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DEVELOPMENT OF CUSTOM LOAD AND RESISTANCE FACTORS FOR DESIGN OF REINFORCED CONCRETE STRUCTURES

THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
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IBRAHIM ELOSTA

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Approval of the Graduate School of Natural and Applied Sciences, Atilim University.

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Prof. Dr. Ender KESKINKILIÇ
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of **Master of Science in Civil Engineering Department, Atılım University**.

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Asst. Prof. Dr. Saman AMINBAKHSH
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This is to certify that we have read the thesis "Development of Custom Load and Resistance Factors for Design of Reinforced Concrete Structures" submitted by Ibrahim Elosta and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

DEVELOPMENT OF CUSTOM LOAD AND RESISTANCE FACTORS
FOR DESIGN OF REINFORCED CONCRETE STRUCTURES

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The load and resistance factors used nowadays in the design of reinforced concrete structures were developed before this century. Using these factors from the past significantly penalizes the design of reinforced concrete structures constructed using materials having better quality control and loads having better predictions of occurrences today. The purpose of this study was to develop a tool that determined the load and resistance factors depending on the statistical data (bias and covariance) related to current materials (concrete and steel) and current prediction of loads (dead, live, etc.) and the target reliability index. The First Order Second Moment (FOSM) and Monte Carlo Simulation (MSC) were the methods used as the structural reliability models. The first method was used to determine the resistance parameters for different failure modes. These resistance parameters (biases and covariances) were calculated using 20 million random variables using MCS Method to determine reliability index values. Finally, a program using Microsoft Excel Software was developed to determine custom load and resistance factors to design reinforced concrete members. Based on the input date for biases and covariances of resistance, dead, and live loads; failure modes of the beams and column members; and the target reliability index, the produced program selects custom load factors for your project.

Keywords: Reliability, Load and Resistance Factors, Reliability Index, Safety Level, Monte Carlo Simulation, First Order Second Moment.

ÖZ

BETONARME YAPILARIN TASARIMI İÇİN ÖZEL YÜK VE DİRENÇ KATSAYILARININ GELİŞTİRİLMESİ

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Danışman: Dr. Öğr. Üyesi Halit Cenan MERTOL
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Betonarme yapıların tasarımında kullanılan mevcut yük ve direnç (dayanım) katsayıları bu yüzyıldan önce geliştirilmiştir. Geçmişten gelen bu katsayıların kullanılması, daha iyi kalite kontrolüne sahip malzemeler ve daha iyi tahminlere sahip yükler kullanılarak inşa edilen betonarme yapıların tasarımını günümüzde önemli ölçüde cezalandırmaktadır. Bu çalışmanın amacı, mevcut malzemeler (beton ve çelik) ile ilgili istatistiksel verileri (bias ve kovaryans) ve yük tahminlerini (ölü, canlı vb.) kullanarak istenilen hedef güvenilirlik endeksine göre yük ve dayanım (direnç) katsayılarını belirleyen bir araç geliştirmektir. Birinci Derece İkinci Moment Moment (BDİM) ve Monte Carlo Simülasyonu (MSC) yapısal güvenilirlik modelleri olarak kullanılan yöntemlerdir. Farklı göçme modları için direnç (dayanım) parametrelerini belirlemek için ilk yöntem kullanılmıştır. Bu direnç parametreleri güvenilirlik indeksi değerlerini belirlemek için MCS Metodu kullanılarak 20 milyon rastgele değişken kullanılarak hesaplanmıştır. Son olarak, betonarme elemanlar tasarlamak için özel yük ve direnç katsayılarını belirlemek için Microsoft Excel Yazılımı kullanılarak bir program geliştirilmiştir. Bu programı kullanılarak, direnç (dayanım), zati ve hareketli yük verileri kullanılarak kiriş ve kolon elemanlarının göçme modlarına ve hedef güvenilirlik indekslerine göre, kendi projeniz için özel yük katsayıları seçilebilmektedir.

Anahtar Kelimeler: Güvenilirlik, Yük ve Direnç (Dayanım) Katsayıları, Güvenilirlik İndeksi, Güvenlik Seviyesi, Monte Carlo Simülasyonu, Birinci Dereceden İkinci Moment.

To my family

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LIST OF ABBREVIATIONS

FOSM	First Order Second Moment Method
RC	Reinforced Concrete
C.D.F.	Cumulative distribution function
c.o.v.	Coefficient of variation
LRFD	Load and Resistance Factor Design
FOSM	First Order Second Moment Method
TS 500	Turkish Standard for the Requirements of Design and Construction of Reinforced Concrete Structures
MCS	Monte Carlo Simulation

LIST OF SYMBOLS

A_s	Area of the steel reinforcement
A_{sw}	Cross sectional area of stirrups
bw	Member width
c_b	Depth of neutral axis at the balanced case in reinforced concrete cross section
c	Distance from the neutral axis to outer compressive fiber in a T cross section
D	Dead load effect
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
M	Safety margin
M_r	Bending moment capacity
$\bar{N}$	Correction factor
N	Axial load
P_f	Probability of failure
P(E)	Probability of event E
P_s	Survival probability (reliability)
R	Resistance
s	Spacing of stirrups
U	Effect of factored load
V	Total design base shear

V_d	Maximum design shear force
V_r	Shear strength
V_w	Resistance of shear reinforcement
X	Basic random variable
X'	Nominal of X
$\bar{X}$	Mean of X
β	Reliability index
β_T	Target reliability index
Δ	Prediction of uncertainty
ε_{cu}	Ultimate strain in concrete
μ	Mean value
δ	Basic variability
ρ_b	Balanced steel ratio
ρ	Steel ratio
ϕ	Generalized resistance factor
σ	Standard deviation
Ω	Total variability

CHAPTER 1

INTRODUCTION

1.1 Background Information

The main priority in the design of civil engineering structures is to provide a structure that is safe, serviceable, and economical. These features are the main foundation for a structural engineer when it comes to the design of any structure. To achieve safe, serviceable and economical designs, a safety requirement shall be integrated in engineering designs based on the risks associated against failure.

Limit State Design (LSD), may also be named as Load and Resistance Factor Design (LRFD), is a method used in design of structures. When this stage is reached, the structure is on the limit of becoming unsuitable for the use and it is the condition immediately before the collapse. During the LSD approach, all loads that are expected to occur during the life time of a structure are evaluated.

By using a combination of the LSD and a probabilistic approach, the LRFD considers the uncertainty of the parameters related to load and resistance. Serviceability Limit State (SLS) and Ultimate Limit State (ULS) are the two types of limit state design [1]. The SLS is concerned with functional use of the structure under normal service loads, while the ULS is related to the design for the safety of the structure which may include the failure or collapse of a structural system [1]. This occurs when the load effects on the structure are equal to or greater than the resistance of structure. Therefore, the ULS must be identified and prevented.

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

Design codes provide safe and economical guidance in engineering designs. They provide a probabilistic approach in design, due to the shortcomings of the deterministic approach in solving structural safety issues. Since the design of structures needs to be performed in the presence of some uncertainties, a probabilistic approach is used to identify these uncertainties for safety purposes.

The reliability of the structure is determined by comparing the impact of the load with the effect of the resistance for structural elements such as beams, columns, etc. In a probabilistic approach the load and resistance parameters are considered as random variables and safety is determined using an acceptable reliability index. The reliability index is an indicator for the probability of safety of the structure. The higher the reliability index, the lower the probability of failure. The reliability of the entire structure is calculated by adding the reliability of all the individual structural members.

To calibrate and develop new load and resistance factors, different potential failure modes shall be taken into consideration for each of the failure mode. Ratio of mean to nominal value, called as bias, and total uncertainties (covariance) are considered to calculate the reliability index. The amount by which the prediction (adjusted for all data sets) varies from the anticipated regression function is indicated by bias. Covariance is a measure of how much specific data sets diverge from the average. Low bias and low covariance results in the best estimate whereas high bias and high covariance results in the worst estimate. A graphical definition of bias and covariance is shown in Figure 1-1.

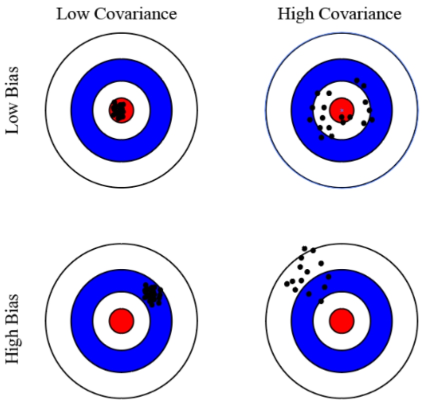


Figure 1-1 – Definition of Bias and Covariance

1.2 Problem Statement

The safe and economical design of buildings require an understanding of the factors applied for loads (dead, live, etc.) and resistance and their uncertainties. As a result, this goal may be achieved by defining the most effective design factors and using risk-based design methodologies. It is critical to reduce or increase the level of design safety (reliability index). For instance, the safety level of a nuclear power plant must be greater than that of a regular structure since the effect of failure of each will produce different levels of hazards.

The design codes such as ACI 318 (2019) [2] and Turkish Building Code, TS 500 (2000) [3] are using the probabilistic data related to the reliability index based on the material strength such as compressive strength of concrete and yield/ultimate strength of steel. Various sources of uncertainties associated with load and resistance parameters were investigated and quantified using reliability method in Turkey and worldwide. The proposed criteria and the resulting load and resistance factors produced a rational basis that reflected the effects of uncertainties directly on designs.

1.3 Purpose of Study

The strength of material for concrete can vary through the years due to the development of technology and adequacy of production of the machines from industries and yield strength of steel can vary from different factories which will be used in reinforced concrete buildings.

The load and resistance factors used nowadays in the design of reinforced concrete structures were developed before this century. The quality control on the materials nowadays is much better compared to that of 50 years ago. Furthermore, the loads on the structures can be predicted much better than the predictions performed 50 years ago. Using these factors from the past significantly penalizes the design of reinforced concrete structures constructed using materials having better quality control and loads having better predictions of occurrences today.

The purpose of this study was to develop a tool that determined the load and resistance factors depending on the statistical data (bias and covariance) related to current materials (concrete and steel) and current prediction of loads (dead, live, etc.) and the target reliability index. Initially for a reinforced concrete structure, beams and columns having various dimensions and reinforcement configurations were analyzed for various resistance factors and for different failure modes. These results were compared to the load effects resulting from various factors and combinations of dead and live loads. The reliability index for each comparison was calculated to evaluate the effects of load and resistance factors on the variation of safety (reliability index).

A program was developed to automatically calculate these parameters using Visual Basic for Applications. In addition, this program could calculate a set of load and resistance factors for the design of reinforced concrete structural elements such as beams and columns under different failure mode using probabilistic approaches and by taking the effects of uncertainty which are related to material properties for beams and columns. The Turkish Design Code (TS 500, 2000) [3] and American Design Code [2] were used as the guides in this study.

The First Order Second Moment (FOSM) Method and Monte Carlo Simulation (MCS) were the methods used as the structural reliability models. The first method was used to determine the resistance parameters for different failure modes. These resistance parameters (biases and covariances) were calculated using 20 million random variables for MCS Method to determine reliability index values and to develop the load and resistance factors.

1.4 Scope of Study

The scope of custom determination of load and resistance factors for reinforced concrete design using reliability approaches is a great work which includes every member of reinforced concrete structures, every type of load applied on the members, various methods in finding the factors, etc. To limit the scope of this research, reinforced concrete rectangular beams and columns were analyzed for flexural, combined flexure and axial, and shear failure modes subjected to dead and live loads.

The material and geometrical properties were limited to the configurations explained in this study.

1.5 Outline of Study

This thesis is organized into 6 chapters:

Chapter 1 provides brief information on LRFD based design procedure and reliability analysis. In addition, this chapter includes the problem statement, purpose, and scope of the study.

Chapter 2 provides a literature review of the studies which have been conducted on reliability analysis of the reinforcement concrete elements. This chapter also introduces the concept of structure reliability, probability theory, and different methods used to compute the reliability index. Furthermore, load and material parameters are explained.

The failure modes used in this study are presented in Chapter 3.

In Chapter 4, the procedures to determine the biases and covariances of resistance, dead, and live loads are explained in detail. Conversions of these biases and covariances to normally distributed functions and establishment of limit state functions are presented. Calculation of reliability index is described also in this chapter.

Detailed analytical study is given in Chapter 5. Sensitivity analyses of reliability index are explained for resistance, dead and live load parameters and load and resistance factors are deeply discussed. The program is presented to calculate the custom load and resistance factors.

Conclusions and future studies are presented in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 Calibration of Design Codes

At the beginning of the twentieth century, the concept of probability estimation of structural safety was first implemented. The American Concrete Institute (ACI) Building Code, ACI 318 (2019) [2] adopted an ultimate limit condition design approach in the early 1960s, which used load factors to maximize the load effects and strength factors to minimize the resistance. This design approach became known as the Ultimate Strengths Design (USD). These factors were developed using elementary statistical analysis resulting from incomplete data and insufficient information. Remarkable research in this area started in the late 1960s, and there has been a growing interest in structural reliability since then.

Extensive studies were carried out in the 1970s to calculate load and resistance parameters for steel buildings. During that time, it was recognized that the use of multiple load and resistance variables in the design of steel and reinforced concrete structures might cause design confusion. In 1976, the ACI Building Code Committee passed a motion endorsing the concept of nearly identical load factors for all materials in a manner that all committee members agreed on. To suggest a comprehensive set of load and resistance factors for use in building design, the Building Technology Center at the National Bureau of Standards brought in Drs. Cornell, Ellingwood, Galambos, and MacGregor together in 1979.

Rackwitz (2000) examined the process of code generation by minimizing the overall cost based on reliability. This research examined Rosenblueth and Hasofer's [4, 5] contributions in detail and applied their principles to a variety of failure modes, including ultimate limit state failure under ordinary and severe conditions, serviceability failure, fatigue and other deterioration, and obsolescence. This study also considered the maintenance and reconstruction costs. Rackwitz (2000) argued that comprehensive rebuilding or repair was the only reasonable reconstruction policy for virtually all civil engineering facilities. The failure rate (failure intensity, renewal density) was discovered to be the most important criteria for establishing safety/reliability goals for high reliability systems [6].

Aktas (2001) examined the calibrations of the structural design code using cost optimization based on reliability, where predefined target reliability levels were not used [7]. In his study, Aktas (2001) concluded that to achieve an optimum code, increasing the failure cost factor (when the effects of failure are raised) affected both the safety index and the load factors, requiring higher values for both. Due to the fact that as the cost of failure have risen, higher reliability levels were required [7].

The Building Code Requirement for Structural Concrete ACI 318 (2019) [2] was calibrated based on the study conducted by Nowak & Szerszen (2007) [8, 9]. The aim of this research was to find resistance factors that were compatible with the load and load combination factors defined in the ASCE 7-98 Standard [10]. The study was performed in two parts. The first part (2003a) discussed the topic of statistical model for the resistance parameters, which created the basis of the selection of the resistance factors (strength reduction factors). In their analysis, the parameters of the resistance "R" have been calculated for ordinary concrete, high-strength concrete, and light-weight concrete, cast-in-place and plant-cast, and a wide range of reinforcing steel bars, from 9.5 mm to 34.5 mm. The determination of the parameters of the resistance such as bias and covariance was performed according to material properties, fabrication factor and professional factor [9].

The second paper Szerszen and Nowak (2003) discussed reliability analysis and methods of selecting a resistance factor for reinforced and prestressed concrete elements. The researchers in this study stated that there was no need to minimize the resistance factor in ACI 318 (2019) [2] . However, the combination factors for dead and live loads were recommended to be increased since the reliability indices for load combinations with a dead load of about 80-90 percent of the total load and a live load of 10-20 percent were determined to be low [8].

The probabilistic modelling approach is not well recognized in Turkey. As a result, there was no literature on probabilistic design. Moreover, Turkish Design Code (TS 500, 2000) [3] was using a deterministic approach, and reliability-based design was not included in its calibration. In Turkey, Yücemen and Gülkan (1989) were the first to conduct substantial research on the reliability-based design subject. Their research focused on proposing a series of load and resistance factors depending on the reliability for the design of reinforced concrete beams by using FOSM method [11].

Later, Kömürcü and Yücemen (1996) established a reliability-based design criterion to be used in the design practice in Turkey for singly reinforced concrete beams in flexural failure mode. Many sources of uncertainties related to loads, material properties, dimensions, workmanship, and design methods were considered in their analysis. Moreover, the FOSM method was used as the reliability model. In this study, it was stated that the optimal load factors which minimize the deviations from selected target reliabilities were found to yield to low resistance factor for D+L combination and to resistance factors that are greater than 1 for D+L+E combination. Hence in order to overcome this issue, the dead load factor was set to 1.3 and resistance factor to 0.9 or 1.0 in all of the load combinations [12].

Fırat (2007) established a reliability-based design for T-shaped reinforced concrete beams and rectangular columns. The purpose of this study was to develop a set of load and resistance factor for reinforced concrete beams and columns using the local condition in Turkey. The FOSM was used as the reliability method. The data gathered from various concrete production plants in Turkey between 1998 and 2004 combined with the data published in the international literature was used in the analysis. The resulting data from Turkey consisted of the 28-day concrete compressive strength tests of approximately 11000 cubic specimens. The mean value of cubic compressive strength was determined to be 29.87 N/mm^2 and the covariance was equal to 0.105. The cubic compressive strength of concrete data was converted to the standard cylinder strength value resulting in a mean value of 24.87 N/mm^2, a bias and a covariance of 1.25 and 0.18, respectively. The parameters related to steel were calculated using the yield strength data obtained from various steel manufacturing plants in Turkey. The mean value was estimated as 501.37 N/mm^2, the bias and covariance were 1.24 and

0.09, respectively. The author performed reliability analyses on singly reinforced flanged concrete beams and columns considering various failure modes specified by Turkish codes and design practices. The author stated that the reliability indices ranged from 2.41 to 2.90 for dead and live (D+L) combination. In this analysis, the optimum load and resistance factors for flexural failure mode of beams were estimated as 1.2D+1.74L and 0.90R, for shear failure mode as 1.2D+1.38L and 0.70R. Furthermore, for columns in the combined flexure and axial load failure mode, the factors were determined as 1.2D+1.8L and 0.8R, in the shear failure mode as 1.2D+1.39L and 0.7R [13].

When the latest and previous researches on reliability analysis of reinforcement concrete elements were examined, it was discovered that all reliability analyses and estimation of resistance and load factor were conducted on beams and columns having single reinforcement. Moreover, the reliability analysis of the previous studies used direct reliability method in determination of the reliability index which might result in approximate values of reliability index. As previously explained, the current Turkish Building Code (TS 500, 2000) [3] was established using directly the old data from various sources of uncertainty.

2.2 Reliability Theory

Structural engineers mainly focus on some characteristics of designed structures such as safety, stability, cost, etc. However, the uncertainties in the properties of the materials that are used in structures may result and increase in construction costs. Structural components must meet the criteria related to the acceptable and reliable standards. All designs must ensure that the structure is reliable. The reliability of the structure is the ability of the building to perform the required functions to which it was designed for under various conditions. These structures shall not require any significant maintenance, essential repairs and reconstructions during its lifetime.

Design codes provides simple, economical, and safe methods for the design of civil engineering structures. Design codes optimize the resources such as the materials, equipment, and workmanship. Design codes use design equations as the basis, and the reliability of a given design can be checked by comparing the resistances to load effects. Since the loads and resistances are subjected to uncertainties, structural design must be performed considering the presence of these uncertainties.

Many sources of uncertainty are inherent in structural design. The parameters of loading and the load carrying capacities of structural members are not deterministic quantities. They are random variables and thus absolute safety cannot be achieved. Consequently, structures must be designed to serve their functions with a finite probability of failure. Safety is required if the hazard is intended to be kept under control and the risk is limited to an acceptable level. Reliability is defined as the probability a member or the structure to perform its intended function for a specific period, under specific conditions. The probability of failure is the inverse of reliability. If the reliability is defined as P_s and the probability of failure is defined as P_f, the relation between these two concepts can be expressed as $P_s = 1 - P_f$. In contrast to safety, reliability can be measured by means of probability.

2.2.1 The Concept of Structural Reliability

Structural reliability concept is concerned with the rationalistic treatment of uncertainties which is related to design of safe and serviceable civil engineering structures. Structural reliability is related to the calculation of the probability of a structure to exceed the limit states used in design under its specific period.

The limit state in reliability theory is defined as the boundary between desired and undesired structural performance and it is represented by the limit state function. The limit states of a structure can be classified into three groups namely collapse, serviceability and fatigue limit states.

The determination of the reliability of the load carrying capacity of a column can be given as an example of structural reliability and limit state condition. In this case, reliability is a function of the maximum applied load on the column and the strength of the column. The risk can be expressed as the probability that a column subjected to a maximum applied load that exceeds the capacity of that column. The structure will be safe when the capacity is greater than the applied loads as expressed in the equation below:

$$\text{Safety} \rightarrow \text{Resistance (Capacity)} > \text{Load Effects (Applied Loads)} \quad (2\text{-}1)$$

The safety examination and risk evaluation can be classified into three groups:

- The partial safety factor based on the approach of basic design variables.
- Checking safety and economy at some selected locations on the failure boundaries and limit states considering uncertainties.
- Exact estimation of safety based on the details related to probabilistic analysis of the structural systems.

2.2.2 Theory of Probability

The theory of the probability can be defined as an outcome A which occurs T times in N mutually exclusive, equally likely, and exhaustive trials. The probability of occurrence of A can be expressed as:

$$\mathrm{P[A]} = \frac{\mathrm{T}}{\mathrm{N}} \quad (2\text{-}2)$$

For mathematicians, probabilities are numbers in the range between 0 and +1 that determine the probability of a certain random event or uncertain.

$$0 \leq \mathrm{P[A]} \leq 1 \quad (2\text{-}3)$$

In civil engineering, the probability can be considered as the design of a structure. After design and construction, only two outcomes are possible either safety or failure. Both are mutually exclusive; they are also called exhaustive.

$$\mathrm{P[safety]} + \mathrm{P[failure]} = 1 \quad (2\text{-}4)$$

The probability of the safety of the structure ($\mathrm{P[safety]} = R$) is defined as

$$R = 1 - P_f \tag{2-5}$$

2.2.3 Random Variable and Probability Distributions

Random variables (X) are variables which always connected with a probability of occurrence, and it is expressed using various probability distributions according to statistical theory. Also, the numerical values of the random variables cannot be determined or expected with certainty without performing experiments.

The basic identifiers of a random variable are defined by the probability density function parameters such as mean value, standard deviation, and coefficient of variation. For example, the probability of an X random variable with a known probability density function to take a value in the interval (a, b). It can be expressed in the following equation. The graph of a that probability density function is shown in Figure 2-1.

$$P(a \leq X \leq b) = \int_a^b f_x(x)dx \tag{2-6}$$

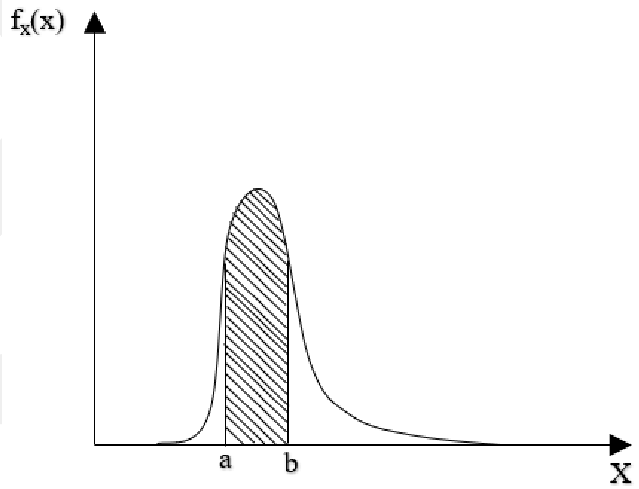


Figure 2-1 – Probability Density Function of Random Variable X

Examples of probability distributions are normal, lognormal, exponential, gamma, etc. In this study, normal distribution is used in the analyses

The random numbers were produced using the function (=RAND()) readily available in Microsoft Excel Software in this study. These random numbers were homogeneously distributed between 0 and 1.0. The bar chart produced from 4000 random variable created using the above function is shown in Figure 2-2.

