension in Turkey will be used here. Semih & Firat, (2014), conducted a research on column dimension computed a value of 0.032, 0.024 and 0.025 as the prediction error related to the width, depth and effective depth of column, and a mean to nominal ratio result of 1.02, 1.03 and 1.01 for the width, depth and effective depth, respectively. The total variability is computed by combining the inherent variability and prediction error they were found as 0.044, 0.038 and 0.074.

In a similar study conducted by (Yücemen and Gulkan, 1989), and (Kömürcü, 1995), on column dimension also computed the same value for the prediction error. In this study the same values will be taken as the prediction error and the total variability related to the width, depth and effective depth.

Table 6. Statistical analysis results on column dimensions (Firat, 2007)

Column Dimension	Inherent variability	Prediction Error	Mean to nominal ratio	Total Variability
Width (W_c)	0.032	0.03	1.02	0.044
Depth (d_c)	0.024	0.03	1.03	0.038
Effective depth (d_{ec})	0.025	0.07	1.01	0.074

3.5. Area of Reinforcing Steel Bars

In the study conducted by (Mirza and MacGregor, 1976) on reinforcing steel bars with sizes ranging from 9.5 mm to 35 mm diameters found a value of 0.97 and 0.024 for the mean to ratio and c.o.v, respectively. In another study, (Firat, 2007) computed a value of 1 as the mean to nominal ratio and a total variability $\Omega_{AS,}$ as 0.03. In this study a value of 1 and a total variability of 0.03 is taken solely due to the fact that most reinforcing steel bars used in North Cyprus are imported from Turkey.

Chapter 4

ANALYSIS OF THE RESISTANCE PARAMETER CONSIDERING DIFFERENT FAILURE MODES.

4.1 Introduction

In this chapter results computed from the previous chapter (Chapter 3) on the uncertainties related to the resistances variables is utilized in order to analyze reinforced concrete structural members considering different failure mode. The mean to nominal ratio and the total uncertainties are computed for structural members in different failure mode, which would be used in the coming chapters for the purpose of calibration.

4.2 Failure Modes in Reinforced Concrete Beams

Beam like other reinforced concrete structures are monolithic in nature, their main purpose is to carry transvers load which creates flexural moments in the beam and shear forces, beams are also subjected to axial load. Therefore two failure modes will be considered for beams in this study (i.e. flexural and shear failure), since they are the most influential parameters governing the beam design.

4.2.1 Shear Strength in Beams

Shear failure in beams are mostly influence by the size of the member and the ratio of shear span to depth. This type of failure occurs due to in adequate shear reinforcement in that structural member, which can be prevented by providing adequate shear reinforcement for the member to attain its maximum limit state in flexure. Thus, the following condition should be satisfied for the design of structural members in shear.

$$V_r \geq V_d \tag{4.1}$$

Where;

V_r: Shear strength in the beam member.

V_d: Maximum shear force in design (Calculated).

The shear strength of a reinforced concrete member can be defined as a summation of the concrete resistance and the shear reinforcement resistance. Here the concrete reinforcement is denoted by V_c, which is considered as 80% of the cracking shear strength in concrete for safety purpose. The shear reinforcement resistance is denoted by V_w. Eq. (4.2 – 4.4) is used to compute these values as specified in TS500 (2000).

$$V_c = 0.80(0.65 f_{ctd} b_w d \psi) \tag{4.2}$$

$$\psi = 1 + 0.007 \frac{N_d}{A_c} \tag{4.3}$$

$$V_w = \frac{A_{sw}}{S} f_{ywd} d \tag{4.4}$$

Where,

A_c : Area of concrete

N_d : Design Axial load

V_c: Cracking shear strength.

F_{ctd}: Design tensile strength of concrete

A_{sw}: Cross-sectional area of one stirrup

f_{ywd}: Design yield stress of shear reinforcement

S: Stirrup spacing.

The total shear force resisted by the reinforced concrete beams can be computed as;

$$V_r = \frac{A_{sw}}{S} f_{ywd} d + 0.52 f_{ctd} b_w d \psi \tag{4.5}$$

Given the above relationship in Eq. (4.5) the mean to nominal ratio of the shear failure of the beam can be computed by dividing the mean and the nominal values of the shear force in Eq. (4.6) below;

$$\frac{\overline{V}_r}{V'_r} = \frac{\frac{\overline{A}_{sw}}{\overline{S}}\overline{f}_{ywd}\overline{d}+0.52\overline{f}_{ctd}\overline{b}_w\overline{d\psi}}{\frac{A'_{sw}}{S'}f'_{ywd}d'+0.52f'_{ctd}b'_w d'\psi'} \tag{4.6}$$

The conformity of the design practice in North Cyprus and that of Turkey made easier to use the result of the analysis carried out on the shear strength in reinforced concrete beam in Turkey by (Firat, 2007). The results of the analysis on shear strength in beams yielded a result of the ratio of mean to nominal value of 1.24. Nowak & Szerszen (2003a) computed a value 1.23. Ellingwood et al., (1980) found a value of 1.09 as the mean to nominal value for the shear force in reinforce concrete beams.

The average total uncertainty was reported to be 0.17 (Firat, 2007). In a similar study conducted by (Ellingwood et al., 1980), reported a value of 0.115 as the total uncertainty value. Nowak & Szerszen (2003a) found a value of 0.11 as the total uncertainty.

Here, the value of mean to nominal and the total uncertainty related to shear strength in beams (Ω_{V_r}) are taken as 1.24 and 0.17, respectively.

4.2.2 Flexural Strength of Beam

The effect of flexure in beams is that it creates bending stress in the beam member, if the bending moment is positive it produces tensile stress and compressive strain in the bottom and top of the beam, respectively, the opposite happens when the bending moment turns out to be negative. Therefore, for a beam to be safe it should be able to resist these stresses and strain caused due to flexure. To avoid structural failure, proper reinforced steel bars are placed in the tension zone; this is because the concrete strength in tension is lower than that of the compression.

The nominal flexural strength can be computed for a rectangular beam by using the equivalent rectangular stress block due its simplicity. Figure 8 shows the equivalent rectangular block.

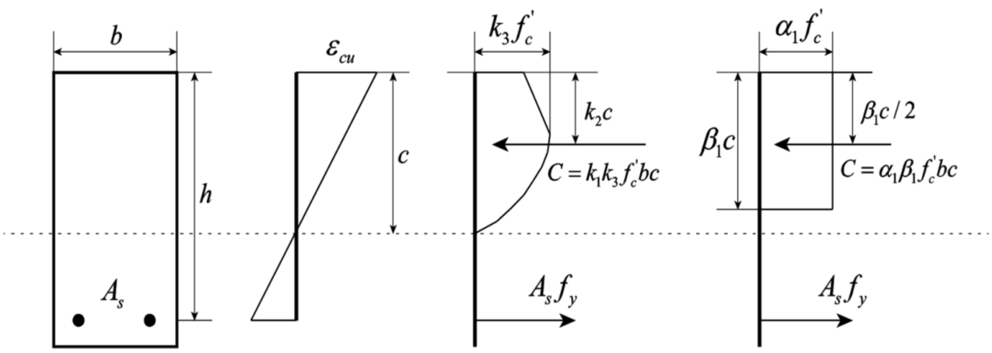

Figure 8. Equivalent Rectangular Stress Block

Where the value of “a” is determined by the relationship given in Eq. (4.7).

$$a = k_1 c \tag{4.7}$$

Where,

c = distance from the outer fiber to the neutral axis in the compression zone.

K_1 = this factor depends on the type of concrete strength to be used.

$$k_1 = \begin{cases} 0.85 \text{ For C16 – C25} \\ 0.82 \text{ for C30} \\ 0.79 \text{ For C35} \end{cases}$$

The average stress is taken as $0.85f_c$, which is assumed to act in the compressive region. The strain (ε_{cu}) in concrete is assumed to be equal to 0.003 TS500 (2000).

Failure of a beam in flexure can occur due to over-reinforcement, under-reinforcement or balance failure. Design codes forbid the design of over-reinforced beams, due to the fact that they behave in a brittle manner and the failure is sudden. The balanced case of a reinforced concrete beam is important in checking whether or not if a beam is over or under reinforced. In order to check with the balanced case, the balanced steel ratio is computed and compared with the calculated steel ratio. Eq. (4.8 – 4.10) give the relationship used in computing the balanced steel ratio and the existing steel ratio.

$$P_b = \frac{0.85 f_{cd}}{f_{yd}} k_1 \frac{c_b}{d} \tag{4.8}$$

$$c_b = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} d \tag{4.9}$$

$$\rho = \frac{A_s}{b_w d} \tag{4.10}$$

Where c_b is the depth from the outer fiber to the neutral axis for the balanced case. A_s is the area of steel

For the case where a beam is double reinforced (ρ- ρ') is computed and compared with the balanced case in order to check whether the design is under or over reinforced. Where, ρ' is the ratio of the steel in the compression zone. The moment capacity of a beam is denoted by Mr, its magnitude can be computed using the equivalent rectangular stress block with the help of the relationships given by Eq. (4.11 – 4.12).

$$M_r = A_s f_{yd} \left(d - \frac{k_1 c}{2}\right) \tag{4.11}$$

$$k_1 c = \frac{A_s f_{yd}}{0.85 f_{cd} b} \tag{4.12}$$

Using the equations above the value of mean to nominal value can be computed by using any method or with the help of computer programs. Firat (2007) computed a value of 1.24 as the ratio of mean to nominal value for reinforced concrete beam under flexure failure mode. Kömürcü and Yücemen (1996) computed a value of 1.19 for the flexural strength in beams in accordance with the local data in Turkey.

The total uncertainty was found to be 0.13 by (Firat, 2007). In a similar study conducted by (Kömürcü and Yücemen, 1996) computed the total uncertainty as 0.16. Here, in this study the value of mean to nominal ratio and total uncertainty is considered as 1.24 and also 0.13, respectively.

4.3 Failure Modes in Reinforced Concrete Columns

A column is vertical member that supports beams and slabs; it basically helps in transferring load from the upper part of a structure to the foundation then to the ground. Columns are normally compression members and can either be classified as braced or unbraced, short or slender base on structural or dimensional factors.

Columns should be designed carefully due to the failure of columns in a structure is catastrophic in terms of human lives and economic point. Columns are named according to how the reinforcing steel bars are arranged, generally there are four types of columns, composite columns, tied columns, concrete-filled pipe columns and columns.