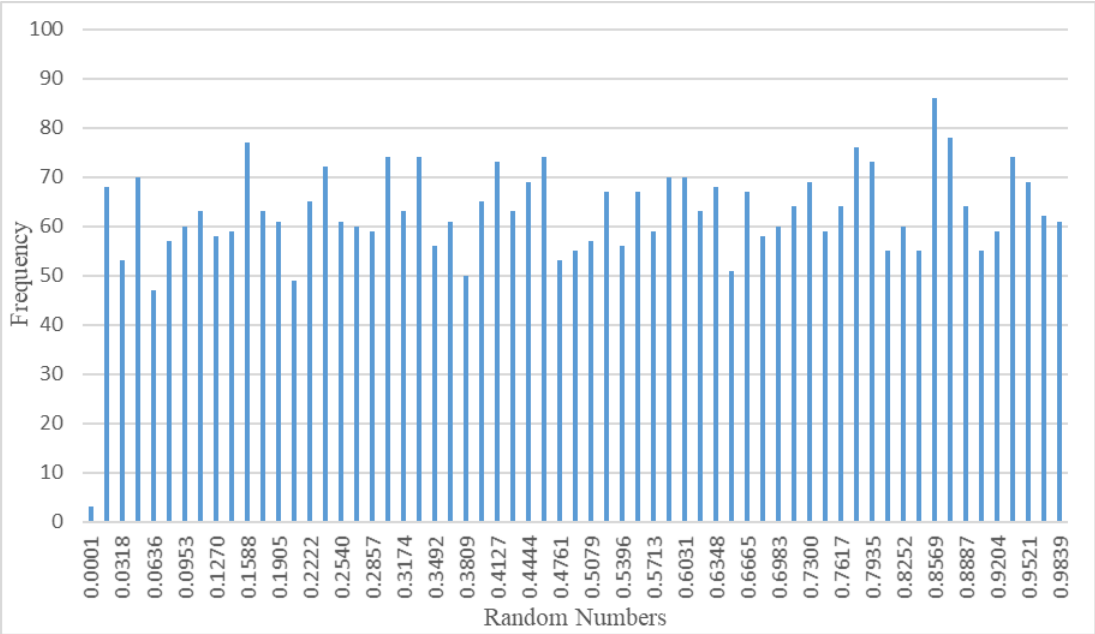


Figure 2-2 – Bar chart of 4000 Random Numbers Produced using Microsoft Excel Software

2.2.3.1 Normal Distribution

Normal distribution of random variables (also called as Gaussian Distribution) is widely used in civil engineering for variables such as loads that do not change with time, material strengths, and structural element sizes.

The Probability Distribution Function (PDF) of the random variable X can be defined as [14]:

$$f_x(X) = \left(\frac{1}{\sigma\sqrt{2\pi}}\right)\exp\left[-\frac{(x-\mu)^2}{2\sigma^2}\right] = \Phi\left(\frac{x-\mu}{\sigma}\right) \quad (2\text{-}7)$$

where μ and σ are the mean and standard deviation of X, respectively. It can be defined as the standard normal distribution PDF (Φ) and the function above ranges form −∞ to ∞. The value of $f_x(X)$ ranges from −∞ to ∞.

Cumulative Distribution Function (CDF) can be derived using the equation below [14]:

$$\Phi(\mathrm{x}) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{\mathrm{x}} \mathrm{e}^{\frac{-\mathrm{x}^2}{2}}\,\mathrm{dx} = \frac{1}{2}\left[1 + \mathrm{erf}\left(\frac{\mathrm{x}}{\sqrt{2}}\right)\right] \tag{2-8}$$

Figure 2-3 can be drawn using the PDF above.

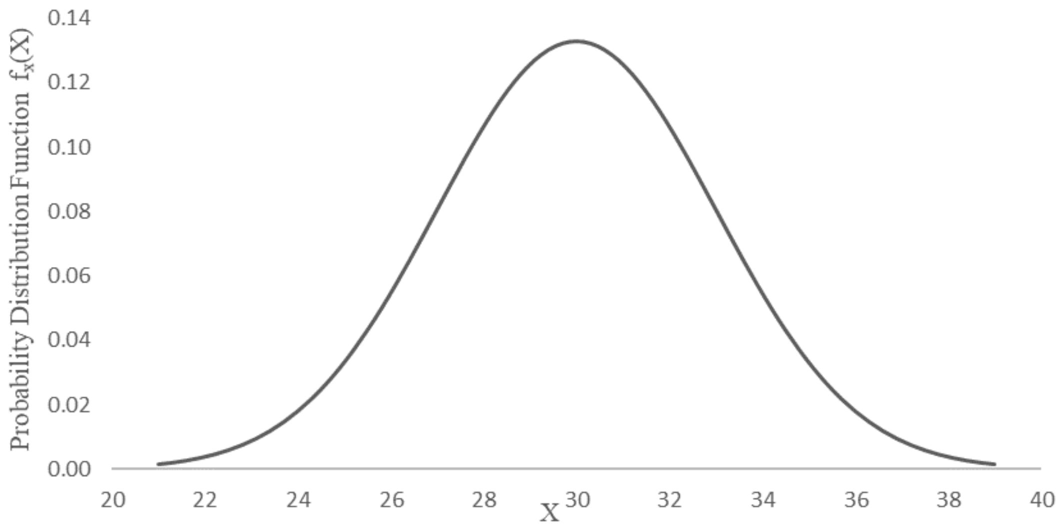


Figure 2-3 – Probability Density Function for Normal Distribution

The coefficient of variation is a measure of dispersal. The value of dispersion is calculated by dividing the standard deviation (σ) into mean (μ) , and it is used as a measure for comparison of change or variance and does not depend on the units used for the measurement of the data.

2.2.4 The Estimation of Structural Reliability

Reliability of a structural system is defined as the probability of any member of the structure not exceeding the limit state. In other words, the intended design and desired performance will be obtained during the lifetime of structure. The risk associated with the concept of reliability is the opposite of this condition. The more reliable a system is, the more risk-free it is.

The limit state function of the overall load effects acting on a structural system may be denoted by the random variable “Q” and the composition of all the elements forming the strength (capacity) of the structural system may be denoted using the random variable “R”. The purpose of the reliability analysis is to ensure that the resistance (R) is greater than the applied load effects (Q) during the lifetime of the structural system or during its specified duration. This may be expressed as R > Q.

The mathematical expression of reliability and risk of this expression can be written as the probability P (R > Q) and P (R < Q), respectively. A function (Z) can be obtained related to the capacity-demand (Z = R - Q) using these expressions. Note that, this function is also a random variable. In addition, the actual state of a structure depends on more than two random variables. These random variables can be a value such as compressive strength, length, bending moment, earthquake force, demand functions, etc.

The condition of resistance smaller than the load effects (R < Q) occurs in the following cases:

- When the load carrying capacity is less than the applied loads.
- When the allowable deflection of the member is exceeded.
- For columns, when the columns buckle due to the applied loads which are greater than the buckling strength.
- For concrete elements, the failure occurs when concrete crushes.

The boundaries between safe behavior and failure can be defined as the limit sate function g(X) which contains all random variables and is a mathematical expression of the system's performance. The limit state function can be written as [15]:

$$Z = g(X) = g(X_1, X_2, \dots, X_n) \tag{2-9}$$

When Z is greater than 0, the structural system is in the safe state; when Z is equal to 0, the structural system is in equilibrium; and when it is less than 0, the structural system collapses.

Figure 2-4 shows the PDF of the load effects, resistance, and limit sate function. In addition, it also shows the safe state when the limit state is greater than zero and the failure zone when limit state function is less than zero.

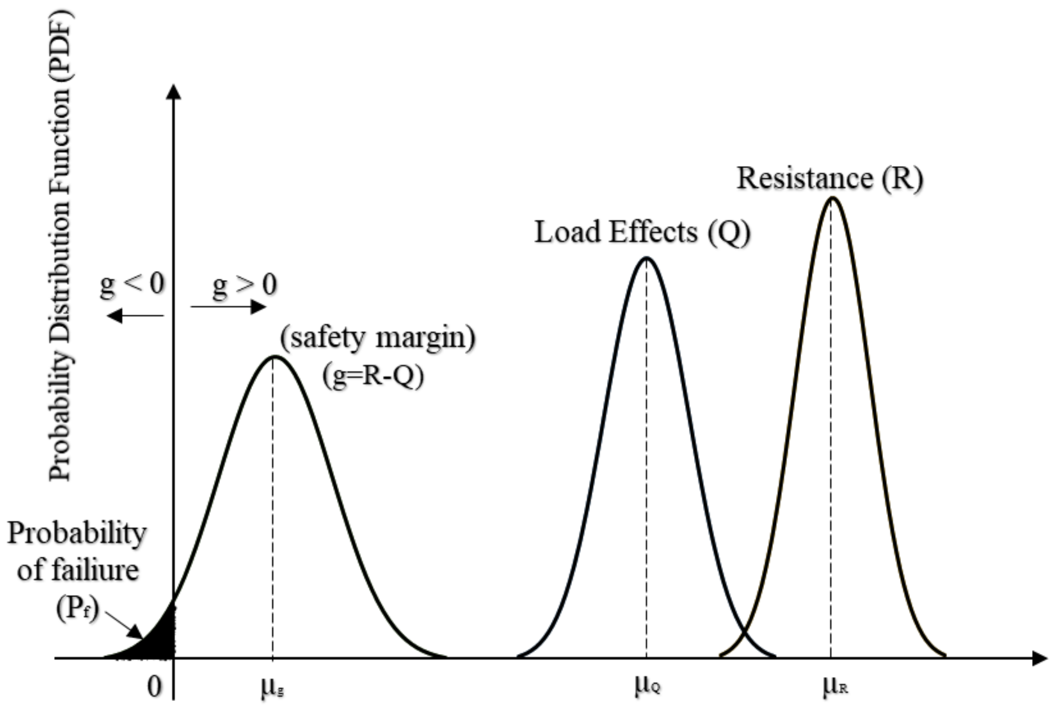


Figure 2-4 – Determination of Probability of Failure using PDF

The function below shows the probability of failure when both load effects and resistance are normally distributed [15].

$$p_f = \Phi\left(\frac{0-\mu_g}{\sigma_g}\right) = \Phi\left[-\frac{\mu_g}{\sigma_g}\right] \qquad (2\text{-}10)$$

where, $\mu_g = \mu_R - \mu_Q$ and $\sigma_g = \sqrt{\sigma_R^2 + \sigma_Q^2}$.

where mean (μ) is a measure of central tendency and standard deviation (σ) is a measure of amount of variation over range of data.

When the both load and resistance are lognormally distributed the function below is used to determine the probability of failure.

$$p_f = \Phi\left(\frac{\ln\left(\frac{\mu_{\ln R}}{\mu_{\ln Q}}\right)}{\sqrt{\sigma_{\ln R}^2+\sigma_{\ln Q}^2}}\right) \tag{2-11}$$

Two methods, namely First Order Second Moment (FOSM) and Monte Carlo Simulation (MCS), were used to estimate the structural reliability in this thesis. FOSM was used to determine the reliability of resistance parameters in different failure modes. MCS was used to determine the reliability index for various combinations.

2.2.5 Methods for Determining Reliability

2.2.5.1 Monte Carlo Simulation (MCS)

MCS method is an alternative for the integration of a common probability equation relevant to the domain of random variables corresponding to failure. MCS method can be considered as one of the exact probabilistic approach methods. In the first step of this simulation, pseudo numbers between 0 to 1 for each of the components in the limit state are generated. The generation of such numbers may be performed by using build-in functions provided by most of the programming languages. In the second step, the outcomes of the pseudo random numbers are transformed to standard normal random numbers which can be performed by the help of the inverse of cumulative distribution function. Later, the standard normal random numbers can be transformed to real distributions of random variables according to mean and standard deviation.

In MCS, each of the random variables in the limit state function is generated several times to represent its actual distribution based on its probabilistic characteristics and its parameters such as bias (ratio of mean to nominal) and covariance. Solving these problems for each random value deterministically is known as simulation cycles. In addition, each of the results of the limit state function is checked whether or not it produces a positive value. The total number of simulations are counted (n_f) and after N simulations, the probability of failure, P_f, is estimated through $P_f = [g (X) > 0] / N$.

Let the variables of a performance function Z = g (X) of a structural system have elements such as $X_1, X_2, ..., X_n$. If the probability distributions of variables can be defined and their parameters can be determined, random numbers specific to their distributions can be generated for all of them. Consequently, independent sets of random numbers can be generated as $(X_{11}, X_{21}, ..., X_{m1})$, $(X_{12}, X_{22}, ..., X_{m2})$, and $(X_{1n}, X_{2n}, ..., X_{mn})_n$ [16].

In this study, 20 million random numbers were used to converge the variation of reliability index. Less random numbers (e.g. 100000, 500000, 1000000, etc.) were used initially in the reliability index analyses however, the results did not converge to a single value with a sufficient variation (the value of the sufficient variation was selected as 0.001). As an example, when 100 thousand random numbers were used, the resulting reliability indexes were ranging from 3.43 to 3.48 for each run. However, for 20 million sample, the reliability index was exactly equal to 3.54 with variation of 0.001 for each performed analyses.

2.2.5.1.1 Pseudo-Random Numbers

The pseudo-random numbers are the numbers that are actually nondeterministic and produced using various methods. Methods such middle square, additive and multiplicative methods are used to produce these numbers. Note that, the randomness of these generated numbers shall be checked using randomness tests to determine whether they are truly random. Excel random number generator function (=RAND()) was used to avoid the repetition of random numbers.

2.2.5.2 Advanced First Order Second Moment (AFOSM)

AFOSM reliability method is also known as “Hsofer Lind” method. In this method, all the variables and limit state function are transferred to a standard normal space. The design point is determined by specifying a minimization method that results in a mark on the limit state surface and the shortest distance from the origin. The reliability index in this method which is referred as β_{HL} is also called as Hasofer Lind reliability index [5].

As seen below, the basic variables X_i's are first translated into the uniform variables Z_i's, which have zero means and unit variances:

$$Z_i = \frac{X_i - \bar{X}_i}{\sigma_{X_i}}\ ,\ \ i = 1,2,\dots,\mathrm{n} \qquad (2\text{-}12)$$

The failure surface of x-coordinate system is converted to a failure surface of z-coordinate system. In the z-coordinate system, the failure surface separates the z space into a failure zone and a safe region. The reliability index β_2 in the z-coordinate system is defined as the shortest distance from the origin to the failure surface. On the failure surface, the design point is known as the point with the shortest distance to the origin. In Figure 2-5, the reliability index, β_2, and the design point are illustrated for the two basic variables cases.

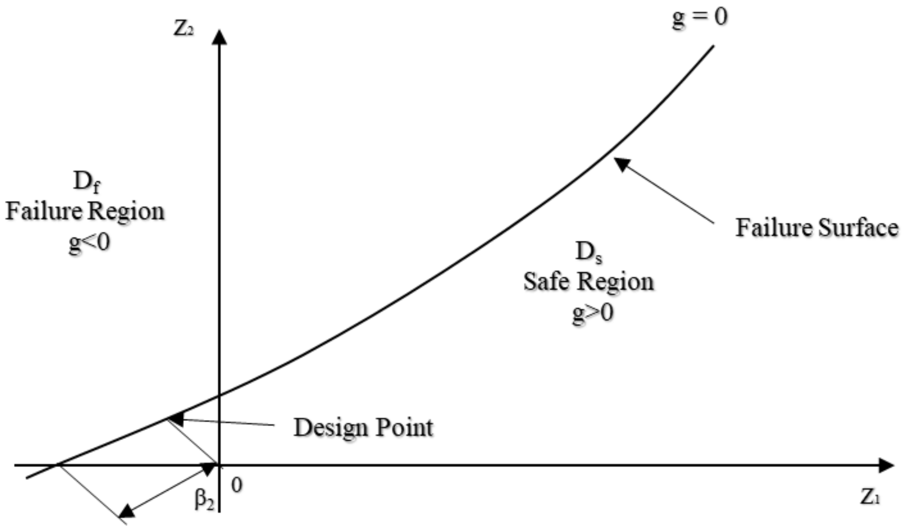


Figure 2-5 – Reliability Index β_2 [17]

This reliability index, β_2, can be formulated as follows [17]:

$$\beta_2 = min(\sum_{i=1}^{n} z_i^2)^{1/2} \tag{2-13}$$

The iterative algorithm can be used to find the design point and calculate the reliability index when the failure surface is nonlinear. According to the iterative procedure, the design point on the failure surface is determined by solving the following system of equations given from 3.21 to 3.23 [17]:

$$\alpha_i = \frac{-\frac{\partial g}{\partial z_i}}{\left(\Sigma\left(\frac{\partial g}{\partial z_i}\right)^2\right)^{1/2}} \text{ , i=1, 2, …, n} \tag{2-14}$$

$$Z_i^* = \alpha_i \beta_2 \tag{2-15}$$

$$g(z_i^*, \ldots, z_n^*) = 0 \tag{2-16}$$

where α_i's are the directional cosines that minimize β_2 and at the design point the derivatives are evaluated since the design point is related to the standardized coordinate system. Hence, the basic variables on the design point must be converted to original variable space and it can be carried out by using the expression below [17]:

$$X_i^* = \overline{X}_i(1 - \alpha_i \beta\, \Omega_i) \tag{2-17}$$

where Ω_i is the coefficient of variance which represents the total amount of uncertainty in X_i.

2.2.5.3 First Order Second Moment Reliability Method (FOSM)

This method is also called as the partial safety coefficients approach. The value of the partial safety coefficients can be determined by a probabilistic approach as a function of the coefficients of variation for material strengths and applied loads for a given risk of failure. From the expression for P_f (probability of failure), when the load and resistance are independent and normally distributed, the following expression can be derived [18]:

$$P_f = \Phi\left[-\frac{\mu_g}{\sigma_g}\right] = \Phi[-\beta] \quad (2\text{-}18)$$

Where, $\mu_g = \mu_R - \mu_Q$ and $\sigma_g = \sqrt{\sigma_R^2 + \sigma_Q^2}$

In the formula above, β is the reliability index for limit state function, g (X), which was proposed by Ang and Cornell (1974) [19] when the limit state function is linear. Also, it is known as Cornell's Reliability Index and a typical graph is shown in Figure 2-6.

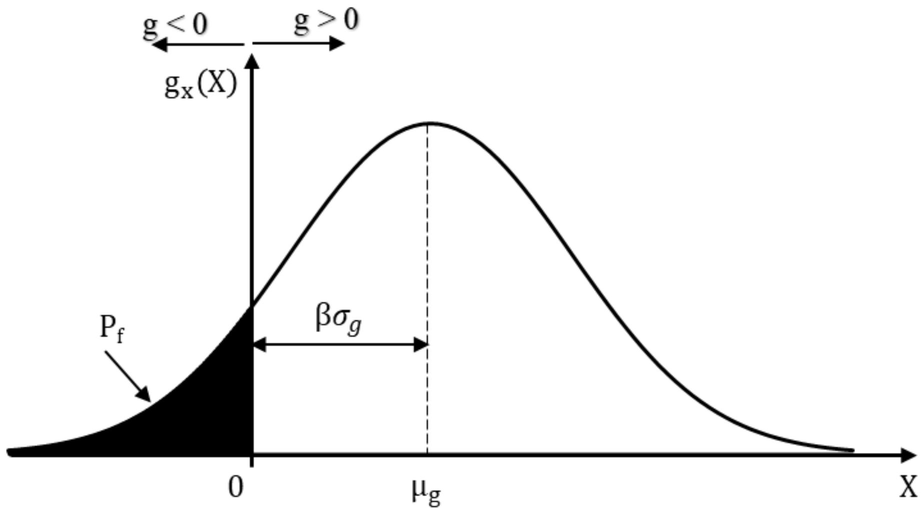


Figure 2-6 – Cornell's Reliability Index

The random variables are defined by their first two moments which are μ_g and σ_g. This is achieved by first order approximation of the nonlinear function g (X) and hence, this method is called as FOSM reliability method. In Equation 3.25, the limit state is considered as linear combination of random variables.

When the limit state function is nonlinear, the Taylor's series expression shall be used to estimate the first two moment by series of expression of limit state function g (X) about any point of X= (X_1, X_2,…..X_n) as shown in the equations 3.26 to 3.29 [19]:

$$LS = g(X_1^*, X_2^*, \ldots\ldots X_n^*) + \sum_{i=1}^{n} (\frac{\partial g}{\partial X_i}) (X_i - X_i^*) + \sum_{i=1}^{n} \left(\frac{\partial^2 g}{\partial x_i^2}\right) \frac{(X_i - X_i^*)^2}{2} + .. \quad (2\text{-}19)$$

$$\mu_g = E[g(X)] \cong g(\mu_1, \mu_2, \ldots, \mu_n) + 0 \quad (2\text{-}20)$$

$$Var(g) = \sigma_g^2 \cong \sum_{i=1}^{n} \sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i}\right) \left(\frac{\partial g}{\partial X_j}\right) cov(X_i, X_j) \quad (2\text{-}21)$$

$$p_f = \Phi\left[-\frac{\mu_g}{\sigma_g}\right] = \Phi[-\beta] \quad (2\text{-}22)$$

2.3 Load Parameters

A reliability-based design cannot be entirely computed without identifying the implications of loads on structural elements. Loads acting on a structure can take several various forms. Typically, they are classified as primary or secondary loads (Lemaire, 2009). Primary loads are loads that related to the structure's internal weight, the occupants' and furniture's weight, varying weather conditions, loading conditions experienced by the structure during construction. Secondary loads are the loads result from the change in temperature, material shrinkage, foundation settlement, etc. For the scope of this study, only two loads are considered, dead load and live load.

Dead load is defined as the weight of elements and dead load is assumed to be constant during the life time of structure. As a result, in order to evaluate the uncertainties in dead load, the dimensions of the components and the type materials used to construct the member must be considered.

Live loads, which also known as variable actions, are related to other components which are not elements that make up the structure such as people, furniture and machines.

2.3.1 Dead Loads

The weights of structure elements such as beams, columns, slabs, shear walls and roof covers, all kinds of materials in the building constitute the dead load. Compared to other loads, the amount of uncertainty in dead loads is very small. However, there will be uncertainties arising from the errors and variations in the dimensions of the building elements, the differences between the actual and predicted weights of the materials used. The main characteristics of dead loads can be explained as follows [20]:

- Dead load remains unchanged over the life of the structure, and the probability of existence of dead load at arbitrary point of time is one.
- The variation of dead load throughout time is usually insignificant.

Taking the previous statements into account, the following factors lead to the variability of the dead loads:

- Estimation of lengths, defects, and tolerances,
- Actual and nominal unit weights of construction materials,
- Weight variation due to non-structural products such as roofing and partitions,
- Approaches such as tributary area, which is a rough approximation of the actual situation, etc.

The weight of these non-structural elements causes the most difference in dead loads. As a result, Ellingwood et al. (1980) concluded that the materials which made up the structural system had only a minor impact on the dead load variability [21].

Many studies have been carried on dead load to determine the type of the probability distribution and it was stated that the dead loads followed a normal distribution. Kömürcü (1995) determined the value of mean to nominal ratio of the dead loads as 1.05 and the total uncertainty as 0.10 [22]. In another study, Szerszen and Nowak (2003) used a value of 1.05 as bias and 0.10 as covariance in their calculations [9]. Fırat (2007) used the same values offered by Kömürcü (1995) [13]. Therefore, the bias and covariance was used as 1.05 and 0.10, in this study respectively. Values for bias and covariance of dead loads suggested by various researchers are shown in Table 2-1.

Table 2-1 – Bias and Covariance for Dead Load in Different Studies

References	Bias (λ)	Covariance (Ω)
Galambos & Ravindra (1973)	1	0.08
Allen (1976)	1	0.1
Ellingwood (1978)	1	0.1
Lind et al. (1978)	1	0.05
Ellingwood et al. (1980)	1.03	0.1
Kömürcü (1995)	1.05	0.1
Fırat (2007)	1.05	0.1
Abdulhamid (2014)	1.05	0.1

2.3.2 Live Load

The moving load, storage materials, light partition walls that do not carry loads, cranes, machinery, tools, equipment, people, furniture weights, etc. are generally expressed in the structure as live loads. Live loads can exhibit different characteristics in different structures such as hospitals, hotels, shopping malls, factories, places of worship, residences, office areas, etc. In some of these structures, the human concentration in certain areas is the main source of the variation in live load.

Most of the live load studies were conducted an average of 30 years ago, and due to the lack of data and inadequacies in statistical analyses in these studies, this load could not be revealed more scientifically despite this time. It is natural that the moving load varies according to the countries, the traditions in the countries, the regions, and geographical regions within the country. However, it is known that this amount of change is insignificant compared to loads such as snow, wind, and earthquake loads. In a study by Kumar (2002), it was stated that the general characteristics and variability of the statistical analysis of live load performed in England, USA, Austria, and India were very similar [23].

The variation of the live loads during the life of a structure may be huge. It may not be predicted correctly for some conditions while designing the structures. Also, the live load may change randomly with respect to time and areas within the structure.

Live load was computed based on history data. A maximum of 50 years live load survey data was considered by Szerszen and Nowak (2003). They determined a value of 1 as bias and the covariance of 0.18 [8]. Kömürcü (1995) suggested the value of mean to nominal ratio as 1.0 and the total uncertainty as 0.27 [22]. This selection was performed based on the research carried out on live loads for a span of 50 years. Fırat (2007) used the same value that was offered by Kömürcü (1995) [13]. In this study, a value of 1.00 is used as the value of the ratio of mean to nominal and the total uncertainty related to live load was considered as 0.27. Values for bias and covariance of dead loads suggested by various researchers are shown in Table 2-2.

Table 2-2 – Bias and Covariance for Live Load in Different Studies

References	**Bias (λ)**	**Covariance (Ω)**
Ellingwood (1978)	1	0.27
Kömürcü (1995)	1	0.27
Fırat (2007)	1	0.27
Abdulhamid (2014)	1	0.27

2.3.3 Combination of Dead and Live Loads

When determining the resistance of a system, the mode of combining these loads is essential. To evaluate load combinations, there are three major approaches [7]:

- Turkstra’s Rule,
- Ferry-Borges Model, and
- Wen’s Load Coincide Method.

Turkstra's Rule is used in this study since it is mostly used in code calibration procedures of most of the modern codes due to its simplicity in implementation. Furthermore, this load combination rule is based on observations that errors are often triggered by only one load exceeding its maximum value. In other words, when one load is at arbitrary point in time and other load is at its maximum value.

The total load effect of this rule is denoted as U(t) which is calculated using the equations below. The maximum load is selected using Equation (2-24) which includes all the load combinations available for the analyses.

$$U_{(t)} = X_1(t) + X_2(t) +, \dots, + X_n(t) \tag{2-23}$$

$$\max U = max \begin{Bmatrix} X_{1\,max} + X_2 +, \dots, +X_n \\ X_1 + X_{2\,max} +, \dots, +X_n \\ . \\ . \\ . \\ . \\ X_1 + X_2 +, \dots, +X_{n\,max} \end{Bmatrix} \tag{2-24}$$

where X_i is the arbitrary point-in-time load, X_{imax} is the maximum load of the ith level.

To apply the calibration process, it is essential to consider the safety criteria for the dead and live load. The safety of reinforced concrete structural members can be ensured using following equation:

$$\phi R \geq \gamma_D D + \gamma_L L \tag{2-25}$$

where γ_D and γ_L are the load factors for dead and live loads, ϕ is the resistance factor, D is the dead load, and R is the resistance.

2.4 Material Parameters

This section addresses the determination of uncertainties of resistance parameters at materials level. The considered materials include:

- Concrete,
- Reinforcing Steel Bars.

The mean to nominal ratio (bias) and total uncertainties (covariance) for each resistance variable were calculated for calibration purposes.

2.4.1 Concrete

Concrete is made of aggregates and cement paste. The cement paste is composed of cement and water and the aggregates are composed of sand, gravel, and crushed stones. The quality of concrete is critical for the safety of reinforced concrete buildings and it can be determined by examining its characteristics such as serviceability, compressive strength, durability, performance, creep, shrinkage, etc.

The data obtained from characteristics tests of concrete such as compressive strength must be analyzed statistically to evaluate the quality of concrete. The mean (μ) and coefficient of variation (c.o.v.) shall be determined. A high mean value and a low coefficient of variation result in a better quality concrete. The characteristics of the materials used during the mixing stage play an essential role in the variations of concrete quality. Also, the temperature, mixing methods, and proportions of mixing have vital roles in altering the concrete quality in concrete mixing stage.

A variety of factors influence concrete consistency and these factors have a significant impact on the mixture. The following measures can be taken to produce high-quality concrete:

- Using well graded, hard, and durable aggregates,
- Using required amounts of water and cement for the aimed strength,
- Mixing concrete thoroughly for improved uniformity,
- Curing concrete properly (at least for 28 days).

Compressive strength of concrete is the most significant property that determines the production quality. As a result, the compressive strength evaluations using cylindrical and cubic specimens based on the related standards are used to determine the quality of concrete.

The variations in concrete strength may be caused by a variety of factors, such as:

- Changes in the testing process,
- Variation in the mixing,
- Concrete mixture ratios,
- Method of construction, and curing, etc.

Additional uncertainties apart from the inherent variability may be caused by some other factors. These factors will be considered according to the international literature since there is not much local information to quantify these factors. An example of such factors is that the consideration of in-situ concrete strength in a structure is lower than that of a cylinder specimen of the same concrete tested in the laboratory environment. Other factors may be explained as follows:

- Curing and placing processes,
- Segregation of concrete in deep member,
- Strain effect in concrete caused by micro cracking and creep,
- Flaw due to the absence of a standard testing process and poor timing, as well as inadequate machine and human accuracy,
- Size and shape, and
- Stress conditions.

For the design of a structural member, the concrete compressive strength must be obtained for all the concrete classes using many specimens (cubical or cylindrical) which will be used in the construction. For this purpose, numerous concrete specimens are tested in the laboratory environment. For each specimen, the maximum load achieved in testing is divided by the area of the specimen under load to determine the strength of each specimen. As a result of numerous tests, the data is analyzed statistically to obtain the mean and coefficient of variation. The resulting data is normally distributed. As an example, the normally distributed concrete compressive strength laboratory test results having a mean value of 30 MPa and a coefficient of variation of 0.1 (a standard deviation of 3 MPa) is shown in Figure 2-7.

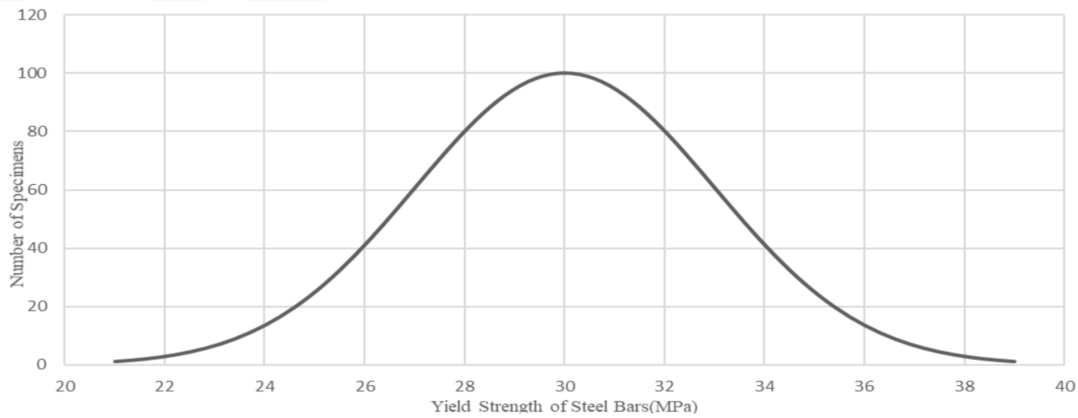


Figure 2-7 – Normally Distributed Concrete Compressive Strength Laboratory Test Data

The in-situ concrete compressive strength is different than that of laboratory strength. Therefore, the laboratory test data shall be converted to in-situ strength data used in the construction. For this purpose, the normally distributed concrete compressive strength laboratory test results shall be converted to normally distributed in-situ concrete compressive strength values. This is performed by converting the mean ($\dot{f_c}$) and coefficient of variation ($\dot{\Omega}_c$) of the laboratory results into mean ($\bar{f_c}$) and coefficient of variation ($\overline{\Omega}_c$) of in-situ data.

The magnitude of in-situ strength differences within structural members varies randomly from one member to another due to the factors which has been mentioned above. For these reasons, the strength of the concrete tested in the laboratory environment is greater than the in-situ-strength. To account for this strength variation, a correction factor (N_{c1}) having a coefficient of variation (Δ_{c1}) shall be applied to the concrete compressive strength. Ellingwood and Ang (1972) performed tests to evaluate the magnitude of the difference between the in-situ concrete compressive strength and the cylindrical strength. The correction factor (N_{c1}) was determined as in range from 0.83 to 0.92 and coefficient of variation (Δ_{c1}) as 0.16 [24]. Mirza and MacGregor (1979) suggested the correction factor (N_{c1}) between 0.74 and 0.94 and coefficient of variation (Δ_{c1}) as 0.1 [25]. Fırat (2007) recommended the correction factor (N_{c1}) and coefficient of variation (Δ_{c1}) as 0.86 and 0.13, respectively [13].

Another factor that causes the observed in-situ concrete compressive strength to vary is the strain effect in concrete caused by micro cracking and creep. As the rate of loading increases, the apparent strength of concrete increases. Because of micro cracks and creep, the strain in concrete increases over time, resulting in a decrease in observed strength. This can be considered using the correction factor for strain effect ($\mathrm{N_{c2}}$) and a coefficient of variation (Δ_{c2}). The value of $\mathrm{N_{c2}}$ was offered by Mirza and MacGregor (1979) as shown in Equation 5.1 [25]:

$$\mathrm{N_{c2}} = 0.89 \times (1 + 0.08 \log(\mathrm{R})) \qquad (2\text{-}26)$$

where R is the rate of loading. In the study performed by (Mirza and MacGregor, 1979), the correction factor for the strain effect was assumed as 0 [25]. Kömürcü (1995) suggested the value of this correction factor for strain effect as 0.88 without any coefficient of variation [22]. However, in the study performed by Fırat (2007), the correction factor and coefficient of variation were assumed as 0.87 and 0, respectively [13]. The values of N_{c2} and Δ_{c2} were calculated as 0.89 and 0 by Mahmud (2017), respectively [26].

Errors due to the absence of a standard testing procedure and poor timing, as well as inadequate machine and human accuracy are other factors that shall be considered. The factor for the correction of these issues is defined as N_3 and the coefficient of variation of this factor is denoted as Δ_{c3}. Kömürcü (1995) suggested the value of N_3 and Δ_{c3} as 0.95 and 0.05, respectively [22].

To convert the mean concrete compressive strength ($\dot{f_c}$) of the laboratory to that of the in-situ strength ($\bar{f_c}$), the following equation is used:

$$\bar{\mathrm{f}}_{\mathrm{c}} = N_{\mathrm{fc}} \times \dot{\mathrm{f}}_{\mathrm{c}} \qquad (2\text{-}27)$$

where N_{fc} is the total correction factor. The total correction factor is calculated by multiplying the three explained correction factors as shown below:

$$\mathrm{N}_{\mathrm{fc}} = \mathrm{N}_{\mathrm{c1}} \times \mathrm{N}_{\mathrm{c2}} \times \mathrm{N}_{\mathrm{c3}} \qquad (2\text{-}28)$$

There are two types of uncertainties that are used in the calculation of coefficient of variation ($\bar{\Omega}_c$) of in-situ data. The first one is the inherent uncertainty which is the coefficient of variation ($\dot{\Omega}_c$) of the laboratory tests. The inherent uncertainty is uncontrollable and unpredictable. The second uncertainty results from the prediction errors (Δ_{fc}) of the concrete compressive strength. To convert the coefficient of variation ($\dot{\Omega}_c$) of the laboratory results into the coefficient of variation ($\bar{\Omega}_c$) of in-situ data, the following equation is used:

$$\overline{\Omega}_c = \sqrt{\left(\dot{\Omega}_c\right)^2 + (\Delta_{\mathrm{fc}})^2} \tag{2-29}$$

where the prediction errors (Δ_{fc}) may be calculated by taking the square root of the sum of the squares of the coefficient of variations of the correction factors as shown below:

$$\Delta_{\mathrm{fc}} = \sqrt{\Delta_{\mathrm{c1}}^2 + \Delta_{\mathrm{c2}}^2 + \Delta_{\mathrm{c3}}^2} \tag{2-30}$$

As an example, the total correction factor for concrete compressive strength ($\mathrm{N_{fc}}$) shall be determined first. Using the data in the literature, if $\mathrm{N_1}$, $\mathrm{N_2}$, and $\mathrm{N_3}$ are selected as 0.85, 0.87, and 0.90, respectively, the total correction factor will be calculated using Equation 5.3 as:

$$\mathrm{N_{fc}} = 0.85 \times 0.87 \times 0.90 = 0.665 \tag{2-31}$$

If numerous concrete cylinders are tested under compression and the resulting mean cylindrical concrete compressive strength obtained from the laboratory tests ($\dot{f_c}$) is equal to 30 MPa and a coefficient of variation ($\dot{\Omega}_c$) of 0.1 (standard deviation of 3 MPa), the mean value of in-situ concrete compressive strength ($\bar{f_c}$) can be calculated using Equation 5.5 as:

$$\bar{\mathrm{f}}_{\mathrm{c}} = N_{\mathrm{fc}} \times \dot{\mathrm{f}}_{\mathrm{c}} = 0.665 \times 30 = 19.95 \text{ MPa} \tag{2-32}$$

Later the coefficient of variation ($\overline{\Omega}_c$) of in-situ data shall be calculated. Using the data in the literature, if Δ_{c1}, Δ_{c2}, and Δ_{c3} are selected as 0.13, 0, and 0.05, respectively, the coefficient of variation ($\overline{\Omega}_c$) of in-situ data will be calculated using Equation 5.3 as:

$$\overline{\Omega}_c = \sqrt{\left(\dot{\Omega}_c\right)^2 + (\Delta_{\mathrm{fc}})^2} = \sqrt{(0.1)^2 + \left(\sqrt{0.13^2 + 0^2 + 0.05^2}\right)^2} = 0.1715 \tag{2-33}$$

As a result, the concrete compressive strength obtained from the laboratory tests (having a mean ($\dot{f_c}$) equal to 30 MPa and a coefficient of variation ($\dot{\Omega}_c$) equal to 0.1 (standard deviation of 3 MPa)) is converted to that of in-situ data (having a mean ($\bar{f_c}$) equal to 19.95 MPa and a coefficient of variation ($\bar{\Omega}_c$) equal to 0.1715 (standard deviation of 3.42 MPa)) as shown in Figure 2-8.

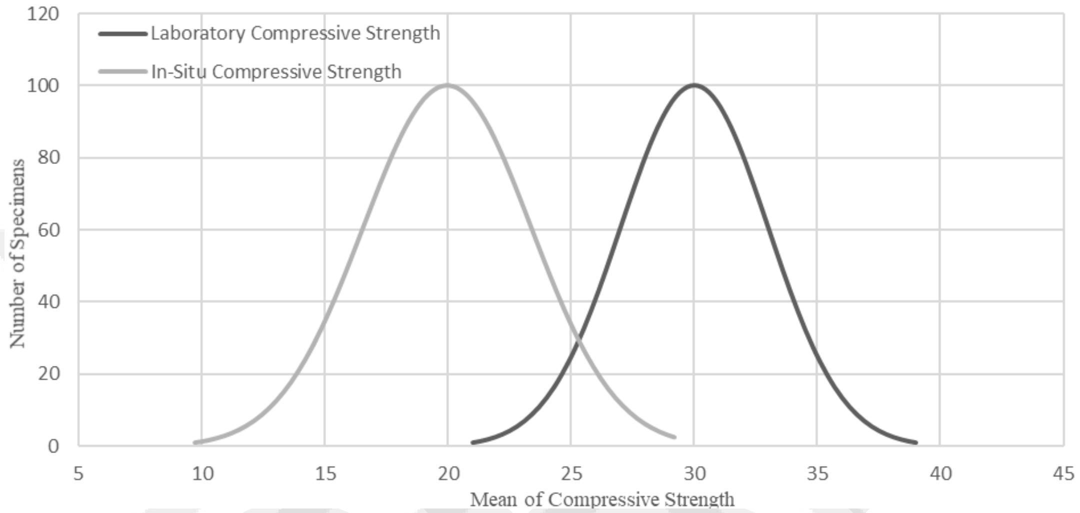


Figure 2-8 – Concrete Compressive Strength Laboratory Test Data Converted to In-Situ Data

The design concrete compressive strength (nominal) can be calculated by dividing the average cylindrical concrete compressive strength obtained from the laboratory tests ($\dot{f_c}$) by the material factor for concrete (γ_{mc}=1.5) specified by TS 500 (2000) [3] as follows:

$$\mathrm{f_{cd}} = \frac{\dot{f_c}}{1.5} = \frac{30}{1.5} = 20\ MPa \qquad (2\text{-}34)$$

The ratio (bias, λ) of the in-situ concrete compressive strength ($\bar{f_c}$) to the design concrete compressive strength ($\mathrm{f_{cd}}$) can be calculated as:

$$\lambda_\mathrm{c} = \frac{\bar{\mathrm{f}}_\mathrm{c}}{\mathrm{f_{cd}}} = \frac{19.95}{20} = 0.9975 \qquad (2\text{-}35)$$

Values for bias and covariance for concrete suggested by various researchers are shown in Table 2-3.

Table 2-3 – Bias and Covariance for Concrete in Different Studies

Bias (λ_c)	**Covariance (Ω_c)**	**References**
1.12-1.38	0.1	Nowak & Szerszen (2003a)
1.25	0.18	Fırat (2007)
1.41	0.24	Mahmud (2014)

2.4.2 Reinforcing Steel

The main characteristics which are used to identify reinforcing steel bars are their mechanical properties, such as their strength and deformation parameters. The variations in properties can be seen as a source of the uncertainty for the strength parameters (i.e., yield and ultimate strength). The below are the sources of variation in yield and ultimate strength of reinforcing steel bar:

- The variation of rate of loading and the strength of material itself.
- The use of different cross-sectional area of the bars and their effects on mechanical properties of bars.
- Effect of the strain at which yield is defined.
- The effect of usage steel bars from different steel industries.

For the design of a structural member, yield strength of the steel reinforcement must be obtained for all the steel types which will be used in the construction. For this purpose, numerous steel reinforcement bars with different dimensions are tested in the laboratory environment. This is usually performed in the steel manufacturing plants. For each specimen, the maximum load achieved in testing is divided by the area of the specimen under tension to determine the yield strength of each specimen. As a result of numerous tests, the data is analyzed statistically to obtain the mean and coefficient of variation. The resulting data is normally distributed. As an example, the normally distributed yield strength of the steel in laboratory test results having a mean value of 620 MPa and a coefficient of variation of 0.03 (a standard deviation of 18.6 MPa) is shown in Figure 2-7.

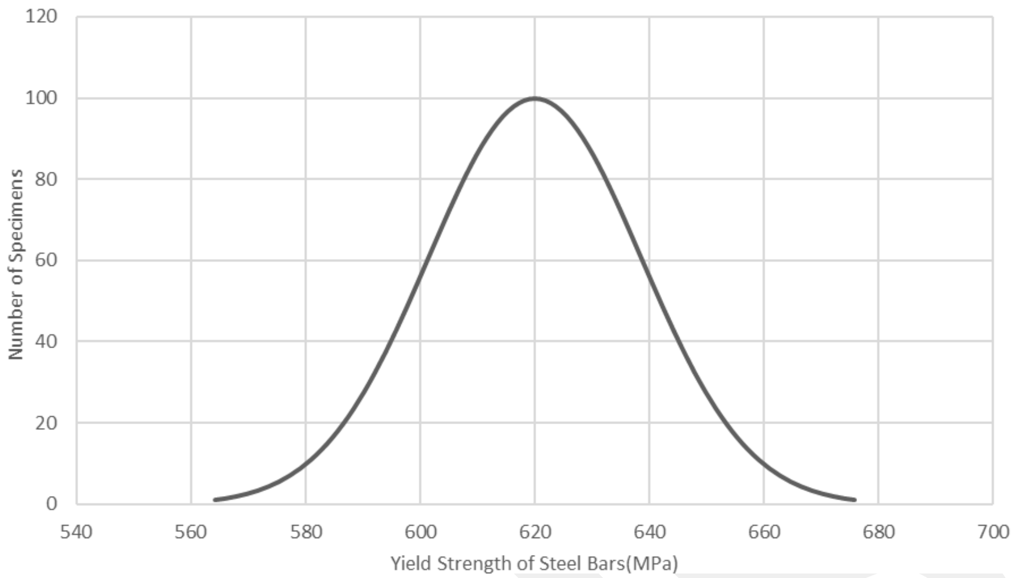


Figure 2-9 – Normally Distributed Yield Strength of Reinforcing Bars for Laboratory Test Data

To apply the uncertainties which have been mentioned above, firstly, the data related to yield and ultimate strengths of various steel types for different bar diameters used in design shall be obtained from the industry. These data include the number of samples, mean and standard deviation of yield strength, etc. The overall mean and standard deviation can be computed using these data.

The observed yield strength of a test specimen is affected considerably by the strain rate and the rate of application of the load. The strain values used in tests on reinforcing steel bars are usually significantly higher than those used in structures subjected to static loads. For this reason, tests on reinforcing steel bars are expected to overestimate the yield strength. These test results shall be corrected using the correction factor (N_{s1}) and the prediction error (Δ_{s1}) shall be taken into consideration to avoid overestimation. Mirza and MacGregor (1979) and Kömürcü (1995) determined the value of the correction factor (N_{s1}) as 0.9 and the prediction error (Δ_{s1}) as ranging from 0.01 to 0.08 [22, 25]. In the study performed by Fırat (2007), the correction factor (N_{s1}) was assumed the same whereas the prediction error (Δ_{s1}) was neglected [13].