Failure in columns is of three types, compression failure, Buckling and combination of buckling and compression failure. In North Cyprus, rectangular tied column are mostly used in design of structures, therefore the rectangular tied columns are considered for this analysis.

4.3.1 Combination of Both Axial Load and Flexural Strength

In this section the influence of flexure and axial load is evaluated as a failure mode of column, since concrete on its own creates eccentricity under only axial load influence due to its non-homogeneous nature, for this reason most codes forbids the design of columns without the consideration of moment and a minimum eccentricity should be provided (Nawy 2005). Hence with respect to this, the influence of both flexure and axial load on columns is considered.

In the analysis of this type of columns that are under the influence of both flexure and axial load, the moment resisted by the column, M_r, and the axial load, N_r, carried by the columns must be computed. Nawy (2005) suggested that the rectangular stress block approach that was used in computing beams can also be utilized here in the design of columns in order to compute the values of M_r and N_r.

Firat (2007), in his study on reinforced concrete columns under the influence of flexure and axial load combination in Turkey utilized the rectangular stress block approach using the relationships given in Eq. (4.13 – 4.15) according to the framework provide by FOSM.

$$N_r = 0.85 f_{cd} k_1 c_b + \sum_{i=1}^{n} A_{si}\sigma_{si} \qquad (4.13)$$

$$M_r = 0.85 f_{cd} k_1 c \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\sigma_{si}X_I \qquad (4.14)$$

$$\sigma_{si} = 0.003 E_s \left(1 + \frac{X_i - \frac{h}{2}}{c}\right) \leq f_{yd} \qquad (4.15)$$

Where;

A_{si}: Area of steel in the i^{th} layer

σ_{si}: steel stress in the corresponding Area of steel in i^{th} layer

E_s: modulus of elasticity of reinforcement steel bars

X_i: distance between the neutral axis and i^{th} layer of steel.

In this study results from research carried out by (Firat, 2007) in Turkey will be used here due to the similarities in workmanship and type of design code used here in North Cyprus. The mean to nominal values due to the influence of axial load and flexure is found to be 1.24 (Firat, 2007). In the study of (Nowak & Szerszen, 2003a) reported a value of 1.26, in another study conducted by (Ellingwood et al., 1980) on axially loaded columns found a value of 1.10.

The total uncertainty, Ω_{ca}, is computed by combining the prediction error (0.08) and the inherent variability (0.12) as a result of 0.14. In this study, the mean to nominal ratio is considered to be 1.24 and the total uncertainty as 0.14 as computed in the analysis carried out for Turkey.

4.3.2 Shear Strength of Columns

The shear strength of columns are calculated from the analysis of the longitudinal and transverse shear, it is computed using the same approach used in computing the shear strength of beams.

In computing the shear strength of columns the axial load, N_d, that was computed during the analysis of the influence of both flexure and axial load is utilized here. The results from the analysis carried out in Turkey and provide nominal value from TS 500 (2000), are used to compute the mean to nominal ratio and the total uncertainty related to the shear strength in columns. The mean to nominal ratio value and total uncertainty were computed almost same with the values of ratio found in the analysis for beams. Firat (2007) reported a value of 1.24 and 0.17 as the ratio of mean to nominal and the total uncertainty. Nowak & Szerszen (2003a) reported a value of 1.23 and 0.10. Ellingwood et al. (1980) computed a value of 1.09 and 0.115. In the course of study the mean to nominal ratio is consider to be 1.24 and total uncertainty as 0.17.

Chapter 5

LOAD VARIABLES

5.1 Introduction

A reliability based design cannot be fully computed without considering the effect of loads on the structural members. There are several types of load acting on different members of a structure, they are usually classified as either primary or secondary loads. Primary loads are self-weight of the structural members, furnitures, people and other weather conditions like snow, wind and earthquake load. While the secondary loads are caused by settlement in the foundation, temperature change, shrinkage in materials used in constructing a structural member. In the scope of this study only two loads will be considered (i.e. dead and live load).

Dead load in a structure is often assumed constant throughout its life cycle, therefore to quantify the uncertainties in dead load, the uncertainties due to the member's dimension and materials used to construct the member should be considered.

Live load includes the non-structural element such as, people, furnitures and all other movable objects in a structure. The uncertainties in live load is more accurately computed given that there's a long history of data, if there is no such data available a shorter period (A year) can be used to compute the uncertainty related to live load.

5.2 Dead Load

Dead load is defined as the combination of the structural and non-structural members that connected to the structure permanently. (Nowak & Szerszen, 2003b).

Research carried out on dead load shows that dead load follows a normal distribution, with a uniform value for the ratio of mean to value, $\frac{\bar{D}}{D'}$ Nowak & Szerszen (2003b), reported a value of 1.05 as the ratio of mean to nominal and the c.o.v of 0.10. In a similar study (Kömürcü, 1995) took the value of mean to nominal ratio as 1.05 and the total uncertainty as 0.10.

In Table 7 a list of reported values on the ratio of mean to nominal value and the total uncertainties related to dead load is summarized. Here, the ratio of Mean to nominal value is taken as 1.05 and the total uncertainty, Ω_D, related to dead load as 0.10.