Another factor which must be taken into consideration is the upper or lower yielding points and specific strain values may affect the yield strength. For this reason, the correction factor (N_{s2}) and prediction error (Δ_{s2}) shall be introduced. This effect was not included in the study conducted by Mirza and Macgregor (1979) [25]. However, Kömürcü and Yücemen (1995) introduced the correction factor (N_{s2}) and prediction error (Δ_{s2}) as 1.0 and 0.09, respectively [22]. Fırat (2007) suggested the correction factor (N_{s2}) as 1.0 and the prediction error (Δ_{s2}) as 0.05 [13].

Furthermore, the differences of the manufacturing plants of the steel reinforcing bars may introduce some variability. The correction factor (N_{s3}) and prediction error (Δ_{s3}) shall be used to account for this variability. In the study conducted by Ellingwood and Ang (1972), the correction factor (N_{s3}) was predicted as 1.0 and prediction error (Δ_{s3}) was suggested as 0.05 [24]. In another study by Fırat (2014), the correction factor (N_{s3}) and prediction error (Δ_{s3}) were assumed as 1 and 0.06, respectively [13].

To convert the mean laboratory yield strength ($\dot{f}_y$) data into the mean yield strength ($\bar{f}_y$) of steel used in construction, the following equation is used:

$$\bar{f}_y = N_{fy} \times \dot{f}_y \tag{2-36}$$

where N_{fy} is the total correction factor. The total correction factor is calculated by multiplying the three explained correction factors as shown below:

$$N_{fy} = N_{s1} \times N_{s2} \times N_{s3} \tag{2-37}$$

There are two types of uncertainties that are used in the calculation of coefficient of variation ($\bar{\Omega}_s$) data of steel used in construction. The first one is the inherent uncertainty which is the coefficient of variation ($\dot{\Omega}_s$) of the laboratory tests or statistically data analysis. The inherent uncertainty is uncontrollable and unpredictable. The second uncertainty results from the prediction errors (Δ_{fy}) of the yield strength of the steel bars. To convert the coefficient of variation ($\dot{\Omega}_s$) of the laboratory results into the coefficient of variation ($\bar{\Omega}_s$) data of steel used in construction, the following equation is used:

$$\bar{\Omega}_s = \sqrt{(\dot{\Omega}_s)^2 + (\Delta_{fy})^2} \quad (2\text{-}38)$$

where the prediction errors (Δ_{fc}) may be calculated by taking the square root of the sum of the squares of the coefficient of variations of the correction factors as shown below:

$$\Delta_{fy} = \sqrt{\Delta_{s1}^2 + \Delta_{s2}^2 + \Delta_{s3}^2} \quad (2\text{-}39)$$

As an example, the total correction factor for yield strength of steel (N_{fy}) shall be determined first. Using the data in the literature, if N_{s1}, N_{s2}, and N_{s3} are selected as 0.80, 0.81, and 0.93, respectively, the total correction factor will be calculated using Equation 5.3 as:

$$N_{fy} = 0.80 \times 0.81 \times 0.93 = 0.602 \quad (2\text{-}40)$$

If numerous steel bars are tested for yield strength and the resulting mean yield strength obtained from the laboratory tests ($\dot{f}_y$) is equal to 500 MPa and a coefficient of variation ($\dot{\Omega}_s$) of 0.03 (standard deviation of 15 MPa), the mean value of yield strength of steel used in construction ($\bar{f}_y$) can be calculated using Equation 5.5 as:

$$\bar{f}_y = N_{fy} \times \dot{f}_y = 0.602 \times 500 = 301 \text{ MPa} \quad (2\text{-}41)$$

Later the coefficient of variation of steel used in construction ($\bar{\Omega}_s$) shall be calculated. Using the data in the literature, if Δ_{s1}, Δ_{s2}, and Δ_{s3} are selected as 0, 0.05, and 0.06, respectively. The coefficient of variation of steel used in construction ($\bar{\Omega}_s$) will be calculated using Equation 5.3 as:

$$\bar{\Omega}_s = \sqrt{(\dot{\Omega}_s)^2 + (\Delta_{fy})^2} = \sqrt{(0.03)^2 + \left(\sqrt{0^2 + 0.05^2 + 0.06^2}\right)^2} = 0.0836 \quad (2\text{-}42)$$

As a result, the yield strength of the steel obtained from the laboratory tests (having a mean ($\dot{f}_y$) equal to 620 MPa and a coefficient of variation ($\dot{\Omega}_s$) equal to 0.03 (standard deviation of 18.6 MPa)) is converted to that of steel used in construction (having a mean ($\bar{f}_y$) equal to 373.24 MPa and a coefficient of variation ($\overline{\Omega}_s$) equal to 0.0836 (standard deviation of 31.20 MPa)) as shown in Figure 2-10.

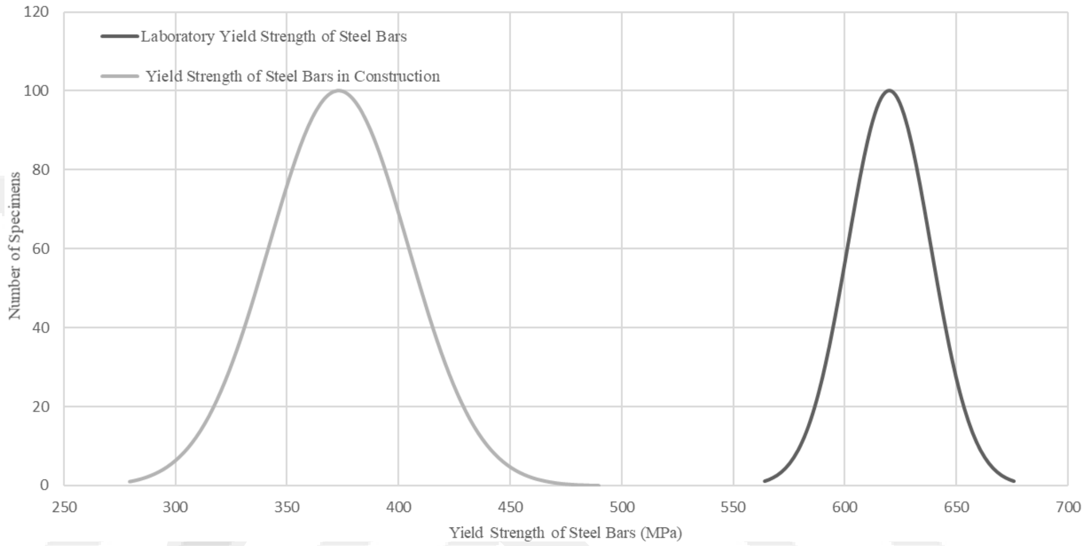


Figure 2-10 – Yield Strength of Steel Bars of Laboratory Test Data and Yield Strength of Steel Bars used in Construction

The design yield strength of the steel (nominal) can be calculated by dividing the average yield strength of steel obtained from the laboratory tests ($\dot{f}_y$) by the material factor for concrete (γ_{ms}=1.15) specified by TS 500 (2000) [3] as follows:

$$f_{yd} = \frac{\dot{f}_y}{1.15} = \frac{420}{1.15} = 365.21 \text{ MPa} \tag{2-43}$$

The ratio (bias, λ) of the mean yield strength of steel ($\bar{f}_s$) to the design yield strength of steel (f_{yd}) can be calculated as:

$$\lambda_s = \frac{\bar{f}_y}{f_{yd}} = \frac{373.24}{365.21} = 1.021 \tag{2-44}$$

Values for bias and covariance for reinforcing steel suggested by various researchers are shown in Table 2-4.

Table 2-4 – Bias and Covariance for Reinforcing Steel in Different Studies

Bias (λ_s)	**Covariance (Ω_s)**	**References**
1.145	0.035-0.065	Nowak and Szerszen (2003a)
1.24	0.17	Fırat (2007)
1.18	0.14	Mahmud (2014)

CHAPTER 3

FAILURE MODES USED IN STUDY

3.1 Introduction

The failure modes used to calculate the capacities of the cross-sections is explained in this chapter. These failure modes were:

- Beams:
  - Flexure, and
  - Shear.
- Columns:
  - Combined flexure and axial load, and
  - Shear.

3.2 Failure Modes in Reinforced Concrete Beams

Flexural and shear failures are the two most common failure modes of reinforced concrete beams. The flexural failure occurs when the applied moment exceeds the resisting moment capacity of the beam. The shear failure occurs when shear resistance of the beam is exceeded by the applied shear forces. Both failure modes may be further subdivided into various types of failure: the flexural failure may occur as a tension failure, compression failure, or balanced failure whereas shear failure may happen with or without axial load.

3.2.1 Flexural Failure in Beams

Flexural compression failure occurs when concrete crushes at the outermost compression fiber without yielding of reinforcement at the outermost steel layer at the tension side. This behavior occurs when the beam is over-reinforced, meaning the longitudinal reinforcement ratio of the beam is greater than the balanced reinforcement ratio. This type of failure results in brittle failure which is a form of failure that occurs suddenly and without warning. In reinforced concrete design, this behavior is undesirable. To prevent this type of failure, reinforcing steel may be added at the compression zone and less reinforcement may be located at the tension side.

Due to its simplicity, the corresponding rectangular stress block can be used to calculate the flexural moment capacity of a rectangular beam. The equivalent rectangular stress block is shown in Figure 3-1.

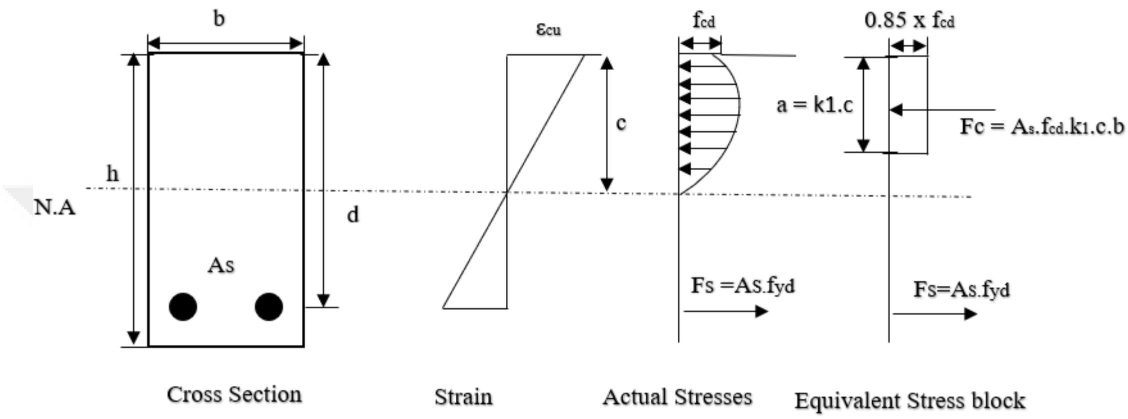


Figure 3-1 – Strain Distribution and Force Equilibrium for Beams

In this figure, h is the height of the rectangular section, b (or b_w) is the width of the section, d is the depth of outermost steel layer from the most compressed fiber, A_s is the area of longitudinal reinforcement, *N.A.* is the neutral axis, ε_{cu} is the ultimate concrete strain, c is the depth of neutral axis, f_{cd} is the design concrete compressive strength, f_{yd} is the design yield strength of longitudinal reinforcement, F_c is the resultant concrete force, Fs is the steel force, a is the height of the equivalent stress block, and k_1 is the factor related to the concrete class based on TS 500 (2000) as follow [3]:

$$k_1 = \begin{bmatrix} 0.85 \text{ for C16 to C25} \\ 0.82 \text{ for C30} \\ 0.79 \text{ for C35} \end{bmatrix} \qquad (3\text{-}1)$$

In this study, the ultimate concrete strain, ε_{cu}, is equal to 0.003 as suggested by TS 500 (2000) [3].

The balanced case of a reinforced concrete beam is critical to determine whether a beam is over- or under-reinforced. The balanced case occurs when the concrete at the most compressed fiber crushes and reinforcement at the outmost steel layer yields at the same time. The relationship used in calculating the balanced steel ratio and the actual steel ratio is given by Equations (3-2) to (3-4) [3].

$$\rho_b = \frac{0.85 f_{cd}}{f_{yd}} k_1 \frac{c_b}{d} \tag{3-2}$$

$$c_b = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} \times d \tag{3-3}$$

$$\rho = \frac{A_s}{bd} \tag{3-4}$$

where c_b is the depth of neutral axis for the balanced case, ρ_b is the balanced reinforcement ratio, ε_y is the yield strain of reinforcement, and ρ is the reinforcement ratio of the section.

When there is less area of tensile reinforcement in a beam than that of the balanced case, the beam behaves in a ductile manner. This type of beam is called under-reinforced beam. The tensile reinforcement yields first, then the concrete crushes.

Using the equivalent rectangular stress block approach, moment capacity for rectangular beam, M_r, can be calculated by using Equations (3-5) to (3-7) [3]:

$$M_r = 0.85\, f_{cd}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \tag{3-5}$$

$$\varepsilon_{si} = 0.003 \left(\frac{c - d_i}{c}\right) \tag{3-6}$$

$$\sigma_i = \begin{bmatrix} f_{yd} \;, \text{ if } |\varepsilon_{si}| \geq \varepsilon_y \\ E_s \times |\varepsilon_{si}| \,, \text{if } |\varepsilon_{si}| < \varepsilon_y \end{bmatrix} \tag{3-7}$$

where i represents the number of steel layers, A_{si} is the total area at the ith steel layer, d_i is the depth of ith steel layer from the most compressed fiber, σ_i is the stress at the ith steel layer, and ε_i is the strain at the ith steel layer.

3.2.2 Shear Failure in Beams

Shear failure occurs when the shear resistance of a beam is less than its flexural strength, and the shear force exceeds the shear capacity of the beam. A shear load is a force that causes a material to slide along a plane parallel to the direction of force action which produces a brittle failure. Since the tensile strength of concrete is very low, the principal tensile stresses induced by shear poses a significant problem in reinforced concrete members [14].

This type of failure can be avoided by supplying sufficient transverse reinforcement to reach the maximum limit state in flexure. As a result, the condition below must be satisfied.

$$V_r \geq V_d \tag{3-8}$$

where V_r is the resisting shear strength of the member, V_d is the maximum design shear force that shall be determined using the load factors and combinations.

In TS 500 (2000) [3] , the shear strength of the reinforced concrete member is equal to the sum of the concrete resistance and the shear reinforcement resistance. The 80% of the cracking strength of concrete (V_{cr}) is considered as the shear strength of concrete (V_c). The shear strength of steel is denoted as V_w. The contribution of concrete to the shear strength of the beam can be calculated using the following equations [3]:

$$V_c = 0.80 \times 0.65 \times f_{ctd} \times b_w \times d \tag{3-9}$$

$$V_w = \frac{A_{sw}}{s} \times f_{ywd} \times d \tag{3-10}$$

where f_{ctd} is the design tensile strength of concrete, A_{sw} is the area of the transverse reinforcement multiplied by the number of legs cut by the shear crack, s is the spacing of transverse reinforcement, and f_{ywd} is the yield strength of transverse reinforcement.

The overall shear resistance of the reinforced concrete beam is the summation of the shear strength of concrete and steel as follows:

$$V_r = \frac{A_{sw}}{s} \times f_{ywd} \times d + 0.52 \times f_{ctd} \times b_w \times d \quad (3\text{-}11)$$

A Microsoft Excel Workbook was produced the automate the capacity calculations for flexural and shear failure of beams. A screenshot of the program is shown in Figure 3-2.

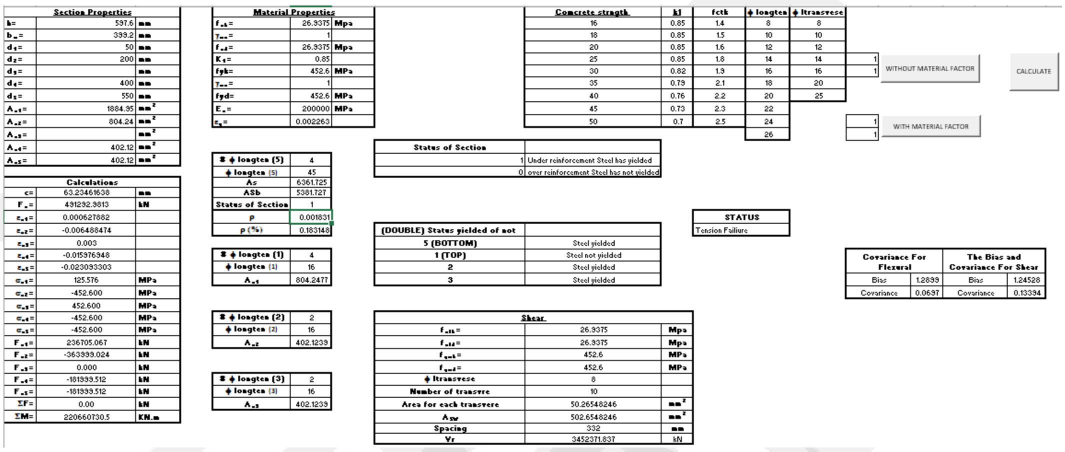


Figure 3-2 – Program for Calculating Flexural and Shear Capacity of Beams

3.3 Failure Modes in Reinforced Concrete Columns

Reinforced concrete column is a compression member and it transfers the loads vertically from the structure to the ground through foundations. The failure of a column in a critical location may cause the connected floors to collapse, resulting in the total collapse of the structure. Concrete members may develop eccentricity under only axial load due to its non-homogeneous characteristics and asymmetrically placed floor loads. Most columns in reinforced concrete buildings are not subjected to only uniaxial loading. In addition, the lateral loads, such as wind and earthquake loads, flexural moment takes place and results in combined axial load and flexure. As a result, the impact of both flexure and axial load on columns shall be considered in design.

Columns can be braced or unbraced, short or slender, depending on structural characteristics and dimensional factors. In this study, the three failure modes (under-, over-reinforced, and balanced) will be considered for combined axial and flexure and shear for rectangular columns.

3.3.1 Combined Flexural and Axial Failure in Columns

Similar to the beams, there are three types of failure modes of columns under combined flexure and axial loading:

- Compression failure occurs when concrete crushes before steel yields. This case is the most common condition in columns that are subjected to axial forces and bending.
- Tension failure occurs when steel yields first then concrete crushes.
- Balanced failure occurs when steel yields and concrete crush at the same time.

To analyze columns under combined axial and flexural loading, equivalent rectangular stress block shall be used, hence the resisting moment capacity (M) and the applied axial load (N) shall be calculated using Figure 3-3 and the following equations [3]:

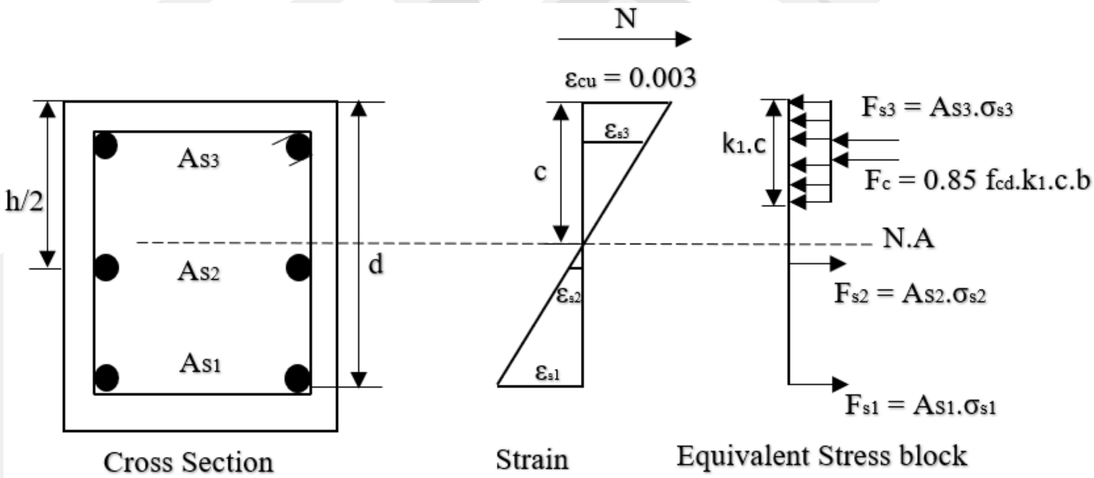


Figure 3-3 – Strain Distribution and Force Equilibrium for Columns

$$N = 0.85\, f_{cd}\, k_1\, c + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \tag{3-12}$$

$$M = 0.85\, f_{cd}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \tag{3-13}$$

$$\sigma_{si} = 0.003\, E_s \left(1 + \frac{d_i - \frac{h}{2}}{c}\right) \leq\ f_{yd} \tag{3-14}$$

In study, A_{s1} is considered as the outermost steel reinforcement layer from the most compressed fiber where n is the number of steel layers.

3.3.2 Shear Failure in Columns

Shear strength of a column can be computed in the same approach as performed in the case of beams. However, for the shear capacity of a column, the values of axial load, (N_d) is considerably larger than that used for the shear capacity of a beam (close to zero, therefore neglected). The shear strength of concrete can be calculated using the following equations [3]:

$$V_c = 0.80 \times 0.65 \times f_{ctd} \times b_w \times d \times \Psi \tag{3-15}$$

$$\Psi = \begin{bmatrix} 1 + 0.3\dfrac{-N_d}{A_c} \;, when \; \dfrac{N_d}{A_c} < 0.5\,MPa \\ 1 + 0.007\dfrac{+N_d}{A_c} \;, when \; \dfrac{N_d}{A_c} > 0.5\,MPa \end{bmatrix} \tag{3-16}$$

$$V_w = \frac{A_{sw}}{s} f_{ywd}\, d \tag{3-17}$$

where Ψ is the factor that revises the shear capacity due to the presence of axial load, N_d is the design axial load (due to factored load) (negative for tension and positive for compression), A_c is the area of the column, concrete's area.

The contribution of concrete to the shear strength of the columns can be calculated using the following equations [3]:

$$V_r = \frac{A_{sw}}{s} f_{ywd}\, d + 0.52 f_{ctd} b_w\, d\, \Psi \tag{3-18}$$

A Microsoft Excel Workbook was produced the automate the capacity calculations for flexural and shear failure of columns. A screenshot of the program is shown in Figure 3-4.

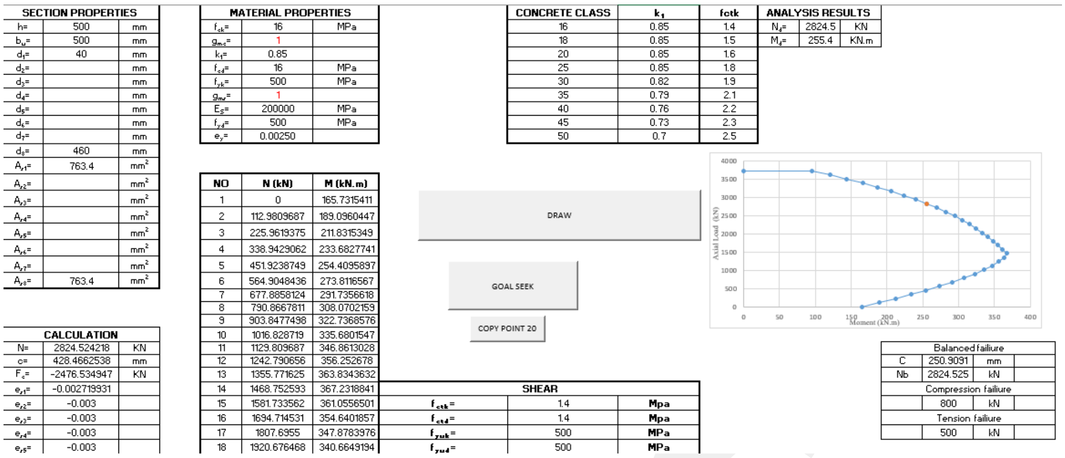


Figure 3-4 – Program for Calculating Flexural and Shear Capacity of Columns

CHAPTER 4

PROCEDURE TO ESTABLISH LIMIT STATE FUNCTION AND DETERMINE RELIABILITY INDEX

4.1 Introduction

The procedure used to establish the limit state function and determine the reliability index is explained in this chapter. Initially the variability (biases and covariances) of the various failure modes based on concrete and steel reinforcement data and dead and live loads were determined. These biases and covariances were converted to normally distributed functions using MSC Method. Later the limit state function is established, and corresponding reliability index was determined.

4.2 Determination of Biases and Covariances of Dead and Live Loads

The biases and covariances of dead and live loads can be determined using observations and experiments. The values proposed by various researchers may also be used.

4.3 Determination of Biases and Covariances of Resistance for Various Failure Modes for Beam and Column Sections

The method to calculate the bias and covariance of failure modes for beam and column sections is explained in steps below:

- First the bias and covariance values (λ_c, Ω_c for concrete and λ_s, Ω_s for steel) obtained from the laboratory test and in-situ strengths for all the concrete class and steel types shall be determined.

- For a selected concrete class and a steel type, the design (nominal) concrete compressive strength and design (nominal) yield strength of steel shall be determined as follows:

$$f_{cd} = \frac{\dot{f}_c}{\gamma_{mc}} \tag{4-1}$$

$$f_{yd} = \frac{\dot{f}_{yd}}{\gamma_{ms}} \tag{4-2}$$

where $\dot{f}_c$ is the selected concrete compressive strength, $\dot{f}_{yd}$ is the selected yield strength of steel, f_{cd} is the design concrete compressive strength and f_{yd} is the design yield strength of steel, γ_{mc} is the material factor for concrete (1.5 based on TS 500 (2000)) [3], γ_{ms} is the material factor for steel (1.15 based on TS 500 (2000)).

- The mean concrete compressive strength (f_{cdm}) and mean yield strength of steel (f_{ydm}) shall be calculated by multiplying the design strengths by bias of each material as follow:

$$f_{cdm} = f_{cd} \times \lambda_c \tag{4-3}$$

$$f_{ydm} = f_{yd} \times \lambda_s \tag{4-4}$$

- Then the capacities of the rectangular section shall be calculated for each failure mode twice, one using the design (nominal) strengths of concrete and steel; and other using the mean strengths of concrete and steel. These capacity calculations are explained in the previous section and summarized in Table 4-1.

Table 4-1 – Capacity Calculations for Various Failure Modes using Design and Mean Strengths of Concrete and Steel

	Using Design (Nominal) Strengths	Using Mean Strengths
Beam Flexure	$M_n = 0.85\, f_{cd}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \left(\frac{h}{2} - d_i\right)$ ($\sigma_{si} = f_{yd}$ *if steel has yielded*)	$M_m = 0.85\, f_{cdm}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \left(\frac{h}{2} - d_i\right)$ ($\sigma_{si} = f_{ydm}$ *if steel has yielded*)
Beam Shear	$V_n = \frac{A_{sw}}{s} f_{ywd}\, d + 0.52\, f_{ctd}\, b_w\, d$	$V_m = \frac{A_{sw}}{s} f_{ywdm}\, d + 0.52\, f_{ctdm}\, b_w\, d$
Column Combined Flexure and Axial	$M_n = 0.85\, f_{cd}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \left(\frac{h}{2} - d_i\right)$ ($\sigma_{si} = f_{yd}$ *if steel has yielded*)	$M_m = 0.85\, f_{cdm}\, k_1\, c\, b \left(\frac{h}{2} - \frac{k_1 c}{2}\right) + \sum_{i=1}^{n} A_{si}\, \sigma_{si} \left(\frac{h}{2} - d_i\right)$ ($\sigma_{si} = f_{ydm}$ *if steel has yielded*)
Column Shear	$V_n = \frac{A_{sw}}{s} f_{ywd}\, d + 0.52\, f_{ctd}\, b_w\, d\Psi$	$V_m = \frac{A_{sw}}{s} f_{ywdm}\, d + 0.52\, f_{ctdm}\, b_w\, d\Psi$

- The bias (λ) for the capacities for all the failure modes shall be calculated by dividing the mean by design (nominal) value of the capacity.

$$\lambda_{Beam\ Flexure} = \frac{M_m}{M_n} \tag{4-5}$$

$$\lambda_{Beam\ Shear} = \frac{V_m}{V_n} \tag{4-6}$$

$$\lambda_{Column\ Flexure\ and\ Axial\ Load} = \frac{M_m}{M_n} \tag{4-7}$$

$$\lambda_{Column\ Shear} = \frac{V_m}{V_n} \tag{4-8}$$

- Then standard deviations (σ) of the mean capacities of the rectangular section are calculated using the FOSM method [13] for each failure mode as shown in Table 4-2.

Table 4-2 – Standard Deviation Calculations for Various Failure Modes using Mean Strengths of Concrete and Steel

Beam Flexure	$\sigma_{M_m} = \sqrt{\left(\frac{\partial M_m}{\partial A_{si}}\right)^2 \sigma_{A_{si}}^2 + \left(\frac{\partial M_m}{\partial f_{ym}}\right)^2 \sigma_{f_{cdm}}^2}$
Beam Shear	$\sigma_{V_m} = \sqrt{\left(\frac{\partial V_m}{\partial A_s}\right)^2 \sigma_{A_s}^2 + \left(\frac{\partial V_m}{\partial f_{ywdm}}\right)^2 \sigma_{f_{ywdm}}^2 + \left(\frac{\partial V_m}{\partial f_{ctdm}}\right)^2 \sigma_{f_{ctd}}^2}$
Column Combined Flexure and Axial	$\sigma_{M_m} = \sqrt{\left(\frac{\partial M_m}{\partial A_{si}}\right)^2 \sigma_{A_{si}}^2 + \left(\frac{\partial M_m}{\partial f_{ym}}\right)^2 \sigma_{f_{cdm}}^2}$
Column Shear	$\sigma_{V_m} = \sqrt{\left(\frac{\partial V_m}{\partial A_s}\right)^2 \sigma_{A_s}^2 + \left(\frac{\partial V_m}{\partial f_{ywdm}}\right)^2 \sigma_{f_{ywdm}}^2 + \left(\frac{\partial V_m}{\partial f_{ctdm}}\right)^2 \sigma_{f_{ctd}}^2}$

- The covariances (Ω) for all the failure modes shall be calculated by standard deviation by the mean capacities as shown below:

$$\Omega_{Beam\ Flexure} = \frac{\sigma_{M_m}}{M_m} \tag{4-9}$$

$$\Omega_{Beam\ Shear} = \frac{\sigma_{V_m}}{V_m} \tag{4-10}$$

$$\Omega_{Column\ Flexure\ and\ Axial\ Load} = \frac{\sigma_{M_m}}{M_m} \tag{4-11}$$

$$\Omega_{Column\ Shear} = \frac{\sigma_{V_m}}{V_m} \tag{4-12}$$

Using this method, the bias and covariances for different failure modes were calculated for one selected concrete compressive strength ($\dot{f}_c$ = 30 MPa) and one selected yield strength of steel ($\dot{f}_{yd}$ = 420 MPa) for the rectangular beam cross-sections for numerous times. Three types of sections were used in the analyses namely singly, doubly, and multi-layer reinforced sections (Figure 4-1). For singly reinforced cross-section, the flexural capacities were calculated using the design (nominal) and mean strengths of materials for one cross-section (300×600 mm) having 20 different cross-sectional areas of tensile reinforcement ranging from 750 to 3800 mm^2. A total of 20 analyses were performed for this case. The shear capacities of singly reinforced cross-sections were calculated using the design (nominal) and mean strengths of materials for two different diameters of transverse reinforcement having 20 different spacings ranging from 75 to 170 mm. A total of 20 analyses were performed for this case. For the other section types of beams, the details of the variables for the analyses are shown in Table 4-3. A screenshot of the produced program used to calculate the bias and covariance of the flexural failure mode is shown in Figure 4-2.

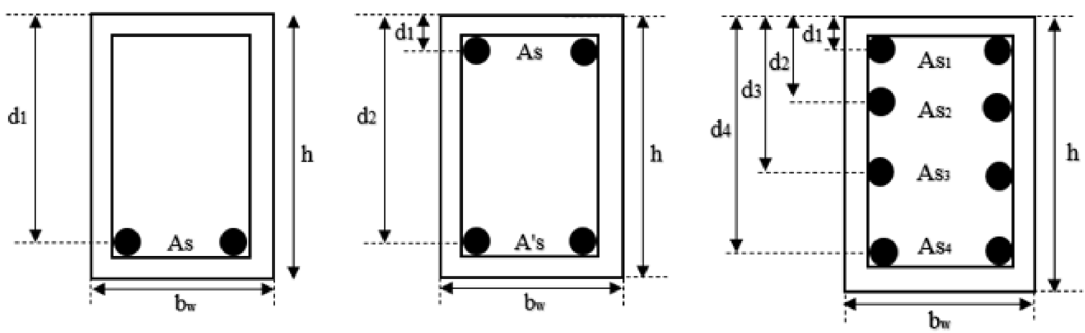


Figure 4-1 – Types of Beam Sections Used in Analyses

Table 4-3 – Analyses Scheme for Beams

Section Type	**Section Dimensions (mm)**	**Longitudinal Reinforcement Area (mm^2)**	**Transverse Reinforcement**
Singly Reinforced	300×600	A_s=750-3800 (20 different values) (Total 20 analyses)	ϕ8 and ϕ10 # of legs=4 s=75-170 mm (20 different values) (Total 20 analyses)
Doubly Reinforced	250×400 350×500	A_s=2463 A_s'=942 and 1556 (Total 4 analyses)	ϕ8 and ϕ20 (7 different values) # of legs=4 s=75-170 mm (10 different values) (Total 36 analyses)
Multi-Layer Reinforced	400×600	A_{s1}=1885 A_{s2}=402 A_{s3}=402 A_{s4}=804 (Total 1 analysis)	ϕ8 and ϕ20 (7 different values) # of legs=4 s=75-170 mm (10 different values) (Total 36 analyses)

	h(mm)	bw(mm)	d(mm)	As (mm^2)	Failiure Status	Tenstion Steel strain	ε_y	Mn (N.mm)	Mr (N.mm)	(Mn/Mr)	σ	σ/Mr
1	600	300	550	3800	Tension Failiure	-0.00446247	0.001825	814271177.6	637113464.1	1.278	36680176.089	0.045
2	600	300	550	3648	Tension Failiure	-0.00477341	0.001825	788513789.2	616457208.5	1.279	36071978.538	0.046
3	600	300	550	3496	Tension Failiure	-0.00511138	0.001825	762188612.7	595398596	1.280	35421816.164	0.046
4	600	300	550	3344	Tension Failiure	-0.00548008	0.001825	735295647.9	573937626.6	1.281	34730592.818	0.047
5	600	300	550	3192	Tension Failiure	-0.00588390	0.001825	707834894.9	552074300.2	1.282	33999365.296	0.048
6	600	300	550	3040	Tension Failiure	-0.00632809	0.001825	679806353.6	529808617	1.283	33229372.103	0.049
7	600	300	550	2888	Tension Failiure	-0.00681904	0.001825	651210024.2	507140576.8	1.284	32422068.967	0.050
8	600	300	550	2736	Tension Failiure	-0.00736455	0.001825	622045906.4	484070179.8	1.285	31579172.797	0.051
9	600	300	550	2584	Tension Failiure	-0.00797422	0.001825	592314000.5	460597425.8	1.286	30702716.333	0.052
10	600	300	550	2432	Tension Failiure	-0.00866011	0.001825	562014306.3	436722314.9	1.287	29795116.306	0.053
11	600	300	550	2280	Tension Failiure	-0.00943745	0.001825	531146823.9	412444847.1	1.288	28859258.674	0.054
12	600	300	550	2128	Tension Failiure	-0.01032584	0.001825	499711553.3	387765022.3	1.289	27898605.430	0.056
13	600	300	550	1976	Tension Failiure	-0.01135091	0.001825	467708494.4	362682840.7	1.290	26917328.488	0.058
14	600	300	550	1824	Tension Failiure	-0.01254682	0.001825	435137647.3	337198302.1	1.290	25920477.212	0.060
15	600	300	550	1672	Tension Failiure	-0.01396017	0.001825	401999012	311311406.6	1.291	24914187.014	0.062
16	600	300	550	1520	Tension Failiure	-0.01565618	0.001825	368292588.4	285022154.2	1.292	23905936.489	0.065
17	600	300	550	1368	Tension Failiure	-0.01772909	0.001825	334018376.6	258330544.9	1.293	22904858.890	0.069
18	600	300	550	1216	Tension Failiure	-0.02032023	0.001825	299176376.6	231236578.7	1.294	21922108.333	0.073
19	600	300	550	1064	Tension Failiure	-0.02365169	0.001825	263766588.3	203740255.6	1.295	20971268.952	0.080
20	600	300	550	750	Tension Failiure	-0.03480986	0.001825	188819440.8	145664522.1	1.296	19182258.291	0.102

Figure 4-2 – Screenshot of Program to Calculate Bias and Covariance of Flexural Failure Mode

The same procedure was performed to determine the bias and covariances for different failure modes for columns using the selected concrete compressive strength ($\dot{f_c}$ = 30 MPa) and selected yield strength of steel ($\dot{f}_{yd}$ = 420 MPa) for the rectangular columns cross-sections for numerous times. Two types of sections were used in the analyses namely doubly and multi-layer reinforced sections (Figure 4-3). For doubly reinforced cross-section, the combined shear and flexural capacities were calculated using the design (nominal) and mean strengths of materials for two cross-sections (300×400 and 300×500 mm) having cross-sectional areas of tensile reinforcement of 763.4 mm^2 both at the top and bottom of the section. The shear capacities of doubly reinforced cross-sections were calculated using the design (nominal) and mean strengths of materials for two different diameters of transverse reinforcement having 36 different spacings ranging from 75 to 170 mm. A total of 36 analyses were performed for this case. For the other section type of columns, the details of the variables for the analyses are shown in Table 4-4.

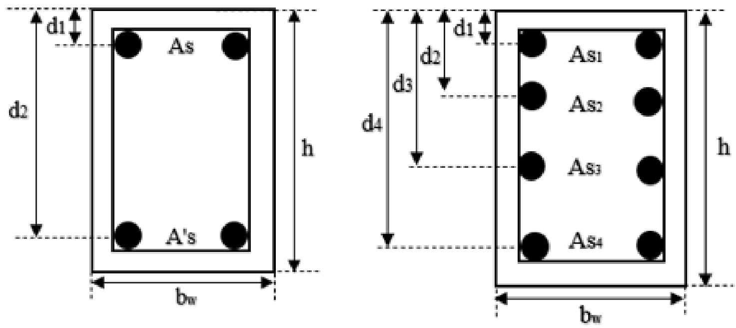


Figure 4-3 – Types of Column Sections used in Analyses

Table 4-4 – Analyses Scheme for Beams

Section Type	Section Dimensions (mm)	Longitudinal Reinforcement Area (mm^2)	Transverse Reinforcement
Doubly Reinforced	300×400 300×500	A_s=763.4 A_s'=763.4 (Total 2 analyses)	ϕ8 and ϕ20 (7 different values) # of legs=4 s=75-170 mm (10 different values) (Total 36 analyses)
Multi-Layer Reinforced	300×400 300×500	A_{s1}=1593, A_{s2}=763, A_{s3}=763, A_{s4}=1593 A_{s1}=1571, A_{s2}=628, A_{s3}=628, A_{s4}=1571 (Total 4 analyses)	ϕ8 and ϕ25 (7 different values) # of legs=4 s=100-332 mm (10 different values) (Total 36 analyses)

At the end of these analyses, the average bias and covariances values were calculated for each failure mode. The userforms used to perform these actions and provide the bias and covariances for beams and columns for each failure mode are shown in Figure 4-4 and Figure 4-5.

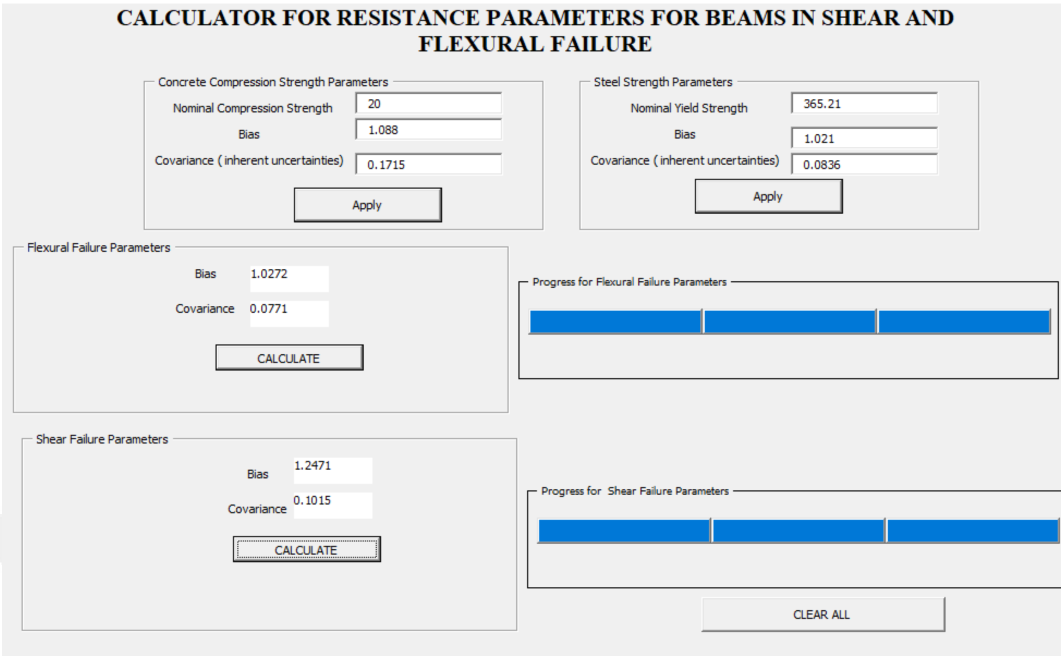


Figure 4-4 – Userform to Calculate Bias and Covariance of Beams for Each Failure Mode when Material Data is Provided

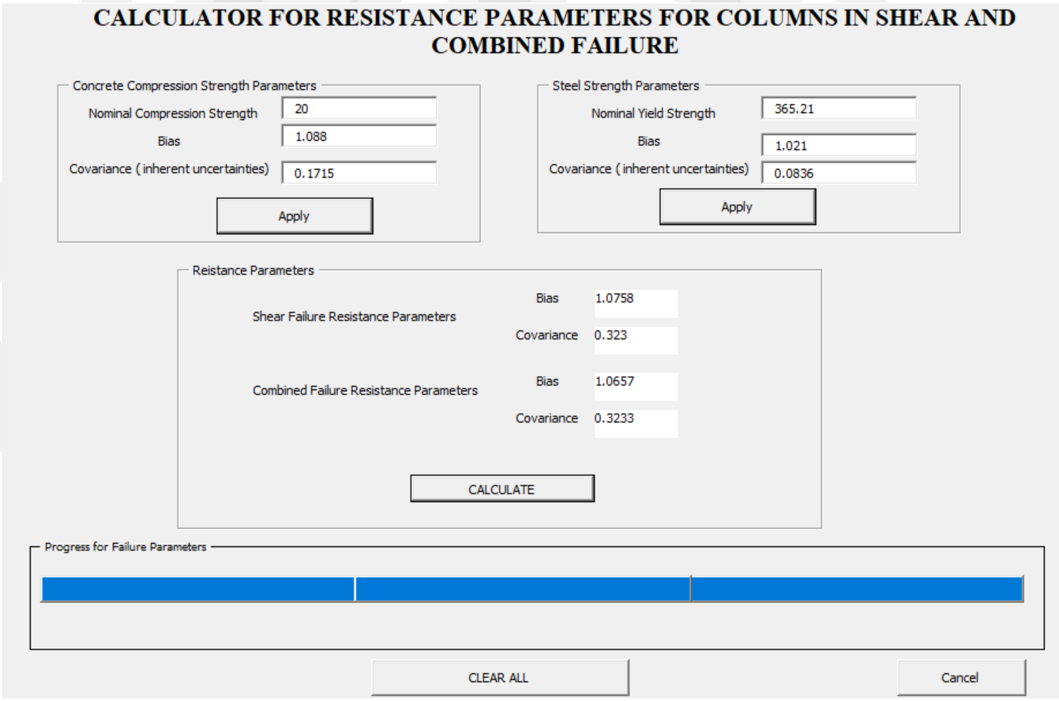


Figure 4-5 – Userform to Calculate Bias and Covariance of Columns for Each Failure Mode when Material Data is Provided

For this study, the bias and covariance for the beam in flexural failure were 1.0727 and 0.0771, respectively. The bias and covariance for the beam in shear failure were 1.2471 and 0.1015, respectively. The bias and covariance for columns in combined axial load and flexure failure were 1.0758 and 0.323, respectively. The bias and covariance for columns in shear failure were 1.0657 and 0.323, respectively.

Using similar procedures, various researchers suggested values for bias and covariance of failure modes of beams and columns as shown in Table 4-5 and Table 4-6.

Table 4-5 – Bias and Covariance for Failure Modes of Beams in Different Studies

Failure Mode	**Bias (λ)**	**Covariance (Ω)**	**References**
Flexural	1.19	0.16	Kömürcü and Yücemen (1996)
	1.02	0.06	Nowak and Szerszen (2003a)
	1.24	0.13	Fırat (2007)
	1.24	0.13	Mahmud (2014)
Shear	1.09	0.115	Ellingwood et al. (1980)
	1.23	0.11	Nowak and Szerszen (2003a)
	1.24	0.17	Fırat (2007)
	1.24	0.17	Mahmud (2014)

Table 4-6 – Bias and Covariance for Failure Modes of Columns in Different Studies

Failure Mode	**Bias (λ)**	**Covariance (Ω)**	**References**
Combined Flexure and Axial Load	1.1	0.12	Ellingwood et al. (1980)
	1.24	0.14	Fırat (2007)
	1.24	0.1442	Mahmud (2014)
Shear	1.09	0.115	Ellingwood et al. (1980)
	1.23	0.1	Nowak and Szerszen (2003a)
	1.24	0.17	Fırat (2007)
	1.24	0.17	Mahmud (2014)

4.4 Conversion of Biases and Covariances into Normal Distribution Functions

After having the biases and covariances for the resistance of the failure modes and loads, these biases and covariances were used to determine their normal distribution when MSC is applied for 20 million random numbers between 0 and 1.0.

MCS Method was applied to resistance, dead, and live loads. As an example, typical application of the MSC Method on the dead load was explained in this section. Similar procedure was applied to live loads and the resistance of each failure mode. Initially, the bias (λ) and covariance (Ω) of the dead load were converted to mean (μ) and standard deviation (σ) values of the dead load using the following equations:

$$\mu = L_n \times \lambda \tag{4-13}$$

$$\sigma = L_n \times \lambda \times \Omega \tag{4-14}$$

where L_n is the nominal (unfactored) dead load value.