Table 7. Uncertainty analysis on dead load (from Firat, 2007)

References	$\bar{\mathbf{D}}/\mathbf{D}'$	$\Omega_{\mathbf{D}}$
Galambos & Ravindra, 1973	1.0	0.08
Allen, 1976	1.0	0.10
Ellingwood, 1978	1.0	0.10
Lind et al., 1978	1.0	0.05
Lind, 1976	1.05	0.09
Ellingwood et al. 1980	1.03	0.10
Kömürcü, 1995	1.05	0.1
Firat, 2007	1.05	0.1

5.3 Live Load

As live load is computed based on history data, a maximum of 50 years live load survey data was considered by (Nowak & Szerszen, 2003b) after the analysis on live load reported a value of 1.00 as the ratio of mean to nominal value, $\frac{\bar{L}}{L'}$ and the total uncertainty, Ω_L , as 0.18.

In Table 8 gives a summary of previous research on the uncertainty of live load is given with proper references.

Table 8. Uncertainty analysis on live load

References	$\bar{L}/L'$	Ω_L
Ellingwood et al. 1980	1	0.27
Kömürcü (1995)	1	0.27
Firat, (2007)	1	0.27

The results from the table yielded a total uncertainty slightly higher than the value found by (Nowak & Szerszen, 2003b), this is due to the effect of transforming the uniformly distributed load to an equivalent load effect and also the uncertainty in load modeling.

In this study, a value of 1.00 is taken as the value of the ratio of mean to nominal and the total uncertainty related to live load as 0.27. This selection is done based on research carried out on live load for a span of 50 years, since no data on live load is available in North Cyprus.

Chapter 6

CALIBRATION AND SELECTION OF LOAD AND RESISTANCE FACTORS

6.1 Introduction

Chapters 3, 4 and 5 of this study has laid the proper background for the consideration of the load and resistance factor for reinforced concrete structural members in different failure mode, within the framework of reliability-based design.

The calibration process will be carried out based on the design practice in North Cyprus and with the use of the Turkish design codes, for the purpose of calibration three Turkish design codes will be utilized in order to access the safety level in these reinforced concrete structural members, they include;

- ❖ Requirements for Design and Construction, (TS 500, 2000).
- ❖ Specification for Structures to be built in Disaster Area (2007).
- ❖ Design loads for Building, (TS 498, 1997).

In probability based resistance and load design, it is required that the factor resistance should be greater or equal to the factor of load acting on the structure. Normally the resistance factors are less than one and the load factors are greater than one. For the purpose of developing a probability based resistance and load design, these factor are computed in a manner that keeps the probability of a combination that is not favorable for the load and resistance variable small on an acceptable level.

This is done for all variable of a design combination that would be in the equations used to check the safety of the structure.

6.2 Load Parameters

During the life span of a structure different types of loads act on it. Some of those includes;

- Dead load.
- Live load.

The mode of combining these loads is important when trying to determine the tolerance or resistance of the structure. There are mainly three approaches used in load combination determination, (Aktas et al., 2001) as;

- Turkstra's rule.
- Ferry-Borges Model.
- Wen's load coincide method.

Turkstra's rule being one of the most used approach in the development of codes due to the application of this approach practically therefore making it suitable for this study. The turkstra's rule uses a procedure that determines the maximum of a load combination occurs when one of the combine load is at a maximum while the other is at their arbitrary point-in-time (Apt) value. The total load effect of this rule is denoted as $U_{(t)}$ which is computed as given in Eq. (6.1), and the maximum load is selected with the aid of Eq. (6.2), which is done from the analysis of all the combined loads.

$$U_{(t)} = X_1(t) + X_2(t) +, \ldots, + X_n(t) \tag{6.1}$$

$$\max U = \max \begin{Bmatrix} X_{1\,max} + X_2 +, \ldots, + X_n \\ X_1 + X_{2\,max} +, \ldots, + X_n \\ . \\ . \\ . \\ . \\ X_1 + X_2 +, \ldots, + X_{n\,max} \end{Bmatrix} \tag{6.2}$$

Here; X_i's: the arbitrary point-in-time load, $X_{i\,max}$ are the maximum load of the i^{th} level.

The calibration process will not be successful if the safety criterion is not considered. The safety of reinforced concrete structural members can be guaranteed if it follows the relationship given by Eq. (6.3) below;

$$\phi R \geq \gamma_D D + \gamma_Y Q + \sum \gamma_J Q_{apt\,j} \tag{6.3}$$

Where; γ is the load factor, φ is the resistance factor, D is the dead load, R is the nominal capacity, Q represent the principle variable load and, Q_{apt} represent the arbitrary point-in-time value of the variable loads.

Ersoy & Özcebe (2004) in their study stated that the relationship given in Eq. 6.3 for the safety criterion is based on the fundamental formula for load combination given in TS 500 (2000). Eq. (6.4) gives the formula below;

$$\text{Factored Load Effects} = U\left\{\gamma_D D' + \gamma_Q\left(Q_i + \textstyle\sum_{i \neq j} \alpha_{0j} Q_j\right)\right\} \tag{6.4}$$

Where Q_i is the characteristic value of the principle load variable, Q_j represents the characteristic value of other variable loads and α_{oj} is the ratio of arbitrary point-in-time value of the i^{th} load to the nominal value.