Using the function "RAND()" available in Microsoft Excel Software, twenty million random numbers were produced between 0 and 1. Later these random values were converted into normal distribution function using the function "NORM.DIST()" available also in Microsoft Excel Software and mean (μ) and standard deviation (σ) values of the dead load. The screenshot of the produced program used to perform these steps for resistance, dead, and live loads is shown in Figure 4-6.

Live					
L	20	kN	μ_L	20	kN
λ_L	1		σ_L	5.4	kN
V_L	0.27				

No	Random	Nor.L
1	0.012505007	7.90
2	0.290437964	17.02
3	0.746434961	23.58
4	0.282236146	16.89
5	0.337838933	17.74
6	0.479245433	19.72
7	0.887726508	26.56
8	0.21082794	15.66
9	0.226357378	15.95
10	0.736356622	23.41
11	0.916634963	27.47
12	0.86460783	25.95
13	0.749791114	23.64
14	0.50019428	20.00
15	0.852187486	25.65
16	0.325223356	17.55
17	0.906111266	27.11
18	0.367192651	18.17
19	0.739252276	23.46
20	0.203033284	15.51
21	0.953197858	29.05
22	0.429104286	19.04
23	0.610687131	21.52
24	0.62589901	21.73
25	0.192972847	15.32
26	0.441403974	19.20
27	0.456973622	19.42
28	0.161206968	14.66
29	0.354958271	17.99
30	0.932572514	28.07
31	0.716072514	23.08
32	0.953189354	29.05
33	0.924681836	27.76

Dead					
D	120	kN	μ_D	126	kN
λ_D	1.05		σ_D	12.6	kN
V_D	0.1				

No	Random	Nor.D
1	0.416880809	123.36
2	0.947576513	146.43
3	0.724729417	133.52
4	0.675039344	131.72
5	0.773895944	135.47
6	0.36192819	121.55
7	0.294096665	119.18
8	0.475339715	125.22
9	0.862636172	139.76
10	0.638373144	130.46
11	0.340466226	120.82
12	0.020109291	100.15
13	0.03761569	103.58
14	0.810699744	137.09
15	0.838840227	138.47
16	0.415300994	123.30
17	0.257369972	117.79
18	0.633209184	130.29
19	0.600610213	129.21
20	0.019638125	100.03
21	0.904268893	142.46
22	0.674025922	131.68
23	0.974706254	150.63
24	0.890427134	141.48
25	0.175704574	114.26
26	0.003832106	92.40
27	0.283639367	118.79
28	0.596504956	129.08
29	0.109641976	110.52
30	0.401194348	122.85
31	0.370632335	121.84
32	0.211612492	115.91
33	0.6850789	132.07

Resistance					
R	350	kN	μ_R	357	kN
λ_R	1.02		σ_R	107.1	kN
V_R	0.3				

No	Random	Nor.R
1	0.523622903	363.35
2	0.27332406	292.44
3	0.06540668	195.18
4	0.203068029	268.03
5	0.508067017	359.17
6	0.721725336	419.97
7	0.495815592	355.88
8	0.617052732	388.89
9	0.856025416	470.81
10	0.950272838	533.45
11	0.949415346	532.56
12	0.994754054	631.09
13	0.067019855	196.53
14	0.910658504	501.03
15	0.191095389	263.41
16	0.221510493	274.84
17	0.005982314	87.84
18	0.891754501	489.37
19	0.775214091	437.98
20	0.735328082	424.37
21	0.686576846	409.07
22	0.815209753	453.10
23	0.059729425	190.24
24	0.122076716	232.26
25	0.611344809	387.29
26	0.265518767	289.91
27	0.113420433	227.57
28	0.898915077	493.59
29	0.176456297	257.51
30	0.050118772	180.96
31	0.735123575	424.30
32	0.074339781	202.32
33	0.190773568	263.28

Figure 4-6 – Screenshot of Program to Convert Biases and Covariances of Resistance, Dead, and Live Loads to Normal Distribution Functions

4.5 Establishing Limit State Function and Calculation of Reliability Index

The strength limit state requires that the resistance of the structure (*R*) shall be greater than or equal to the load effects (*F*) as shown in the following equation:

$$R \geq F \tag{4-15}$$

The limit state function is established by moving the right-hand side of this expression to the left-hand side as follows:

$$R - F \geq 0 \tag{4-16}$$

The load effects evaluated in the scope of this study were only the dead and live loads. Therefore, the expression can be written as follows where D is the dead loads and L is the live loads:

$$G = R - D - L \geq 0 \tag{4-17}$$

This expression is named as the limit state function (G) and is calculated for each of the 20 million values of resistance, dead and live loads. This procedure can be explained in detail using Figure 4-6. The normal distribution values of dead and live load (Nor. D and Nor. L) were subtracted from the normal distribution value of resistance (Nor. R) for each randomly produced values. As an example, for the first randomly produced value (No 1), the value of Nor. D (123.36 kN) and Nor. L (7.90) is subtracted from Nor. R (363.35 kN) and the value of the limit state function is determined as 232.09 kN. This procedure is repeated for 20 million times. For each value, the probability of occurrence in 20 million values were calculated using the function "NORMSDIST()" readily available in Microsoft Excel Software. The inverse of the probability of occurrence (P_f) is the reliability index (β). The reliability index (β) can also be calculated using the function "NORMSINV()" readily available in Microsoft Excel Software. The resulting values of all these limit state functions were sorted from smallest to largest. The limit state function value versus reliability index graph were drawn for 20 million values as shown in Figure 4-7. Finally, the intersection of the 20 million values and the y-axis is determined as the value of the reliability index of this limit state function. For the example shown in the figure, the reliability index is equal to 3.5.

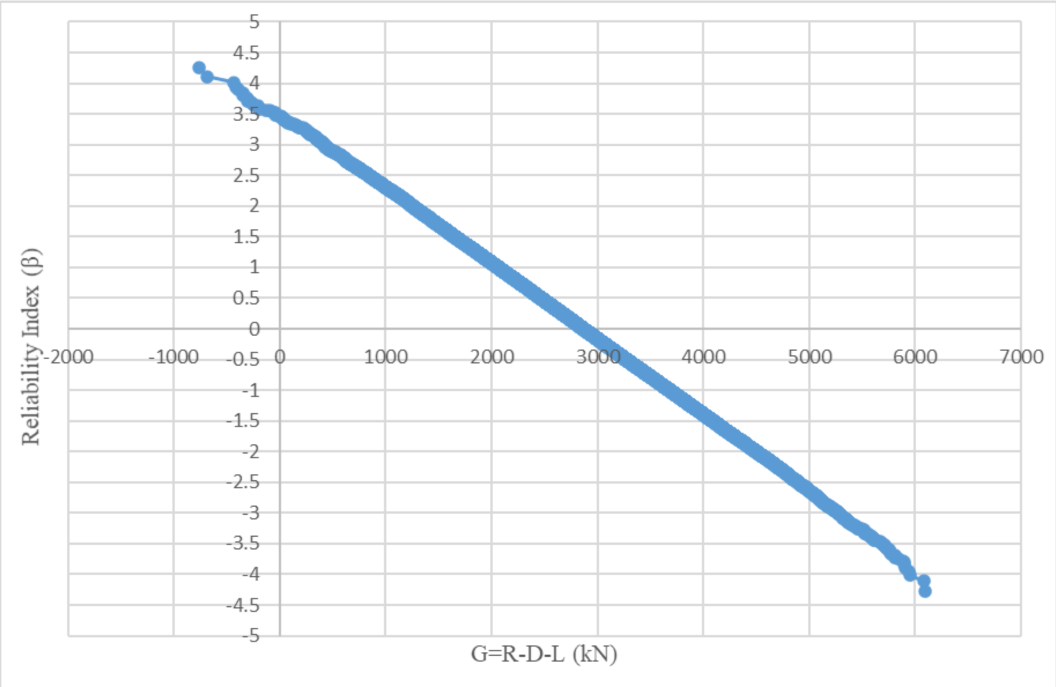


Figure 4-7 – The Limit State Function and Reliability Index

CHAPTER 5

ANALYTICAL STUDY

5.1 Introduction

The evaluation and results used to calculate the custom load and resistance factors for the design of reinforced concrete structures is explained in this chapter.

5.2 Sensitivity Analyses of Reliability Index based on Load and Resistance Parameters

It is essential to analyze how the parameters for resistance, dead, and live loads affect the reliability index. For this reason, analyses were performed using MCS for various ratios of live to dead loads (L/D) ranging from 0.1 to 1.0 and ratios of resistance to dead load (R/D) ranging from 1.6 to 2.5 as shown in Table 5-1. The sensitivity analyses were performed for 6 load and resistance load combinations as shown in Table 5-2. These load and resistance combinations were the combinations available in TS 500 (2000) and ACI 318 (2019) [3, 2].

Table 5-1 – Ratios for L/D and R/D

r = L/D	R/D
0.1	1.6
0.2	1.7
0.3	1.8
0.4	1.9
0.5	2
0.6	2.1
0.7	2.2
0.8	2.3
0.9	2.4
1	2.5

Table 5-2 – Combinations Used in Sensitivity Analyses

Dead Load Factor	Live Load Factor	Resistance Factor
0.85	1.4	1.6
0.9	1.2	1.6
0.65	1.2	1.6
0.7	1.4	1.6
1	1.2	1.6
1	1.4	1.6

5.2.1 Effect of Dead Load Parameters

Analyses for dead load parameters (bias and covariance) for beams and columns were performed on 20 million random values for various L/D ratios ranging from 0.1 to 1. The effect of bias (λ) of the dead loads was evaluated for values between 1.01 and 1.3 with increments of 0.01 while the covariance was kept constant as 0.09. The bias and covariance of live load were constant and equal to 1.0 and 0.27, respectively. The bias and covariance for the resistance were also constant and equal to 1.24 and 0.13, respectively. The results of this analysis are shown in Figure 5-1. The figure indicated that as the bias for the dead load increased, the reliability decreased for all the 5 combinations.

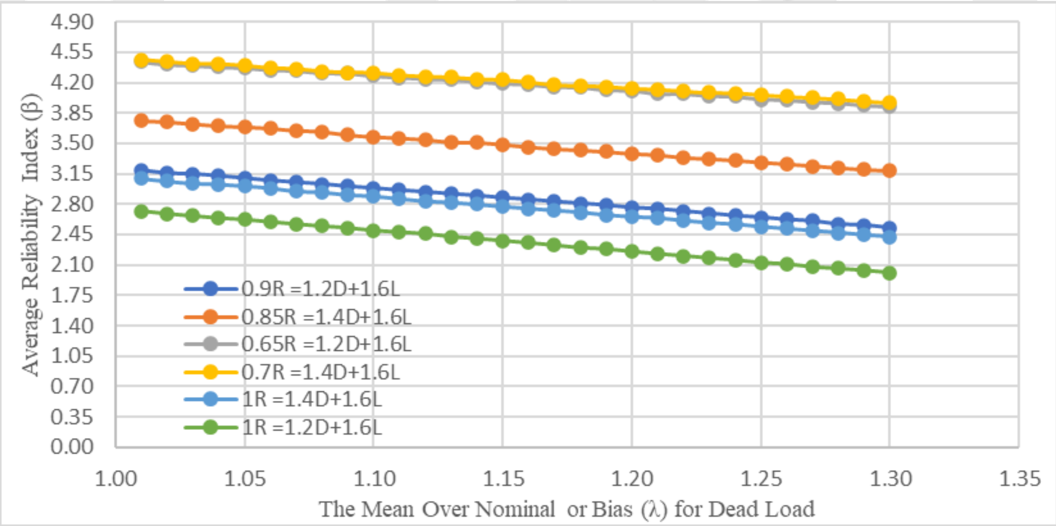


Figure 5-1 – Relationship of Bias (λ) for Dead Load and Reliability Index

Similar analysis was performed to evaluate the effect of covariance (Ω) of the dead loads varying between 0.01 and 0.30 with increments of 0.01 while the bias was kept constant as 1.05. All other variables were similar to the analyses above. The results of this analyses are shown in Figure 5-2. This figure indicated that as the covariance of the dead load increased, the reliability index decreased gradually for all the 6 combinations. Hence as the bias or covariance increased, the reliability index gradually decreased.

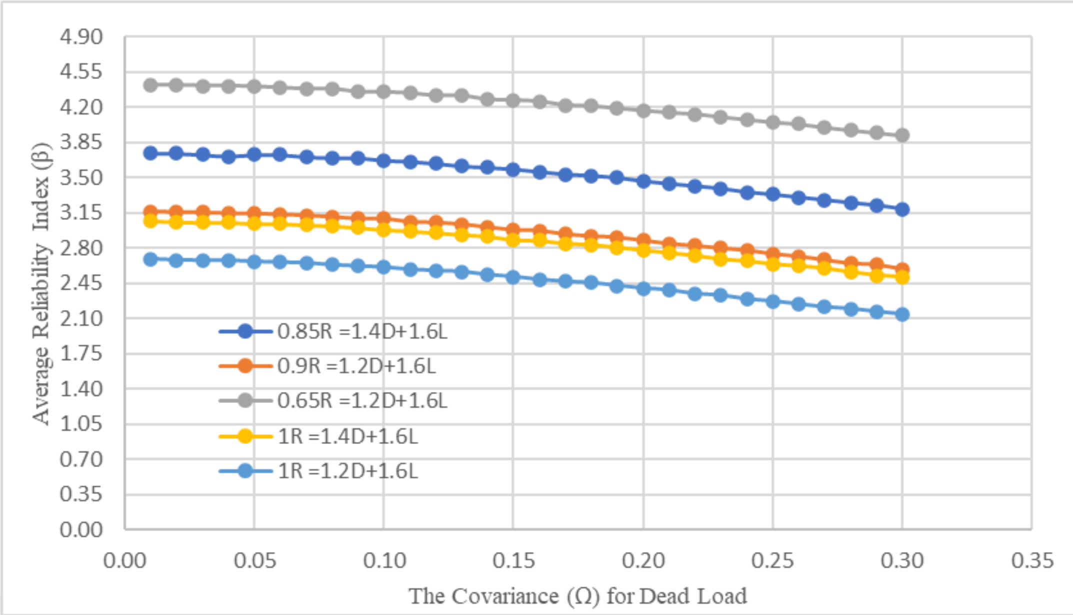


Figure 5-2 – Relation of Covariance (Ω) for Dead Load and Reliability Index

5.2.2 Effect of Live Load Parameters

Analyses for live load parameters (bias and covariance) for beams and columns were performed on 20 million random values for various L/D ratios ranging from 0.1 to 1. The effect of bias (λ) of the live loads was evaluated for values between 1.01 and 1.3 with increments of 0.01 while the covariance was kept constant as 0.09. The bias and covariance of dead load were constant and equal to 1.05 and 0.1, respectively. The bias and covariance for the resistance were also constant and equal to 1.24 and 0.13, respectively. The results of this analysis are shown in Figure 5-3. The figure indicated that as the bias for the live load increased, the reliability index decreased for all the 6 combinations. However, the effects of bias and covariance for dead load on the reliability index was more critical since it produced greater decrease in the reliability index compared to that of live loads.

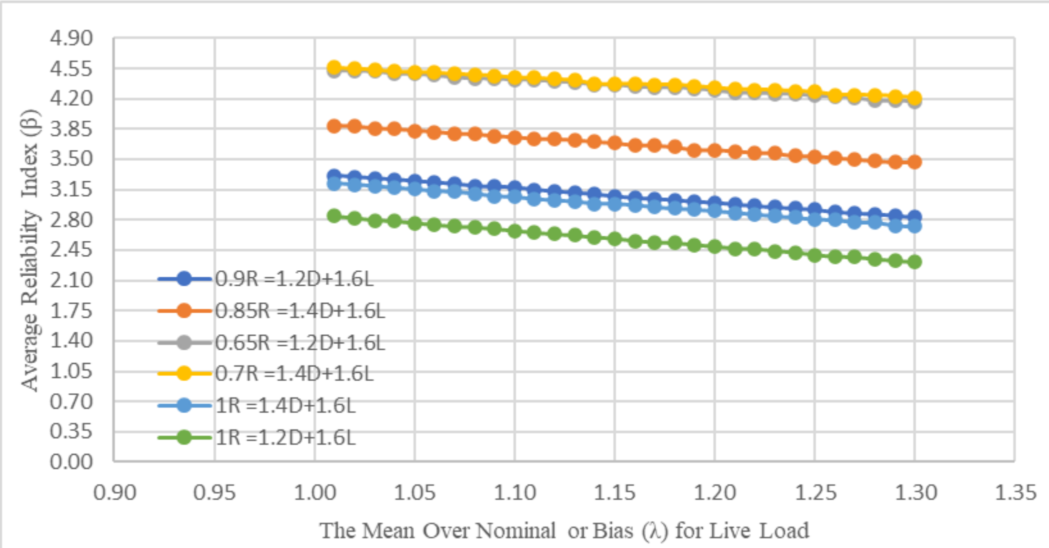


Figure 5-3 – Relationship of Bias (λ) for Live Load and Reliability Index

Similar analysis was performed to evaluate the effect of covariance (Ω) of the live loads varying between 0.01 and 0.30 with increments of 0.01 while the bias was kept constant as 1.16. All other variables were similar to the analyses above. The results of this analyses are shown in Figure 5-4. This figure indicated that as the covariance of the live load increased, the reliability index decreased gradually for all the 6 combinations. Hence, when the bias and covariance increased, the reliability index gradually decreased.

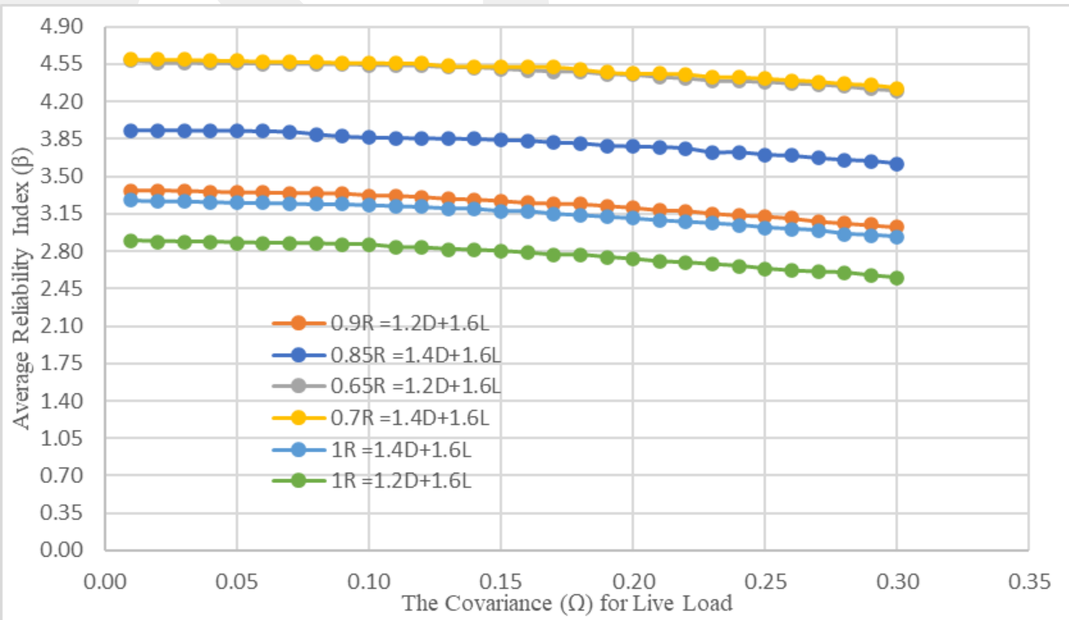


Figure 5-4 – Relationship of Covariance (Ω) for Live Load and Reliability Index

5.2.3 Effect of Resistance Parameters

Analyses for resistance parameters (bias and covariance) for beams and columns were performed on 20 million random values for various L/D ratios ranging from 0.1 to 1. The effect of bias (λ) of the resistance was evaluated for values between 1.01 and 1.3 with increments of 0.01 while the covariance was kept constant as 0.09. The bias and covariance of dead load were constant and equal to 1.05 and 0.1, respectively. The bias and covariance for live load were also constant and equal to 1.0 and 0.27, respectively. The results of this analysis are shown in Figure 5-5. The figure indicated that as the bias for the resistance increased, the reliability index increased for all the 6 combinations.

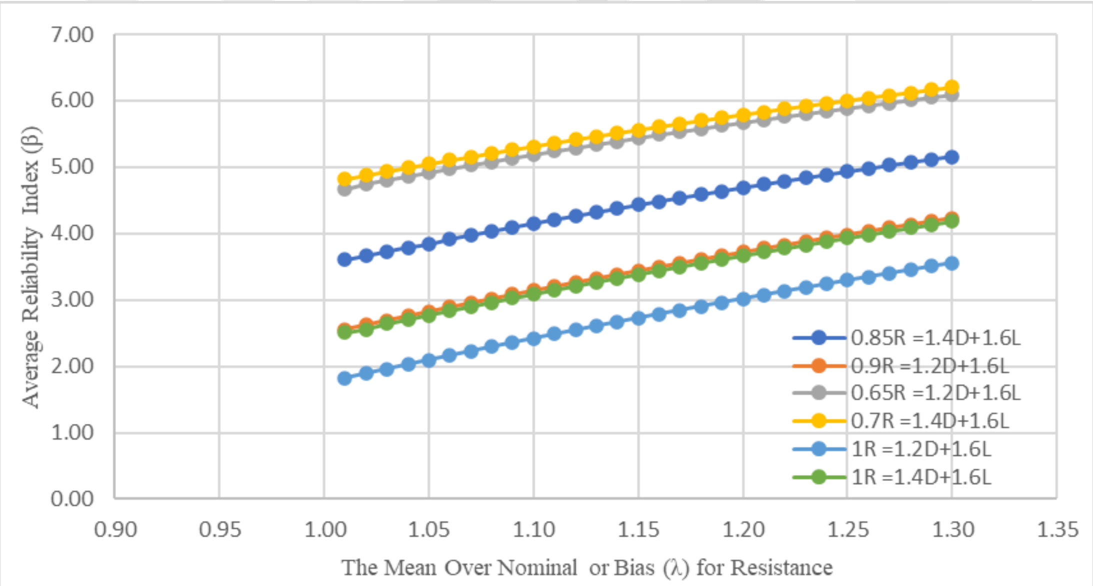


Figure 5-5 – Relation of Bias (λ) for Resistance and Reliability Index

Similar analysis was performed to evaluate the effect of covariance (Ω) of the resistance varying between 0.01 and 0.30 with increments of 0.01 while the bias was kept constant as 1.16. All other variables were similar to the analyses above. The results of this analyses are shown in Figure 5-6. This figure indicated that as the covariance of the resistance increased, the reliability index decreased for all the 6 combinations. Hence, as the bias increased, the reliability index also increased. However, as the covariance increased, the reliability index decreased.

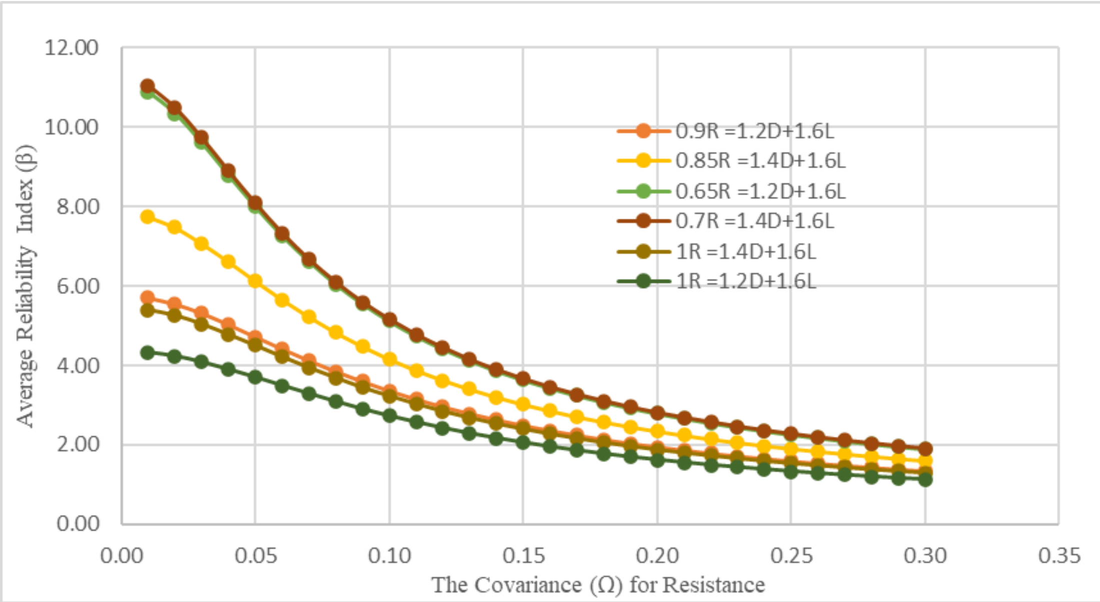


Figure 5-6 – Relationship of Covariance (Ω) for Resistance and Reliability Index

5.3 Sensitivity Analyses of Reliability Index based on Load and Resistance Factors

The investigation of sensitivity analysis of reliability index based on load and resistance factors were performed for the ratio of L/D which is denoted as "r", ranging from 0.1 to 1.00. MCS Method was used for 20 million samples to determine the effect of load and resistance factors on the reliability index.

The variations of live and dead load factors with respect to the reliability index are shown in Figure 5-7. This figure indicated that when the live and dead factors increased, the reliability index also increased. The live and dead load factors had significant effects on reliability index due to the major increase of reliability index value. However, the maximum reliability index for dead load factors was greater than that of live load factors.

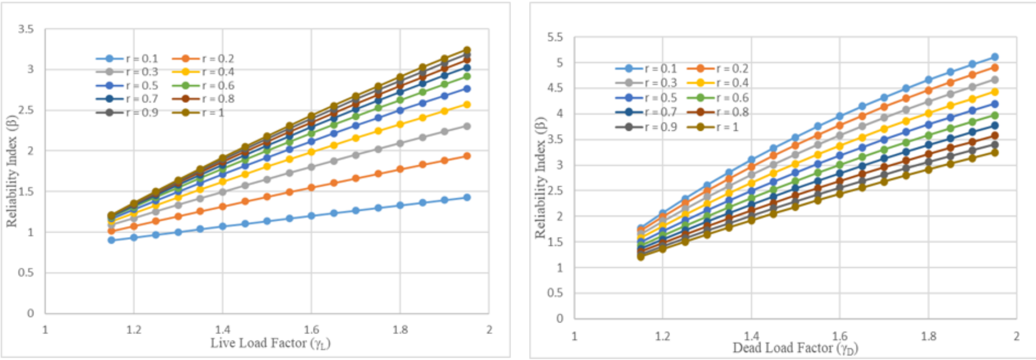


Figure 5-7 – Relationship of Dead and Live Loads wrt. Reliability Index

The variation of resistance factor with respect to the reliability index is shown in Figure 5-8. The ratio of live to dead load was ranging from 0.1 to 1. This figure indicated that. as the resistance factor increased, the reliability index decreased. Hence, it can be concluded that higher resistance factor would lead to low reliability index.

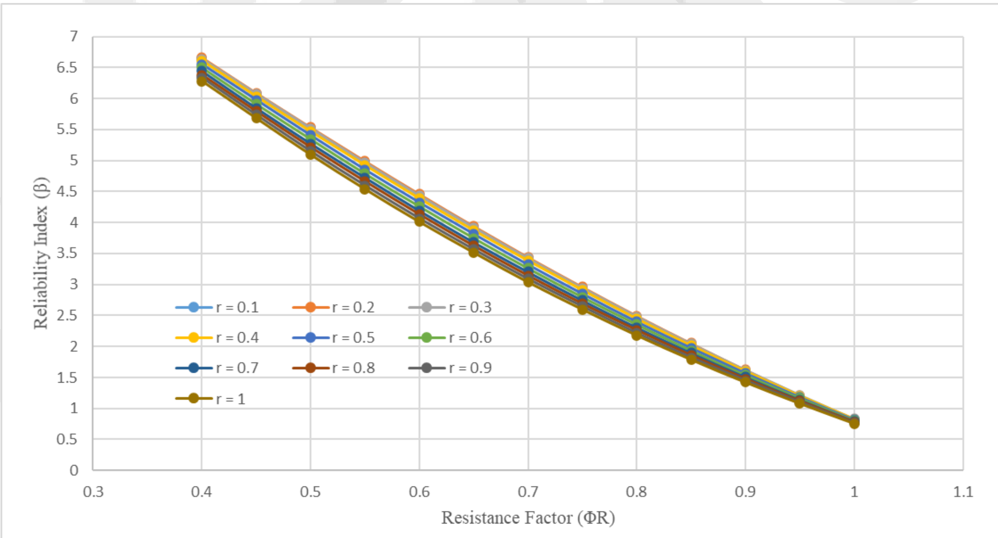


Figure 5-8 – Relation of Resistance Factor wrt. Reliability Index

5.4 Comparison of Reliability Indexes of TS 500 (2000) [3] and ACI 318 (2019) [2]

Turkish Reinforced Concrete Design Code (TS 500, 2000) [3] satisfies the safety criteria using load factors which increases the load effects and material factors which reduces the strength of materials in calculation of resistance. The American Concrete Design Code (ACI 318, 2019) [2] uses load factors which increases the load effect;

however, the resistance is not reduced using the material factors in this code. Instead, the resistance is calculated using the characteristic strengths of materials (no reduction in material strengths), however finally the resistance is reduced by a factor for each failure mode. The resistance factors for each failure mode of beams and columns specified by ACI 318 (2019) [2] is shown in Table 5-3. Using these resistance factors and the procedures explained in Chapter 4, the reliability indexes were calculated for each resistance factor specified by ACI 318 (2019) [2] as shown in this same table.

Table 5-3 – Resistance Factors and Target Reliability Index Based on ACI 318 (2019) [2]

	Failure	Resistance Factor (ϕ)	Minimum Reliability Index (β)
Beams	Compression	0.65	2.832
	Tension	0.9	2.004
	Only Shear	0.75	2.497
Columns	Compression	0.65	2.832
	Tension	0.9	2.004
	Only Shear	0.75	2.497

Even though TS 500 (2000) [3] had a different approach from the ACI 318 (2019) [2], for comparison purposes, the material factors approach in TS 500 (2000) [3] was converted to the resistance factors approach in ACI 318 (2019) [2]. For this purpose, the resistance of each failure mode (flexural, shear, and combined flexure and axial) was calculated twice, once using the reduced material strengths and once using the unreduced material strengths. Then, to evaluate the resistance factor, the resistance calculated using reduced material strengths was divided by the resistance calculated using unreduced material strengths. This procedure was performed for each failure mode using MCS Method on 20 million beam and column cross-sections having various dimensions and reinforcement areas. The diameters of reinforcement used in the analyses were ranging from 8 to 26 mm for both the longitudinal and transvers bars. The concrete type and steel type was ranging from C16 to C50, S420 to S500, respectively. The geometric properties of beams and columns used in these analyses are shown in Table 5-4 to Table 5-7. A screenshot of the produced program used to calculate resistance factors for TS 500 (2000) [3] combined flexure and axial failure of columns is shown in Figure 5-9.

Table 5-4 – Geometric Properties of Singly Reinforced Concrete Beams

h (mm)	b_w (mm)	d (mm)	A_s (mm^2)
400	250	350	400-1901
600	300	550	750-3800

Table 5-5 – Geometric Properties of Doubly Reinforced Concrete Beams

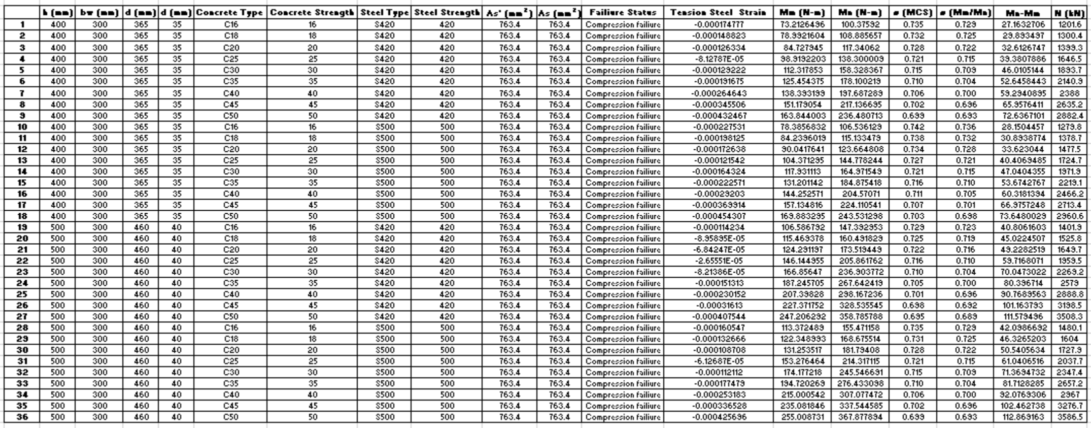
h (mm)	bw (mm)	d_1 (mm)	d_2 (mm)	A_{s1} (mm^2)	A_{s2} (mm^2)
400	250	65	350	1257	2463
600	300	65	450	942	2463

Table 5-6 – Geometric Properties of Multi-Layer Reinforced Concrete Beams

h (mm)	b_w (mm)	d_1 (mm)	d_2 (mm)	d_3 (mm)	d_4 (mm)	As_1 (mm^2)	As_2 (mm^2)	As_3 (mm^2)	As_4 (mm^2)
600	400	50	200	400	500	1885	804	402	402

Table 5-7 – Geometric Properties of Multi-Layer Reinforced Concrete Columns

h (mm)	b_w (mm)	d_1 (mm)	d_2 (mm)	A_{s1} (mm^2)	A_{s2} (mm^2)
400	300	35	365	763	763
500	300	40	460	763	763
400	400	35	365	763	763
500	500	40	460	763	763



	h (mm)	bw (mm)	d (mm)	d (mm)	Concrete Type	Concrete Strength	Steel Type	Steel Strength	As' (mm^2)	As (mm^2)	Failure Status	Tension Steel Strain	Mm (N-m)	Mn (N-m)	φ (MCS)	φ (Mm/Mn)	Mn-Mm	N (kN)
1	400	300	365	35	C16	16	S420	420	763.4	763.4	Compression failure	-0.000174777	73.2126496	100.37592	0.735	0.729	27.1632706	1201.6
2	400	300	365	35	C18	18	S420	420	763.4	763.4	Compression failure	-0.000148823	78.9921604	108.885657	0.732	0.725	29.893497	1300.4
3	400	300	365	35	C20	20	S420	420	763.4	763.4	Compression failure	-0.000126334	84.727945	117.34062	0.728	0.722	32.6126747	1399.3
4	400	300	365	35	C25	25	S420	420	763.4	763.4	Compression failure	-8.12787E-05	98.9192203	138.300009	0.721	0.715	39.3807886	1646.5
5	400	300	365	35	C30	30	S420	420	763.4	763.4	Compression failure	-0.000129222	112.317853	158.328367	0.715	0.709	46.0105144	1893.7
6	400	300	365	35	C35	35	S420	420	763.4	763.4	Compression failure	-0.000191675	125.454375	178.100219	0.710	0.704	52.6458443	2140.9
7	400	300	365	35	C40	40	S420	420	763.4	763.4	Compression failure	-0.000264643	138.393199	197.687289	0.706	0.700	59.2940895	2388
8	400	300	365	35	C45	45	S420	420	763.4	763.4	Compression failure	-0.000345506	151.179054	217.136695	0.702	0.696	65.9576411	2635.2
9	400	300	365	35	C50	50	S420	420	763.4	763.4	Compression failure	-0.000432467	163.844003	236.480713	0.699	0.693	72.6367101	2882.4
10	400	300	365	35	C16	16	S500	500	763.4	763.4	Compression failure	-0.000227531	78.3856832	106.536129	0.742	0.736	28.1504457	1279.8
11	400	300	365	35	C18	18	S500	500	763.4	763.4	Compression failure	-0.000198125	84.2396019	115.133479	0.738	0.732	30.8938774	1378.7
12	400	300	365	35	C20	20	S500	500	763.4	763.4	Compression failure	-0.000172638	90.0417641	123.664808	0.734	0.728	33.623044	1477.5
13	400	300	365	35	C25	25	S500	500	763.4	763.4	Compression failure	-0.000121542	104.371295	144.778244	0.727	0.721	40.4069485	1724.7
14	400	300	365	35	C30	30	S500	500	763.4	763.4	Compression failure	-0.000164324	117.931113	164.971549	0.721	0.715	47.0404355	1971.9
15	400	300	365	35	C35	35	S500	500	763.4	763.4	Compression failure	-0.000222571	131.201142	184.875418	0.716	0.710	53.6742767	2219.1
16	400	300	365	35	C40	40	S500	500	763.4	763.4	Compression failure	-0.00029203	144.252571	204.57071	0.711	0.705	60.3181394	2466.2
17	400	300	365	35	C45	45	S500	500	763.4	763.4	Compression failure	-0.000369914	157.134816	224.110541	0.707	0.701	66.9757248	2713.4
18	400	300	365	35	C50	50	S500	500	763.4	763.4	Compression failure	-0.000454307	169.883295	243.531298	0.703	0.698	73.6480029	2960.6
19	500	300	460	40	C16	16	S420	420	763.4	763.4	Compression failure	-0.000114234	106.586792	147.392953	0.729	0.723	40.8061603	1401.9
20	500	300	460	40	C18	18	S420	420	763.4	763.4	Compression failure	-8.95895E-05	115.469378	160.491829	0.725	0.719	45.0224507	1525.8
21	500	300	460	40	C20	20	S420	420	763.4	763.4	Compression failure	-6.84247E-05	124.291197	173.519449	0.722	0.716	49.2282519	1649.7
22	500	300	460	40	C25	25	S420	420	763.4	763.4	Compression failure	-2.65551E-05	146.144955	205.861762	0.716	0.710	59.7168071	1959.5
23	500	300	460	40	C30	30	S420	420	763.4	763.4	Compression failure	-8.21386E-05	166.85647	236.903772	0.710	0.704	70.0473022	2269.2
24	500	300	460	40	C35	35	S420	420	763.4	763.4	Compression failure	-0.000151313	187.245705	267.642419	0.705	0.700	80.396714	2579
25	500	300	460	40	C40	40	S420	420	763.4	763.4	Compression failure	-0.000230152	207.39828	298.167236	0.701	0.696	90.7689563	2888.8
26	500	300	460	40	C45	45	S420	420	763.4	763.4	Compression failure	-0.00031613	227.371752	328.535545	0.698	0.692	101.163793	3198.5
27	500	300	460	40	C50	50	S420	420	763.4	763.4	Compression failure	-0.000407544	247.206292	358.785788	0.695	0.689	111.579496	3508.3
28	500	300	460	40	C16	16	S500	500	763.4	763.4	Compression failure	-0.000160547	113.372489	155.471158	0.735	0.729	42.0986692	1480.1
29	500	300	460	40	C18	18	S500	500	763.4	763.4	Compression failure	-0.000132666	122.348993	168.675514	0.731	0.725	46.3265203	1604
30	500	300	460	40	C20	20	S500	500	763.4	763.4	Compression failure	-0.000108708	131.253517	181.79408	0.728	0.722	50.5405634	1727.9
31	500	300	460	40	C25	25	S500	500	763.4	763.4	Compression failure	-6.12687E-05	153.276464	214.317115	0.721	0.715	61.0406516	2037.7
32	500	300	460	40	C30	30	S500	500	763.4	763.4	Compression failure	-0.000112112	174.177218	245.546691	0.715	0.709	71.3694732	2347.4
33	500	300	460	40	C35	35	S500	500	763.4	763.4	Compression failure	-0.000177479	194.720269	276.433098	0.710	0.704	81.7128285	2657.2
34	500	300	460	40	C40	40	S500	500	763.4	763.4	Compression failure	-0.000253183	215.000542	307.077472	0.706	0.700	92.0769306	2967
35	500	300	460	40	C45	45	S500	500	763.4	763.4	Compression failure	-0.000336528	235.081846	337.544585	0.702	0.696	102.462738	3276.7
36	500	300	460	40	C50	50	S500	500	763.4	763.4	Compression failure	-0.000425696	255.008731	367.877894	0.699	0.693	112.869163	3586.5

Figure 5-9 – Screenshot of Program to Calculate Resistance Factors for TS 500 (2000)

The results of this analysis indicated that the resistance factor for beams in tension controlled flexural failure was ranging from 0.810 to 0.876, hence the average value was taken as 0.86. For beams in compression controlled flexural failure, the resistance factor was ranging from 0.72 to 0.80 and its average was equal to 0.76 of the capacity of the beam calculated using unfactored material strengths. For beams under shear, the resistance factor was equal to 0.82 of the shear strength capacity of the beam calculated using unfactored material strengths. For combined flexure and axial load for columns, the resistance factor in tension failure was ranging from 0.80 to 0.85, the average was 0.81. For balanced failure, the resistance factor was ranging from 0.72 to 0.79 and the average was 0.74. For compression failure, the resistance factor was ranging from 0.7 to 0.78 and average was 0.76. The MCS was applied to determine the minimum reliability index between 0.1 and 1 for live and dead loads in different failure modes using bias and covariance for dead load as 1.05 and 0.1, respectively. The bias and covariance for live load were 1 and 0.18, respectively. The bias and covariance for resistance were 1.16 and 0.2, respectively. These results are tabulated in Table 5-8. Using these resistance factors and the procedures explained in Chapter 4, the reliability indexes were calculated for each resistance factor specified by TS 500 (2000) [3], as shown in this same table.

Table 5-8 – Resistance Factors and Target Reliability Index Based on TS 500 (2000) [3]

	Failure	**Resistance Factor (Φ)**	**Minimum Reliability Index (β)**
Beams	Compression	0.76	2.463
	Tension	0.86	2.134
	Only Shear	0.82	2.265
Columns	Compression	0.76	2.463
	Tension	0.81	2.298
	Only Shear	0.70	2.664

As seen from Table 5-3 and Table 5-8, TS500 (2000) [3] and ACI 318 (2019) [2] have some differences in the resistance factor which resulted in variation of minimum reliability indexes. For beams in compression controlled flexural and shear failures, ACI 318 (2019) [2] was safer. For beams in tension controlled flexural failure, TS 500 (2000) [3] was safer due to the fact that the reliability index was greater.

For columns in combined flexure and axial load and shear failures, the minimum reliability indexes in compression failure for ACI 318 (2019) [2] were greater than TS 500 (2000) [3] which resulted in safer deign whereas for columns in tension failure. TS 500 (2000) [3] was safer due to the fact that the reliability index was greater.

5.5 Determination of Custom Load and Resistance Factors

A program was developed to determine custom load and resistance factors to design reinforced concrete members (Figure 5-10). In this program, the user may enter the biases and covariances of resistance, dead, and live loads, manually (Figure 5-10(b)). These parameters may also be entered by selecting the related previous study in the literature (Figure 5-10(a)). Then the user shall select the member type (beam or column), failure mode (flexure, combined flexure, and axial load, shear), and the reinforced concrete code (TS 500, 2000 or ACI 318, 2019) [3,2] (Figure 5-10(c)). Based on these selections, the reliability index is calculated by the program as explained in Chapter 4. The user may also specify the target reliability index, without performing the selections (Figure 5-10(d)). For each step, the program performs the MSC Method using 20 million random variable and procedures explained in Chapter 4 and calculates custom load factors and combinations which results in the intended reliability index (Figure 5-10(e)). This procedure is performed many times for D/L ratios ranging from 0.1 to 1, for different load and resistance combination until the reliability index is equal to target reliability index entered by the user. The outcome of the program provides the user with a list of combination which has the same target reliability index (Figure 5-10(e)). The minimum target reliability and probability of failure are also provided in the outcome (Figure 5-10(e)). The time needed for a full run requires approximately 18 hours.

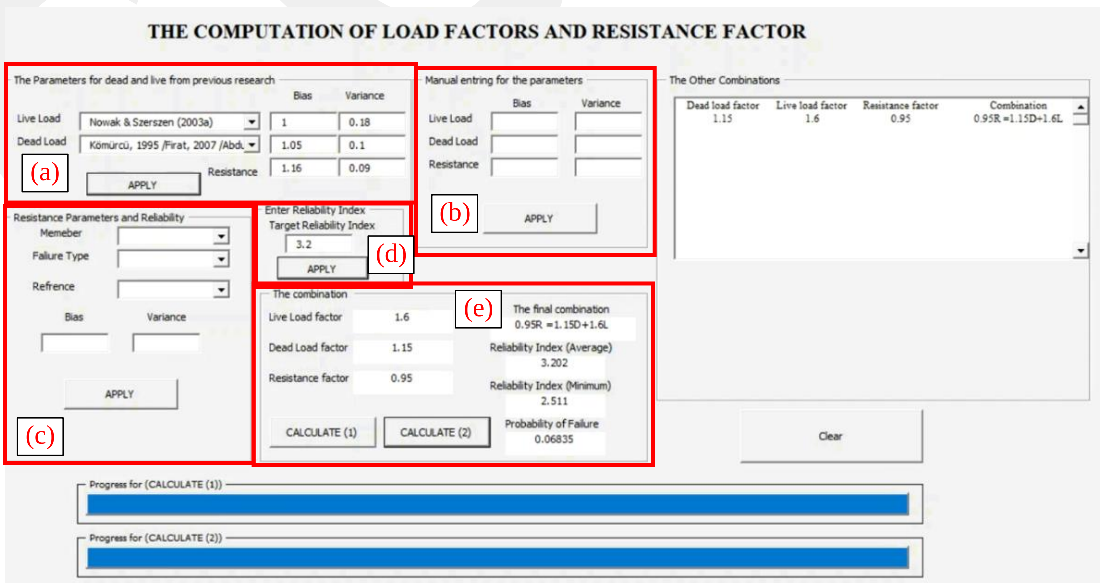


Figure 5-10 – Program for Determining Custom Load and Resistance Factors

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary and Conclusions

The load and resistance factors used nowadays in the design of reinforced concrete structures were developed before this century. The quality control on the materials nowadays is much better compared to that of 50 years ago. Furthermore, the loads on the structures can be predicted much better than the predictions performed 50 years ago. Using these factors from the past significantly penalizes the design of reinforced concrete structures constructed using materials having better quality control and loads having better predictions of occurrences today.