In this study two loads will be considered which include dead load and live load, therefore the load combination that will be considered for this study is given in Eq. (6.5.)

$$U = \gamma_D D' + \gamma_L L' \tag{6.5}$$

The previous chapter was devoted in computing the ratio mean to nominal value of the load variable for the calibration. Table 9 gives the whole summary of the analysis on load variable.

Table 9. Statistical analysis on dead load and live load

Load Component	**Distribution Type**	**Bias Factor**	**c.o.v**	**Mean Value (KN-m)**	**Standard Deviation (KN-m)**
Dead load (D)	Normal	1.05	0.10	429.20	42.9
Live Load (L)	Normal	1	0.27	847.85	139.73

6.3 Resistance Parameters

The resistance parameters were evaluated based on the local data collected in North Cyprus and published data in international literature. For the resistance parameter concrete class that includes C16, C20, C25, C30 and C35, were evaluated. The ratio of mean to nominal value was computed for the purpose of calibration. The nominal values were taken from TS 500 (2000), and the mean values were computed from the statistical analysis on test results of 28 days compressive strength. The mean value on yield strength of 420(a) reinforcing steel bars were analyzed statistically by considering reinforcing steel bars with diameter of 8, 10, 12, 14, 16, 18, 20, 22 and 24 mm. Dimensions of some reinforced concrete structural members were also considered and the ratio of mean to nominal value and the total uncertainties related with the dimension of beams and columns were computed. Using the results from the analysis carried out on the resistance parameters from chapters 3 and 4, probability based resistance criterion design for reinforced concrete Structural members (beams and column) in different failure modes will be examined in this chapter.

6.4 Selection of Target Reliability Index

Target reliability indexes are level of risk that is acceptable from a reliability based safety point of view. Target reliability indexes are assessed based on the cost of safety and effect of failure.

Some sets of target reliability indexes are computed and proposed by past researchers that used the safety levels to propose these reliability indexes for both different load combination and different failure modes in different structural members. In Table 10, a summary of computed target reliability index is shown with references, for the combination of live load and dead load in flexure and shear failure modes.

Table 10. Target reliability indexes for different load combinations and different structural members according to different studies

Structural Member	**Failure Mode**	**Load Combination**	**β**	**β_T**	**Reference**
BEAM	Flexure capacity	D′ + L′	2.62-3.58	3	Ellingwood et al. (1980)
			2.38	2.7	Kömürcü(1995)
			3.83	3.5	Nowak and Szerszen (2003b)
	Shear Capacity	D′ + L′	1.99-2.45	3	Ellingwood et al. (1980)
			3.78	3.5	Nowak and Szerszen (2003b)
COLUMN	Axial Capacity	D′ + L′	2.98-3.49	3	Ellingwood et al. (1980)
		D′ + L′	4.68	4	Nowak and Szerszen (2003b)

In this study with the aid of Table 10, target reliability levels are selected with the consideration of safety levels integrated in the Turkish design code for different failure modes. For load combinations involving only gravity loads (D + L), in flexure and shear failure modes of beam β is selected as 3, and for the combined action of flexure and axial load in column under gravity loads is selected as 3.2. This selection is due to the reported data from research done on target reliability index and the data used frequently in recent research.

6.5 Reliability Analysis for each Failure Mode.

A reliability-based design criterion for reinforced concrete structural members is carried out under the consideration of main factors;

- Loads.
- Structural strength (Resistance).
- Methods of reliability analysis.

The method of reliability analysis adopted for this study is provided by classical reliability theory. The approach adopted for this study is given at the beginning of this dissertation (chapter 2), it involves the use of limit state design. The probability based limit state design usually operates by using load or load effects, which are normally multiplied by the corresponding load factor, and the resistance of structural members are multiplied by resistance factors. It is required that the factored resistance of a structural member to be greater than the factored load effects in order to check the safety or serviceability of the structure.

Reliability index for a normally distributed random variable is computed by first defining the limit state function to be used, in this study the limit state function used is given in Eq. (6.6) below.

$$G = (\phi R) - (\gamma_{DL} DL) - (\gamma_{LL} LL) \quad (6.6)$$

Where ϕ is the resistance factor

γ_{DL} = the dead load factor

γ_{LL}= the live load factor

In computing the reliability index the dead load factor, γ_{DL}, is set to 1.2 in order to compute the rest of the missing factors. The reliability indexes are obtained by analyzing the existing Turkish design code and the limit state given above, using the target reliabilities selected in the previous section to calibrate new load and resistance factors.

The reliability index, β, is computed with the aid of the relationship given by Eq. (6.7) for load and resistance variables that are normally distributed and the limit state function is linear (e.g. Eq. (6.6)).

$$B = \frac{\bar{R} - \bar{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (6.7)$$

Where $\bar{R}$ is given as the mean of the result from multiplying the resistance by the resistance factor (i.e. (ϕR)). While $\bar{Q}$ is given as the summation of the load effects (i.e. $(\gamma_{DL} DL) + (\gamma_{LL} LL)$).

6.5.1 Procedure for the Selection of Load and Resistance Factors

The selection of load and resistance factor can be performed by judgement, curve fitting, optimization or by combining everything together. Optimization of new loads and resistance factors is normally done in the order stated below;

- ✓ Identify the type of failure modes.
- ✓ Checking the closeness
- ✓ Selecting the optimal load and resistance factors.

In chapter 4, various failure mode were analyzed which includes flexure and shear failure in reinforce concrete beams and columns, the uncertainties related to these failure are quantified in for the purpose of calibration.

Checking the closeness, in this context means the safety factors should be selected such that the computed reliability indexes corresponding to each failure mode is as close as possible to target reliability index. The relationship used to check the closeness is given as;

$$\min W(\gamma) = \sum_{j=1}^{L} w_j \big(\beta_j(\gamma) - \beta_T\big)^2 \leq 0.2 \tag{6.8}$$

Where w_j has a factors of j = 1- - -L

The closeness is determined from the resulted computed from Eq. (6.8) the combination with better closure is preferred.

Selecting the optimal load and resistance factors, this is done by computing the reliability indexes according to the framework provided by AFOSM method and the checking the on with the best closure to the selected target reliability index, the corresponding computed loads and resistance factors are taken as the result.

In this study the reliability indexes are carefully computed according to each failure mode and the closeness is check for the purpose of calibration. The coming sections in this chapter will give outputs of computed reliability indexes and the corresponding selected load and resistance factors.

6.6 Reliability Index and Selection of Load and Resistance Factors

Target reliabilities selected in the previous section (6.4) will be used as a guideline in the computation of the reliability index and in the development of the new loads and resistance factors. Β value is computed for different failure mode in reinforced concrete structures and with the aid of safety level present in the current Turkish design codes and specifications.

6.6.1 Reinforced Concrete Beam in Flexure Failure Mode under Gravity Loads.

Gravity loads in this study is considered as the sum of dead load and the maximum live load (i.e. D + L) as the load combination. Therefore, relating it with the provided combination in TS 500 (2000), the load combination is given as;

$$\mathrm{U} = 1.4\mathrm{D}' + 1.6\mathrm{L}' \tag{6.9}$$

Based on Eq. (6.9) the reliability index for flexure failure mode in reinforced concrete beam is computed according to the sets of nominal loads and the results is given in Table 11. The optimal load and resistances values are also computed simultaneously by a trial and error sequences and compared with the target reliability indexes. The dead load factor is set to 1.2 constant and the other factors are been iterated until convergence.

Table 11. Resistance factors and recommended load combination for beam in flexure failure mode

Load combination	**Resistance Factor**	**Recommended load combination**	**β**	$\bar{\beta}$	β_T
D′ + L′	0.93	1.2 D′ + 1.48 L′	2.83	2.87	3
	0.95	1.2 D′ + 1.52 L′	2.95		
	0.96	1.2 D′ + 1.59 L′	2.84		

Kömürcü, (1995) reported an average value of 2.38 for β. In the study of (Ellingwood et al., 1980) 2.9 was computed as the average value of the reliability index in accordance with the design practice in the USA (ACI Code, 1997, 1983, 1989).

6.6.2 Reinforced Concrete Beam in Shear Failure Mode under Gravity Loads.

As given in section (6.5.1), the load considered for gravity loads are dead load and the maximum live load (D + L) and the load combination will be used as given in Eq. (6.6).

Table 12. Resistance factors and recommended load combination for beam in flexure failure mode

Load combination	**Resistance Factor**	**Recommended load combination**	**β**	$\bar{\beta}$	β_T
D′ + L′	0.90	1.2 D′ + 1.48 L′	2.43	2.64	3
	0.88	1.2 D′ + 1.35 L′	2.61		
	0.77	1.2 D′ + 1.7 L′	2.88		

The result for the reliability index is computed for the shear failure mode and the result is given in Table 12, again trial and error and increased number of iterations are used here towards computing the resistances and load factors.

6.6.3 Reinforced Concrete Column in Combined Action Failure Mode under Gravity Loads.

The load combination in TS 500 (2000), will be used for gravity loads in this failure mode to compute the reliability indexes, the results of this analysis is given in Table 13.

Table 13. Resistance factors and recommended load combination for column in combined action failure mode

Load combination	**Resistance Factor**	**Recommended load combination**	**β**	$\bar{\beta}$	β_T
D′ + L′	0.90	1.2 D′ + 1.46 L′	2.50	2.77	3
	0.95	1.2 D′ + 1.52 L′	2.96		
	0.96	1.2 D′ + 1.59 L′	2.86		

6.6.4 Reinforced Concrete Column in Shear Failure Mode under Gravity Loads.

In this section since the same value was computed for shear failure in beams the results of the resistances factor and the recommended load combination is taken the same.

6.6.5 Reinforced Concrete Members in all Failure Modes under Gravity Loads

In this section the resistance factor, φ, is taken as 1 in accordance to the provision of TS 500 (2000). The load factor is computed according to the specified resistance factor. The result is given in Table 14 below.

Table 14. Resistance factors and recommended load combination for structural members in all failure modes

Load combination	**Resistance Factor**	**Recommended load combination**	**β**	**β_T**
D′ + L′	1	1.2 D′ + 1.72 L′	2.92	3

At the end of the analysis in the study the suggested load factors for the combination of D + L is given as the results from Table 14 above, considering all the failure modes with a resistances factor equal to 1.