The purpose of this study was to develop a tool that determined the load and resistance factors depending on the statistical data (bias and covariance) related to current materials (concrete and steel) and current prediction of loads (dead, live, etc.) and the target reliability index. Initially for a reinforced concrete structure, beams and columns having various dimensions and reinforcement configurations were analyzed for various resistance factors and for different failure modes. These results were compared to the load effects resulting from various factors and combinations of dead and live loads. The reliability index for each comparison was calculated to evaluate the effects of load and resistance factors on the variation of safety (reliability index).

The FOSM and MCS were the methods used as the structural reliability models. The first method was used to determine the resistance parameters for different failure modes. These resistance parameters (biases and covariances) were calculated using 20 million random variables using MCS Method to determine reliability index values and to develop the load and resistance factors.

Following conclusions were reached at the end of this study:

- The reliability analysis for dead and live loads showed that, as the bias and covariance increased, the reliability index decreased. However, the variation of the dead load parameters was more critical, since the percentage of occurrence of dead loads in a structure was greater than that of the live loads.

- For the resistance parameters, as the bias increased, the reliability index increased as well. When the reliability index decreased, the covariance increased. Hence the bias for the resistance had a significant effect on the reliability index.
- The results from the analysis of load and resistance factors using Monte Carlo Simulation revealed that the dead load factors had greater effects on the reliability index due to their occurrence was larger than that of the live loads. Also, the live load factors had effects on reliability index, however the maximum reliability index for live load factors was approximately 3.25, whereas it was approximately 5 for the dead load factor.
- A program was developed to determine custom load and resistance factors to design reinforced concrete members. In this program, the user may enter the biases and covariances of resistance, dead, and live loads, manually. These parameters may also be entered by selecting the related previous study in the literature. Then the user shall select the member type (beam or column), failure mode (flexure, combined flexure, and axial load, shear), and the reinforced concrete code (TS 500, 2000 or ACI 318, 2019) [3, 2]. Based on these selections, the reliability index is calculated by the program as explained in Chapter 4. The user may also specify the target reliability index, without performing the selections. For each step, the program performs the MSC Method using 20 million random variable and procedures explained in Chapter 4 and calculates custom load factors and combinations which results in the intended reliability index. This procedure is performed many times for D/L ratios ranging from 0.1 to 1, for different load and resistance combination until the reliability index is equal to target reliability index entered by the user. The outcome of the program provides the user with a list of combinations which has the same target reliability index. The minimum target reliability and probability of failure are also provided in the outcome. The time needed for a full run requires approximately 18 hours.

6.2 Future Research Recommendations

- The methods and the developed program used in this analysis, as well as the methodological evidence on loads, can be applied to various building materials, such as metal frames, pre-stressed concrete, and engineered masonry, as well as structural members such as slabs and shear walls.
- The various loads such as earthquake, wind, soil pressure, etc. may be added to the combinations for future calibrations.

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APPENDIX A

Resistance Factor	Live Load Factor	Dead Load Factor	r = 0.1	r = 0.2	r = 0.3	r = 0.4	r = 0.5	r = 0.6	r = 0.7	r = 0.8	r = 0.9	r = 1
0.4	1	1	6.646	6.662	6.640	6.607	6.552	6.499	6.444	6.383	6.338	6.282
0.45	1	1	6.081	6.091	6.076	6.030	5.977	5.911	5.848	5.795	5.738	5.682
0.5	1	1	5.521	5.536	5.518	5.466	5.407	5.345	5.275	5.216	5.151	5.093
0.55	1	1	4.974	4.994	4.967	4.921	4.855	4.788	4.727	4.665	4.598	4.538
0.6	1	1	4.443	4.461	4.436	4.386	4.325	4.262	4.193	4.131	4.067	4.011
0.65	1	1	3.925	3.943	3.923	3.871	3.817	3.750	3.683	3.624	3.567	3.514
0.7	1	1	3.424	3.443	3.424	3.383	3.322	3.263	3.205	3.144	3.089	3.038
0.75	1	1	2.937	2.962	2.944	2.908	2.853	2.798	2.745	2.690	2.644	2.597
0.8	1	1	2.475	2.497	2.483	2.447	2.402	2.358	2.308	2.263	2.216	2.179
0.85	1	1	2.025	2.055	2.043	2.014	1.979	1.936	1.897	1.857	1.821	1.787
0.9	1	1	1.601	1.624	1.621	1.600	1.572	1.539	1.507	1.477	1.448	1.422
0.95	1	1	1.189	1.214	1.218	1.205	1.184	1.163	1.141	1.119	1.098	1.079
1	1	1	0.796	0.823	0.833	0.830	0.820	0.805	0.793	0.782	0.767	0.757
1	1.15	1	0.902	1.012	1.090	1.137	1.168	1.186	1.199	1.205	1.210	1.212
1	1.2	1	0.936	1.075	1.170	1.237	1.281	1.308	1.329	1.343	1.351	1.358
1	1.25	1	0.970	1.136	1.254	1.334	1.391	1.430	1.457	1.476	1.492	1.502
1	1.3	1	1.004	1.196	1.335	1.432	1.499	1.546	1.584	1.611	1.628	1.642
1	1.35	1	1.039	1.257	1.415	1.528	1.607	1.665	1.706	1.737	1.762	1.780
1	1.4	1	1.074	1.315	1.492	1.621	1.713	1.779	1.828	1.865	1.895	1.917
1	1.45	1	1.104	1.376	1.572	1.714	1.816	1.890	1.947	1.990	2.024	2.049
1	1.5	1	1.138	1.434	1.649	1.808	1.918	2.001	2.065	2.112	2.150	2.179
1	1.55	1	1.171	1.494	1.726	1.897	2.019	2.109	2.178	2.235	2.275	2.307
1	1.6	1	1.204	1.550	1.800	1.985	2.116	2.217	2.294	2.350	2.396	2.434
1	1.65	1	1.237	1.605	1.875	2.071	2.215	2.322	2.403	2.466	2.514	2.557
1	1.7	1	1.268	1.662	1.950	2.159	2.312	2.424	2.512	2.580	2.636	2.676
1	1.75	1	1.301	1.718	2.021	2.243	2.404	2.527	2.619	2.691	2.749	2.793
1	1.8	1	1.332	1.774	2.093	2.326	2.500	2.623	2.723	2.800	2.860	2.911
1	1.85	1	1.364	1.830	2.164	2.407	2.589	2.723	2.825	2.906	2.970	3.027
1	1.9	1	1.397	1.884	2.235	2.490	2.676	2.820	2.929	3.013	3.081	3.138
1	1.95	1	1.429	1.938	2.303	2.572	2.766	2.915	3.026	3.119	3.187	3.244
1	1	1.15	1.767	1.718	1.650	1.574	1.499	1.430	1.367	1.311	1.257	1.213
1	1	1.2	2.061	1.992	1.901	1.803	1.713	1.627	1.546	1.477	1.414	1.357
1	1	1.25	2.341	2.250	2.141	2.026	1.917	1.816	1.722	1.642	1.568	1.502
1	1	1.3	2.605	2.498	2.374	2.240	2.118	2.000	1.897	1.803	1.720	1.642
1	1	1.35	2.855	2.734	2.593	2.450	2.310	2.182	2.063	1.960	1.862	1.782
1	1	1.4	3.099	2.964	2.809	2.649	2.498	2.357	2.227	2.113	2.009	1.916
1	1	1.45	3.328	3.181	3.012	2.843	2.675	2.526	2.386	2.261	2.150	2.052
1	1	1.5	3.543	3.388	3.208	3.025	2.852	2.688	2.542	2.412	2.288	2.178
1	1	1.55	3.754	3.585	3.398	3.206	3.024	2.851	2.693	2.552	2.423	2.311
1	1	1.6	3.954	3.776	3.581	3.375	3.187	3.005	2.839	2.689	2.556	2.435
1	1	1.65	4.143	3.955	3.755	3.543	3.343	3.157	2.984	2.828	2.685	2.554
1	1	1.7	4.321	4.135	3.922	3.707	3.501	3.306	3.127	2.960	2.809	2.678

	h (mm)	bw (mm)	d (mm)	As (mm^2)	Failiure Status	Tenstion Steel strain	ε_y	Mn (N.m)	Mr (N.m)	(Mn/Mr)	σ	σ/Mr
1	600	300	550	3800	Tension Failiure	-0.0022	0.00182605	598402819	574467009	1.042	41975413	0.070
2	600	300	550	3648	Tension Failiure	-0.0024	0.00182605	581413848	558739089	1.041	41493033	0.071
3	600	300	550	3496	Tension Failiure	-0.0026	0.00182605	563845948	542406939	1.040	40911645	0.073
4	600	300	550	3344	Tension Failiure	-0.0029	0.00182605	545699120	525470560	1.038	40231271	0.074
5	600	300	550	3192	Tension Failiure	-0.0031	0.00182605	526973364	507929950	1.037	39451941	0.075
6	600	300	550	3040	Tension Failiure	-0.0034	0.00182605	507668679	489785110	1.037	38573689	0.076
7	600	300	550	2888	Tension Failiure	-0.0038	0.00182605	487785065	471036040	1.036	37596562	0.077
8	600	300	550	2736	Tension Failiure	-0.0042	0.00182605	467322523	451682740	1.035	36520615	0.078
9	600	300	550	2584	Tension Failiure	-0.0046	0.00182605	446281053	431725210	1.034	35345919	0.079
10	600	300	550	2432	Tension Failiure	-0.0051	0.00182605	424660654	411163449	1.033	34072564	0.080
11	600	300	550	2280	Tension Failiure	-0.0056	0.00182605	402461327	389997459	1.032	32700663	0.081
12	600	300	550	2128	Tension Failiure	-0.0062	0.00182605	379683071	368227239	1.031	31230364	0.082
13	600	300	550	1976	Tension Failiure	-0.0069	0.00182605	356325887	345852789	1.030	29661860	0.083
14	600	300	550	1824	Tension Failiure	-0.0077	0.00182605	332389774	322874108	1.029	27995410	0.084
15	600	300	550	1672	Tension Failiure	-0.0087	0.00182605	307874733	299291198	1.029	26231366	0.085
16	600	300	550	1520	Tension Failiure	-0.0099	0.00182605	282780763	275104057	1.028	24370221	0.086
17	600	300	550	1368	Tension Failiure	-0.0113	0.00182605	257107865	250312687	1.027	22412684	0.087
18	600	300	550	1216	Tension Failiure	-0.0131	0.00182605	230856038	224917086	1.026	20359815	0.088
19	600	300	550	1064	Tension Failiure	-0.0154	0.00182605	204025283	198917256	1.026	18213263	0.089
20	600	300	550	750	Tension Failiure	-0.0231	0.00182605	146765335	143293702	1.024	13496401	0.092

Average target reliability	Minimum reliability index	MAX reliability index
6.505	6.282	6.662
5.923	5.682	6.091
5.353	5.093	5.536
4.803	4.538	4.994
4.272	4.011	4.461
3.762	3.514	3.943
3.274	3.038	3.443
2.808	2.597	2.962
2.363	2.179	2.497
1.941	1.787	2.055
1.541	1.422	1.624
1.161	1.079	1.218
0.801	0.757	0.833
1.132	0.902	1.212
1.239	0.936	1.358
1.344	0.970	1.502
1.448	1.004	1.642
1.550	1.039	1.780
1.650	1.074	1.917
1.748	1.104	2.049
1.845	1.138	2.179
1.941	1.171	2.307
2.035	1.204	2.434
2.127	1.237	2.557
2.218	1.268	2.676
2.307	1.301	2.793
2.394	1.332	2.911
2.481	1.364	3.027
2.566	1.397	3.138
2.650	1.429	3.244
1.479	1.213	1.767
1.689	1.357	2.061
1.892	1.502	2.341
2.090	1.642	2.605
2.279	1.782	2.855
2.464	1.916	3.099
2.641	2.052	3.328
2.812	2.178	3.543
2.980	2.311	3.754
3.140	2.435	3.954
3.295	2.554	4.143
3.447	2.678	4.321

APPENDIX B

Subroutines used in Calculations

```
Private Sub CommandButton1_Click()
Sheets("sheet1").Visible = True
 Sheets("sheet1"). Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Worksheets("Sheet1"). Range("AW33:BB3208").ClearContents
Range("bg27"). Value = "0.55"
Dim i As Integer
i = 1
Do Until i > 8
Range("BG27"). Value = Range("BG27"). Value + 0.05
i = i + 1
Dim sh As Worksheet
Set sh = Sheets("sheet1")
Dim m As Long
Dim iTotalSheet As Integer
Dim pctDone As Single
Dim iLabelWidth As Integer
iTotalSheet = 355
iLabelWidth = 648
For m = 32 To iTotalSheet
Range ("BE" & m). Copy Range("AX2")
Range ("BF" & m). Copy Range("AX1")
Range ("BG" & m). Copy Range("AX3")
Range("AX1").Copy
Range("AW3090").End(xlUp).Offset(1, 0).PasteSpecial xlPasteValuesAndNumberFormats
Range("AX2").Copy
Range("AX3090").End(xlUp).Offset(1, 0).PasteSpecial xlPasteValuesAndNumberFormats
Range("AX3").Copy
Range("AY3090").End(xlUp).Offset(1, 0).PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _ :=False,
Transpose:=False
Range("AY2").Copy
Range("AZ3090").End(xlUp).Offset(1, 0).PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _:=False,
Transpose:=False
Range("BC18").Copy
Range("BA3090").End(xlUp).Offset(1, 0).PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _:=False,
Transpose:=False
Range("BC19").Copy
Range("BB3090").End(xlUp).Offset(1, 0).PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _ :=False,
Transpose:=False
 pctDone = m / iTotalSheet
 With UserForm1
.lblProgress.Width = pctDone * iLabelWidth
End With
DoEvents
Next m
Loop
Dim shmM As Worksheet
Set shmM = Sheets("sheet1")
Dim zz As Long
For zz = 33 To shmM.Range("bb3080").End(xlUp).Row
If Range("Bb" & zz).Value = Range("Bc" & zz).Value Then
Range("AW" & zz).Copy
Range("br1000").End(xlUp).Offset(1, 0).PasteSpecial xlPasteValuesAndNumberFormats
Range("ax" & zz).Copy
Range("bs1000").End(xlUp).Offset(1, 0).PasteSpecial xlPasteValuesAndNumberFormats
Range("ay" & zz).Copy
Range("bt1000").End(xlUp).Offset(1, 0).PasteSpecial xlPasteValuesAndNumberFormats
Range("az" & zz).Copy
```

```
Range("BU1000").End(xlUp).Offset(1, 0).PasteSpecial Paste:=xlPasteValues, Operation:=xlNone, SkipBlanks _ :=False,
Transpose:=False
End If
Next zz
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton10_Click()
Sheets("sheet1").Visible = True
 Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
 Calculation = Application.Calculation
 EnableEvents = Application.EnableEvents
 DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
 Application.ScreenUpdating = False
  Application.EnableEvents = False
  ActiveSheet.DisplayPageBreaks = False
Range("Br21").Copy
Range("aX1").PasteSpecial Paste:=xlPasteValues
Range("Bs21").Copy
Range("Ax2").PasteSpecial Paste:=xlPasteValues
Range("BT21").Copy
Range("ax3").PasteSpecial Paste:=xlPasteValues
Dim iTotalSheetm As Integer
Dim pctDonem As Single
Dim iLabelWidthm As Integer
iTotalSheetm = 16
iLabelWidthm = 648
For q = 7 To iTotalSheetm
Range("aW" & q).Copy Range("J2")
Range("aX" & q).Copy Range("C2")
Range("BB" & q).Copy
Range("Y2").PasteSpecial Paste:=xlPasteValues
TotalM = 0
For m = 1 To 10
Range("ad6:ad100005").Copy
Range("ap6").PasteSpecial Paste:=xlPasteValues
Range("ap6:ap100005").Sort _
Key1:=Range("ap6"), Order1:=xlAscending
TotalM = TotalM + Range("AK24").Value
avgg = TotalM / m
Next m
Range("AK28").Value = avgg
TotalE = 0
For E = 1 To 10
Range("ad6:ad100005").Copy
Range("ap6").PasteSpecial Paste:=xlPasteValues
Range("ap6:ap100005").Sort _
Key1:=Range("ap6"), Order1:=xlAscending
TotalE = TotalE + Range("AK24").Value
avggE = TotalE / E
Next E
Range("AK29").Value = avggE
Range("ak30").Copy
Range("BD" & q).PasteSpecial xlPasteValuesAndNumberFormats
pctDonem = q / iTotalSheetm
With UserForm1
.lblProgress3.Width = pctDonem * iLabelWidthm
End With
DoEvents
Next q
Range("aX1").Copy Range("BQ18")
Range("AX2").Copy Range("BR18")
Range("aX3").Copy
Range("BS18").PasteSpecial xlPasteValuesAndNumberFormats
Range("aX3").Copy
Range("BS18").PasteSpecial xlPasteValuesAndNumberFormats
Range("AY2").Copy
Range("BT18").PasteSpecial xlPasteValuesAndNumberFormats
Range("BD18").Copy
Range("BV18").PasteSpecial xlPasteValuesAndNumberFormats
Range("BD19").Copy
Range("BU18").PasteSpecial xlPasteValuesAndNumberFormats
Label43.Caption = Worksheets("Sheet1").Range("br18").Value
```

```
Label44.Caption = Worksheets("Sheet1").Range("bq18").Value
Label45.Caption = Worksheets("Sheet1").Range("bs18").Value
Label46.Caption = Round(Range("bu18").Value, 3)
Label49.Caption = Round(Range("bv18").Value, 3)
Label47.Caption = Worksheets("Sheet1").Range("bt18").Value
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton2_Click()
Sheets("sheet1").Visible = True
Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Worksheets("Sheet1").Range("y3").Value = TextBox5.Value
Worksheets("Sheet1").Range("y4").Value = TextBox6.Value
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton4_Click()
Sheets("sheet1").Visible = True
Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Worksheets("Sheet1").Range("c3").Value = TextBox1.Value
Worksheets("Sheet1").Range("c4").Value = TextBox2.Value
Worksheets("Sheet1").Range("j3").Value = TextBox3.Value
Worksheets("Sheet1").Range("j4").Value = TextBox4.Value
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton6_Click()
Sheets("sheet1").Visible = True
Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Worksheets("Sheet1").Range("c3").Value = TextBox8.Value
Worksheets("Sheet1").Range("c4").Value = TextBox9.Value
Worksheets("Sheet1").Range("j3").Value = TextBox7.Value
Worksheets("Sheet1").Range("j4").Value = TextBox10.Value
Worksheets("Sheet1").Range("y3").Value = TextBox12.Value
Worksheets("Sheet1").Range("y4").Value = TextBox13.Value
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton8_Click()
Sheets("sheet1").Visible = True
Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Me.lblProgress.Width = 0
Me.lblProgress3.Width = 0
```

```
Worksheets("Sheet1").Range("AW33:BB3208").ClearContents
Worksheets("Sheet1").Range("BR21:BU531").ClearContents
TextBox1.Value = ""
TextBox2.Value = ""
TextBox3.Value = ""
TextBox4.Value = ""
TextBox6.Value = ""
TextBox5.Value = ""
TextBox15.Value = ""
TextBox8.Value = ""
TextBox9.Value = ""
TextBox7.Value = ""
TextBox10.Value = ""
TextBox12.Value = ""
TextBox13.Value = ""
Label43.Caption = ""
Label44.Caption = ""
Label45.Caption = ""
Label47.Caption = ""
Label46.Caption = ""
Label49.Caption = ""
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub CommandButton9_Click()
Sheets("sheet1").Visible = True
Sheets("sheet1").Select
ScreenUpdating = Application.ScreenUpdating
Calculation = Application.Calculation
EnableEvents = Application.EnableEvents
DisplayPageBreaks = ActiveSheet.DisplayPageBreaks
Application.ScreenUpdating = False
Application.EnableEvents = False
ActiveSheet.DisplayPageBreaks = False
Worksheets("Sheet1").Range("bb1").Value = TextBox15.Value
Application.ScreenUpdating = True
Sheets("sheet1").Visible = False
End Sub

Private Sub UserForm_Activate()
Me.lblProgress.Width = 0
 Me.lblProgress3.Width = 0
End Sub

Private Sub UserForm_Initialize()
Dim sh As Worksheet
Set sh = Sheets("sheet1")
'declare variable
Dim x As Long
For x = 2 To sh.Range("cl200").End(xlUp).Row
If Application.WorksheetFunction.CountIf(sh.Range("cl2", "cl" & x), sh.Cells(x, 90)) = 1 Then
Me.ComboBox2.AddItem sh.Cells(x, 90)
End If
Next x
Dim shm As Worksheet
Set shm = Sheets("sheet1")
'declare variable
Dim m As Long
For m = 2 To shm.Range("ce200").End(xlUp).Row
If Application.WorksheetFunction.CountIf(sh.Range("ce2", "ce" & m), sh.Cells(m, 83)) = 1 Then
Me.ComboBox3.AddItem shm.Cells(m, 83)
End If
Next m
End Sub

Private Sub ComboBox10_Change()
Dim x As Long, xx As Long, sh As Worksheet
Set sh = Sheets("sheet1")
xx = sh.Range("ci" & Rows.Count).End(xlUp).Row
For x = 3 To xx
If Me.ComboBox10.Value = sh.Cells(x, "ci") Then
TextBox5.Value = sh.Cells(x, "cg").Value
TextBox6.Value = sh.Cells(x, "ch").Value
End If
```

```
Next x
End Sub

Private Sub ComboBox9_Change()
Me.ComboBox10.Clear
' set worksheet
Dim shm As Worksheet
Set shm = Sheets("sheet1")
'declare variable
Dim m As Long
For m = 2 To shm.Range("ce200").End(xlUp).Row
If shm.Cells(m, 84) = Me.ComboBox9.Value Then
Me.ComboBox10.AddItem shm.Cells(m, 87)
End If
Next m
End Sub

Private Sub ComboBox3_Change()
Me.ComboBox9.Clear
' set worksheet
Dim shm As Worksheet
Set shm = Sheets("sheet1")
'declare variable
Dim m As Long
For m = 2 To shm.Range("ce200").End(xlUp).Row
If shm.Cells(m, 83) = Me.ComboBox3.Value Then
If Application.WorksheetFunction.CountIf(shm.Range("cf2", "cf" & m), shm.Cells(m, 84)) = 1 Then
Me.ComboBox9.AddItem shm.Cells(m, 84)
End If
End If
Next m
End Sub

Private Sub ComboBox2_Change()
Me.ComboBox4.Clear.set worksheet
Dim sh As Worksheet
Set sh = Sheets("sheet1")
'declare variable
Dim x As Long
For x = 2 To sh.Range("cl200").End(xlUp).Row
If sh.Cells(x, 90) = Me.ComboBox2.Value Then
If Application.WorksheetFunction.CountIf(sh.Range("cm2", "cm" & x), sh.Cells(x, 91)) = 1 Then
Me.ComboBox4.AddItem sh.Cells(x, 91)
End If
End If
Next x
End Sub

Private Sub ComboBox4_Change()
Dim x As Long, xx As Long, sh As Worksheet
Set sh = Sheets("sheet1")
xx = sh.Range("ci" & Rows.Count).End(xlUp).Row
For x = 2 To xx
If Me.ComboBox4.Value = sh.Cells(x, "cm") Then
TextBox1.Value = sh.Cells(x, "cn").Value
TextBox2.Value = sh.Cells(x, "co").Value
TextBox3.Value = sh.Cells(x, "cp").Value
TextBox4.Value = sh.Cells(x, "cq").Value
End If
Next x
End Sub
```