Chapter 7

CONCLUSION

In this study, an experimental and analytical approach is adopted in order to develop a reliability-based criterion for reinforce concrete structural members under different failure modes considering local data and design practice in North Cyprus. A probabilistic approach was used as the method of analysis within the theoretical frame work of Advanced First Order Second Moment (AFOSM) approach. Turkstra's rule is the model adopted for the combination of load, which gave the basis for the limit state function.

Local data available in North Cyprus and data published in international literature are used to quantify the uncertainties related to the resistance parameters. In addition, results of the analysis conducted on different failure modes of reinforced concrete structural members in Turkey, was also utilized in the evaluation of the uncertainties related to the resistance parameters. Results found in this study are compared with results recorded by other researchers.

Target reliability index is selected with the consideration of the values reported from other studies conducted for different load combinations, which is used as guideline in the computation of the reliability index corresponding to the design practice in North Cyprus for reinforced concrete beams and columns in flexure failure mode, shear failure mode and combined action of flexure and axial load (column) failure mode under the influence of gravity loads (D + L). A new set of load and resistance factors are selected in accordance with the target reliability index and safety levels.

Using the proper limit state function and the reliability index relationship to compute the reliability index by making a trial and error sequences which was used to determine the set of optimal factor for the load and resistance. Firat (2007) utilized Fortran 77 as the software used to compute the value of the optimum load and resistance factors. According to this study the following conclusion and recommendations are deduced:

Data collected from the laboratory of Civil Engineering Department of EMU on the compressive strength of concrete came with random concrete age ranging from 3 to 55 day, but for the purpose of this study 28-day concrete compressive strength is preferred for the statistical analysis on the resistance parameter. A relationship was developed in Microsoft excel software which is used to convert the compressive strength of concrete from any age to an approximate 28-day compressive strength. Then analysis on the data was carried out yielding a mean compressive strength of 40.87 N/mm^2 with a c.o.v of 19%, when the cubic compressive strength is converted to the standard cylinder compressive strength a value of 33.92 N/mm^2 with a c.o.v equal 19%. The mean to nominal ratio for the compressive strength is computed as 1.41 and the total uncertainty is computed as 0.24 by combining the prediction error (epistemic uncertainty) and the inherent variability (Aleatory uncertainty).

Data on reinforcing steel bars was obtained from the laboratory of Mechanical Engineering Department of EMU on the yield strength, ultimate strength and elongation, At the completion of the statistical analysis, the mean to nominal ratio and the total uncertainty for the yield and ultimate strength was found to be 478.78 N/mm^2 and 640.52 N/mm^2 with a c.o.v as 0.12 and 0.096, respectively. The mean to nominal ratio and the total uncertainty related to the yield strength and ultimate strength of reinforcing steel bars was found to 1.18, 1.33, 0.14 and 0.12, respectively.

The resistance parameters were analyzed for different reinforced concrete structural members considering different failure modes and the ratio of mean to nominal ratio was found to be 1.24 and the total uncertainty was found to be 0.13 and 0.17 corresponding to the different failure modes and the type of structural member.

For the purpose of this study, two type of loads were considered namely, dead load and live load. For the quantification of the loads, since there are no local data available, results reported in international literature was used. The mean to nominal ratio and total uncertainty of dead load was reported as 1.05 and 0.10, respectively. The mean to nominal ratio related to live load is considered to be 0.28 and 1as a maximum value and the total uncertainty as 0.70 and 0.27, respectively.

The current design practice for reinforced concrete structural members in North Cyprus is assumed depending on the Turkish design codes which includes; TS 500 (2000), TS 498 (1997) and Specifications for Structures to be Built in Disaster Area (1998). The safety levels inherent in the design practice for reinforced concrete structural members considering different failure modes is computed by using the AFOSM method, with regards to the failure modes the values of β is ranging from 2.34 to 2.90 for D + L load combination in North Cyprus.

Optimal factors for loads which minimize the alteration from the selected target reliabilities, they are found to yield low resistance factor for D + L combination. In order to compute these factors the dead load factor was set to γ_D = 1.2 and the resistance factor was separately selected for each failure mode.

- U = 1.2 D′ + 1.53 L′ ϕ = 0.95 for beams in the flexural failure mode
- U = 1.2 D′ + 1.51 L′ ϕ = 0.85 for beams in the shear failure mode
- U = 1.2 D′ + 1.52 L′ ϕ = 0.93 for column under combined action failure mode
- U = 1.2 D′ + 1.51 L′ ϕ= 0.85 for column in the shear failure mode

Since in TS 500 (2000), resistance factor are not considered rather safety factors of materials are used. Therefore, ϕ is equated to 1 and new sets of load were selected for the resistance factor equal to 1 is computed as;

- U = 1.2 D′ + 1.72 L′ ϕ= 1 for all failure mode

The methodology and the statistical data on loads provided in this study can be extended to the limit state design of different construction materials which includes metal structures, pre-stressed concrete and engineered masonry and also structural forms like slabs and shear walls

Values proposed in this study for the load and resistance factors are open to future improvement when new local and international data becomes available and when more knowledge and research on reliability-based design criterion is acquired.

REFERENCES

American Concrete Institute, (2005). *Building Code Requirements for Reinforced Concrete*, ACI 318/05, ACI, Detroit, Michigan, USA.

Aktas, E. (2001). *Structural Design Code Calibration Using Reliability Based Cost Optimization, Ph. D. Thesis*, The University of Pittsburgh, Pittsburgh.

Ang, A. H.S & Tang, W.H. (1984). *Probability Concepts in Engineering Planning and Design, Volume II*, John Wiley and Sons Inc., New York.

Atadero, R. a., & Karbhari, V. M. (2006). *Development of Load and Resistance Factor Design for FRP Strengthening of Reinforced Concrete Structures*. Vol. 1, pp. 17-26

Bulleit, W. M. (2008). Uncertainty in Structural Engineering. *Practice Periodical on Structural Design and Construction*, *13*(1), 24–30.

Ellingwood, B.R., Galambos, T.V., MacGregor, J.G. & Cornell, C.A. (1980). *Development of a Probability Based Load Criterion for American National Standards* A58, NPS Special Publication 577.

Ellingwood, B. & Galambos, T.V. (1982). *Probability-Based Criteria for Structural Design*, Structural Safety, No.1, pp. 15-26.

Ellingwood, B. & Ang. A. H. S. (1972). *Probabilistic Study of Safety Criteria for Design, Structural Research Series No. 387*, Department of Civil Engineering, University of Illinois, Urbana.

Ellingwood, B.R. (1996). *Reliability-based condition assessment and LRFD for existing structures*, Structural Safety, Vol. 18, pp 67-80.

Ersoy, U. & Özcebe, G. (2004). *Reinforced Concrete*, Evrim Yayinevi, Ankara.

Ergün, A., & Kürklü, G. (2012). *Assessing the relationship between the compressive strength of concrete cores and molded specimens*. Gazi University Journal of Science, 25(3), 737–750.

Firat, F.K. (2007). Development of Load and Resistance Factors for Reinforced Concrete Structures in Turkey, The Graduate School of Natural and Applied Sciences, Middle East Technical University, Doctorate Thesis, Ankara.

Firat, F. K. (2016). *Mechanical Properties of Reinforcing Steel in R/C: Uncertainty Analysis and Proposal of a New Material Factor*. Arabian Journal for Science and Engineering, 41(10), 4019–4028.

Kömürcü, A.M. (1995). A Probabilistic Assessment of Load and Resistance Factors for Reinforced Concrete Structures Considering the Design Practice in Turkey, M. Sc. Thesis, Department of Civil Engineering, METU, Ankara.

Kömürcü, A.M. & Yücemen, M.S. (1996). Load and resistance factors for reinforced concrete beams considering the design practice in Turkey, Concrete Technology for Developing Countries, Fourth International Conference, Eastern Mediterranean University, Gazimağusa, North Cyprus.

Mirza, S. A. & MacGregor, J.G. (1979). *Variations in Dimensions of Reinforced Concrete Member*, Journal Of The Structural Division, ASCE, Vol. 105, No. ST4, pp. 921-937.

Mirza, S.A. & MacGregor, J.G. (1979). *Variability of Mechanical Properties of Reinforced Bars,* Journal of the Structural Division, ASCE, Vol. 105, No. ST4, pp. 751-766.

Mirza, S.A. & MacGregor, J.G. (1982). *Probabilistic study of strength of Reinforced concrete members*, Can. J. Civ. Engr., Ottawa 9, pp. 431-438.

Nawy, E.G. (2005). *Reinforced Concrete* A Fundamental Approach, Pearson Education, Inc., New Jersey, USA.

Nowak, A.S. & Szerszen, M.M. (2003a). *Calibration of Design Code for Buildings (ACI318): Part 1- Statistical Models for Resistance,* ACI Structural Journal, May-June, pp. 377-382.

Nowak, A.S. and Szerszen, M.M. (2003b). *Calibration of Design Code for Buildings (ACI318): Part 2- Reliability Analysis and Resistance Factors,* ACI Structural Journal, May-June, pp. 383-391.

Öztemel, A., & Şensoy, S. (2004). Mathematical Model for the Probability Distribution of in-Situ Concrete Compressive Strength of in-Situ Concrete Compressive Strength. *29th Conference on Our World In Concrete & Structures*, 387–395.

Rackwitz, R. (2000). *Optimization- the Basis of Code-Making and Reliability Verification*, Structural Safety, Vol. 22, pp. 27-60.

Salgado, R. (2008). *The Engineering of Foundations*. McGraw-Hill.

Semih, M., & Firat, F. (2014). *Determination of New Load and Resistance Factors for Reinforced Concrete Structural Members* † 1, *25*(3), 1785–1808.

Toft-Christensen, P. and Baker, M. J. (1982). *Structural Reliability Theory and Its Applications*, Springer Verlag, Berlin.

TS 498; (1997). *Turkish Standard Institution, Ankara, Turkish Standard Institution, Ankara*,

TS-500; (2000). *Requirements for Design and Construction of Reinforced Concrete Structures, Turkish Standard Institution, Ankara*.

Yücemen, M.S. and Gülkan, P. (1989). *Betonarme Yapilar _cin Yük ve Dayanim Katsayilarinin Belirlenmesi*, 10. Teknik Kongre, Cilt 2, _nsaat Mühendisleri Odasi, s. 637-651.